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Statens vegvesen

# Strait Crossings 2013

Proceedings

STATENS VEGVESENS RAPPORTER

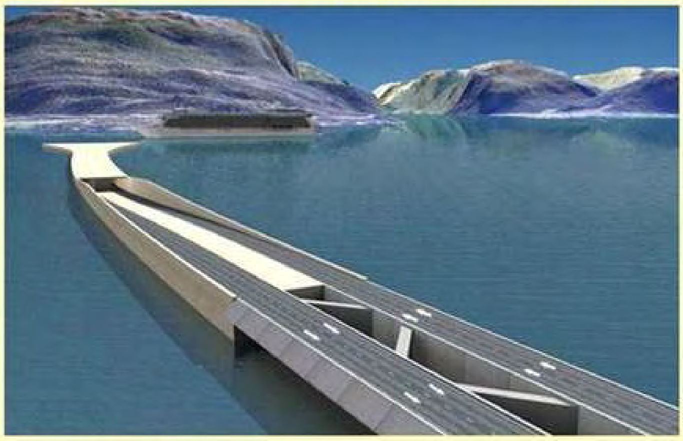
Nr. 231

Strait Crossings 2013

Sixth Symposium on Strait Crossings

STRAIT CROSSINGS NORWAY 2013

*Extreme Crossings and New Technologies*



16 – 19 June 2013
Radisson Blu Royal Hotel, Bryggen
Bergen, Norway

Statens vegvesen
Norwegian Public Roads Administration

Tekna

## Statens vegvesens rapporter

**Tittel**
Strait Crossings 2013

**Undertittel**
Proceedings

**Forfatter**
Tor Erik Frydenlund, Kaare Flaate, Håvard Østlid

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Olav Egil Ellevset

**Godkjent av**
Olav Egil Ellevset

**Emneord**
Bruer, Tunneler, Ferjer, Regional utvikling

**Sammendrag**
Presentasjon av symposiets styringsorganer, program og godkjente bidrag levert til symposiet.

**Antall sider** xxviii + 1051

**Dato** 2013-08-21

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Norwegian Public Roads Administration

**Title**
Strait Crossings 2013

**Subtitle**
Proceedings

**Author**
Tor Erik Frydenlund, Kaare Flaate, Håvard Østlid

**Department**

**Section**
Director General's Staff

**Project number**
603248

**Report number**
No. 231

**Project manager**
Olav Egil Ellevset

**Approved by**
Olav Egil Ellevset

**Key words**
Bridges, Tunnels, Ferries, Regional development

**Summary**
Presentation of the Organising bodies, the Symposium Programme and approved papers submitted to the symposium.

**Pages** xxviii + 1051

**Date** 2013-08-21

**PROCEEDINGS OF THE SIXTH SYMPOSIUM ON STRAIT CROSSINGS, BERGEN, NORWAY JUNE 16TH–19TH, 2013.**

# Strait Crossings 2013

Bergen, Norway

Tor Erik Frydenlund
Kaare Flaate
Håvard Østlid

Editors

Cover illustration, courtesy of:
The Norwegian Public Roads Administration



ISSN 1893-1162 paper version

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## Table of approved papers

# Preface

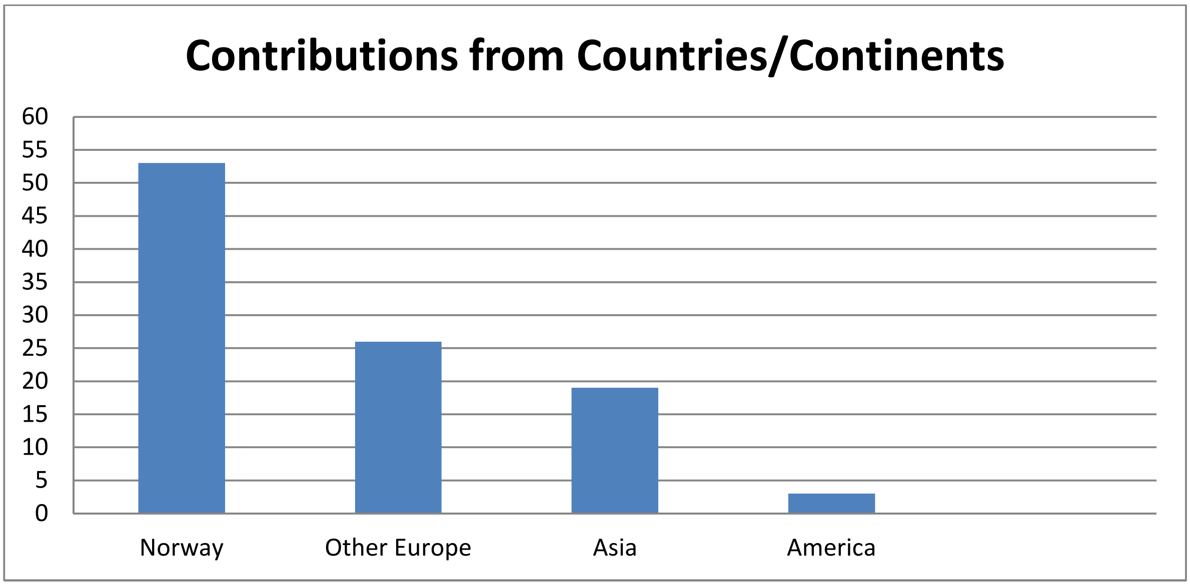
Symposiums on Strait Crossings have been arranged on five previous occasions, all of them in Norway. The first took place in Stavanger in 1986, the second in Trondheim in 1990, the third in Ålesund in 1994, the fourth in Bergen in 2001 and the fifth in Trondheim in 2009.

Strait Crossings will remain an important subject as long as straits, fjords and sounds represent restrictions to road and rail transport. There is a continuous demand for more time efficient, cost effective and environmentally friendly solutions for strait crossings.

Improved connections can be obtained through advanced design of bridges and tunnels as well as by new ferry technology. In addition studies of socio economic effects, environmental conditions and traffic safety are required.

We are pleased to present the proceedings for the Sixth Symposium on Strait Crossings held in Bergen, Norway, June 19 - 19, 2013. The main themes of the symposium are Bridges, Tunnels, Ferries and Regional development with focus on Extreme Crossings and New Technologies.

The proceedings include approved 101 papers, with 9 papers presented in plenary sessions, 82 in parallel sessions (3 not presented) and 9 Poster presentations.  The opening address was given by the Director General of the Norwegian Public Roads Administration, Mr. Terje Moe Gustavesen (PowerPoint presentation). For the first plenary lecture given by Professor Victor Norman at the Norwegian School of Economics there is no written record. Authors from 21 different countries have contributed with a distribution as shown below.




We convey our great appreciation and thanks to the authors for their contributions and for efforts made to share their knowledge and experience with us. We are confident that these proceedings represent a valuable source of information for owners, contractors, engineers and planners of strait crossings.

A total of 246 participants from 26 countries registered for the symposium and 241 participants attended. Copies of the symposium papers were presented to participants on memory sticks in pdf-fomat. The present version of the Symposium Proceedings has been prepared after the symposium

Tor Erik Frydenlund
Kaare Flaate
Håvard Østlid

Editors

# Organisation

The Sixth Symposium on Strait Crossings, held on June 16$^{th}$ – 19$^{th}$ 2013, at Radisson Blu Royal Hotel, Bryggen, Bergen was organised by The Norwegian Public Roads Administration in cooperation with Tekna - The Norwegian Society of Graduated Technical and Scientific Professionals.

**Organising Committee**

| | | |
|---|---|---|
| Olav Egil Ellevset (Chair) | Jan Eirik Henning | Jan Olav Skogland |
| Kjell Inge Davik | Bernt Jakobsen | Lidvard Skorpa |
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| Kaare Flaate | Birgitte Rodum | Jinquan Zhang, China |
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| | | |
|---|---|---|
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| Kaare Flaate | Frode Oset | Håvard Østlid |
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| | | |
|---|---|---|
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| Heinz Erbahr, Switzerland | Svein Erik Jakobsen, Norway | Peter O'Neill, USA |
| Yufang Fu, China | Shunji Kanie, Japan | Yoshikazu Ota, Japan |
| Walter Grantz, USA | Satoshi Kashima, Japan | Frederico Perotti, Italy |
| Lok Home, USA | Casper Paludan-Müller, Denmark | Ilkka Vähäaho, Finland |
| | | Yingxin Zhou, Singapore |

**Symposium Secretariat**
Tekna: Siri Ebro Engen
Organising and Programme Committees: Tor Erik Frydenlund – Secretary (S)

**Acknowledgements**

Our thanks and appreciation goes to the symposium sponsors and cooperating organisations listed in the symposium programme for supporting the event.

Olav Egil Ellevset
Chairman of the Organizing and Programme Committees

# Programme

## for the

# 6th Symposium

## on

# Strait Crossings

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# Extreme Crossings

## and

# New Technologies

Statens vegvesen
Norwegian Public Roads Administration

Tekna

Secretariat: Tekna – The Norwegian Society of Graduate Technical and Scientific Professionals

16 – 19 June 2013 Radisson Blu Royal Hotel, Bryggen, Bergen, Norway

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Lilleakerveien 4, N-0283 Oslo, Norway

| | | | | | |
|---|---|---|---|---|---|
| **Sunday 16 June** | | | | | |
| **Time** | **Topic** | **Activity** | | | |
| 18:00 - | Registration | | | | |
| 19:00 - | Reception | Dinner a la carte available * | | | |
| **Monday 17 June** | | | | | |
| **Time** | **Topic** | **Activity** | | | |
| 08:00 - | Registration cont. | **Session chair: Christian Ingerslev** **Cochair: Ian Markey** | | | |
| 08:30 - | Plenary session | Welcome speech: Director General Terje Moe Gustavsen, Norwegian Public Roads Administration | | | |
| 08:45 - | | Impact of fixed links on regional development, Professor Victor Norman, Norwegian School of Economics | | | |
| 09:30 - | | 121 Coastal Highway Route E 39, Project Manager Olav Ellevset, Norwegian Public Roads Administration | | | |
| 10:10 | | 22 The Fehmarnbelt Fixed Link, Director Steen Lykke, Fehmarnbelt, Denmark | | | |
| 10:45 | Coffee break | | | | |
| 11:10 - | Plenary session | 11 Akashi Kaikyo Bridge Project, Shatoshi Kashima, Japan Bridge Engineering Center, Japan | | | |
| 11:40 - | | 73 SR 520 Floating Bridge and Landings Project, Lake Washington, Seattle, Arnfinn Rusten, Berger ABAM Inc. | | | |
| 12:05 - | | 88 The key to efficient ferry operations, Anker Grøvdal, Fjord 1, Norway | | | |
| 12:30 | Lunch | | | | |
| | **Location** | **Room 2** | **Room 3** | **Room 4** | **Room 1** |
| | | **Bridges A**<br>Session chair:<br>Yufang Fu | **Tunnels**<br>Session chair:<br>Steen Lykke | **Ferries**<br>Session chair:<br>Anker Grøvdal | **Regional development**<br>Session chair:<br>Olav Ellevset |
| 13:30 - | Paper presentations | 37 Floating bridge | 22 Immersed | 116 Battery operated | 122 Impact of fixed links |
| | | 87 Floating bridge | 67 Immersed | 89 Efficiency | 24 Impact of fixed links |
| | | 10 Floating bridge | 30 Immersed | 79 Terminals | 105 Regional development |
| | | 16 Floating bridge | 71 SFT | 42 Terminals | 120 Regional development |
| 15:10 | Coffee break | | Tommy Olsen | | |
| 15:35 - | Paper presentations | 84 Floating bridge | 1 SFT | | 4 Procurement |
| | | 47 Floating bridge | 9 SFT | | |
| | | 69 Floating bridge | 56 SFT | | |
| | | 97 Floating bridge | 48 SFT | | |
| 17:15 | End of presentations | | | | |
| 18:00 - | | Social event: Venue in front of the hotel entrance ** | | | |

* Not included in symposium fee. ** Guided walking tour of Bergen including the Hanseatic Wharf, Ride with the Fenicular to Fløien and dinner at Fløien.

For details regarding parallel sessions see pages xvi-xiii for **Bridges**, xix-xxi for **Tunnels**, xxii for **Tunnels/Bridges**, xxiii for **Ferries** and xxiv for **Regional development.** For **Room plan** see page xxv.

| Tuesday 18 June | | | | | |
|---|---|---|---|---|---|
| **Time** | **Topic** | **Activity** | | | |
| | **Location** | **Room 1 and 2** | **Room 3** | **Room 4** | |
| | | **Bridges A**<br>Session chair:<br>Naeem Hussain | **Tunnels**<br>Session chair:<br>Marco Ramoni | **Tunnels/bridges**<br>Session chair:<br>Ismail bin Mohamed Taib | |
| 08:30 | Paper presentations | 108 Floating bridge | 102 SFT Fire | 36 Feasibility study | |
| | | 110 Floating bridge | 28 SFT | 39 Design conditions | |
| | | 6 Cable stayed | 101 SFT | 44 Moorings | |
| | | 41 Cable stayed | 38 Subsea - experience | 86 Site investigations | |
| 10:10 | Coffee break | | | | |
| 10:30 | Paper presentations | 26 Suspension Bridge | 49 Subsea – oil fields | 81 Competitive dialogue | |
| | | 27 Suspension Bridge | 117 Subsea - Gibraltar | 98 Combined solution | |
| | | 54 Dynamics | 21 Subsea – ultra long | 82 Feasibility study | |
| | | 20 Dynamics | 32 Subsea - feasibility | 111 Feasibility study | |
| | | 113 Dynamics | 62 Subsea - Xiamen Xiang | 57 Energy | |
| 12:35 - | Lunch | | | | |
| | | **Bridges**<br>Session chair:<br>Arnfinn Rusten | **Tunnels**<br>Session chair:<br>Martin Knights | **Tunnels/Bridges**<br>Session chair:<br>Jonathan Baber | |
| 13:30 | Paper presentations | 119 Design analysis | 96 Subsea – Tallin Helsinki | 80 Lighting | |
| | | 19 Wind | 35 Subsea | 91 Material properties | |
| | | 29 Wind | 18 Subsea | 5 Corrosion | |
| | | 52 Wind | 31 Subsea | 58 Floating bridge | |
| 15:10 | Coffee break | | | | |
| 15:35 | Paper presentations | 118 Cables | 40 Subsea - maintenance | 70 Deep sea foundations | |
| | | 14 Cables | 17 Lining | 106 Construction | |
| | | 107 Cables | 2 TBM | 90 Monitoring | |
| | | 12 Concrete | 50 TBM | | |
| 17:15 | End of presentations | | | | |
| 19:00 - | | Banquet: Venue in front of the hotel entrance for a 5 min. walk to the Håkonshallen banquet hall. | | | |

For details regarding parallel sessions see pages xvi-xiii for **Bridges**, xix-xxi for **Tunnels**, xxii for **Tunnels/Bridges**, xxiii for **Ferries** and xxiv for **Regional development.** For **Room plan** see page xxv.

| **Wednesday 19 June** | | | | | |
|---|---|---|---|---|---|
| **Time** | **Topic** | **Activity** | | | |
| | **Location** | **Room 1 and 2** | **Room 3 and 4** | | |
| | | **Bridges**<br>Session chair: Vanja Samec | **Tunnels**<br>Session chair: Bjørn Nilsen | | |
| 08:30 – | Paper presentations | 43 Concrete | 75 Knowledge transfer | | |
| | | 77 Concrete | 76 Requirements | | |
| | | 15 Concrete | 103 Rescue tunnels | | |
| | | 61 Life cycle | 104 Subsea – Bjorøy | | |
| 10:10 | Coffee break | **Session chair: Walter Grantz** | **Cochair: Håvard Østlid** | | |
| 10:35 | Plenary session | 100 The past, present and future of the Seikan Tunnel, Motohiro Sato, Japan | | | |
| 11:10 | | 63 Environmental footprint in early planning of coastal road sections, Johan Berg Pettersen,, MiSA, Norway | | | |
| 11:45 | | 115 FOR - Forever open roads, Steve Phillips, Secretary General FEHRL | | | |
| 12:20 | Symposium closure | Project Manager Olav Ellevset, Norwegian Public Roads Administration | | | |
| 12:35 | Lunch | | | | |
| 14:00 – | | Departure for technical visits | | | |

**Poster presentations:** The following papers have been approved for poster presentations: 3, 7, 34, 55, 74, 92, 93, 95 and 99.

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**Technical visits:** A) Bergen – Vallavik – Hardanger bridge – Fykse/Steinstø – Steindalsfossen - Return to Bergen at 22:00 hours

**Short technical tours:** B) Bergen - Nordhordland bridge - Return to Bergen at 17:00 hours
C) Bergen – Bjorøy-tunnel - Return to Bergen at 17:00 hours

**Sightseeing in Bergen:** Social event Monday night.

**Touristic tours:**

| | |
|---|---|
| Sognefjord in a nutshell | http://www.norwaynutshell.com/ |
| Norway in a nutshell | http://www.norwaynutshell.com/ |
| Island hopping on the west coast of Norway | http://www.fjordkysten.no/en/WHAT-TO-DO/Island-Hopping/ |

Touristic tours must me booked separately through tour operators.

# Sessions on Bridges – Room 2

| | | | |
|---|---|---|---|
| **Sunday 16th of June** | | | |
| **Time** | **Topic** | **Activity** | |
| 18:00 - | Registration | | |
| 19:00 - | Reception | Dinner a la carte available * | |
| **Monday 17th of June** | | | |
| **Time** | **Topic** | **(paper no.) Presenter** | **Paper** |
| 08:00 - | Registration continued | **Session chair: Christian Ingerslev** | **Cochair: Ian Markey** |
| 08:30 - | Plenary session | Welcome speech: Director General Terje Moe Gustavsen, Norwegian Public Roads Administration | |
| 08:45 - | | Impact of fixed links on regional development, Professor Victor Norman, Norwegian School of Economics | |
| 09:30 - | | 121 Coastal Highway Route E 39, Project Manager Olav Ellevset, Norwegian Public Roads Administration | |
| 10:10 - | | 22 The Fehmarnbelt Fixed Link, Director Steen Lykke, Fehmarnbelt, Denmark | |
| 10:45 – | Coffee break | | |
| 11:10 - | Plenary session | 11 Akashi Kaikyo Bridge Project, Shatoshi Kashima, Japan Bridge Engineering Center, Japan | |
| 11:40 - | | 73 SR 520 Floating Bridge and Landings Project, Lake Washington, Seattle, Arnfinn Rusten, Berger ABAM Inc. | |
| 12:05 - | | 88 The key to efficient ferry operations, Anker Grøvdal, Fjord 1, Norway | |
| 12:30 – | Lunch | **Session chair: Yufang Fu** | **Cochair: Børre Stanesvold** |
| 13:30 - | Floating bridge | 37 Jan Olav Skogland | The Nordhordland bridge – twenty years in service |
| | | 87 Jan Scheie | Re-furbishment of The Bergsøysund Floating Bridge (2011-2013) |
| | | 10 Birger Opgård | Chained floating bridge |
| | | 16 Svein Erik Jakobsen | Concept development of a Sognefjord floating bridge crossing |
| 15:10 – | Coffee break | | |
| 15:35 – | Floating bridge | 84 S. Randrup-Thomsen | Bridge crossings at Sognefjorden - Ensuring technical feasible and safe solutions |
| | | 47 Bernt Jakobsen | Ship impacts on the floating pontoons supporting a multiple span suspension bridge |
| | | 69 Rolf Magne Larsen | Deep Sea Floating Foundations for Strait Crossings |
| | | 97 Terje Norddal | Bridge across the Trondheimsfjord |
| 17:15 - | End of presentations | | |
| 18:00 - | | Social event: Venue in front of the hotel entrance ** | |

* Not included in symposium fee. ** Guided walking tour of Bergen including the Hanseatic Wharf, Ride with the Fenicular to Fløien and dinner at Fløien.

For details regarding parallel sessions see pages xvi-xiii for **Bridges**, xix-xxi for **Tunnels**, xxii for **Tunnels/Bridges**, xxiii for **Ferries** and xxiv for **Regional development.** For **Room plan** see page xxv.

# Sessions on Bridges – Room 1 and 2

| Tuesday 18th of June | | | |
|---|---|---|---|
| **Time** | **Topic** | **(paper no.) Presenter** | **Paper** |
| | | **Session chair: Naeem Hussain** | **Cochair: Svein Erik Jakobsen** |
| 08:30 - | Floating bridge | 108 Inge-Bertin Almeland | TLP technology experiences in the North Sea used as foundations for a bridge tower. |
| | | 110 Richard Monster | TLP technology: The tether system. |
| | Cable stayed bridge | 6 Steve Kite | Pushing the Limits of Cable Stayed Bridges – the Partially Earth Anchored Solution |
| | | 41 Tina Vejrum | Long Span Cable Stayed Bridges for Railway |
| 10:10 – | Coffee break | | |
| 10:30 – | Suspension bridge | 26 Kristian Berntsen | A 3700 m single span suspension bridge |
| | | 27 Volkert Oosterlaak | How to cross the 7500 m wide Boknafjord? |
| | Dynamics | 54 Lars Jensen | Izmit Bay Bridge - seismic design |
| | | 20 Ole Øiseth | Prediction of wave induced dynamic response in time domain using the finite element method |
| | | 113 Federico Perotti | Feasible control strategies in the protection of long span bridges against external dynamic loads |
| 12:35 – | Lunch | **Session chair: Arnfinn Rusten** | **Cochair: Tor Ole Olsen** |
| 13:30 – | Design analysis | 119 Vanja Samec | Consistent structural analysis and design of long-span bridges |
| | Wind | 19 Ole Øiseth | Prediction of wind-induced dynamic response and flutter stability limit of long-span bridges using the finite element method |
| | | 29 Jesna B. Jakobsen | Wind-induced vibrations of dry inclined stay cables in the critical Reynolds number range |
| | | 52 Bjørn Isaksen | Effects of topography on gusty wind action for long-span suspension bridges |
| 15:10 – | Coffee break | | |
| 15:35 – | Cables | 118 Martin Laube | New test methods for cable systems |
| | | 14 Jagan Mohanraj | Development of sheathed strands for bridges and permanent mooring applications |
| | | 107 Howard Wilkinson | Access solutions for suspension bridge cable maintenance |
| | Concrete | 12 Ismail bin Mohamed Taib | Sustainability through innovation in design and construction: Second Penang Bridge, Malaysia |
| 17:15 | End of presentation | | |
| 19:00 - | | Banquet: Venue in front of the hotel entrance for a 5 min. walk to the Håkonshallen banquet hall. | |

For details regarding parallel sessions see pages xvi-xiii for **Bridges**, xix-xxi for **Tunnels**, xxii for **Tunnels/Bridges**, xxiii for **Ferries** and xxiv for **Regional development.** For **Room plan** see page xxv.

# Sessions on Bridges – Room 1 and 2

| Wednesday 19th of June | | | |
|---|---|---|---|
| **Time** | **Topic** | **Presenter (paper no.)/Activity** | **Paper** |
| | | **Session chair: Vanja Samec Cochair: Johannes Veie** | |
| 08:30 – | Concrete | 43 Yu-Fang Fu | Monitoring of Effective Stresses in Prestressing Steel Strand for PC Bridges |
| | | 77 Gongyi Xu | Innovative Design for Saivan Double-deck Bridge with Twin PC Girder in Macau |
| | | 15 Sturla Rambjør | The Tverland Bridge. A four lane Free Cantilever Bridge in northern Norway |
| | Life cycle | 61 Johanne Hammervold | Life Cycle Assessment of bridges; Accomplishment and implementation |
| 10:10 | Coffee break | **Session chair: Walter Grantz Cochair: Håvard Østlid** | |
| 10:35 | Plenary session | 100 The past, present and future of the Seikan Tunnel, Motohiro Sato, Japan | |
| 11:10 | | 63 Environmental footprint in early planning of coastal road sections, Johan Berg Pettersen,, MiSA, Norway | |
| 11:45 | | 115 FOR - Forever open roads, Steve Phillips, Secretary General FEHRL | |
| 12:20 | Symposium closure | Project Manager Olav Ellevset, Norwegian Public Roads Administration | |
| 12:35 | Lunch | | |
| 14:00 – | | Departure for technical visits | |

For details regarding parallel sessions see pages xvi-xiii for **Bridges**, xix-xxi for **Tunnels**, xxii for **Tunnels/Bridges**, xxiii for **Ferries** and xxiv for **Regional development.** For **Room plan** see page xxv.

# Sessions on Tunnels – Room 3

| Time | Topic | Activity | |
|---|---|---|---|
| **Sunday 16th of June** | | | |
| **Time** | **Topic** | **Activity** | |
| 18:00 - | Registration | | |
| 19:00 - | Reception | Dinner a la carte available * | |
| **Monday 17th of June** | | | |
| **Time** | **Topic** | **(paper no.) Presenter** | **Paper** |
| 08:00 - | Registration continued | **Session chair: Christian Ingerslev** | **Cochair: Ian Markey** |
| 08:30 - | Plenary session | Welcome speech: Director General Terje Moe Gustavsen, Norwegian Public Roads Administration | |
| 08:45 - | | Impact of fixed links on regional development, Professor Victor Norman, Norwegian School of Economics | |
| 09:30 - | | 121 Coastal Highway Route E 39, Project Manager Olav Ellevset, Norwegian Public Roads Administration | |
| 10:10 - | | 22 The Fehmarnbelt Fixed Link, Director Steen Lykke, Fehmarnbelt, Denmark | |
| 10:45 – | Coffee break | | |
| 11:10 - | Plenary session | 11 Akashi Kaikyo Bridge Project, Shatoshi Kashima, Japan Bridge Engineering Center, Japan | |
| 11:40 - | | 73 SR 520 Floating Bridge and Landings Project, Lake Washington, Seattle, Arnfinn Rusten, Berger ABAM Inc. | |
| 12:05 - | | 88 The key to efficient ferry operations, Anker Grøvdal, Fjord 1, Norway | |
| 12:30 – | Lunch | **Session chair: Steen Lykke** | **Cochair: Kjersti Kvalheim Dunham** |
| 13:30 - | Immersed tunnel | 22 Antonius Hemel | The Fehmarnbelt tunnel crossing: The world longest IMT |
| | | 67 Tommy Olsen | Planning and construction of complex immersed tunnels |
| | | 30 Ian Markey | The Bjørvika Immersed Tunnel - assessments of calculated and measured settlements over 3 year period |
| | SFT | 71 Andreas Saur Brandtsegg | Development of a submerged floating tunnel concept for crossing of the Sognefjord |
| 15:10 – | Coffee break | **Session chair: Tommy Olsen** | **Cochair: Kjersti Kvalheim Dunham** |
| 15:35 – | SFT | 1 Walter Grantz | The Immersed Bridge Tunnel |
| | | 9 Birger Opgård | Submerged floating tunnel in steel for Sognefjorden |
| | | 56 Arne Instanes | Crossing of Wide Fjords by Double Walled Submerged Floating Tunnel (SFT) |
| | | 48 Bernt Jakobsen | Various SFT concepts for crossing wide and deep fjords |
| 17:15 - | End of presentations | | |
| 18:00 - | | Social event: Venue in front of the hotel entrance ** | |

* Not included in symposium fee. ** Guided walking tour of Bergen including the Hanseatic Wharf, Ride with the Fenicular to Fløien and dinner at Fløien.

For details regarding parallel sessions see pages xvi-xiii for **Bridges**, xix-xxi for **Tunnels**, xxii for **Tunnels/Bridges**, xxiii for **Ferries** and xxiv for **Regional development.** For **Room plan** see page xxv.

# Sessions on Tunnels – Room 3

| **Tuesday 18th of June** | | | |
|---|---|---|---|
| **Time** | **Topic** | **(paper no.) Presenter** | **Paper** |
| | | **Session chair: Marco Ramoni** | **Cochair: Bernt Jakobsen** |
| 08:30 - | SFT Fire | 102 Haukur Ingason | New challenges for the fire safety in submerged floating tunnels |
| | SFT | 28 Federico Perotti | The non-linear dynamic response of Submerged Floating Tunnels to earthquake and seaquake excitation. |
| | | 101 Myung Sagong | Simplified collision analyses for floating tunnel using the theory of beam with elastic foundation |
| | Subsea | 38 Jan Eirik Henning | Thirty years of experience with subsea road tunnels in Norway |
| 10:10 – | Coffee break | | |
| 10:30 – | Subsea | 49 Eivind Grøv | Subsea tunnels to oil field developments in northern Norway. TBM-tunnelling at 300 m water depth in sedimentary rock |
| | | 117 Yves Boissonnas | The Gibraltar Crossing |
| | | 21 Gareth Mainwaring | Ultra-Long Undersea Tunnels |
| | | 32 Bjørn Nilsen | Methodology for predicting and handling challenging rock mass conditions in hard rock subsea tunnels |
| | | 62 Ziping Huang | Design and construction of Xiamen Xiang'an subsea tunnel |
| 12:35 – | Lunch | **Session chair: Martin Knights** | **Cochair: Eivind Grøv** |
| 13:30 – | Subsea | 96 Ossi Ikävalko | Soil and bedrock conditions to be expected in Tallinn - Helsinki -tunnel construction |
| | | 35 Tor Geir Espedal | The Rogfast project |
| | | 18 Usko Anttikoski | Europe's northern dimension from a transportation perspective |
| | | 31 Gunnar Eiterjord | Rv. 13 Ryfast – world longest subsea tunnel combined with E39 Eiganestunnelen |
| 15:10 – | Coffee break | | |
| 15:35 – | Subsea | 40 Gunnar Gjæringen | Factors affecting operation, maintenance and costs in Subsea tunnels |
| | Lining | 17 Marco Ramoni | Inner lining in traffic tunnels |
| | TBM | 2 Marco Ramoni | On the applicability of tunnel boring machines for the excavation of long strait crossing tunnels |
| | | 50 Eivind Grøv | Tunneling Rogfast with TBM at 390 m below sea level? |
| 17:15 | End of presentation | | |
| 19:00 - | | Banquet: Venue in front of the hotel entrance for a 5 min. walk to the Håkonshallen banquet hall. | |

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For details regarding parallel sessions see pages xvi-xiii for **Bridges**, xix-xxi for **Tunnels**, xxii for **Tunnels/Bridges**, xxiii for **Ferries** and xxiv for **Regional development.** For **Room plan** see page xxv.

# Sessions on Tunnels – Room 3 and 4

| **Wednesday 19th of June** | | | |
|---|---|---|---|
| **Time** | **Topic** | **Presenter (paper no.)/Activity** | **Paper** |
| | | **Session chair: Bjørn Nilsen** | **Cochair: Gunnar Eiterjord** |
| 08:30 – | Knowledge transfer | 75 Ruth Gunlaug Haug | The Academy of Tunnelling - an expert program for improving future tunnelling |
| | Requirements | 76 Kjersti Kvalheim Dunham | Modern road tunnels - the future of long and deep tunnelling - is there any limit? |
| | Rescue tunnels | 103 Jan K.G. Rohde | What is the real cost of a rescue tunnel |
| | Subsea | 104 Gunnar Gjæringen | The Bjorøy tunnel – blasting on the seabed above the tunnel running through the “Bjorøyzone” |
| 10:10 | Coffee break | **Session chair: Walter Grantz Cochair: Håvard Østlid** | |
| 10:35 | Plenary session | 100 The past, present and future of the Seikan Tunnel, Motohiro Sato, Japan | |
| 11:10 | | 63 Environmental footprint in early planning of coastal road sections, Johan Berg Pettersen,, MiSA, Norway | |
| 11:45 | | 115 FOR - Forever open roads, Steve Phillips, Secretary General FEHRL | |
| 12:20 | Symposium closure | Project Manager Olav Ellevset, Norwegian Public Roads Administration | |
| 12:35 | Lunch | | |
| 14:00 – | | Departure for technical visits | |

For details regarding parallel sessions see pages xvi-xiii for **Bridges**, xix-xxi for **Tunnels**, xxii for **Tunnels/Bridges**, xxiii for **Ferries** and xxiv for **Regional development.** For **Room plan** see page xxv.

# Combined Tunnel/Bridge sessions – Room 4

| **Tuesday 18th of June** | | | |
|---|---|---|---|
| **Time** | **Topic** | **(paper no.) Presenter** | **Paper** |
| | | **Session chair: Ismail bin Mohamed Taib Cochair: Ruth Gunlaug Haug** | |
| 08:30 - | Feasibility | 36 Ove Solheim | Crossing the Oslofjord - an early strategic analysis |
| | Design conditions | 39 Ove T Gudmestad | Design basis for Strait Crossings |
| | Moorings | 44 Morten Bjerkås | Mooring concept for deep water crossings |
| | Site investigations | 86 Harald Sysytad | Sub-bottom investigations for a floating structure across Bjørnafjorden - anchoring conditions |
| 10:10 – | Coffee break | | |
| 10:30 – | Competitive dialogue | 81 Kasper Nordmelan | Experience from use of Competitive Dialogue in complex projects |
| | Combined solution | 98 Kristen Olav Dahl | Combined floating bridges and submerged floating tunnel |
| | Feasibility study | 82 Lidvard Skorpa | Crossing the deep and wide fjords on the western coast of Norway with fixed connections |
| | | 111 Yiqiang Xiang | Challenges and solutions of future channels construction over straits or bays in China coastal area. |
| | Energy | 57 Mohammed Hoseini | Integrating renewable energy into bridge construction |
| 12:35 – | Lunch | **Session chair: Jonathan Baber Cochair: Lidvard Skorpa** | |
| 13:30 – | Lighting | 80 Per Ole Wanvik | Tunnel Lighting |
| | Material properties | 91 Yiqiang Xiang | Preliminary Design and Comparison of SFT tube with different high performance fiber concrete materials |
| | Corrosion | 5 Shigeki Kushuhara | Development of Anticorrosion System for Underwater Steel Structures of Long-Span Bridges |
| | Floating bridge | 58 Geir L. Kjersem | Floating bridge with high bridge – a conceptual presentation |
| 15:10 – | Coffee break | | |
| 15:35 – | Deep sea foundations | 70 Ketil Aas-Jakobsen | Deep Sea Foundations for Strait Crossings |
| | Construction | 106 Naeem Hussain | Review of recent long span cable supported bridges across estuaries and straits and proposals for application to future crossings |
| | Monitoring | 90 Yuan Zhong | Structural health monitoring of the Wusu cable-stayed bridge |
| | | | |
| 17:15 | End of presentation | | |
| 19:00 - | | Banquet: Venue in front of the hotel entrance for a 5 min. walk to the Håkonshallen banquet hall. | |

For details regarding parallel sessions see pages xvi-xiii for **Bridges**, xix-xxi for **Tunnels**, xxii for **Tunnels/Bridges**, xxiii for **Ferries** and xxiv for **Regional development.** For **Room plan** see page xxv.

# Session on Ferries - 4

| | | | |
|---|---|---|---|
| **Sunday 16th of June** | | | |
| **Time** | **Topic** | **Activity** | |
| 18:00 - | Registration | | |
| 19:00 - | Reception | Dinner a la carte available * | |
| **Monday 17th of June** | | | |
| **Time** | **Topic** | **(paper no.) Presenter** | **Paper** |
| 08:00 - | Registration continued | **Session chair: Christian Ingerslev Cochair: Ian Markey** | |
| 08:30 - | Plenary session | Welcome speech: Director General Terje Moe Gustavsen, Norwegian Public Roads Administration | |
| 08:45 - | | Impact of fixed links on regional development, Professor Victor Norman, Norwegian School of Economics | |
| 09:30 - | | 121 Coastal Highway Route E 39, Project Manager Olav Ellevset, Norwegian Public Roads Administration | |
| 10:10 - | | 22 The Fehmarnbelt Fixed Link, Director Steen Lykke, Fehmarnbelt, Denmark | |
| 10:45 – | Coffee break | | |
| 11:10 - | Plenary session | 11 Akashi Kaikyo Bridge Project, Shatoshi Kashima, Japan Bridge Engineering Center, Japan | |
| 11:40 - | | 73 SR 520 Floating Bridge and Landings Project, Lake Washington, Seattle, Arnfinn Rusten, Berger ABAM Inc. | |
| 12:05 - | | 88 The key to efficient ferry operations, Anker Grøvdal, Fjord 1, Norway | |
| 12:30 – | Lunch | **Session chair: Anker Grøvdal Cochair: Edvard Sandvik** | |
| 13:30 - | Battery operated | 116 Narve Mjøs | Battery powered ships - economic and greener |
| | | 89 Hallgeir Kleppe | How can ferries be a good and lasting alternative in crossing fjords? |
| | Efficiency | 79 Mike Howie | Automatic mooring - a step toward automated terminals |
| | Terminals | 42 Tor Ole Olsen | Floating concrete structures |
| 15:10 – | Coffee break | | |
| 15:35 – | Terminals | | |
| | | | |
| | | | |
| | | | |
| 17:15 - | End of presentations | | |
| 18:00 - | | Social event: Venue in front of the hotel entrance ** | |

* Not included in symposium fee. ** Guided walking tour of Bergen including the Hanseatic Wharf, Ride with the Fenicular to Fløien and dinner at Fløien.

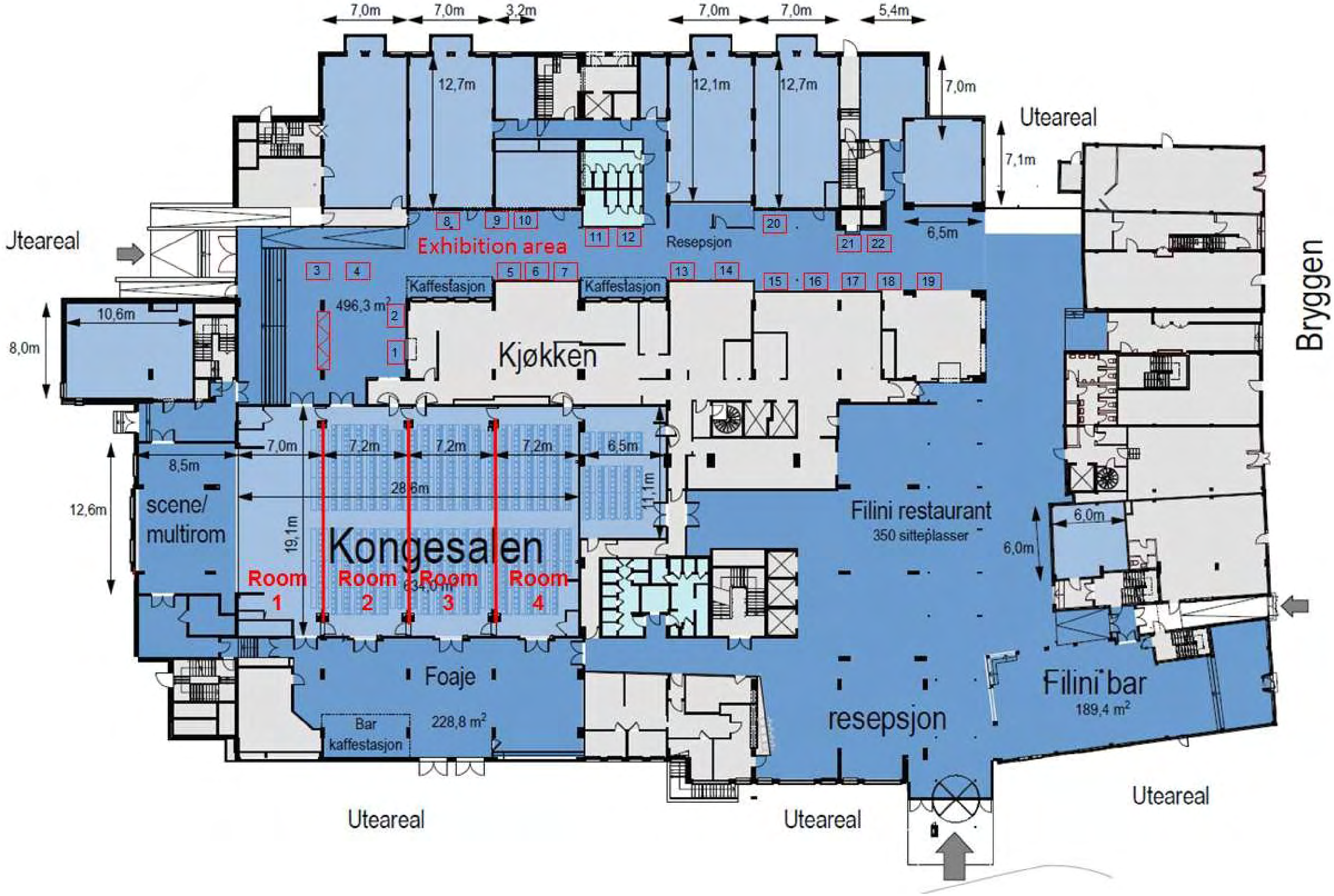
For details regarding parallel sessions see pages xvi-xiii for **Bridges**, xix-xxi for **Tunnels**, xxii for **Tunnels/Bridges**, xxiii for **Ferries** and xxiv for **Regional development.** For **Room plan** see page xxv.

# Session on Regional development - Room 1

| **Sunday 16th of June** | | | |
|---|---|---|---|
| **Time** | **Topic** | **Activity** | |
| 18:00 - | Registration | | |
| 19:00 - | Reception | Dinner a la carte available * | |
| **Monday 17th of June** | | | |
| **Time** | **Topic** | **(paper no.) Presenter** | **Paper** |
| 08:00 - | Registration continued | **Session chair: Christian Ingerslev Cochair: Ian Markey** | |
| 08:30 - | Plenary session | Welcome speech: Director General Terje Moe Gustavsen, Norwegian Public Roads Administration | |
| 08:45 - | | Impact of fixed links on regional development, Professor Victor Norman, Norwegian School of Economics | |
| 09:30 - | | 121 Coastal Highway Route E 39, Project Manager Olav Ellevset, Norwegian Public Roads Administration | |
| 10:10 - | | 22 The Fehmarnbelt Fixed Link, Director Steen Lykke, Fehmarnbelt, Denmark | |
| 10:45 – | Coffee break | | |
| 11:10 - | Plenary session | 11 Akashi Kaikyo Bridge Project, Shatoshi Kashima, Japan Bridge Engineering Center, Japan | |
| 11:40 - | | 73 SR 520 Floating Bridge and Landings Project, Lake Washington, Seattle, Arnfinn Rusten, Berger ABAM Inc. | |
| 12:05 - | | 88 The key to efficient ferry operations, Anker Grøvdal, Fjord 1, Norway | |
| 12:30 – | Lunch | **Session chair: Olav Ellevset Cochair: Signe Eikenes** | |
| 13:30 - | Impact | 122 Peter O'Neill | Regional impacts of transport investments |
| | | 24 Trude Tørset | Can we estimate the impacts of fixed links using current transport models? |
| | Regional development | 105 Olbert Aasan | Bridge creates new opportunities for Fosen and Trondheim |
| | | 120 Anders Jordbakke | More efficient transport across the Oslofjord - a feasibility study at an early stage |
| 15:10 – | Coffee break | | |
| 15:35 – | Procurement | 4 Andrew J. Yeoward | Procurement of Major Crossings |
| | | | |
| | | | |
| | | | |
| 17:15 - | End of presentations | | |
| 18:00 - | | Social event: Venue in front of the hotel entrance ** | |

* Not included in symposium fee. ** Guided walking tour of Bergen including the Hanseatic Wharf, Ride with the Fenicular to Fløien and dinner at Fløien.

For details regarding parallel sessions see pages xvi-xiii for **Bridges**, xix-xxi for **Tunnels**, xxii for **Tunnels/Bridges**, xxiii for **Ferries** and xxiv for **Regional development.** For **Room plan** see page xxv.


Exhibition area
Kaffestasjon
Kaffestasjon
496,3 m²
Resepsjon
Kjøkken
Uteareal
Jteareal
Bryggen
scene/
multirom
Kongesalen
Room 1
Room 2
Room 3
Room 4
634,0 m²
Foaje
Bar
kaffestasjon
228,8 m²
Filini restaurant
350 sitteplasser
Filini bar
189,4 m²
resepsjon
Uteareal
Uteareal
Uteareal
7,0m
7,0m
3,2m
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7,0m
5,4m
12,7m
12,1m
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7,0m
7,1m
6,5m
10,6m
8,0m
8,5m
12,6m
7,0m
7,2m
7,2m
7,2m
6,5m
28,6m
19,1m
11,1m
6,0m
6,0m

# Exhibitions

| Stand No. | Firm | Contact | E-mail |
|---|---|---|---|
| | | | |
| 1 | Multiconsult AS | Birger Opgård | birger.opgaard@multiconsult.no |
| 2 | WSP Genivar (Multiconsult AS) | Birger Opgård | birger.opgaard@multiconsult.no |
| 3 | Bentley Systems Austria | Myriam Derry | Myriam.Derry@bentley.com |
| 4 | TDA | Myriam Derry | Myriam.Derry@bentley.com |
| 5 | Norwegian Geotechnical Institute | Roger Olsson | roger.olsson@ngi.no |
| 6 | COWI AS | Per Arnesen | par@cowi.no |
| 7 | Norconsult AS | Alexander Kyte | alexander.kyte@norconsult.com |
| 8 | The Spencer Group | Jim Mawson | jim.mawson@cspencerltd.co.uk |
| 9 | Alpin Technik und Ingenieurserv. | Monika Hermann | hermann@alpintechnik.de |
| 10 | Alpin Technik und Ingenieurserv. | Monika Hermann | hermann@alpintechnik.de |
| 11 | Giertsen Tunnel AS | Jørn Reite | jorn.reite@giertsen.no |
| 12 | Cavotec Norge AS | Sofus Gedde-Dahl | sofus.gedde-dahl@cavotec.com |
| 13 | Goodwin Steel Castings Ltd | Peter Stokoe | pstokoe@goodwingroup.com |
| 14 | Jernbaneverket | Ingvild Eikeland | ingvild.eikeland@jbv.no |
| 15 | Bridon International | Jennie Ferguson | fergusonj@bridon.com |
| 16 | *fib* Intern. federation for structural concrete | Petra Schumacher | petra.schumacher@epfl.ch |
| 17 | Bridge design&engineering | Helena Russel | H.Russell@hgluk.com |
| 18 | FEHRL | Steve Phillips | Steve.Phillips@fehrl.org |
| 19 | NFF/ITA | Thor Skjeggedal | thor@skjeggedal.com |
| 20 | Statens vegvesen | Liv Bulling | liv.bulling@vegvesen.no |
| 21 | HRC Europe | Thomas Kaiser | thomas.kaiser@hrc-europe.com |
| 22 | Materialprüfanstalt für das Bauwesen | Daniela Klar | D.Klar@ibmb.tu-bs.de |

For location of exhibition stands please see floor plan on page xxvi.

# Cooperating Organisations

Norsk Bergmekanikkgruppe (NBG)
Norwegian Group for Rock Mechanics

fib CEB-FIP
International Federation for Structural Concrete
fédération internationale du béton
www.fib-international.org

ITA AITES
Association Internationale des Tunnels et de l'Espace Souterrain
International Tunnelling and Underground Space Association

IABSE Norwegian Group
International Association for Bridge and Structural Engineering

FEHRL

NFF Norwegian Tunnelling Society

NFS
stålforeningen.no

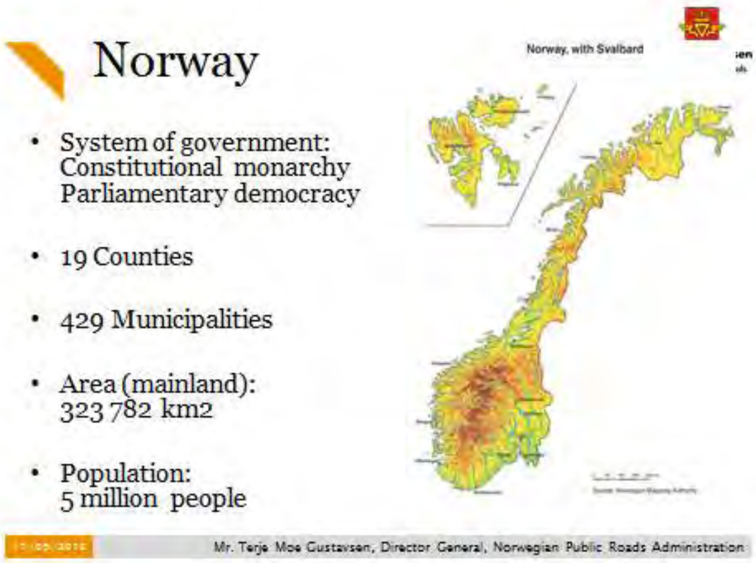
nvf The Nordic Road Association

NGF 1950
Norsk Geoteknisk Forening
Norwegian Geotechnical Society

NB The Norwegian Concrete Association

**WELCOME SPEECH by Director General Mr. Terje Moe Gustavsen**

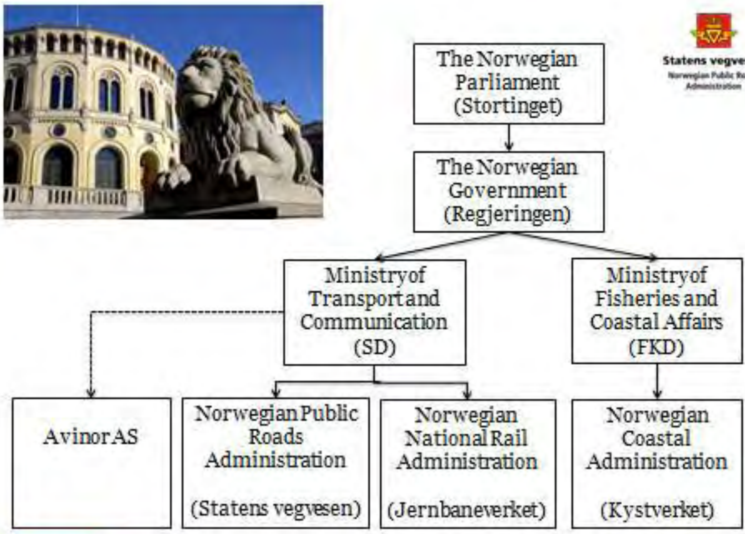




## Key figures

| | | |
|---|---|---|
| Public roads, total | 93 000 | km |
| National roads | 10 500 | km |
| County roads | 44 000 | km |
| Municipal roads | 38 500 | km |
| Road tunnels | 1 000 | tunnels |
| Road bridges | 18 200 | bridges |
| Rail network | 4 000 | km |
| Airports | 49 | airports |
| National ports | 30 | ports |

17/06/2013 Mr. Terje Moe Gustavsen, Director General, Norwegian Public Roads Administration




Statens vegvesen
Norwegian Public Roads Administration

Mr. Terje Moe Gustavsen, Director General, Norwegian Public Roads Administration

## National Transport Plan 2014 - 2023

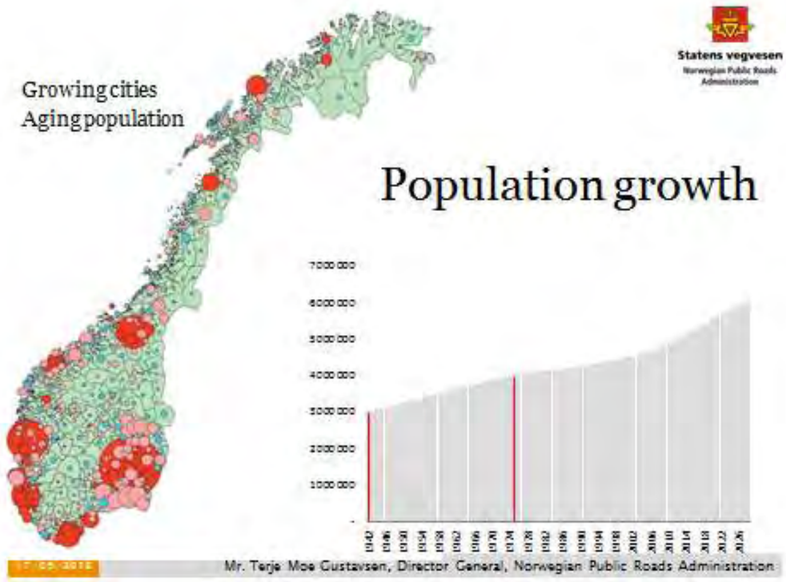
Statens vegvesen
Norwegian Public Roads Administration

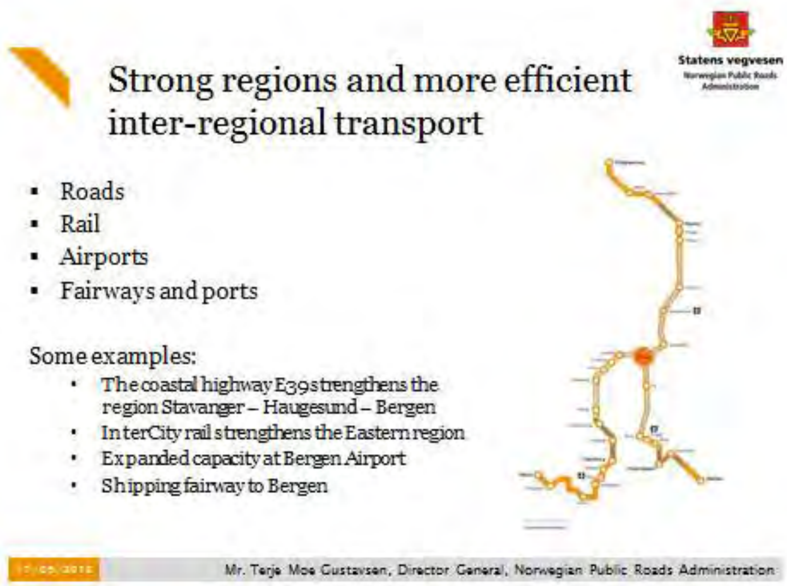
17/06/2013 Mr. Terje Moe Gustavsen, Director General, Norwegian Public Roads Administration

Major challenges:

- Globalisation
- Population growth
- Economic growth
- Traffic safety
- Emissions and climate change


Economic growth
Statens vegvesen
GDP (1970=100)
1600
1400
1200
1000
800
600
400
200
0
1970 1980 1990 2000 2010 2020 2030 2040 2050 2060
Real wages (1970=100)
250
200
150
100
50
0
1970 1980 1990 2000 2010
Mr. Terje Moe Gustavsen, Director General, Norwegian Public Roads Administration


Statens vegvesen
Growing cities
Aging population
Population growth
7000 000
6000 000
5000 000
4000 000
3000 000
2000 000
1000 000
Mr. Terje Moe Gustavsen, Director General, Norwegian Public Roads Administration


Strong regions and more efficient
inter-regional transport
Statens vegvesen
Roads
Rail
Airports
Fairways and ports
Some examples:
The coastal highway E39 strengthens the
region Stavanger – Haugesund – Bergen
InterCity rail strengthens the Eastern region
Expanded capacity at Bergen Airport
Shipping fairway to Bergen
Mr. Terje Moe Gustavsen, Director General, Norwegian Public Roads Administration

National Transport Plan 2014 – 2023

## An Expanding Market

Statens vegvesen

- Road sector + 49 %
- Rail sector + 49 %
- Costal sector + 55 %

compared to NTP 2010-2019

Mr. Terje Moe Gustavsen, Director General, Norwegian Public Roads Administration

«Project with special priority»

## Coastal Highway E39

Statens vegvesen

- Ferry free within 20 years replacing 7 ferryconnections
- Construct roads between the ferries
- 7-8 hours reduced travel time
- Estimated 20.000 MEUR
- Mainly between the fjords first plan period (4 years)
- Rogfast


Mr. Terje Moe Gustavsen, Director General, Norwegian Public Roads Administration

## E-39 Boknafjorden (ROGFAST)

Statens vegvesen

- Worlds longest subsea tunnel 25 km
- Depth: -390 m
- 2 tubes
- Possible start 2015

Mr. Terje Moe Gustavsen, Director General, Norwegian Public Roads Administration


Statens vegvesen
Mr. Terje Moe Gustavsen, Director General, Norwegian Public Roads Administration


Challenging Technical Borders
Statens vegvesen
Norwegian Public Roads Administration
Mr. Terje Moe Gustavsen, Director General, Norwegian Public Roads Administration


National Tourist Routes
Evening at the Atlantic Ocean Road
Thank You for Listening !

# THE IMMERSED BRIDGE TUNNEL (IBT)

Walter C. Grantz, P.E.
Immersed Tunnel Consultant, USA
grantzbw@cox.net

**ABSTRACT**

*For years the writer has argued that the major drawback of the conventional submerged floating tunnel concept has been that if it were to be flooded accidently or through sabotage, the costly facility would be completely and catastrophically destroyed. This possibility might be mitigated for minor flooding by moving the drainage low point to the shore beyond the floating tunnel portion, but a major flood that would cause the tethers to go slack (or sink the pontoons) would cause a complete collapse of the entire structure.*

*With a tethered SFT it would be possible to design the tunnel with enough compartmented flotation (that could not be flooded) to make it float stably even if the roadway/rail ducts were completely filled with water. The same could be done with an SFT supported from pontoons. In both cases however, the amount of tethers and anchorages (or hangars and pontoon size) would be almost doubled. The larger and more complex tunnel cross section would also increase the costs.*
*It occurred to the writer that perhaps he had been thinking in error in proposing drastically increasing the flotation of the tunnel. Instead, the way to provide stability for a flooded tunnel might be to build an immersed tunnel supported on piers instead of the tethered floating concept. Thus, if the piers and footings were designed to take the load and the flooded tunnel was designed to span between the piers, the result would be a stable structure. Depending on local environmental conditions, this could be a feasible alternative for the unsinkable SFT for water depths of up to say 300m.*

*This paper puts forth some rough ideas how this might be done.*

## INTRODUCTION

The main goal of this paper is to show how a tunnel might be built across a watercourse with a depth several times that of a normal, practical immersed tunnel. The deepest immersed tunnel in the world, the Bosphorus Rail Tunnel reaches a maximum depth of almost 60 m. It required concrete reinforcing steel close to practical limits of size and spacing. Modern shipping rarely drafts more than 15m fully loaded. So, setting a navigation clearance depth of 20-50m would provide ample safety from collision while limiting the depth and required structure and keeping the total length of tunnel as short as possible.

Basing this hypothetical design on very conservative assumptions, and the proposed construction methods on years of experience in the field of major immersed tunnel construction, the writer has attempted to work through all the steps needed to build a tunnel across a waterway about 300m. deep. The design is in no way refined, or even aesthetically pleasing perhaps, but it is felt that it ‘does the job’ and illustrates the key features that might be used to build it successfully. No effort has been made to optimize costs or compare them to an equivalent unsinkable SFT. The intent here is to propose an alternative design along with practical construction methods whereby a very deep waterway can be crossed with a tunnel that is stable even if a portion were breached – say due to a structural failure resulting from a vehicle fire for example – and flooded completely.

## HYPOTHETICAL TUNNEL

In this hypothetical tunnel study a basic four-lane rectangular concrete box section has been chosen with the exterior dimensions of 25m. width, 10 m. height and a modular length of 100m. Water is assumed to be one metric ton per cubic meter (fresh) so a 10m.x25m. cross section is assumed to displace 250 tons/m. For simplicity sake we ignore the effects of a reinforced area at the immersion joints and at the piers. For submerged stability a negative buoyancy of ten percent or 25 t/m. of tunnel has been assumed. Taking the unit weight of reinforced concrete as 2.4 metric tons per m3, we can calculate that the total volume of concrete including ballast will be 115 m3. Subtracting this from 250 m2/m we arrive at 135m2/m, the volume of airspace in the empty tunnel. If we flood the tunnel and fill the airspace with water, the tunnel's weight increases by135 t/m. So now the total negative buoyancy of the flooded tunnel is (25+135) = 160 t/m. or 16,000 tons per modular element 100m. long. The normal tunnel weight will be only 25x100 or 2,500 t/m. Initially ignoring the weight of the piers, if we prepare our foundation to take 15 t/m2, its area will only need to be 1,067 m2 to carry the flooded weight of the tunnel. Let us assume a footing area range of 30m.x40 to 30x60m. depending on water depth. The long dimension of the footing would be transverse to the tunnel centerline to provide the best stability in that direction during and after construction.

Roughing out a pier unit buoyant weight of 40t/m., for a 7 m. diameter 250m. tall pier would add 10,000 tons. So if we assume a 30x60m. footing (for the deepest footing) its range of loading would be (2,500+10,000)/1,800 = 6.9 t/m2 (Normal operating condition) and (16,000+10,000)/1,800 = 14.4 t/m2 (Flooded condition).

## PIER CONSTRUCTION

The most difficult aspect of an SFT, or in this case, a IBT, is extreme water depth. The writer is fully aware that what he is proposing is a method that is limited by depth. At some point depth make piers impractical because of column length and stability during and after construction. It is felt however that this method being proposed can satisfy a range of depths of a water crossing that would be too deep for a conventional immersed tunnel but fine for a tunnel where its profile need only provide adequate navigational clearance. Pier heights in the order of 250-300m should be feasible using this method.

Working at depths of more than 50m. with divers becomes very inefficient and difficult. The writer proposes that most of the work can be done remotely from the surface. In preparing the ground to receive each pier footing, first unsuitable soil must be dredged and removed using a clamshell excavator and barges. Such a process would be very slow because of the time it would taken for each bucket load to go down 300 meters, dig, and return to dump into the barge. If environmental regulations would allow, it would be more efficient to simply move the spoil to piles outside of the excavation. This could be done with a regular bucket dredge or it could be done with a special catamaran barge equipped with a 100m. long rail and traveler system controlled from a bridge. The bucket could be equipped with digital sonar so that its precise location and elevation could be monitored as the excavation proceeded. The great depth of excavation would likely cause lateral drift of the barge, cables, or bucket but if the catamaran were equipped with steerable thrusters on each corner, it could automatically station-keep based on the desired 100m track of the bucket. (Figure 1.)

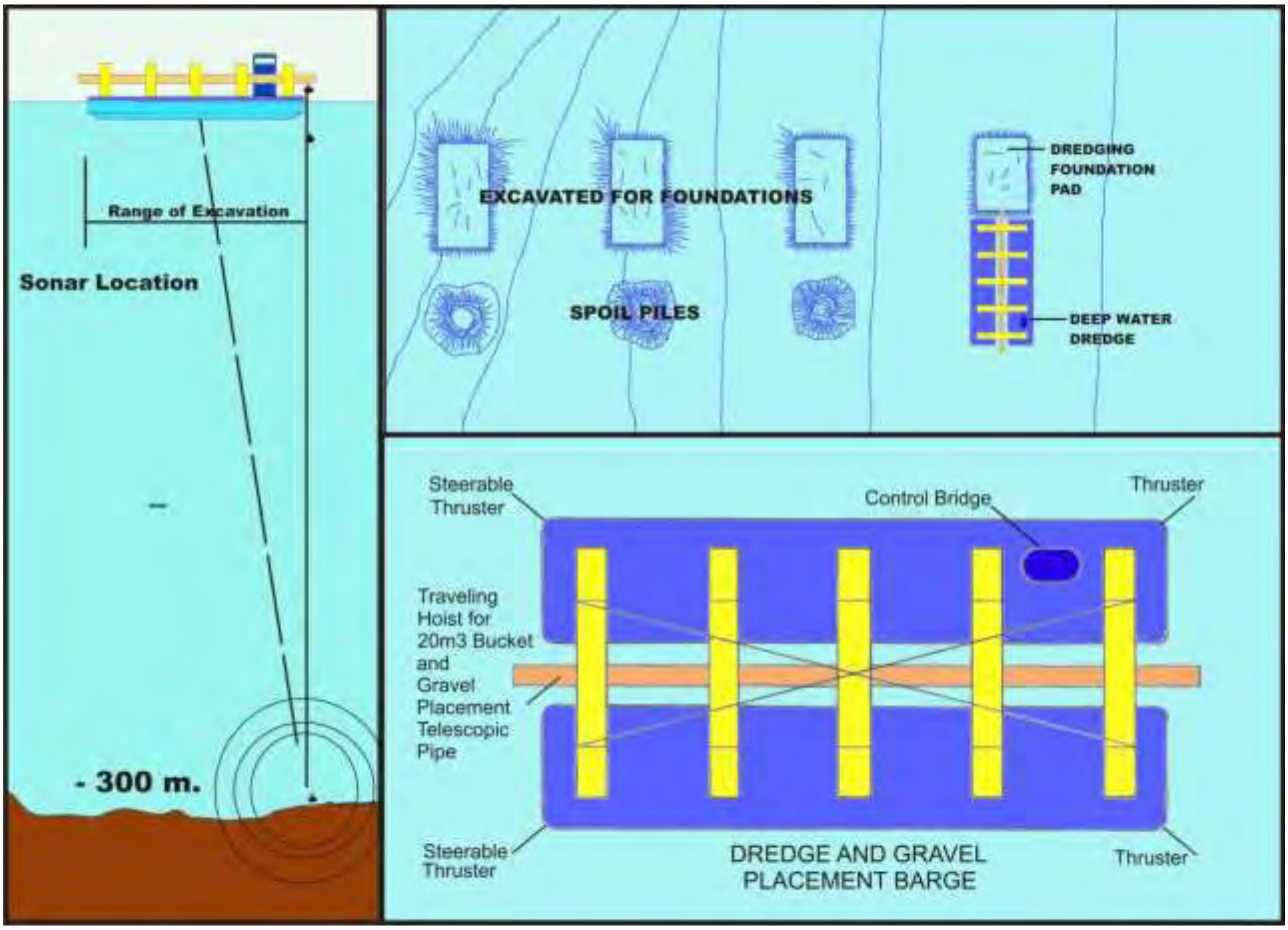



*Figure 1. Possible method for efficient excavation for gravel foundation pads. Clamshell bucket excavates, then moves laterally and deposits material in spoil pile. Bucket never needs to come to the surface during excavation work. Exact location is measured in real time using sonar and programmed to excavate required rectangle in uniform pattern. Barge location is adjusted by station keeping thrusters to best location and orientation taking into account any drift of the bucket due to currents in the waterway.*

Then a base for the pier footings can be installed, perhaps using the same catamaran with telescopic pipes to deposit measured quantities of the gravel foundation course. The surface may then be graded using something similar to what was used during the Bosphorus Rail Tunnel Project. It was a remotely controlled underwater grader. The latter provided a smoothly graded surface. Its operation was monitored with video cameras and grader blade position in three axes was transmitted in real time to the surface using acoustical measurement. (Figure 2.)

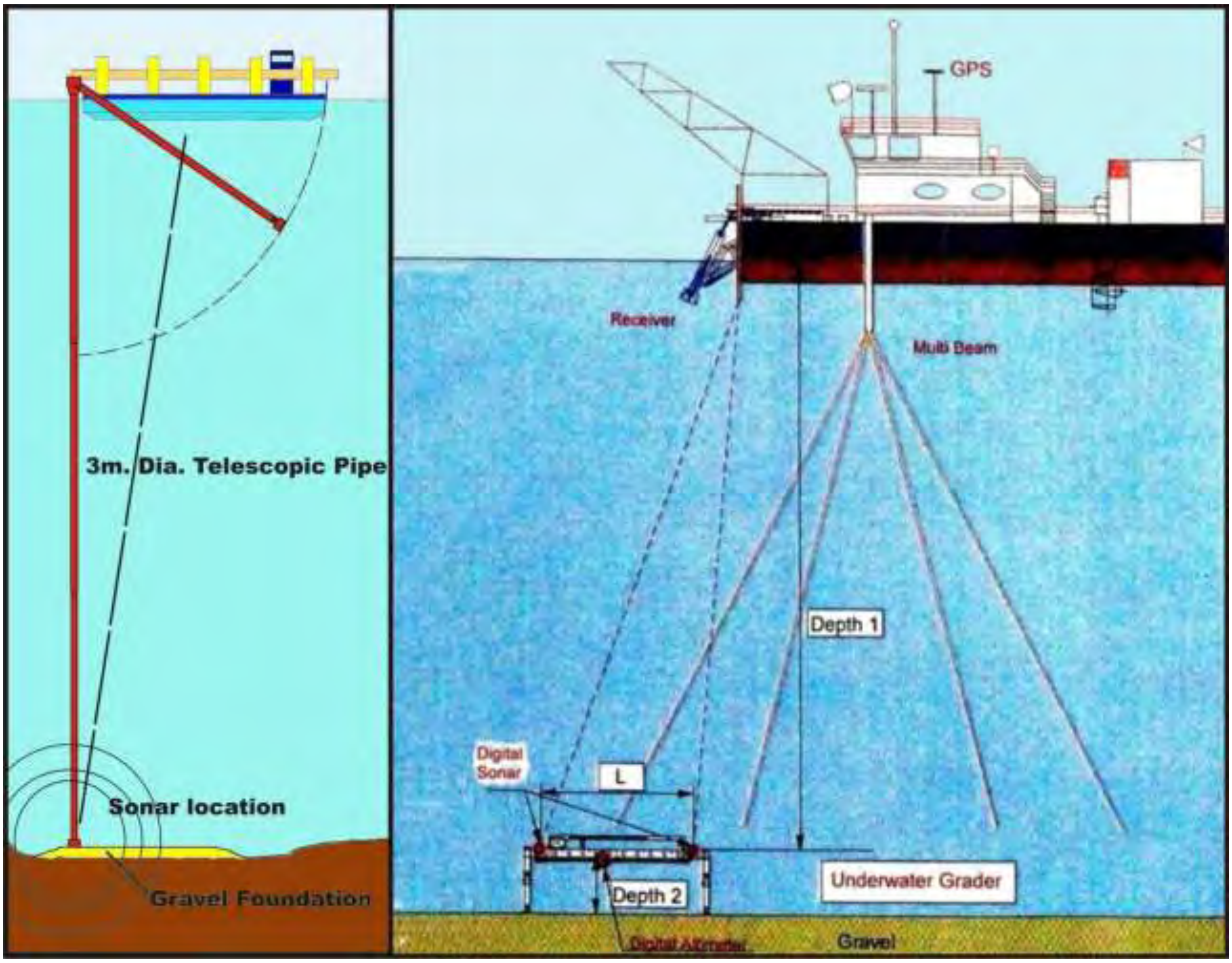



*Figure 2. Foundation material placed by telescopic pipe and graded with remote underwater grader*

The Plan and elevation in Figure 3 show a possible equipment setup for progressively constructing the piers in place. The arrangement is designed for the progressive assembly and lowering of the footing and pier assemblies onto the prepared gravel foundation. Once this was done and the positioning of each pier was verified, they could be backfilled and protected with stone riprap (Figure 6).

The pier construction would first involve a barge carrying the concrete footing (previously cast-in-place on the barge) into position where it can be connected to the four lowering pulley blocks (or "falls") and lifted off the barge with the falls. The barge is then towed away. A match-cast modular section of the pier shaft can then be added and post-tensioned to the footing. Following this operation the combined footing and first shaft section is lowered to a working elevation so that a second shaft module can be post-tensioned to the first. This operation is repeated to build the shaft to the desired height. As this is done, the as-built height of the footing and pier shaft will be carefully measured. The weight during construction can be controlled by building the shaft modules with watertight air filled compartments to make them buoyant. This will also aid in the stability of the shaft as it hangs from the falls. Nearing the completion of a pier shaft/footing it will be lowered onto the gravel foundation for a final elevation check before custom casting and attaching the last shaft section that will support the tunnel. The top elevation and station of each pier will be carefully controlled to a small tolerance in the order of a few centimeters. In this way the tunnel alignment will be maintained accurately. Small variations in tunnel alignment in this order are common in any immersed tunnel.

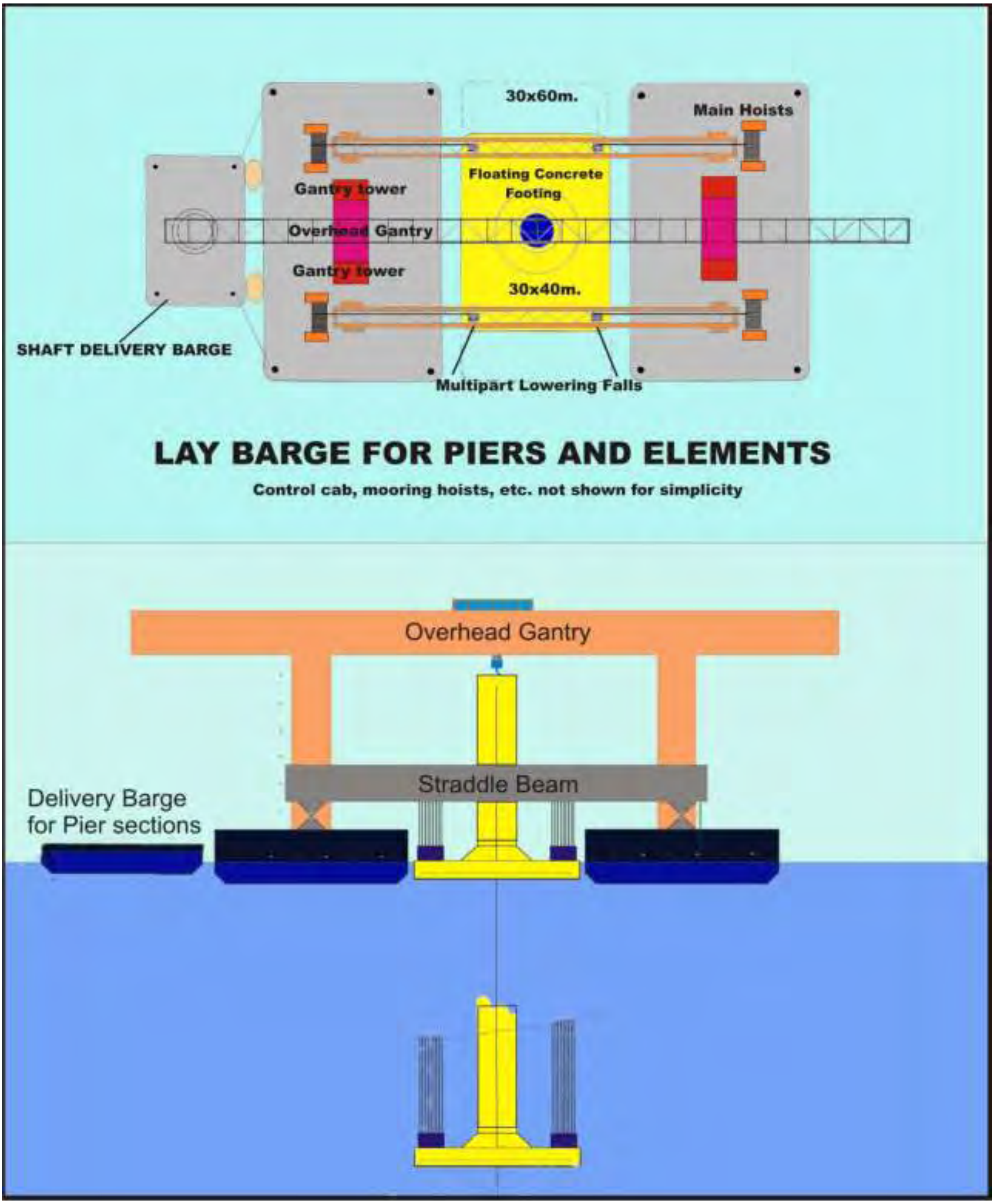



*Figure 3. Plan and Elevation of equipment arrangement for building piers in situ.*

**TUNNEL DESIGN AND INSTALLATION**

The basic premise of this hypothetical tunnel we are proposing is that apart from the piers, this tunnel, to all intents and purposes, would be fabricated and installed by standard immersed tunnel practices.

The elements would be totally or partially fabricated in a dry dock. These units would include temporary watertight bulkheads at each end, a reinforced drop slab and tapered receiver for the pier top (Figure 4), and an immersion joint designed to take full shear and bending forces (Figure 6). Considering the potential for tensile cracking due to bending moments inherent in this bridge like structure, special provisions may be required beyond waterproof concrete to assure watertightness. There are numerous external waterproofing media suitable for this purpose.

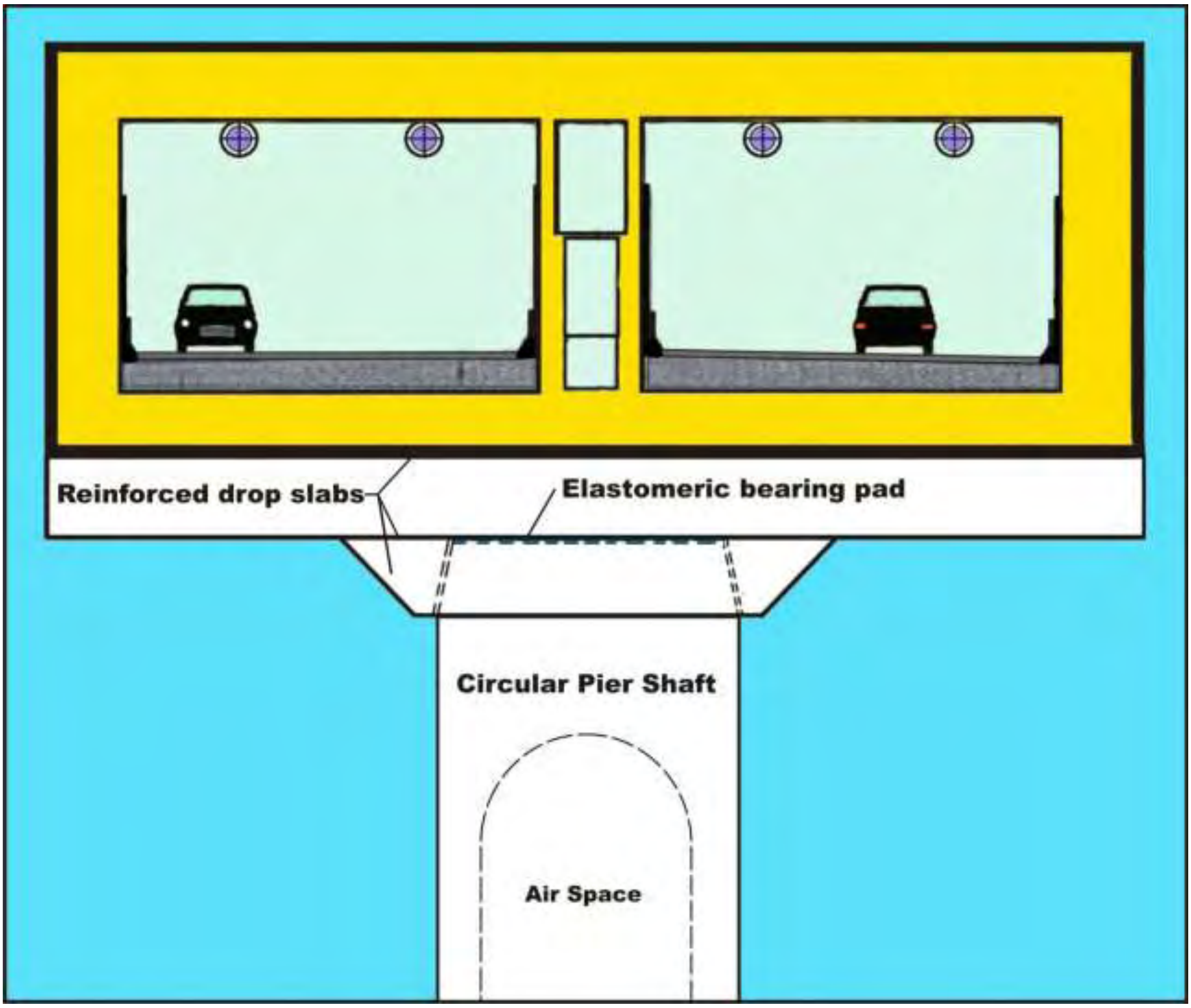



*Figure 4. Detail of tunnel support on pier shaft*

The tunnel elements can be lowered with a purpose-built placing barge or by reconfiguring the catamaran used for constructing the piers (Figure 5.)

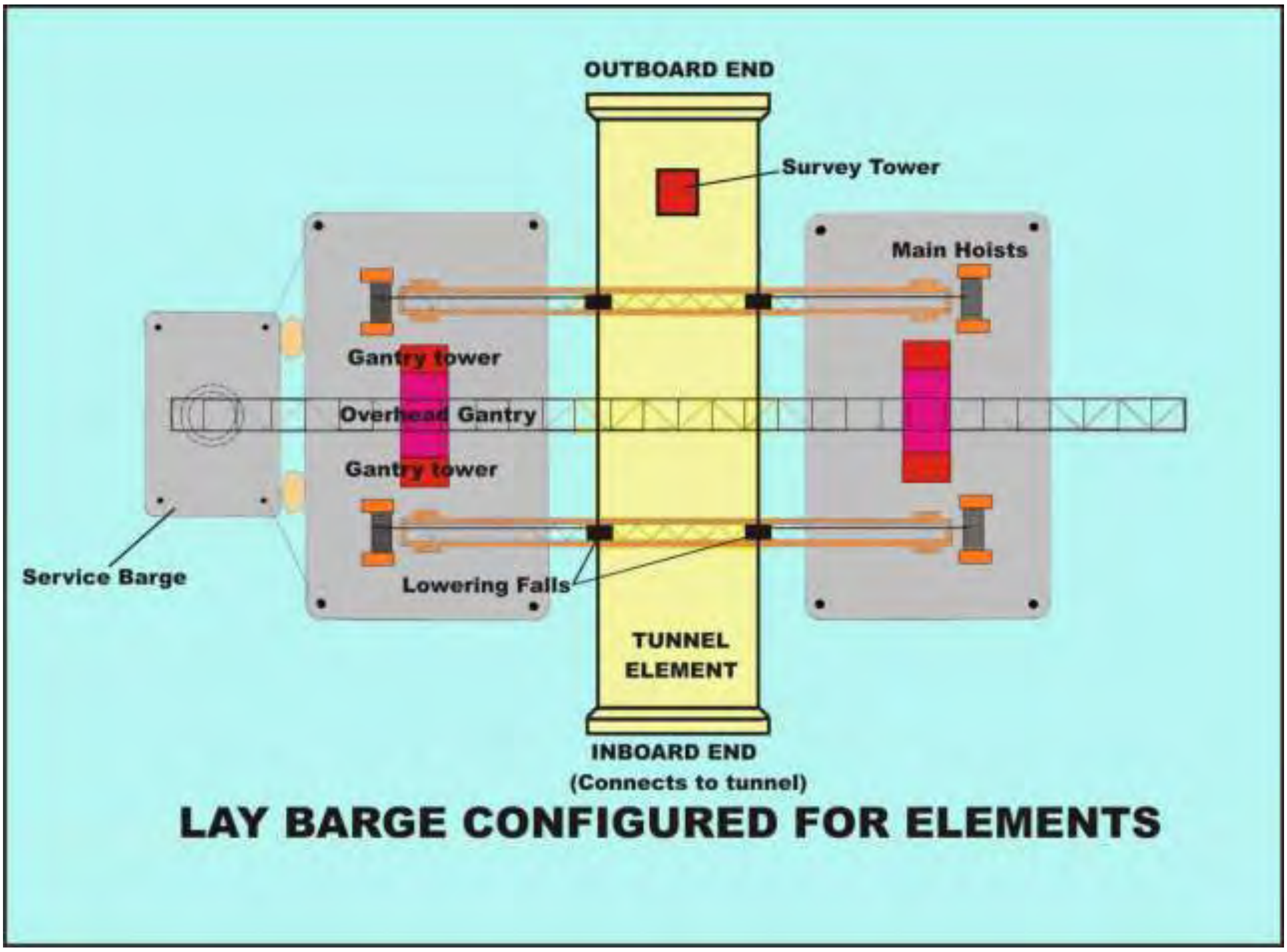



*Figure 5. Catamaran barges reconfigured to place elements.*

In this case, instead of lowering onto a screeded bedding or a set of outboard jacks, the element will be lowered onto a tapered pier top that will fit into a tapered socket built into a reinforced area of the element. The immersion joints will operate in the same way as always. When the element is lowered it will engage guide beams at the joint. Jacks will extend from the tunnel in place to engage the tunnel element being placed. Once engaged these will retract and compress the soft nose of the GINA to obtain an initial seal while at the same time, the element will be sitting lightly in on the pier top. With the initial seal verified, the water pressure in the joint space will be pumped down and the pressure in the joint space reduced to atmospheric. This will mobilize thousands of tons of force acting on the outboard end of the new element to fully compress the GINA making the joint structurally stable and completely watertight. This will allow a survey crew to enter the new element to verify its alignment and elevation. Corrections of out-of-tolerance horizontal alignment can be made with differential jacks in the joint at the horizontal axis. The need for correction in vertical alignment should be unlikely, however it could be done by removing the new element and adding or removing shims on the pier top.

Always keeping the three forward bulkheads secure, workmen will remove the bulkheads of the previous joint to allow installation of the structural ties between the elements and complete the joint integrity of the tunnel as it progresses.

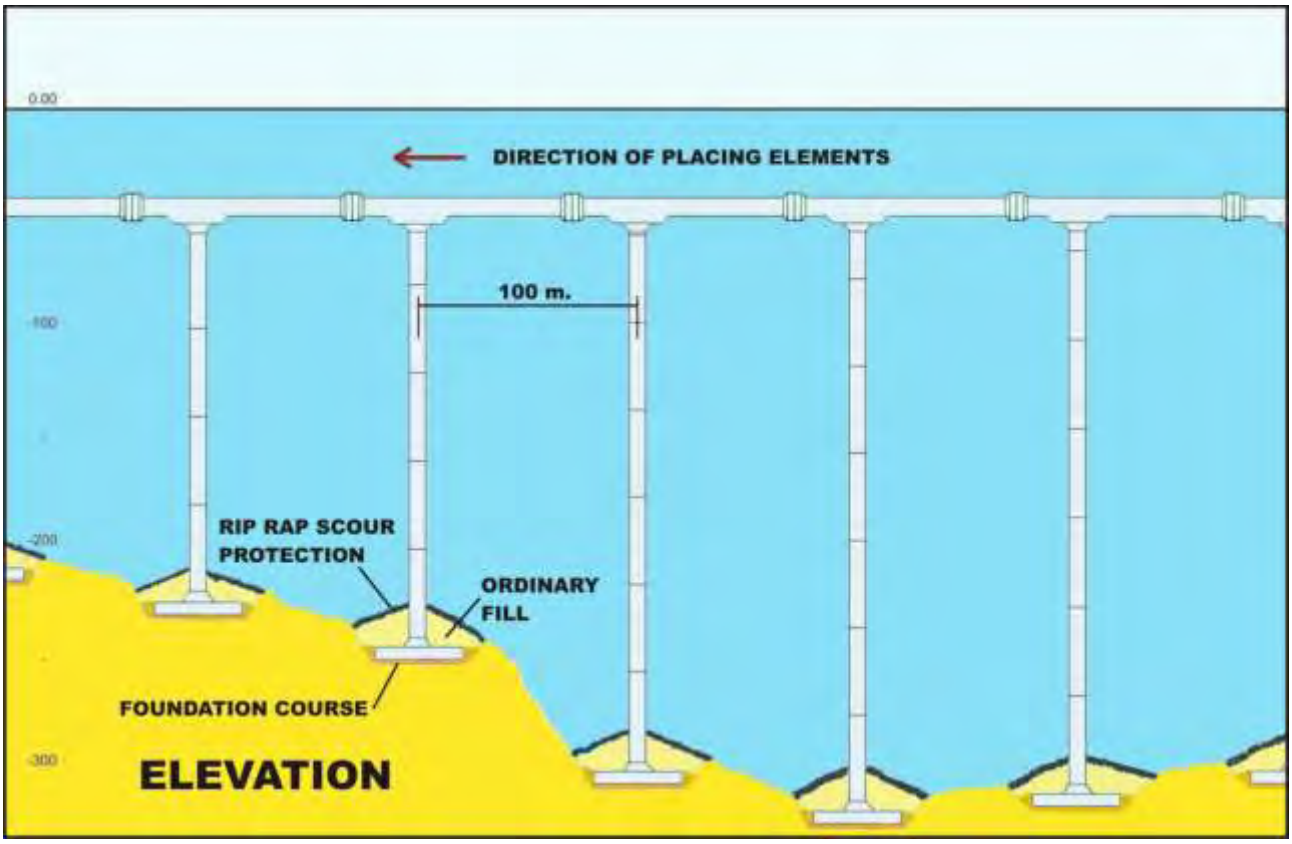



*Figure 6. Elevation of immersed Bridge Tunnel*

## ANOTHER ADVANTAGE

Figure 7 shows a typical transition of an immersed bridge tunnel to a portal structure. One interesting advantage of this immersed bridge concept is how it can be easily adapted to shallow water as an ordinary immersed tunnel. In fact, where a marine crossing consists of a combination of practical water depths for an immersed tunnel as well as an adjacent much deeper channel, such a crossing could combine the two methods. Only the deeper section would need to be supported on piers whereas all the rest of the tunnel could be constructed as a standard immersed tunnel. The tunnel elements would be virtually the same except for the reinforcement and socket for the piers

## QUESTIONS

This paper does not presume to provide answers to all the questions that arise when developing a crossing such as this. The water depth where the method described is feasible will no doubt depend greatly on the environmental conditions to be faced during construction and for the life of the structure. Water currents, wave action, exposure to storm conditions, geotechnical limitations, seismic conditions, and other hazards must be addressed as needed.

The writer is basically just putting another idea on the table on how to develop a water crossing for relatively deep water that is not vulnerable to catastrophic failure in the event of a major flood.

## CONCLUSIONS

The writer has for many years strongly felt that the reason that no Submerged Floating Tunnel (SFT) has so far ever been undertaken is the simple recognition of the fact by a potential owner that if such a tunnel were to be breached, the entire facility would be wiped out. The Immersed Bridge Tunnel proposed herein could also be destroyed under certain unlikely circumstances such as having a ship sink on top of it. With this design however, if a single module were breached by a fire or explosion and the tunnel were flooded, the remaining tunnel structure would stand, could be repaired (subject of another paper!), and put back into service.

It also occurs to the writer that the term "Immersed Bridge - Tunnel" may be less intimidating to the public than "Submerged Floating - Tunnel"

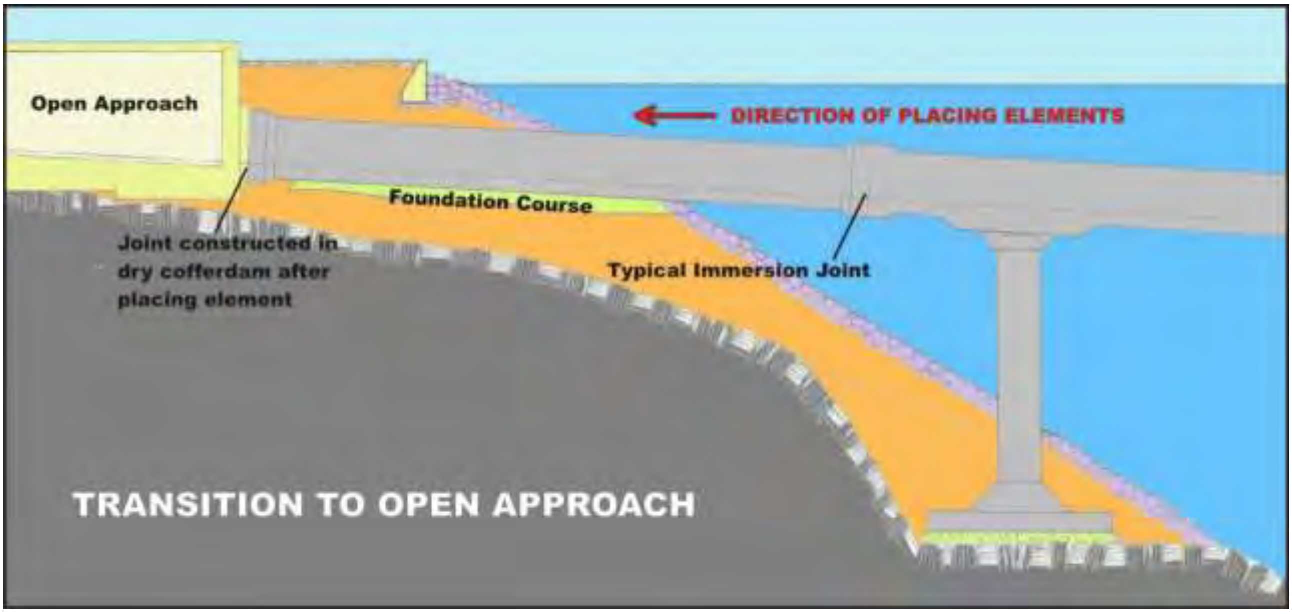



*Figure 7. Detail at end of tunnel illustrates how the immersed bridge can seamlessly transition to an immersed tunnel in shallow water.*

# ON THE APPLICABILITY OF TUNNEL BORING MACHINES FOR THE EXCAVATION OF LONG STRAIT CROSSING TUNNELS

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## ABSTRACT

*Due to the limited accessibility, the excavation of strait crossing tunnels may have to be carried out from the two portals only or from very few points of attack. This may result in long tunnel sections to be excavated from the same starting point. In such a case, tunnel boring machines (TBMs) may be a valuable alternative, as the achievable high advance rates allow for an important reduction of the construction time with costs comparable to a drill-and-blast excavation. An important issue during strait crossing tunnelling is the control of water inflow during excavation and in the long term. In conventional tunnelling, a typical solution consists in the pre-grouting of the rock mass and in the installation of a double-shell lining with a sealing membrane in-between. For tunnelling with shielded TBMs, state-of-the-art solutions exist which allow for a reduction of the required amount of pre-grouting or even for the active control of the water inflow. Furthermore, a watertight single-shell lining can be installed, thus reducing the construction costs. The paper starts reviewing the history of TBM tunnelling in Norway and the hazard scenarios related to the excavation of sub-sea rock tunnels. In the following, the suitable TBM types and their main characteristics as well as technical equipment are shortly illustrated. Finally, the paper compares the excavation methods "TBM" and "drill-and-blast" with respect to construction time, costs, logistics, environmental impact and health and safety, showing that TBM tunnelling is a valuable alternative to drill-and-blast excavation.*

## INTRODUCTION

A limited accessibility is a common feature of strait crossing tunnels. Due to this, the excavation of such a tunnel, which may be quite long, may have to be carried out from the two portals only or, more in general, from very few points of attack. This may result in long tunnel sections to be excavated from the same starting point. In such a case, the use of a tunnel boring machine (TBM) may be a valuable alternative, as the high advance rate allows for a reduction of the construction time with costs comparable to a drill-and-blast excavation. Taking account of the fact that more and more infrastructure projects have to be ready for operation in very short time, a high advance rate and, therefore, a shorter construction time are advantageous.

The present paper, which deals with sub-sea rock tunnels only, starts with some considerations on TBM tunnelling in Norway. Afterwards, the paper reviews the hazard scenarios related to hard rock TBM excavation of strait crossing tunnels. In the following, the suitable TBM types and their main characteristics as well as technical equipment are illustrated shortly, focusing on those technological developments which make TBMs particularly suitable for sub-sea rock tunnels. Finally, the paper compares the excavation methods "TBM" and "drill-and-blast", showing that TBM tunnelling is a valuable alternative to drill-and-blast excavation.

## TBM TUNNELLING IN NORWAY

Drill-and-blast is the usual method for tunnel excavation in Norway and has been applied successfully over years for the excavation of most of the about 5'000 km of Norwegian tunnels [1]. On the contrary, in Norway only 200 km of tunnels have been excavated with TBMs [1]. Most of them were excavated in the 1970s and in the 1980s. One of the first TBM drives was for a sewer tunnel (length: 4.3 km, boring diameter: 2.3 m) in Trondheim, which was bored 1972–1974. Further projects followed (mainly in the hydropower sector but also in the transportation sector) as, for example, a road tunnel (length: 6.9 km, boring diameter: 7.8 m) through Fløyfjellet (Bergen) built in the period 1984–1986 [2]. One of the last TBM applications in Norway was for a water tunnel (length: 10.0 km, boring diameter: 3.5 m) at the Meråker Hydropower Project in 1991–1992 [3]. Due to several reasons (first of all a lack of projects, but also a missing confidence in the TBM technique), a 20 year-long period of TBM tunnelling in Norway came to a temporary stop at the beginning of the 1990s.

In recent times, TBM tunnelling is regaining importance in Norway. In 2012, a contract for the TBM excavation of a water tunnel (length: 12 km, boring diameter: 6.7 m) for the Nedre Røssåga Hydro Project was signed. Also in 2012, the Norwegian National Railway Administration decided that TBMs will be the primary method of construction of the Follo Line Tunnel (length: 19.7 km, boring diameter: 9.8 m) between Oslo Central Station and Ski [4, 5]. Moreover, for the Ulriken Tunnel – a railway tunnel (length: 7.7 km, boring diameter: 8.6 m) near to Bergen – the parallel tendering of TBM and drill-and-blast excavation is planned.

## HAZARD SCENARIOS

TBMs can be applied for the excavation of traffic (road and railway) as well as of utility tunnels and this under mountain, urban or strait crossing conditions. Careful and project specific planning work is required in each case. Several hazard scenarios are the same as for drill-and-blast excavation, while other ones are more specific for TBM tunnelling.

The limited accessibility of strait crossing tunnels is particularly important with respect to the rock mass exploration before construction. Due to the high costs of marine operations, latter is often limited to the portal areas. In account of this, rock mass exploration in advance is mandatory for a continuous update of the evaluation of the rock mass and of the hazard scenarios.

A first group of hazard scenarios to be considered is related to the presence of discontinuities (joints, foliation) in the rock mass: rock fall (Figure 1a), instability of rock wedges (Figure 1b), "loosening" (Figure 1c), face instability (Figure 1d) and slabbing or spalling (Figure 1e). The occurrence and the intensity of these phenomena depend on the orientation, on the spacing and on the extension of the discontinuities. For example, the hazard scenario "loosening" is typical for highly fractured zones, where an instable zone (and not single wedges) develops above the tunnel. Similar considerations apply for the hazard scenario "face instability" which, as a rule, becomes relevant only in weak zones with heavily fractured rock.

For TBM tunnelling, not only the discontinuities but also the compressive strength, the tensile strength and the abrasiveness of the rock are of paramount importance. In fact, high strength and abrasiveness may lead to low penetration rates (slow TBM advance) and major wear of the cutters (Figure 1f) and of the cutter head.

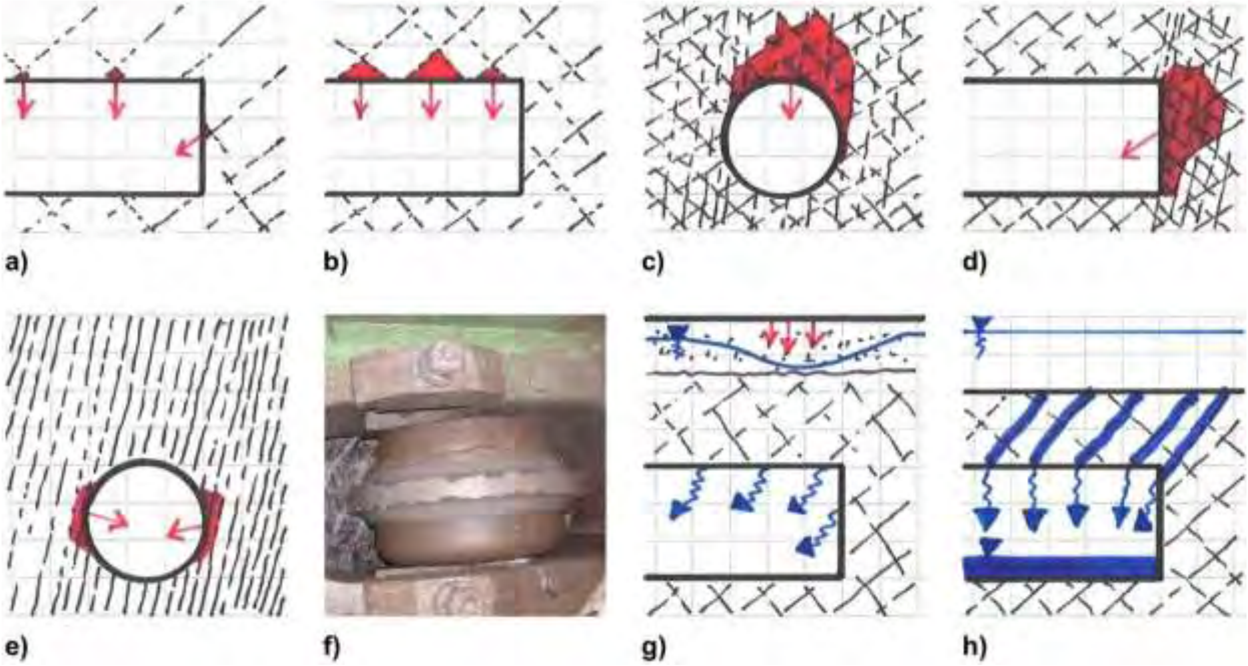


*Figure 1. Typical hazard scenarios (hard rock TBM tunnelling): a) rock fall; b) instability of rock wedges; c) "loosening"; d) face instability; e) slabbing or spalling; f) major wear of the cutters [6]; g) inadmissible surface settlements; h) major water ingress.*

A further group of hazard scenarios is related to water. On the one hand, the water ingress into the tunnel may lead to drainage of the soil deposits (if any) in the project area (for example, in the portal areas of the strait crossing tunnel) and, therefore, may cause loss of valuable humidity-dependent vegetation or inadmissible surface settlements with potential damages to buildings and infrastructure (Figure 1g). On the other hand, major water ingress may lead to a complete flooding of the tunnel in the case of a hydraulic connection to the seabed (Figure 1h), as the water recharge quantity is practically unlimited. Furthermore, in the case of a hydraulic connection to the seabed, the water pressure to be faced may be very high.

## TBM TECHNOLOGY

For hard rock tunnelling, three main types of TBMs exist. These are the so called "gripper TBM", "single shield TBM" and "double shield TBM" (Figure 2).

### Gripper TBM

A TBM excavates the rock by means of cutters which are fitted on the cutter head and pushed against the tunnel face. When a gripper TBM (Figure 2a) is applied, the reaction forces resulting from the applied thrust force and torque are accommodated by means of grippers, which are tended up radially against the tunnel walls. A gripper TBM with a boring diameter in the range of 10 m can dispose of an installed thrust force of 30–35 MN and of a breakout torque of 10–15 MNm resulting in a total gripper force of about 60 MN [7]. In account of this, an efficient TBM advance presupposes sufficient gripper bracing, i.e. the bearing capacity of the rock mass surrounding the grippers has to be high enough for the accommodation of the gripper forces. Considering the generally good rock quality, this is the case in Norway.

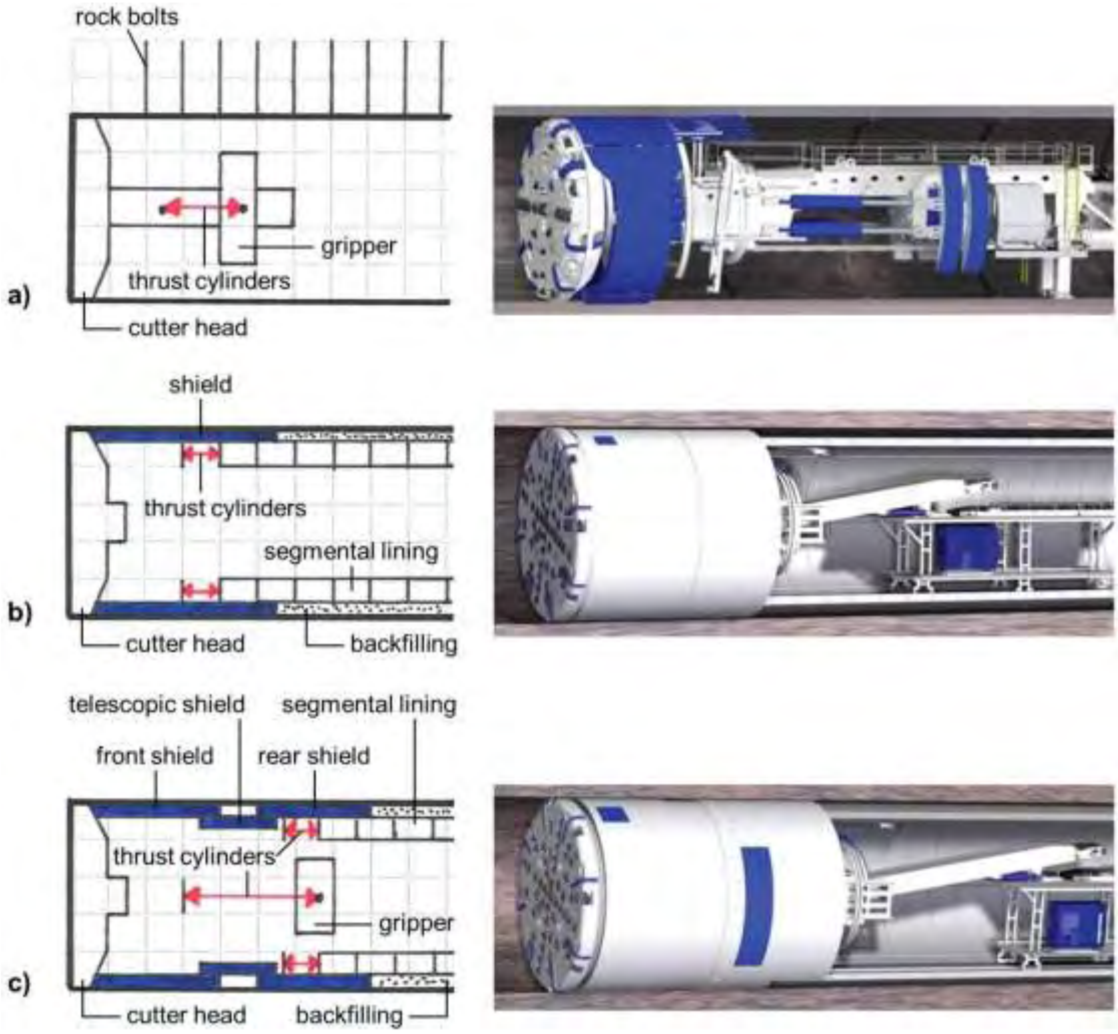


*Figure 2. Hard rock TBMs: a) gripper TBM [8]; b) single shield TBM [8]; c) double shield TBM [8].*

Gripper TBMs are generally equipped with a short shield (canopy, cutter head shield). After the shield (i.e. at a distance of about 5 from the tunnel face), the rock mass is visible and accessible for the installation of support measures. Against instabilities, rock bolts, wire mesh, shotcrete and steel sets can be installed. The quantity of support measures to be applied depends on the encountered rock mass conditions and has an important effect on the TBM advance rate.

As a rule, in tunnels excavated by means of a gripper TBM the tunnel lining consists of two shells (Figure 3a): shotcrete as primary lining and in-situ-casted concrete as final lining; in-between a sealing membrane is installed. Although less usual, a single-shell solution (Figure 3b) is feasible too.

**Single shield TBM**

Instead of being thrusted via grippers, single shield TBMs (Figure 2b) are jacked against a segmental lining. The segmental lining is designed in order to accommodate the thrust force and torque of the TBM (for a boring diameter of about 10 m, 150–200 MN and 30–40 MNm, respectively [7]).

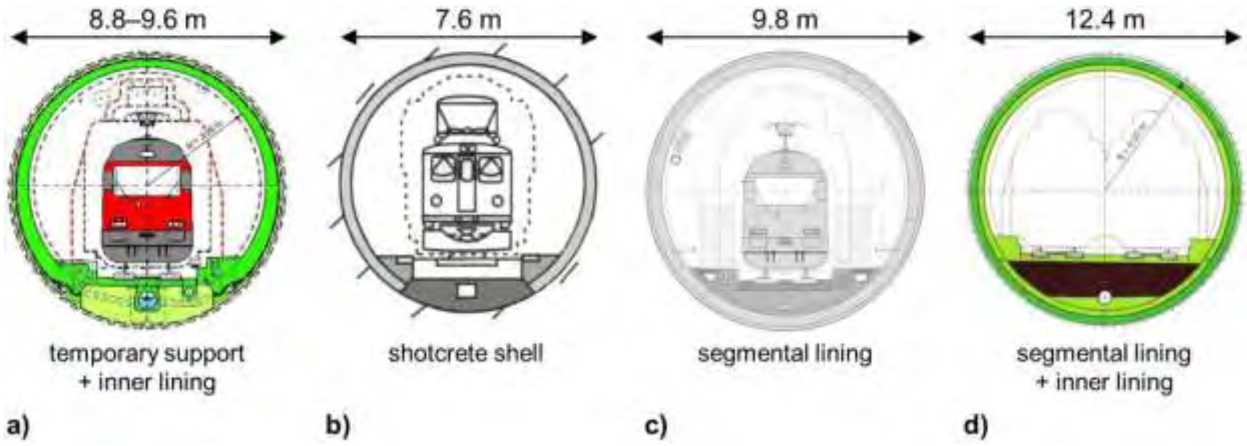


*Figure 3. Typical standard cross-section of TBM tunnels: a) gripper TBM, double-shell lining (Gotthard Base Tunnel, Switzerland) [9]; b) gripper TBM, single-shell lining (Vereina Tunnel Switzerland) [10]; c) shield TBM, single-shell lining (Follo Line Tunnel, Norway) [11]; d) shield TBM, double-shell lining (Zimmerberg Base Tunnel, Switzerland) [12].*

Working with a single shield TBM, the rock mass is not visible behind the cutter head. The shield and the segmental lining continuously protect the miners against instabilities of the tunnel walls. Load transfer between the rock mass and the segments (i.e. a proper support of the tunnel walls) is achieved by a complete and continuous filling of the annulus gap. As the tunnel support (pre-casted segmental lining) is "pre-defined", single shielded TBMs are less sensitive to changes in the encountered geological conditions (e.g. more joints) than gripper TBMs.

The standard solution for tunnels excavated with single shield TBMs is a single-shell lining (Figure 3c), where the joints between the segments are made watertight by means of gaskets. Although less usual, double-shell solutions (Figure 3d) exist.

**Double shield TBM**

A double shield TBM (Figure 2c) consist of three main parts: (i) the front shield (which accommodates the cutter head, the driving unit and the main thrust jacks); (ii) the rear shield (which allocates the auxiliary thrust jacks and the grippers); (iii) the telescopic shield (which connects the front shield and the rear shield and allows for an independent advance of the two other shields. In stable rock conditions, the TBM advance occurs by means of the grippers and the segmental lining is installed simultaneously with boring, thus achieving higher performances than single shielded TBMs. In less stable rock, the bracing by the gripper may not be possible. In this case, TBM advance occurs jacking against the segmental lining with the auxiliary jacks. When instabilities occur around the telescopic part of the shield, additional problems may occur with the extension and compression of the telescopic joint. In this case, one closes the telescopic shield and the double shield TBM is operated in single shield mode.

As for single shield TBMs, the shields and the segmental lining protect the miners continuously against instabilities of the rock mass (which is not visible behind the cutter head) and both single-shell (usual) and double-shell solutions are feasible (Figure 3c and 3d, respectively).

**Wear of the cutters and of the cutter head**

As mentioned above, in hard rock problems like low penetration rates and major wear of the cutters and of the cutter head may arise depending on the geological conditions and the TBM configuration (e.g. spacing and size of the cutters, thrust force, rotational speed of the cutter head). As tunnelling experience as well as empirical studies show, a specific design of the cutter head (e.g. wear protection) for the expected conditions and the application of state-of-the-art technology (e.g. cutters with a diameter of 17–19 inches, which allow a cutter thrust force of 267–312 kN/cutter [13]) are of paramount importance and allow for high penetration rates and for an important reduction of the wear of the cutters and of the cutter head itself.

**Control of water ingress**

The temporary support applied during gripper TBM tunnelling is not watertight. Furthermore, as a rule, for the final lining a drained solution is chosen (Figure 3a). In account of this, with respect to the control of water ingress, a gripper TBM advance is comparable with a drill-and-blast excavation (with the advantage that TBM tunnelling does not cause additional cracks in the rock mass). If a reduction of the permeability is required (for the fulfilment of water leakage limits), the rock mass has to be grouted. In order to avoid inadmissible surface settlements (Figure 1g) or sudden major water ingress (Figure 1h), this has to occur during TBM advance (pre-grouting).

On the contrary, in the case of single shield or double shield TBM tunnelling the temporary support (which is usually at the same time the final one, Figure 3c) can be watertight if designed accordingly. The water tightness of the segmental lining is achieved by means of gaskets placed in the segment joints (according to [14], gaskets which are able to withstand water pressures of 18 bar in the long-term are on the market, while a development towards 20 bar is feasible) combined with the backfilling of the segments with mortar. As the watertight segmental lining is installed near to the tunnel face (10–15 m from the tunnel face), the tunnel is made watertight at an early stage. Therefore, the surface where rock mass drainage can occur is limited to the tunnel face and the tunnel boundary around the shield. In account of this, and assuming that some water ingress will be acceptable in the short-term, the amount of required pre-grouting is much smaller than for gripper TBM tunnelling or drill-and-blast excavation. In fact, if the rock quality is good and the water is infiltrating from single joints, the TBM is able to proceed fast and regularly. As the drainage surface is small and the time available for drainage is short, the potential "damage" is small and pre-grouting is not necessary. On the contrary, in weak zones with potential extensive water inflow and, more in general, to avoid a major water ingress which may endanger tunnel stability or flood the entire tunnel, pre-grouting still remain a valuable countermeasure.

Furthermore, single shield TBMs can temporarily be closed to achieve a watertight system if designed accordingly. This requires, among others, a separation wall behind the cutter head in order to create a working chamber with a limited volume which can be flooded. Additionally, applying state-of-the-art technology of shield tunnelling in soils, a single shield TBM can be enhanced with features – like, e.g., an active face support and water pressure compensation – which enlarges the range of applicability. For example, a dual mode TBM (hard rock TBM and slurry shield) – like the one which is now excavating the Lake Mead Intake Tunnel No. 3 (length: 4.8 km, boring diameter: 7.2 m) in the USA [15] – is able to face not only hard rock but also fault zones containing crushed rock and soil under high water pressures.

**Pre-grouting, auxiliary measures**

Pre-grouting is a valuable countermeasure against inadmissible water ingress. Moreover, grouting can be applied to improve the rock mass and, therefore, to improve stability. The execution of pre-grouting in front of the TBM presupposes that the TBM is designed for this purpose and equipped correspondingly. As an example, the TBM used for the excavation of the Hallandsås Tunnel (length: 8.5 km, boring diameter: 10.6 m) in Sweden can be mentioned [16].


*Figure 4. General arrangement of the drilling pattern for pre-grouting (Hallandsås Tunnel, Sweden) [16]): a) longitudinal section; b) cross-section.*

This TBM was equipped with four drill rigs allowing for the drilling pattern illustrated in Figure 4. Two drill rigs are located inside the shield for the face positions, while a third drill rig is located behind the ring erection area for the periphery positions. An additional fourth drill rig could be temporarily installed on the segment erector. For drilling, the use of preventers (to counter act the water pressure) is possible. The drilling pattern of Figure 4 (length of the grout holes: 35 m) requires 30 inclined channels in the shield skin (in two different angles: 10° and 13°), a large number of ports in the bulkhead and in the submerged wall as well as openings in the cutter head (for 26 outer and 7 inner face positions). The Hallandsås TBM was equipped with one grouting unit. The installation of two identical pre-grouting plants is possible.

Additional to pre-grouting, other auxiliary measures (like, e.g., pipe umbrellas) may be necessary in exceptional cases. The execution of such measures is possible, provided that the TBM is equipped correspondingly (i.e. the auxiliary measures have to be planned in advance).

**Rock mass exploration in advance**

In order to decide if pre-grouting or auxiliary measures are necessary (and to start timely with their execution), systematic rock mass exploration by probe drilling in advance is recommended. This is particularly true for strait crossing tunnels, as the limited accessibility restricts the possibilities for rock mass exploration before construction. The execution of exploration drillings from a TBM is state-of-the-art and does not represent a particular technical problem, provided that the TBM is equipped adequately. The same drill rigs as for the execution of pre-grouting can be used. As a rule, probe drilling is executed during the maintenance shift of the TBM. The drilling must be long enough (e.g. 50 m) in order to cover at least and with enough overlapping the tunnel section excavated by the TBM within one working day.

## TBM VS. DRILL-AND-BLAST

In the following of the paper, the two alternatives "TBM" and "drill-and-blast" are compared briefly and qualitatively (a proper comparison requires project specific and detailed planning works) with respect to the following criteria: construction time, construction costs, logistics, environmental impact and health and safety.

**Construction time**

The topic "construction time" is discussed by means of the example of a 25 km long traffic tunnel. Figure 5 shows the construction schedules for two cases: (i) excavation of the tunnel from the two portals; (ii) excavation of the tunnel from the two portals and from an intermediate point of attack (assumed to be in the middle of the tunnel and 250 m long). For the sake of simplicity, the diagrams show the works for one tube only. If the tunnel consists of two tubes, similar considerations apply. Furthermore, the construction schedules do not include the works for the inner lining. This simplification does not affect the comparison of the construction time if for both TBM and drill-and-blast a single-shell (Figure 3b) or a double-shell (Figure 3d) solution is envisaged. In this case, the works for the inner lining can be carried out after the tunnel excavation (or in parallel to the tunnel excavation with corresponding logistics adjustments) in the same way for both excavation methods. The same does not apply comparing a drill-and-blast excavation with double-shell lining with a TBM excavation with single-shell lining (Figure 3c), as the TBM solution does not require concreting works for the inner lining.

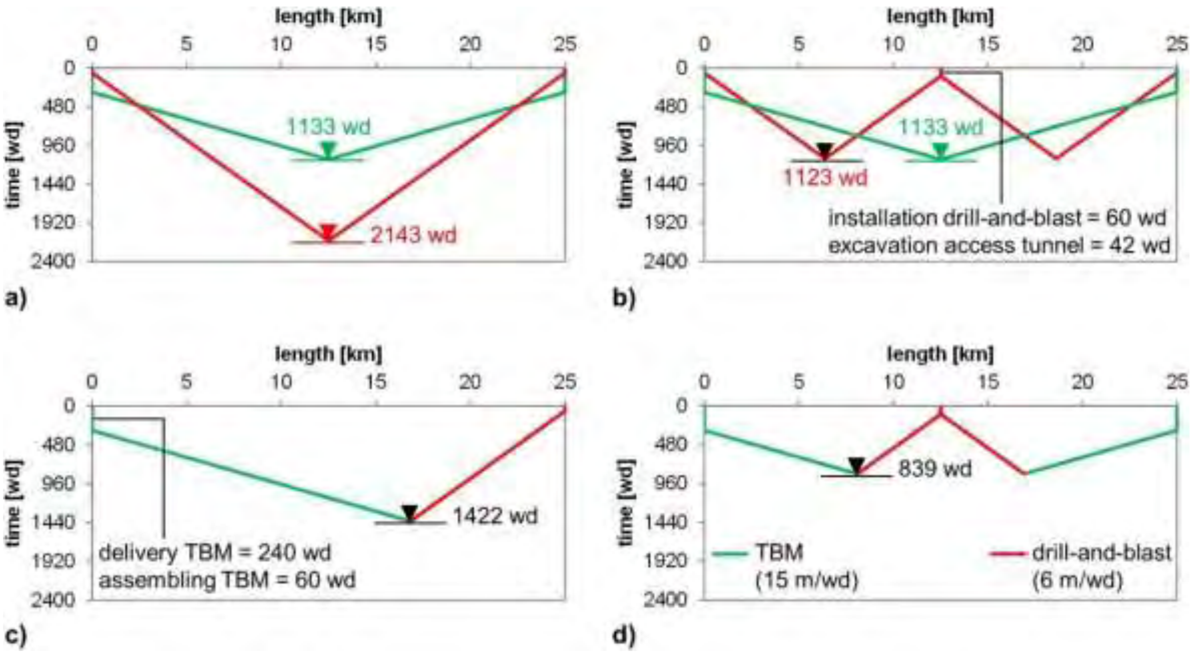


*Figure 5. Construction schedules for a 25 km long tunnel: without (a, c) or with (b, d) intermediate point of attack; without (a, b) or with (c, d) combination of the excavation methods; green: TBM, red: drill-and-blast.*

Excavating the tunnel from the two portals, the excavation time is shorter when applying two TBMs than with two drill-and-blast headings (Figure 5a). The higher advance rate of the TBMs (15 m/wd) compensates their longer mobilization time (during which, e.g., the construction site can be prepared or diversions near to the portals can be excavated). The drill-and-blast headings can be started earlier but are slower (6 m/wd). This is due, among others, to the hindrances resulting from mucking in such long tunnel sections (up to 12.5 km in this example) and to the higher amount of support measures which are necessary (less "gently" excavation than with TBMs). With an intermediate point of attack (Figure 5b), the two excavation methods are equivalent (please note that the application of four TBMs has not been considered due to the high investment costs). Of course, the excavation methods can be combined (Figure 5c and 5d). In this example, the shorter excavation time is achieved combining two TBM drives (from the portals) and two drill-and-blast headings (from the intermediate point of attack), as shown in Figure 5d.

## Construction costs

In the south part of the Lötschberg Base Tunnel (length: 34.6 km, boring diameter: 9.4 m) in Switzerland, along approximately 4 km the two tubes of the tunnel were excavated applying TBM and drill-and-blast in parallel. A comparison [17] showed that the construction costs of the two excavation methods were in the same range. Recently, the same conclusion has been drawn for the Follo Line Tunnel (length: 19.7 km, boring diameter: 9.8 m) in Norway [5].

## Logistics

With respect to logistics, it can be stated that modern solutions exist for both excavation methods. In TBM tunnelling, the degree of industrialization is high. An example is the mucking, which if carried out with conveyor belts is very efficient (the muck is transported directly from the cutter head to the intermediate deposit on the rig area), particularly for long tunnels.

An important difference exists regarding the rig areas. While the dimensions of the rig area of a gripper TBM drive and of a drill-and-blast excavation are comparable, the rig area required applying a single shield or double shield TBM is bigger. This is due to the space requirements for the segment production and storage. In this respect, a possible solution is the outsourcing of the segment production and the "just-in-time" delivery of the segments.

## Environmental impact

Noise and vibrations are two important impacts on the environment (residents and nature). The rotation of the cutter head is the main source of (structure borne) noise during TBM excavation, while the vibrations are in general negligible (< 1 mm/s). The vibrations caused from blasting as well as the (structure borne) noise related to the drilling of holes for pre-grouting and blasting may disturb the environment. For both alternatives the national regulations have to be fulfilled (for example, through a limitation of the working hours). With respect to noise and vibrations, it seems to be that the two excavation methods are more or less equivalent.

The same conclusion does not apply with respect to the process water to be released in a recipient after treatment on the construction site. The quantity of water to be treated and after that released is smaller for a TBM drive than for a drill-and-blast excavation. Furthermore, the nitrogen content of the water of a TBM construction site is smaller. This is because the explosives used for drill-and-blast increase the nitrogen content in the water and, excepted the ammonia removed during pH-lowering, the nitrogen is not removed during the water treatment process.

## Health and safety

With respect to health and safety, the excavation method "TBM" is better than the excavation method "drill-and-blast".

According to the following remarks, it can be stated that the air quality in tunnels excavated by means of TBMs is better than if drill-and-blast is applied: (i) the dust content is low; (ii) explosives materials are used in small quantities only (e.g. for the excavation of cross-passages); (iii) mucking by means of conveyor belts instead of trains or trucks reduces the number of diesel engines to be employed.

The tunnelling equipment for TBM tunnelling is comparable with a factory and it can be assumed that the noise level is similar. One of the most important sources of noise during drill-and-blast excavation, i.e. drilling of boreholes, is less relevant in TBM tunnelling, as the amount of meters of boreholes to be drilled is much smaller.

During drill-and-blast excavation, the workers are exposed to geological hazards until the temporary support is built-in. On the contrary, the short shield of a gripper TBM protects the workers in the machine area, while during shield TBM tunnelling the shield and the segmental lining protect the workers continuously. A positive impact on safety also have the replacement of the cutters from inside the cutter head, the reduced amount of pre-grouting (less work with high pressure devices), the application of conveyor belts (less potential for traffic accidents during mucking) and the high degree of industrialization with the application of standard procedures.

## CLOSING REMARKS

Both excavation methods "TBM" and "drill-and-blast" have advantages and disadvantages. For a given project, both alternatives may be feasible and valuable. The final evaluation depends strongly on the weighting of the comparison criteria. For example, for a long strait crossing tunnel a higher weighting of the criteria "construction time" will lead to the choice of the TBM solution, as this allows the excavation of the tunnel in a shorter time. For both excavation methods, the construction costs are often comparable. In account of this, it is recommended to tender both solutions and to let the market decide which one will be applied for the construction of a given tunnel. However, it is important to note that a double tender is possible only if both solutions have been worked out with a sufficient degree of detailing.

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# BASE TUNNELS IN THE ALPS - HELPFUL EXPERIENCES FOR STRAIT CROSSINGS FROM THE GOTTHARD BASE TUNNEL

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## ABSTRACT

*Strait crossings with excavated undersea tunnels and the crossing of the Alps with Base Tunnels are different in many aspects. Whereas the vertical alignment of a strait crossing by tunnel consists mainly of falling drives, base tunnels have mainly a rising gradient. The high overburden of the alpine base tunnels creates high initial stresses, rock burst phenomena, squeezing and high initial rock temperatures. For strait crossings with subsea tunnels water inflows to the falling drives are an important challenge for the tunnel engineers and miners. Nevertheless many aspects of tunnelling are similar for strait crossings and for alpine tunnelling. Both type of tunnels have to deal with the uncertainties of the ground conditions and the related remaining risks, the logistic challenges of long tunnel drives, the related safety requirements and the need of the protection of the environment.*
*This paper will show the experiences and the lessons learned from the Gotthard Base Tunnel:*

- *Logistics for long and deep tunnel drives*
- *Concepts for exploratory drillings during the tunnel drive*
- *Experiences with drilling and grouting*
- *Safety aspects for long tunnel drives*
- *Environmental aspects, mainly the reuse of the muck for gravel production*
- *Contract management*

### ALPINE BASE TUNNELS AND UNDERSEA TUNNEL - SIMILARITIES AND DIFFERENCES

For many people there may be no big difference in constructing a tunnel under a mountain or under the sea. In both cases an underground space has to be excavated in often not well-known ground conditions. Base tunnels and strait crossings normally belong to the category of Mega-Projects [1] with investments often of more than 1’000 € Mio and attracting a lot of public attention. Both types of project have a common catalogue of project requirements, which has to be fulfilled:
Health and safety requirements
Environmental requirements
Project specific requirements on quality, functionality, time and costs
Requirements on highly professional design and construction processes, project organisation and project management
The requirements of the public opinion

The differences in the two types of projects are the different boundary conditions. Both types of tunnels can be very long (Channel Tunnel 50 km, Gotthard Base Tunnel 57 km). Whereas Alpine Base Tunnels normally are characterised by high overburden (more than 2’000 metres) subsea tunnels have a comparatively low overburden of around 100 metres of rock overburden and up to a few hundred metres water column (Fig. 1).

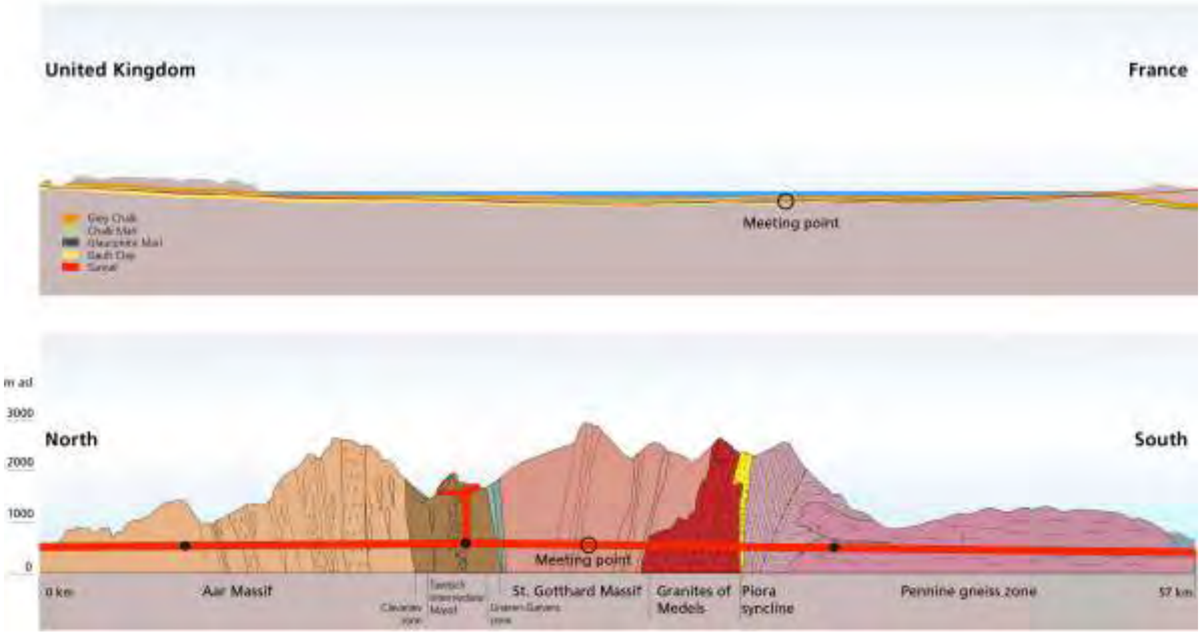


*Fig. 1: Comparison of the longitudinal profile of the Channel Tunnel and the Gotthard Base tunnel in the same scale (AlpTransit Gotthard Ltd. (ATG))*

The high overburden of Base Tunnels makes a previous systematic probe drilling campaign along the future tunnel axis practically impossible. Pre-investigations before the construction can be carried out only at selected locations of special interest. High overburden is related to high initial ground and water pressures with the typical hazard scenarios like rock burst, squeezing rock and high initial ground and water temperatures. Water inflows can occur under very high pressures.

In subsea tunnels all hazards related to water inflows are dominant. Hazards like squeezing rock or rock burst are generally of lower importance. The initial rock temperatures of subsea tunnels are low. Nevertheless working place temperatures have to be controlled due to machines heat. The completely different topographic situation has a big impact on the design and construction procedures of the tunnels. Undersea tunnels can normally not be subdivided in different construction sections by means of intermediate access tunnels, as it is done for Base Tunnels. Risk management is the key factor to control hazards and to take benefit from chances. The procedures for managing risks are the same for both types of tunnels. The implementation of risk management in the earliest project phases and the continuous maintenance is essential for all underground constructions.

## EXPERIENCES FROM THE GOTTHARD BASE TUNNEL

### Project description

The Gotthard Base Tunnel (GBT) is part of the Swiss master plan “New Railway Link through the Alps” (NRLA). The 57 kilometres long tunnel will be the centrepiece of the European transit corridor No. 24 between Rotterdam and Genoa (Fig. 2) and will be used in in a mixed mode (passenger trains and freight trains). The maximum speed in the tunnel will be 250 km/h for passenger trains and 160 km/h for freight trains. The tunnel system consists of two separate single-track tunnels with a minimum axial distance of 40 metres, a maximum slope of 6.76 ‰ and a minimum horizontal radius of 5’000 m. The single-track tunnels are linked every 312.5

metres by cross passages. Multifunction stations, containing technical rooms, ventilation systems and emergency stations are constructed in the third points at Sedrun and Faido.


*Fig. 2: Gotthard Base Tunnel, centrepiece of the European Transport Corridor 24 (ATG)*

The main purpose of the new railway infrastructure is to shift a major part of the heavy transalpine freight traffic through Switzerland from the road to the rail. 220 to 260 freight trains per day will offer higher transport capacities with higher velocities by using the first flat rail link through the Alps (Fig. 1). 50 to 80 passenger trains per day will offer connections linking the economic regions of southern Germany, Switzerland and northern Italy. Faster travel speeds in more comfortable trains will create a real alternative to travelling by car or by airplane.
The project specific standards of the Swiss Federal Office of Transport (FOT) require a lifetime of 100 years for the civil work. No major rehabilitation work with significant operational restrictions (with regard on scope and duration) is allowed during the lifetime.

This requirement can only be achieved with the construction of a double lined tunnel (Fig. 3) with the provisional rock support as outer lining (first lining) and a permanent inner concrete lining (second lining, minimum thickness 30 cm. A drainage system between the two linings reduces permanently the initial water pressures of up to 150 bars. A waterproofing membrane on the entire length of the tunnel (114 kilometres) protects the inner lining against water filtrations. Due to the high initial rock stresses, high water pressures and high initial ground temperatures the development of special design solutions and construction materials was required, in order to fulfil the lifetime requirement.

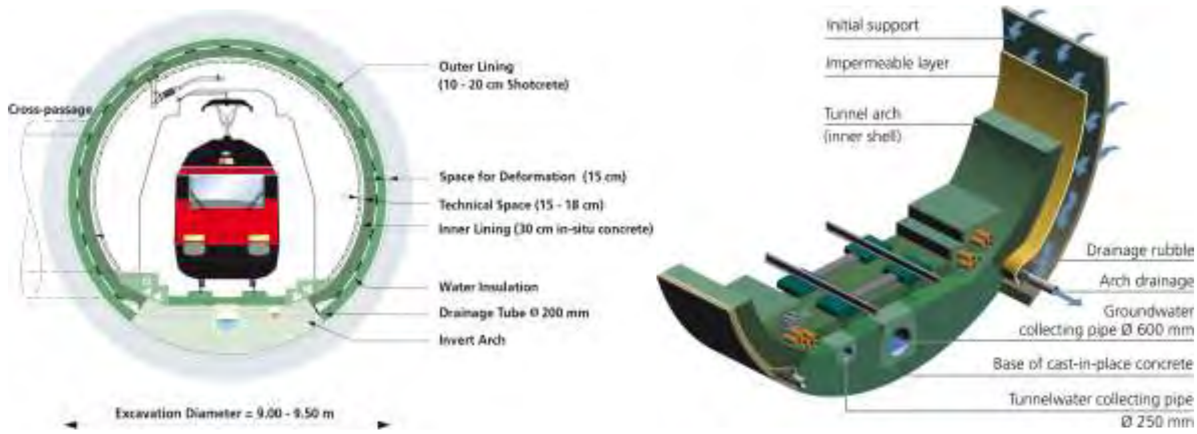


*Fig. 3: Standard cross section: 100 year lifetime - double lined tunnel (ATG)*

The tunnel was divided into five different construction sections, in order to shorten the total construction time by allowing the parallel work on the five different main sites of Erstfeld, Amsteg, Sedrun, Faido and Bodio (from north to south). 64% of the in total 151.1 kilometres long tunnel system were excavated by 4 Tunnel Boring Machines (TBM).
The civil work of the Gotthard Base Tunnel is practically completed. On approx. 60% of the tunnel length the railway technics is under construction or already completed (Fig. 4). The GBT becomes commercially operable in December 2016.

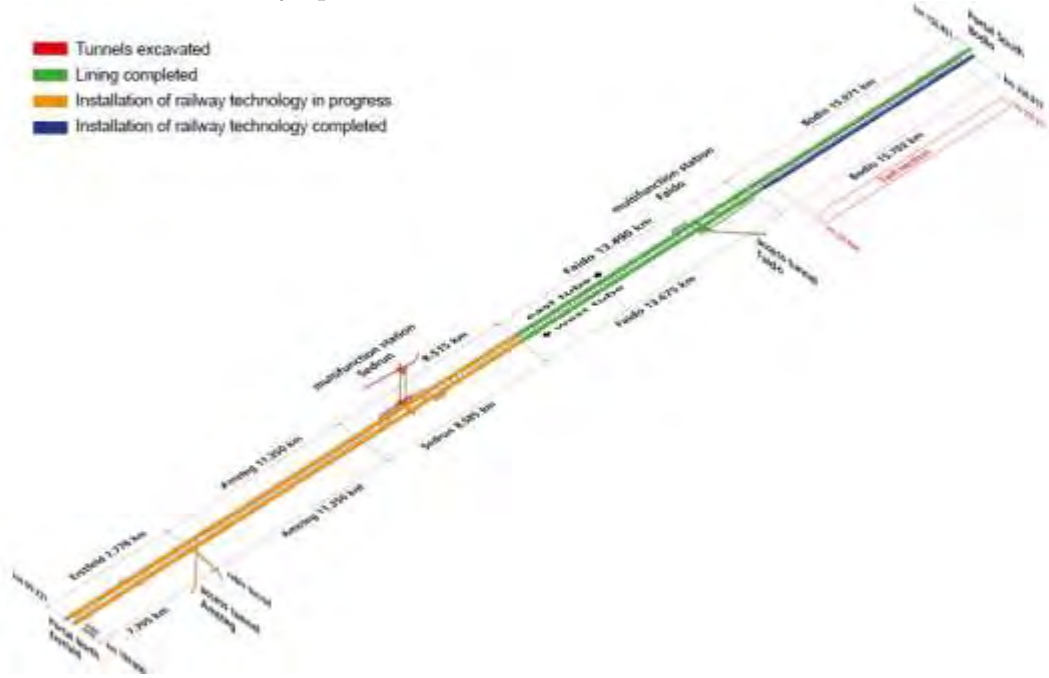


*Fig. 4: Gotthard Base Tunnel, state of the work, End of 2012 (ATG)*

**Uncertainties of the ground conditions**

Underground construction is different from any other type of construction, due to the fact, that the construction material, the ground, is often not well known or may change within a short distance. Additionally the behaviour of the ground is also a result of the interaction with the construction methods applied. Important residual risks are characteristic for large underground projects. The technical measures and the contract models have to take into account these special boundary conditions of underground construction and should allow a fast reaction on changed or unforeseen ground conditions.
At the Gotthard Base Tunnel only the expected most critical areas could be investigated by pre-investigations in the design stage. These investigations helped for a design adapted to the

difficult ground conditions (e.g. Tavetsch Intermediate Massif North, Fig. 1) or showed, that the ground conditions would be better than initially assumed (Piora-Syncline, Fig. 1).

Nevertheless, the forecast of the ground conditions for most of the tunnel length had to be done by extrapolation of the information from the surface and from other neighbouring underground structures to the tunnel level. Therefore AlpTransit Gotthard Ltd. (ATG) as the owner and his engineer took the decision, that each meter of tunnel in both tubes (east and west) should be investigated during the excavation phase by well-adapted examination methods. At Amsteg and Faido certain tests with seismic methods were carried out. Due to the heterogeneous crystalline geology, the results were not satisfying for the purpose of a forecast. Finally probe drillings in the axis of the tunnel carried out from the tunnel-face were the main measures for pre-investigations (percussion drillings 80 to 100 metres, core drillings 150 to 300 metres both types with overlapping lengths) (Fig. 5). Preventer protection was carried out in all cases where high water inflows were expected. Short radial probe-drillings have been used only in few special cases.

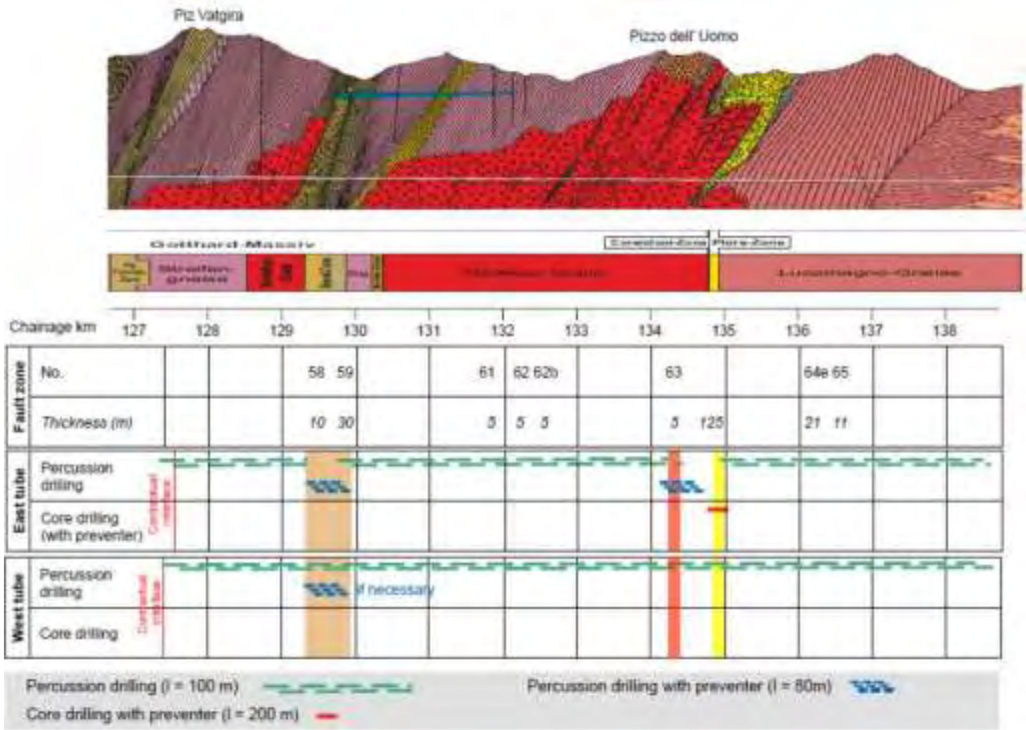


*Fig. 5: Gotthard Base Tunnel, Section Faido, concept for probe drillings*

The project was delivered by applying the design-bid-build process. All contracts were based on detailed bill of quantities ("unit price contract") with a well-defined risk sharing policy (based on code SIA 118/198 [2]) and a detailed catalogue of auxiliary construction methods, such as drainage and grouting.

During the construction unexpected bad ground conditions were encountered in the area of the multifunction station Faido. A delay of two years and additional costs of more than 400 Mio € were the result [3]. Nevertheless the unit price contract gave the required flexibility to handle this critical situation.

In three cases (Amsteg, Sedrun and Faido) fault zones were encountered, which required major grouting campaigns for ground improvement (Amsteg, Faido) and impermeabilisation (Sedrun) [4], [5]. It is an important experience that all three cases occurred in the second tube (western tube) in areas where the leading eastern tube had already passed the critical zones without any major difficulties. The fault zones were detected in advance but there was no forecast of the bad behaviour of the ground in none of the three cases.

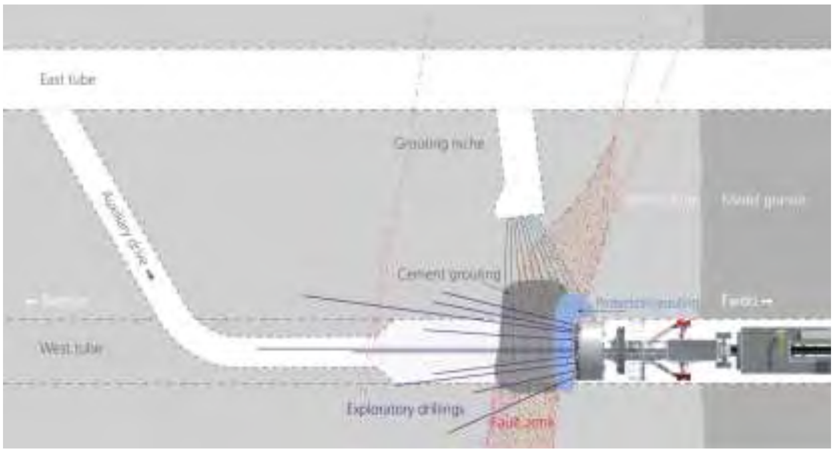


*Fig. 6: Gotthard Base Tunnel, Section Faido, grouting of a fault zone in the western tube*

A standstill of the excavation of 5 months and additional costs in the range of 10 Mio € were the result in all three cases although there was a comparatively easy access to the fault zone area also from the eastern tube. All three incidents belonged to the category of accepted residual risks and could have been avoided only by means of excessive pre-investigations, which would have hindered the daily advance rates. Such measures were not economical in comparison to the residual risks (ALARP-principle of risk management (as low as reasonable practicable)). The additional construction time could be compensated or was included in the contractual construction time, following the principles of Code SIA 118/198. The fault zones caused therefore no delay in the general construction schedule.

**Construction procedures - logistics**

Construction work of the GBT started in 2002 immediately after the signature of three of the four main contracts on the northern side (Amsteg), in the central construction section (Sedrun) and on the southern side (Faido and Bodio) with the goal of a beginning of the commercial operation in December 2014.

For each of the five sections of the Gotthard Base Tunnel large quantities of different categories of material (muck, shotcrete, concrete, rock bolts, wire mesh, steel ribs, impermeabilisation membrane, operating supplies) had to be moved on long distances up to 30 kilometres. In the case of the construction section Sedrun, two 800 metres deep vertical shafts gave the access to the tunnel site (Fig. 7). All the muck, the construction materials and the machinery had to be transported through these shafts by means of a special type of a shaft hoisting installation (Koepe-Machine) known from the mining industry. The supply with electrical energy, water and the ventilation with fresh air, but also the dewatering system and the transport of the exhaust air also used the two shafts.

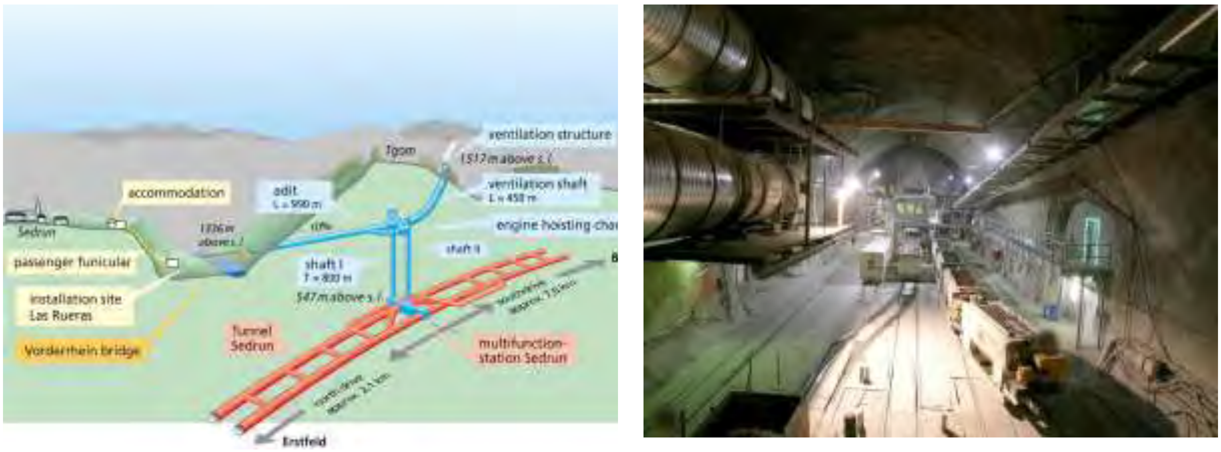


*Fig. 7: Section Sedrun of the GBT: access via 800 metres deep shafts and shaft bottom (ATG)*

Tunnelling means the just in time production of a long horizontal structure. The limited access to the various construction sites is a big challenge for the contractors. The contractors selected different concepts for the transportation system. In the sections Erstfeld, Amsteg and Sedrun the inner lining was poured after the completion of the excavation. These conditions were favourable to the transportation of the muck by belt conveyer, whereas in Bodio and Faido the inner lining followed the excavation in parallel. Taking these restrictions into account, the contractor took the decision to install a railway system for all transports. In the average every 7 minutes a train circulated from the rig area at the southern portal to the different working places (excavation front, cross passages, water proofing, inner lining, multifunction station, maintenance) (Fig. 8). Experienced railway operators managed the entire traffic from a control centre.

After the construction of additional logistic cross passages, the contractors managed finally their tasks successfully. It is recommended to include a sufficient number of logistic cross passages already in the tender design [6].


*Fig. 8: Gotthard Base Tunnel, Section Faido/Bodio, transportation logistics by rail [6]*
*(CP = cross passage, IL = inner lining)*

**Health and Safety**

As everywhere, also in the GBT-Project was stipulated, that each worker should leave the job site at the end of his shift as healthy and safe as he started his work.

In the design phase, different types of fires in the tunnel with inclusion of workers in front of the fire were seen as one of the main safety risks. High initial rock and ground water temperatures of more than 50°C were also a big challenge for creation of acceptable working conditions. ATG, as the owner tendered the health and safety installation measures in detail in order to withdraw them from any speculation. Detailed specifications on the health and safety requirements were given in order to be considered by the contractors in their construction processes (e.g. smoke protection containers close to the excavation front connected to a protected compressed air pipe from outside, cross passages to the 2nd tube excavated 1’500 metres behind the first excavation front etc., layout of the ventilation system taking various fire scenarios into account, cooling systems).

Only very few fires occurred below ground during the construction period, mainly caused by engine failures. Thanks to modern extinguishing systems on all major machines (e.g. locomotives) no severe consequences occurred. A big fire of the conveyer belt system at the rig area of Sedrun showed the high importance of the adequate quality of the conveyer belts below ground. The same accident below ground (with the same, but for underground work inadequate belt quality) would have had disastrous consequences.

The accident risk in the AlpTransit project is defined as the number of accidents per full-time employee per year. This number includes any injury (also bagatelle cases). Since 2002 the accident rates were monitored systematically (Fig. 9).

In a common effort (“stop risk” campaign) the contractors, the site supervision and the client tried to bring this specific rate below the predefined target value of 200 accidents per 1000 workers per year. Even if the accident rates could be substantially reduced since the beginning of the main tunnelling work at the GBT, the monitored monthly accident rate never could generally be reduced below the target value [7].

Regrettably 9 workers lost their lives during the construction of the GBT. Eight of these accidents were related to machinery and transportation; one case was related to a rock-fall from the tunnel face.  Each fatality is one too much and affects dramatically the live of relatives often for the rest of their lives. The fact, that the specific number of fatalities is comparatively low compared to earlier underground projects, indicates that the industry has reached major improvements in the last decades, but the final goal is not reached yet. A further reduction of accidents is a continuous obligation for all partners on the job site. “Target zero” as it is exercised actually in the Crossrail-Project (UK) must be the future state of mind in all Mega-projects.

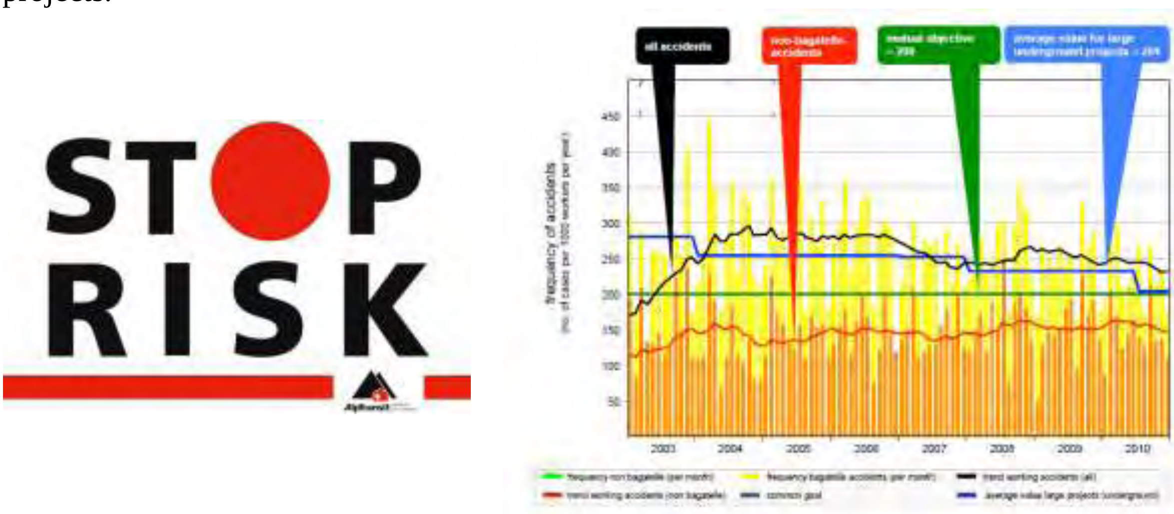


*Fig. 9: Stop risk campaign and specific number of accidents (ATG, [5])*

**Environmental protection**

The main goal of the Gotthard Base Tunnel Project is the improvement of the environmental conditions in the alpine regions by shifting the heavy load trucks from the road to the rail. Air pollution, noise omissions and the risks of accidents should be reduced to a significant lower level in the operational phase. But also in the construction phase dust protection, protection against air pollution, noise protection, water protection, protection of fauna and flora, protecting natural resources by adequate construction methods and the avoidance of damages to third parties properties and rights are also a must during.

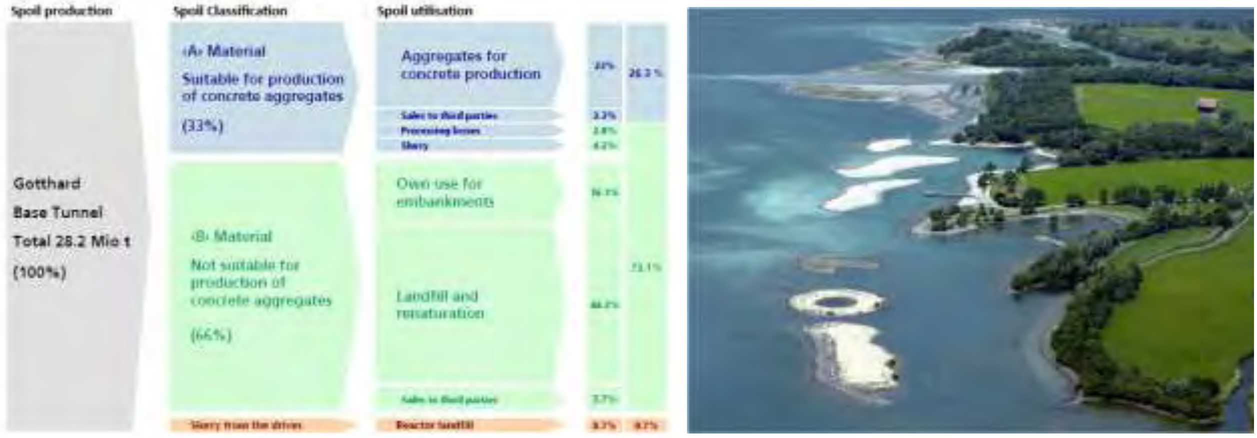


*Fig. 10: Flow chart of the excavated material and excellent example of landfill (artificial islands in the lake of Lucerne for bathing and nature reserve)*

The following measures were taken to protect the environment, to reduce or to compensate the negative impact of the construction:

33% of the spoil was of good quality and could be used for the production of concrete aggregates for the shotcrete (rock support) and the in situ concrete (final lining). 100% of the concrete gravel for the tunnel construction has been produced from excavated rock material with origin from the TBM-drives and the conventional drives! New technologies such as a mica washing process were introduced in order to receive a higher percentage of utilisation.
All mass goods were transported by rail or by conveyer belts (no road transports).
Particle filters were required for all Diesel engines above and below ground long before there was a legal obligation.
Each rig area was equipped with a high tech sewage treatment plant cleaning a broad variety of polluted water.
High investments were made for the treatment of the various qualities of mud.
The inevitable interventions to flora and fauna were compensated by predefined mitigation measures. The goal to increase the biodiversity could be reached thanks to a good collaboration with environmental organisations and specialists.

**3.8 Management of the underground construction contracts**

Contracts for underground structures should take into account the special risk situation of this type of work and should allow a fast reaction on changed or unforeseen ground conditions. Fair risk sharing and partnering are helpful tools to facilitate fast decisions on site. Fair risk sharing between the client and the contractor helps to avoid unnecessary risk surcharges in the contractor’s bid and therefore to reduce the total project costs.

The Swiss Code SIA 118/198 gives a standard solution and follows the widely accepted principle of risk sharing for underground construction work:

The ground belongs to the client. Changed ground conditions outside the contractual limits are therefore client's risk.
Means and methods applied for ground conditions within the contractual limits belong to the contractor's risk sphere.

This principle works only if the client gives a complete and accurate description of the ground conditions and the expected behaviour according to his project. All relevant data from the ground investigation and all other relevant boundary conditions must be accessible to the contractors.

The selection of means and methods, especially the selection of the excavation method is one of the big risk factors for an underground project. The selected methods should be able to overcome the whole range of the expected ground conditions. Together with a catalogue of auxiliary construction methods, the selected excavation methods should also be able to master unexpected changed ground conditions. If the owner has, based on his risk analysis, compelling reasons to favour one method, he should select the excavation method before tendering. Such reasons can be risks related to the ground conditions, but also environmental or logistic requirements. If the ground conditions allow different methods and mainly the economic requirements become decisive, the contractors should be entitled to select his favourite method. Nevertheless the client has to define fair conditions for the competition (e.g. tendering two methods or allowing contractors alternatives in cases where the client tenders only one method).

Partnering is a key factor for success. Partnering means the definition of mutual objectives, the joint monitoring of the performance benchmarks and the common resolution of problems in the case of divergences. Partnering needs a culture of confidence and open discussions on each level. Big deviations from the cost and time targets at the GBT caused important contractual disputes. Solutions could be found after tough and time-consuming negotiations leaving both partners on an equal level of "unhappiness". The detailed unit price contracts, transparent models for the calculation of the unit prices and of the time dependent costs, a clear definition of the risk sharing and a culture of partnering made this result possible.

Not all disputes could be solved directly between the contractual partners. A Dispute Resolution Boards (DRB) was implemented by contractual agreement in each main construction lot following the Swiss Recommendation on Dispute Resolution (VSS 641'510) [8]. The DRB acts only up to limited amounts in dispute as an arbitration panel. In all other disputes the DRB gives a recommendation, which can be accepted or rejected by the contractual partners. In the case of rejection of the DRB's recommendation by one or both contractual partners the doors are open for a court case. The biggest effect of the DRB was the self-made psychological pressure on the site organisation to find an own solution.

Despite the positive experiences with the contract management of the GBT, a general trend to shift differences quickly to the lawyers and to court decision can be observed also in Switzerland. Experiences show, that court decisions are not faster, not cheaper, not better for the project reputation and are substantially often not better than the compromises taken by the contractual partners directly or based on a DRB recommendation. It's time to think about the actual trend and to change the culture back to partnership where the site management with the help of technical specialists and lawyers find fast and stable compromises.

## CONCLUSIONS AND RECOMMENDATIONS

Alpine Base Tunnels and subsea tunnels have to fulfil the same project requirements but with different boundary conditions. Looking back on the history of the execution of the civil works for the Gotthard Base Tunnel, the following experiences may be helpful for future strait crossings:

Professional risk management is the "insurance policy" for the project management and provides the adequate information on the risk situation on time. Risk management has to be introduced in the earliest phases of the project.
Residual risks are immanent to underground constructions and have to be accepted and communicated by the various stakeholders. The contract should contain a clear and fair risk sharing policy.
Well-defined mitigation measures such as a concept for probe-drillings and a catalogue of auxiliary construction measures help to reduce the ground risks.
The design of the tunnel system should take into account all the construction sequences (e.g. logistic cross passages for twin tube systems).
"Target Zero" must be the future state of mind on health and safety requirements (Crossrail).
The reuse of the muck for the production of concrete aggregates is technically feasible and is today a must in order to save the natural, alluvial resources.
The modern tunnelling technique and logistics has proved of value even in the most difficult conditions.
Unit price contracts with fair risk sharing, partnering and the implementation of a dispute resolution board help to avoid time-consuming court cases.

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# PROCUREMENT OF MAJOR CROSSINGS

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## ABSTRACT

*Great attention is rightly paid to the technical challenges that major crossings present to promoters, designers and builders. However, the success or otherwise of a project is often judged in the public's mind by how quickly and economically such infrastructure assets can be built. This in turn is strongly influenced by the way in which they are procured. This paper presents a review of some of the technical considerations associated with procurement of major crossings in recent years, how key technical risks have been addressed and allocated, and how this has influenced overall design and construction. The characteristics of 'major crossings', including strait crossings, that influence this and set them apart from other infrastructure markets are examined. Case studies of well-known major international bridge and tunnel projects are presented including experience in the UK and in Nordic countries, from which 'model' procurement processes can be drawn. This allows for the wide variety of non-technical constraints and stakeholder aspirations that such iconic projects invariably present to those privileged enough to be involved in their development.*

*It is concluded that as innovative procurement methods have continued to be developed over the past 15-20 years, there are increasing numbers of examples of good practice which can act as models for future projects.*

## INTRODUCTION

Over the past 25 years or so the number of major crossings that have been developed and built around the world has greatly increased. These projects are becoming increasingly ambitious in their scope and represent a major investment by governments and private investors. With an ever-growing focus on cost-effective delivery of major civil engineering projects, clients and their advisors are continually seeking news ways in which to procure such projects. Major crossings have a wide range of characteristics which are well-suited to a variety of approaches in their procurement to ensure that the client's needs and requirements are met. This paper will examine these characteristics and review a number of well-known recent sea and river crossings demonstrating alternative approaches to achieving these requirements.

## DEFINITION OF MAJOR CROSSINGS

We should first define what we mean by major crossings, as this can influence the thought process when determining procurement routes. One definition could be:

> *Major Crossings are those projects which are typically procured by clients as 'stand-alone', often requiring separate funding, and set aside from the remainder of the development of their strategic transportation network by reason of their being a high value structure or sea/estuary crossing. They may also be procured separately within a larger programme.*

The technical disciplines most closely associated with major crossings include bridges or tunnels crossing a major geographic obstacle, such as a strait, major river, wetlands, valley or mountain range. Major crossings may also include large scale bridge or tunnel connections between mainland and offshore islands, or linkages between islands or even continents.

Major Crossings incorporate engineering technology of a scale that is not familiar to the owner and is not typically developed and maintained within the local and regional bridge and tunnel engineering communities.

It is important to bear these characteristics in mind as they often govern the thinking of clients when developing procurement strategies. Key considerations to address include

1. What is the function of the crossing, including to what standards should it be designed and operated?
2. Aesthetics
3. How should it be financed?
4. Risk.
5. Design liability, including proof engineering/independent checking
6. Environment

Some points to note on these are set out below.

Within each of these there will be affected and interested parties with varying degrees of influence who will need to be engaged. The organisation of a project will be governed by these needs, such as the involvement of lenders, insurers, their advisors and government oversight.

Davies(1) developed a number of these themes but since the writing of that paper in 1995 some significant developments have taken place on milestone projects. These are examined further in this paper. Potential promoters of major crossings, and in particular bridges, can be guided by these experiences as they develop their own procurement strategies.

**Project Finance**

This paper does not examine the detail of forms of financing, but a key initial decision of course is to decide whether the projects can be developed entirely from the public purse, not at all, or in a ‘public private partnership (‘PPP’ or ‘P3’), noting that private finance brings with it a natural mechanism by which the public owner can obtain an effective life cycle performance warranty of the work for a stipulated number of years and this consideration may in itself be the primary driver for a P3 rather than any cost of finance considerations.

**Aesthetics**

Aesthetic of major bridges have been treated in various different ways and examples are given below. One way of addressing the appearance of an important structure is to carry out a design competition, and a recent publication(2) is now available which gives valuable advice on how such competitions may be carried out. The planning and environmental clearance process sometimes leads a public owner to dictate closely the details of the appearance under P3 and/or design build arrangements. Such constraints may diminish the value of innovations and savings that may otherwise have been achieved. However, procurement mechanisms are available under which visual quality objectives may be specified and managed under P3 arrangements while still permitting flexibility in structural form.

### Risk

Risk is one of the most important influences on how projects are procured, including commercial, physical and technical risks. Risks associated with major bridge projects in particular were described by Fletcher[(3),] which remains an important reference on the subject. Appropriate management of risks, ie allocating them on the basis of who is best placed to handle them, often pays dividends to clients in avoiding excessive risk premium and reducing overt and hidden construction costs.

### Design Liability & Independent Checking

Independent checking has variously been procured by the client, the contractor (in a Design & Build or P3 contract) and by designers. The question of independence within a contractual relationship is naturally of concern, but promoters need also to balance this with the need for clear lines of accountability for the designs that are prepared for major crossings.

## RETROSPECTIVE

Prior to the mid 1980's a traditional approach was usually taken in planning, designing and building major bridges. As an example the Orwell Bridge in UK (figure 1) was developed and designed on behalf of the UK Department of Transport (DoT) by a firm of consulting engineers engaged in the traditional manner. As a pair of box girders, each one acting as a single continuous beam over the full 1300m length of the bridge, the viaducts were constructed in cast-in-situ post-tensioned concrete and the 190m main span as concrete cast-in-place post-tensioned balanced cantilevers. The construction contract was let under the then standard DoT conditions of contract, with risks such as unforeseen ground conditions being covered under clause 12, and carried by the client. Aesthetics were addressed through the direct engagement by the client of a professional firm of architects, working in close collaboration with the design engineers.

*Figure 1 Orwell Bridge*

At tender phase though, bids from contractors included alternative designs such as single concrete box girder and composite steel-concrete box girders. These were not accepted due to their higher tendered costs, but it did demonstrate the appetite for contractors to work more closely with designers to develop competitive solutions. Rowley[(4)] developed this theme and set out the aims of this form of procurement in more detail. Although it had been a form of procurement common on continental Europe for some time, it was relatively new at the time in the UK, but it heralded much more innovative forms of procurement. In the vanguard of these new developments was the third Dartford crossing.

**Dartford Third Crossing (1991)**

The development of a third crossing at Dartford in the late 1980's to complement the existing two tunnel bores was the first of the major infrastructure projects in the UK to be developed by the Design Build Finance Operate (DBFO) model. The selected solution included a cable-stayed bridge over the River Thames (the Queen Elizabeth II Bridge, figure 2) and steel/concrete composite plate girder approach viaducts. The scheme was developed following an unsolicited bid and the UK government invited bids from joint ventures of contractors working with banks/finance houses. The government requirements were contained in a relatively slim firm requirements document with most risks being carried by the concessionaire (or passed on to sub-contractors, suppliers etc). No aesthetic requirements were placed on the concessionaire, and the independent checker was appointed directly by the concessionaire. The concessionaire assumed the responsibility of taking the project through statutory processes, including an Act of Parliament conferring powers to construct it. Much of the success of the project lay in the fact that the government knew the form of the crossing and how much it would cost before obtaining the statutory approvals. The government had also agreed to pay the concessionaire's costs had the statutory procedures failed. The high volume of traffic using the crossing ensured the success of the project, with payback of the financing achieved within 13 years.

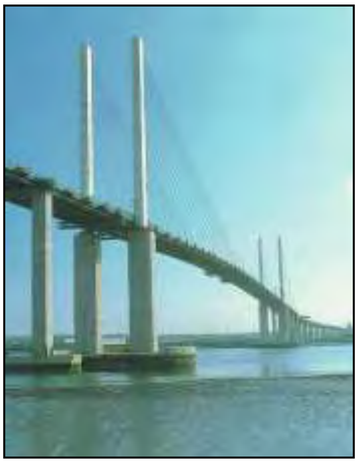

*Figure 2 Dartford Bridge*

Since these early projects, the general DBFO and design-build principles have been developed to take into account project-specific needs and the following case studies pick up on various specific issues that the promoters of each project introduced.

## CASE STUDIES

**Second Severn Crossing (1996)**

Second Severn Crossing (figure 3) rapidly followed on from Dartford Third Crossing and used a very similar form of DBFO procurement**.** The project comprised a 5.2km long estuarial crossing including a 456m main-span cable-stayed bridge and glued segmental precast concrete viaducts. The project was located in a much more environmentally sensitive site than Dartford, and a considerably greater number of studies and investigations were carried out prior to taking the scheme to market[5]. This resulted in a much more comprehensive set of 'Firm Requirements' and related guidance notes. As with Dartford, this included the conventional requirements for use of codes and standards and so forth, but also included specific rules relating to issues such aerodynamics, vessel impact, durability, and environmental restrictions, based on the extensive studies already carried out. Three developments in particular compared with Dartford are worth mentioning:

*Figure 3 Second Severn Crossing*

*Aesthetics* were actively managed within the bidding and delivery process. An architect was engaged directly by the contractor, initially during the tender phase and subsequently during detailed design to work alongside the engineering designers. During the tender design process, presentations were made to the UK's Royal Fine Arts Commission who provided advice to the Department of Transport on the visual acceptability of the proposed structure. Whilst this conferred no contractual obligation on the concessionaire, it did provide some oversight to ensure aesthetics were adequately dealt with. During the detailed design process post-award, the architect continued to maintain an involvement, advising for instance on the suitability of the bridge's signature 'butterfly' upper cross beam, necessitated for structural reasons by the high lateral wind loads on the cables.

An *Illustrative Design* was developed by the government's advisors to demonstrate how the firm requirements could be achieved. These were not contractually binding (and conferred no obligations or liability on the government) but did provide good guidance to tenderers as they developed their options. For instance, the main span of 456m was maintained as being the most appropriate for the topography of the river bed at the main Shoots channel

*Independent checking* was carried out within the contractor's organisation as a sub-consultancy contract to the main design contract. This had the advantage from the point of the view of the government, concessionaire and contractor of putting all the onus of delivering a certified and checked design onto the designer, who was in the best position to manage the design risks.

Again, as with Dartford, the selected bridge arrangement and its cost were known to the government before the statutory procedures were carried out.

**The Øresund Link (2000)**

*Figure 4 Øresund Link*

The Øresund Link (figure 4) marked the development of sophisticated and innovative procurement arrangements promoted by a client with a very clear vision of the end product. The project provides a 15 km long fixed highway and rail link between Denmark and southern Sweden, comprising an immersed tunnel, an artificial island including dredging and reclamation works, and a 7.5km bridge with a 490m main span cable-stayed bridge. The project was promoted through a special purpose joint venture company, Øresundskonsortiet, equally owned by the Danish and Swedish states, which raised the necessary funds on the open market to construct the project.

There are a number of interesting aspects to the way in which the project as a whole was developed and procured. From the outset, the tone of the project was set in the appointment of the in-house consultants, the criteria for whose selection were limited to the technical, aesthetic and environmental aspects of candidates' proposals, which included preliminary design concepts.

The construction of the project was let in a number of detailed design and construct contracts. The details of these are set out elsewhere[6], but a number of aspects were introduced to the bidding process which preserved the client's vision for e.g. aesthetics, whilst managing technical and environmental risk to beneficial effect for both client and contractor. Amongst these:

*Definition drawings* and *Illustrative design.* The principal of an illustrative design reflected the principal established on Second Severn crossing. However, unlike Severn, the preferred aesthetic form of the structures had been defined prior to letting design and construct contracts, so the client introduced definition drawings which defined to the extent necessary the external form of the crossing in order to maintain the aesthetic integrity of the crossing whilst allowing the contractor freedom to determine the detail of the design.

*Reference conditions.* As a means of sharing risk between the client and the contractor, a number of reference conditions were set, which allowed a degree of physical variation, within which margin the contractor takes the risk of variations. Outside these margins, the client picked up the responsibility, the benefit of which was that contractors could sensibly price risk. For example the government carried the residual geotechnical and weather risks

The procurement of the Øresund Link's sister project, the Fehmarnbelt Fixed Link, is currently under development. Of interest here is that, as with many major crossings, the client had not initially decided on a bridge or a tunnel when first embarking on the delivery phase of the project. In order to test properly the viability of each option the client procured separate consultants to develop each concept, in effect creating a competition between them to determine the best solution. As with Øresund, the client will be responsible for financing the project and obtaining the necessary permissions from Denmark and Germany.

**Incheon Bridge (2009)**

Incheon Bridge in Korea (figure 5) comprises a 12 km long sealink with an 800m main span cable-stayed bridge. The crossing was procured on a PPP basis, with a special purpose vehicle (SPV) created to design build finance and operate it, jointly owned by private and government organisations. The project was set up on relatively conventional grounds. Project Performance Requirements (PPR) were set by the government authorities, which were supplemented by the SPV with the Concessionaires Supplementary Requirements (CSR). The selected contractor was responsible for the design and construction but not for providing finance or for obtaining the necessary statutory permissions. The independent checker was engaged directly by the design and build contractor, which kept the design responsibility within that contract, but not directly under the responsibility of the designer.

*Figure 5 Incheon Bridge*

## Goethals Bridge

Opened in 1928, the Goethals Bridge has served as a key link in the New York metropolitan area's transport network, connecting the I-278 and the New Jersey Turnpike to the Staten Island and West Shore Expressways. The ageing structure's deficiencies have however made it functionally and physically obsolete and the client, Port Authority of New York & New Jersey (PANYNJ) are currently in the procurement phase for a new crossing, through a Design Build Finance Maintain method (DBFM) arrangement.

PANYNJ acknowledged the need for an innovative solution to accelerate the development and financing of a new bridge to maximize value to the region and ensure the region's ongoing mobility and economic development. Critical to PANYNJ was that that 'doing it differently' should not come at an unreasonable price (as has arisen on some design competitions) and that the solution should be based on a proven delivery method, adapted as necessary to suit the Authority's circumstances, constraints and project risks.

The project agreement defines the technical specifications obliging the Developer to deliver the replacement bridge and represents a balance between performance requirements (specifying an outcome or service level to be achieved such as ride quality) and prescriptive requirements (mandating a constraint such as a navigational clearance or the use of a particular design code / construction specification).

The client developed its own illustrative design in a similar manner to other projects, but did not dictate the form of the structure. Proposers were therefore able to consider a range of design concepts in preparing their competitive proposals, subject to compliance with the numerous project constraints and other requirements described in the contract. Proposers' initial concepts and interpretations of the specifications and requirements were disclosed to the PANYNJ on a confidential basis during tendering through an Interim Submittal of Proposals ("ISoP") process. The ISoP process is most commonly used in procurements where aesthetics / visual merit are an important feature of a design. The process provides an effective mechanism of conveying a public owner's requirements to Proposers and enables early dialogue, providing confidence that each proposer's design concept interprets the requirements in an acceptable manner. In this respect, the process is a development of the process employed on Second Severn Crossing to ensure acceptable, but economic, treatment of aesthetics.

## Mersey Gateway and New Forth Crossing

Finally, Mersey Gateway Bridge and New Forth Crossing are two major crossings currently being procured or constructed in the United Kingdom, the former as a DBFO and the latter as a design & build contract. On both projects, a competitive dialogue process was entered into with bidders in order that the client could ensure that his requirements were met. In the case of Forth, the structural appearance was very clearly defined prior to inviting DBFO or D & B bids in order to control the aesthetics and to preserve the early development work on the structural form. The statutory approval processes ran in parallel with the competitive D&B procurement process. For Mersey Gateway, the statutory approvals process was also taken to an advanced stage prior to offering the project to the market. Whilst the planning and orders do limit the scope for change, the level of definition was far less prescriptive than on Forth, with opportunities for bidders to add value within certain constraints without threatening the obtained consents.

## CONCLUSIONS

The retrospective and project descriptions in this paper provide examples of how clients are finding solutions to the challenges of ensuring that their requirements are met. These include suitable aesthetic treatment, appropriate allocation of risk and how design liability can be appropriately treated, including independent checking. Key considerations for promoters include:

1. Who should raise the finance;

2. When statutory processes should be carried out;

3. The point at which contractors and their designers should get involved;

4. The appropriate allocation of risk between owner, contractor and other parties;

5. An understanding of the value in different or innovative approaches.

Each new project builds on and refines previous experience and the examples presented give clients who are preparing to procure a major crossing a summary of the current state-of-the-art.

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# DEVELOPMENT OF ANTICORROSION SYSTEM FOR UNDERWATER STEEL STRUCTURES OF LONG-SPAN BRIDGES

**(To ensure soundness for more than 200 years)**

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## ABSTRACT

*Honshu-Shikoku Bridge Expressway Company Limited (HSBE) has been making various efforts to keep the Honshu-Shikoku Bridges (HSBs) in sound condition for more than 200 years. For underwater substructures, the electrodeposition method (ED method) was applied to steel caissons as a basic anticorrosion measure, but it requires time and cost to generate required electrodeposited film. On the other hand, the underwater coating method using "dry box" was developed and applied to splash zone and tidal zone, since ED method is not applicable to these zones. However, the boundary of each coating becomes weak point. Therefore, the cathodic protection method with galvanic anode was applied for the boundary along with above mentioned methods in order to solve this problem. With this additional method, fundamental anticorrosion system for the steel caisson was established.*

*The self-potential of steel caisson was measured one year after the application of this system, and it was confirmed that the potential near the seafloor became protective level only with anodes set in tidal zone. Furthermore, it was also confirmed that the consumption of galvanic anodes, applied to the steel caissons that were coated with paint system at construction stage, was smaller than the assumption, according to the undersea investigation approximately 10 years after completion. From these results, it became clear that the developed anticorrosion system by the combination of ED method and cathodic protection contributed to large cost reduction.*

## INTRODUCTION

Japan consists of four major islands. The Honshu-Shikoku Bridges connect two of them, Honshu and Shikoku, with three routes and17 long-span bridges (Figure 1). The HSBE maintains these bridges with "preventive maintenance" strategy in order to keep them in sound conditions for more than 200 years. The most of the superstructures of the long-span bridges were constructed with steel members with the exception of side spans of some cable-stayed bridges. To keep them in sound condition, various measures were taken to prevent corrosion. Since the superstructures except for cable structures are relatively accessible, heavy duty coating (Figure 2) was applied at the construction stage. Because the surface area in which this coating is applied reaches 4,000,000 $m^2$, it is essential to reduce the Life Cycle Cost of the system. In order to prevent corrosion for a long duration and in an economical manner, only top coat and middle coat are periodically recoated before the consumption of coating layers reaches to under coat which is more vulnerable.

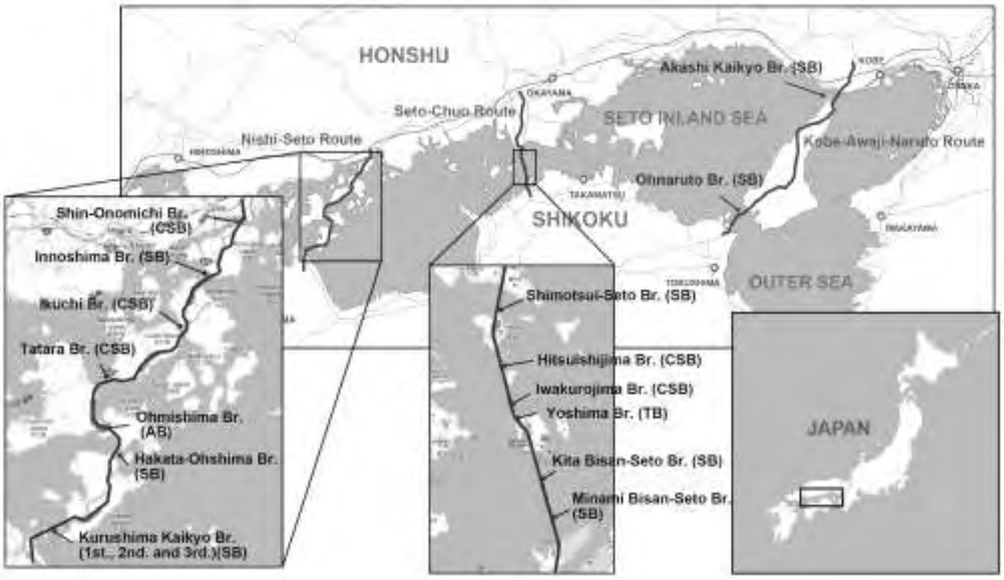



*Figure 1 Honshu-Shikoku Bridges*
(SB: suspension bridge, CSB: cable-stayed bridge, TB: truss bridge, AB: arch bridge)

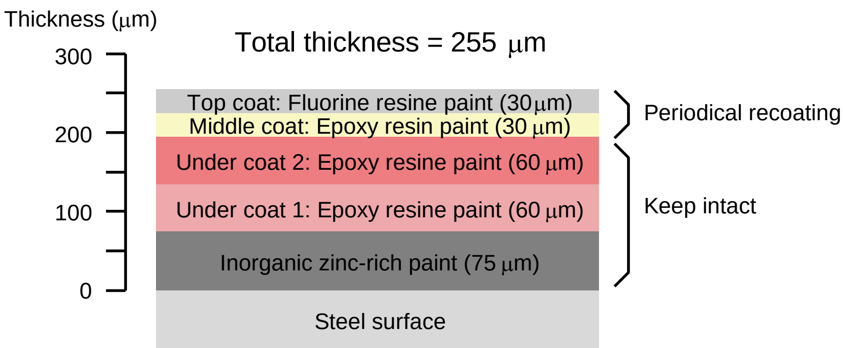



*Figure 2 Heavy-duty coating for Honshu-Shikoku Bridges*

For the main cables of suspension bridges, since they consists of multitude of wires and the inside of them cannot be checked easily, the "Cable Dehumidification System," in which dry air injected into the cables keeps humidity low and protect wires from corrosion, was developed and installed in the actual bridges for the first time in the world [1]. Furthermore, for the suspender ropes of suspension bridges, the "Main Flux Method," in which cross sectional area of the ropes can be quantitatively measured, was developed [2], and control criteria (reduction rate of cross sectional area) were calculated considering actual bridge conditions [3] to keep them in sound state.

On the other hand, for the most of the substructures, steel caissons were used as formworks of concrete foundations. Although role of the steel caissons was considered as formworks for underwater concrete at the construction stage, it was judged that anticorrosion measures had to be taken to keep the foundations in sound condition for more than 200 years and studies had been conducted since then [4]. In this paper, efforts for the development of the anticorrosion measures for these steel caissons by HSBE are presented.

For the Ohnaruto Bridge, steel pipes were used as formworks of multi-column foundations as well. Anticorrosion measure is being installed for the steel pipes since the protection of them was found necessary from the seismic verification result. However, it is not included in this paper.

## CURRENT CONDITION OF STEEL CAISSONS

For foundations of long-span bridges, types of structures are selected depending on conditions of locations where they are set. For the most of the foundations of HSBs, underwater foundations with steel caissons were selected. Years of installation and current anti-corrosion measures for each foundation are summarized in Table 1. Although the role of the steel caissons was initially considered as formworks for underwater concrete which was cast in the seawater at the construction stage, it is advantageous to protect them in order to keep the foundations in sound condition for more than 200 years.

Table 1 Anticorrosion measures for steel caissons (original plan)

| Bridge name | Foundation No. | Anti-corrosion measure (at installation) | | Year and month of installation | Dimensions (m) |
|---|---|---|---|---|---|
| | | Underwater | Splash & tidal zone | | |
| Akashi-Kaikyo *(suspension bridge)* | AB2P | A | E + F | 1989. 3 | ϕ80 × 70 |
| | AB3P | A | F | 1989. 6 | ϕ78 × 67 |
| Hitsuishijima *(cable-stayed bridge)* | HB2P | A + B | B | 1984. 5 | 46 × 25 × 32 |
| | HB3P | A | - | 1983. 9 | 46 × 29 × 30 |
| Iwakurojima *(cable-stayed bridge)* | IB2P | A | - | 1983. 3 | 46 × 18 × 17 |
| | IB3P | A | - | 1983. 8 | 43 × 22 × 26 |
| | IB4P | A | - | 1983. 7 | 32 × 36 × 16 |
| Kita Bisan-Seo & Minami Bisan-Seto *(suspension bridge)* | BB2P | A + B | B | 1982.11 | 57 × 23 × 13 |
| | BB3P | A + B | B | 1981. 2 | 57 × 23 × 13 |
| | BB4A | A | - | 1981.12 | 62 × 57 × 15 |
| | BB5P | A + B | B | 1980.10 | 59 × 27 × 37 |
| | BB6P | A + B | B | 1983. 6 | 59 × 38 × 55 |
| | BB7A | A + B | B | 1982. 3 | 75 × 59 × 55 |
| Tatara *(cable-stayed bridge)* | TB2P | A + C + D | E + F + G | 1993. 9 | 43 × 18 × 45 |
| | TB3P | A + C + D | E + F + G | 1993.12 | 43 × 18 × 45 |
| Kurushima-Kaikyo (1st, 2nd, 3rd) *(suspension bridge)* | KB3P | A + C + D | E + F + G | 1993.10 | 43 × 18 × 45 |
| | KB4A | D | E + F + G | 1992. 2 | 45 × 45 × 34 |
| | KB5P | A + C + D | E + F + G | 1993. 1 | ϕ38 × 39 |

A: Inorganic zinc-rich paint,
B: Chloroprene rubber paint,
C: Tar-epoxy paint,
D: Cathodic protection
E: Organic zinc-rich paint
F: Super thick film type epoxy paint
G: Fluorine resin paint

In 2008, 28 years after the installation of BB7A of the Minami Bisan-Seto Bridge, corrosion survey was conducted. Progress of corrosion was the most severe in the tidal zone and the reduction of the thickness of the steel plate of 10 mm thick was maximum of 3 mm, therefore the corrosion rate is 0.1 mm / year. On the other hand, while the corrosion rate of underwater zone was 0.04 mm / year, local pitting corrosions at which concrete surface under the steel plate was exposed as shown in Figure 3 were observed. If this kind of pitting corrosions are left unrepaired, seawater intruded into the concrete may corrode steel members inside the concrete, expansion pressure of corroded steel members may damage the concrete, and finally, integrity of rigid foundations may be lost. From this scenario, it was judged necessary to protect the steel caissons.

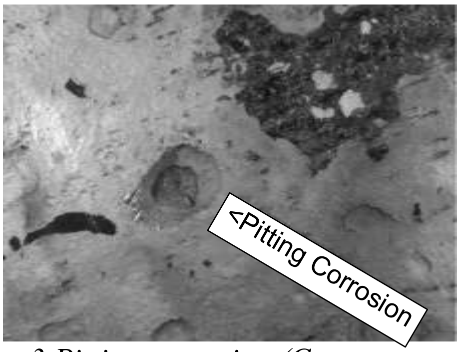



*Figure 3 Pitting corrosion (Concrete exposed)*

## BASIC POLICY OF ANTICORROSION MEASURES

For the steel caissons of HSBs, except for those of the Tatara Bridge and the Kurushima Kaikyo Bridges in which cathodic protection was applied at the construction stage, long-term anticorrosion measures were not taken. As mentioned in the previous section, corrosion of the steel caissons is progressing, and therefore, measures to stop corrosion were studied. As a result of comparison of various existing methods, ED method (Figure 4), in which calcium and magnesium contents in the seawater are deposited on the surface of the steel caissons and form protective layer, was tested in the actual structures because the method does not require diver work and therefore is considered as safe and economical.

After the in-situ tests under various conditions, the following was selected as a basic procedure. That is, firstly remove sea creatures and rust adhered on to the surface of the steel caissons by cleaning machine equipped with water jet (Figure 5), and then form ED film of required thickness (5 mm in average, and minimum of 2 mm) by the facility shown in Figure 6. For the formation of ED film, 1.0 A/m$^2$ was selected as base current density and at the beginning and the end of the application duration, the current density was reduced to form stiff film with more calcium content and enhance adhesion and hardness of the surface. Basic current application pattern is shown in Figure 7. However, after the beginning of real scale applications, it was found that the formation of ED film with design thickness requires about 2 years because of the difference of seawater temperature between summer and winter and tidal force. The method turned out to be costly and time-consuming.

On the other hand, since the splash zone and tidal zone are not always in the seawater, the ED method may not be applicable. For these zones, an anticorrosion measure with coating system which is applicable to wet surface is selected in the HSBs. In order to make long-lasting stable coating, rust on the surface of the steel caissons must be removed completely by blasting. To ensure proper work, “dry box” shown in Fig. 8 is used from surface treatment to top coating.

Furthermore, at the boundary of each method, neither ED film nor abovementioned coating may be applied and it becomes weak point. Therefore, for this boundary, cathodic protection method was applied with aluminum alloy anodes. Thus, problems were addressed in a step-by-step manner, and long-lasting anticorrosion system for steel caissons was established by the application of adequate methods for underwater, splash, and tidal zones and their boundaries.

For shallow foundations, since the ED facility may be excessive and uneconomical, cathodic protection method was selected for under water zone. And for the deep foundations such as 2P and 3P of the Akashi Kaikyo Bridge and 4A, 6P, and 7A of the Minami Bisan-Seto Bridge, because the application is considered to be difficult by the existing ED facilities, safe and economical method has to be studied. Especially for the Akashi Kaikyo Bridge, take into consideration the fact that the foundations are gigantic (diameter of 80 m) and the steel caissons are double-wall structure, necessity of anticorrosion measure is being reevaluated.

Anticorrosion strategy for the steel caissons after the above mentioned considerations is shown in Figure 9.

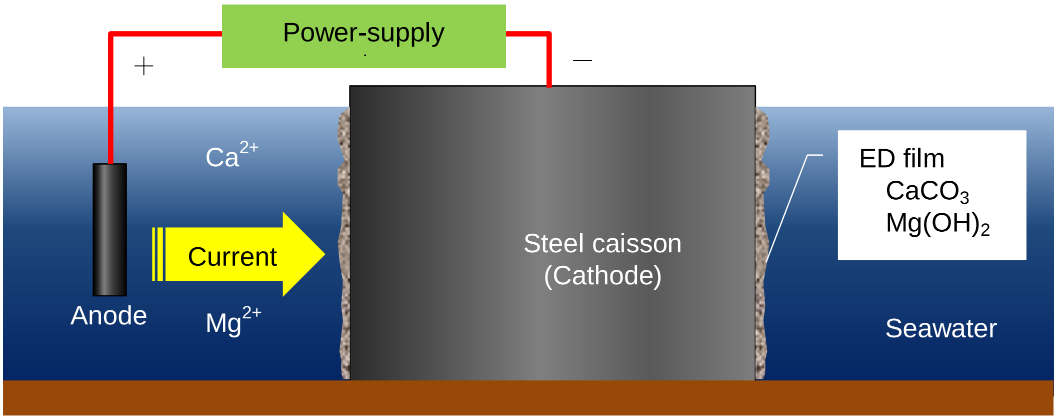



*Figure 4 Concept of ED method*

*Figure 5 Cleaning machine for caisson wall*

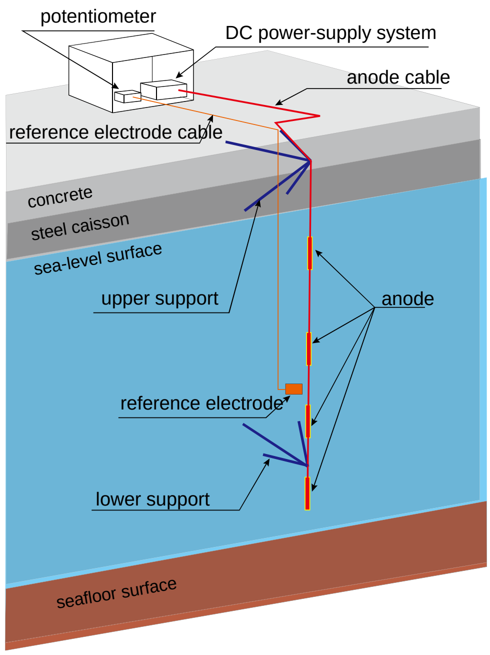



*Figure 6 Basic plan of ED facility*

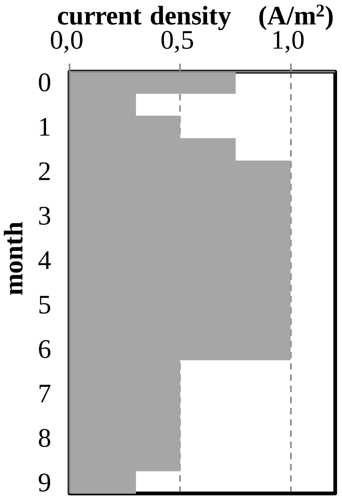



*Figure 7 Basic pattern of current density*

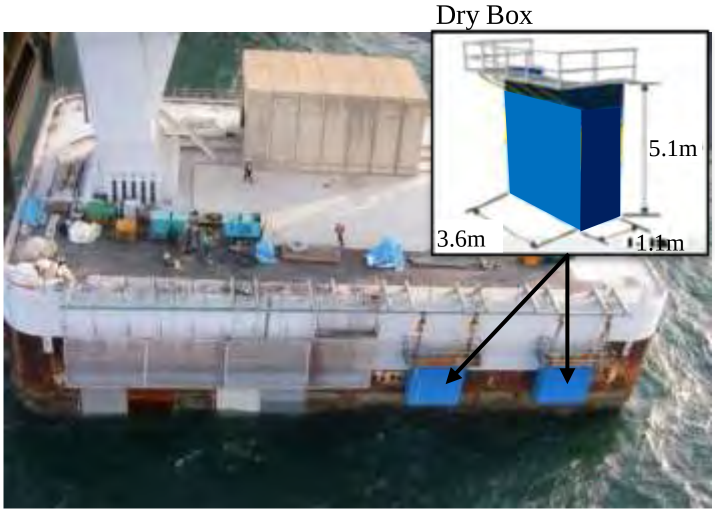
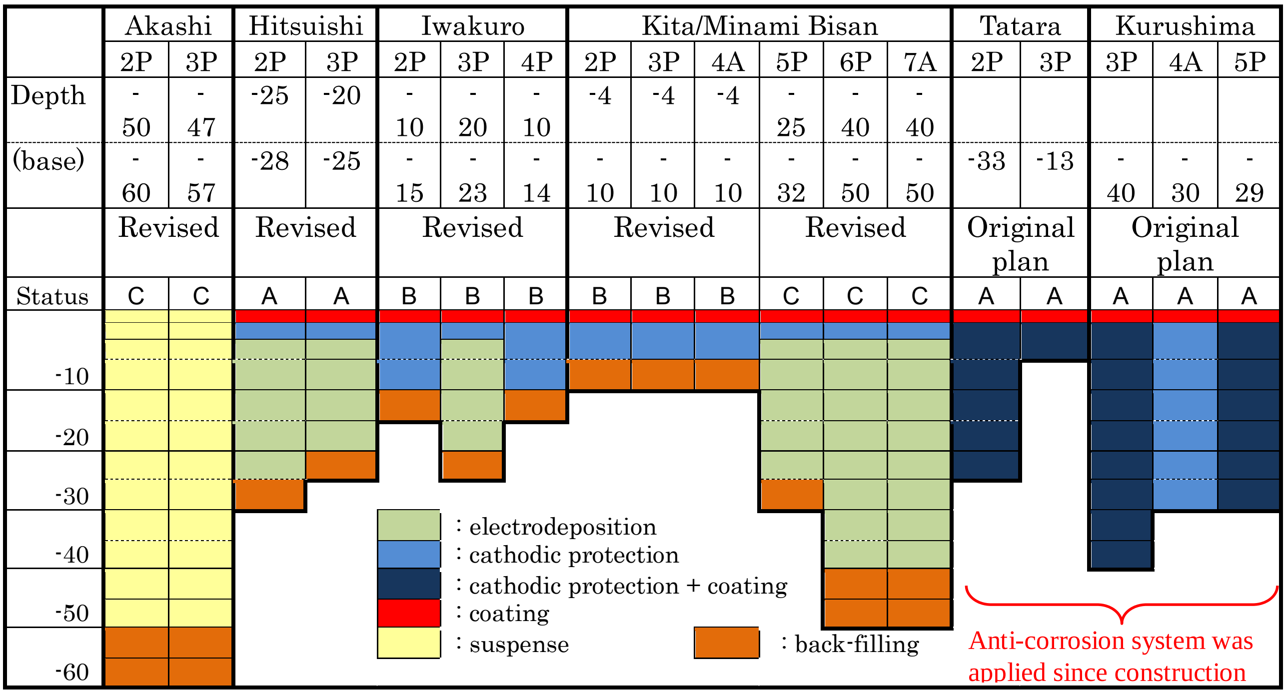



*Figure 8 Coating with use of Dry Box*




Status A : Completed B : Under construction C : Under consideration

*Figure 9 Anticorrosion strategy for the steel caissons of HSBs (Revised)*

## LATEST FINDINGS FROM ACTUAL WORK

Based on the basic anticorrosion strategy mentioned in the previous section, application work has been conducted between 2008 and 2010 for HB3P of the Hitsuishijima Bridge. After the completion of ED application, transition of self-potential was measured. As a result, it was found that the potential reached protective level (-780 mV vs. Ag/AgCl) until the bottom of the caisson by the effect of the sacrificial anodes, which were installed for the protection of the boundary area of ED film and coating, one year after the installation of the anodes (Figure 10). For the horizontal direction, protective potential was kept even at the middle point between two anodes (Figure 11). From these facts, it was confirmed that the anodes set in the tidal zone can protect the whole steel caisson from corrosion.

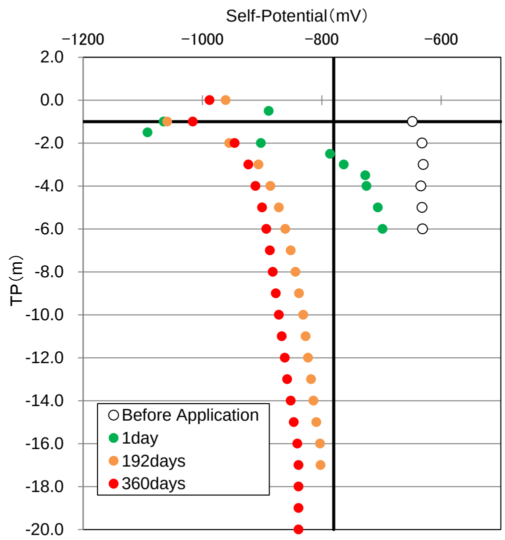



*Figure 10 Distribution of self-potential in vertical direction*
**TP: Tokyo Pail (Tokyo Bay Mean Sea Level)*

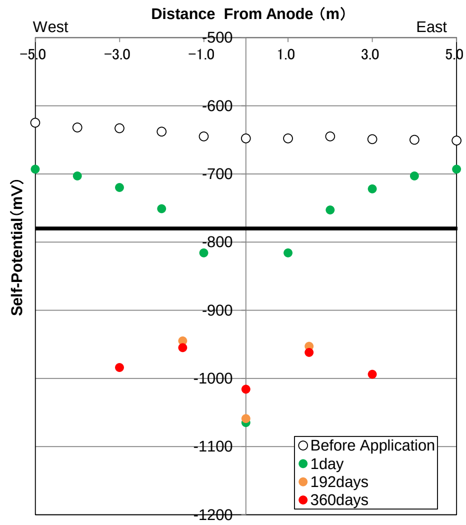



*Figure 11 Distribution of self-potential in horizontal direction*

Additionally, for the case with only sacrificial anodes (without ED layer), extent of area kept in protective potential was measured. As shown in Figure 12, though self-potential near anode had a tendency to improve, the area where potential was lower than the protective potential did not change much. This result implies the fact that current from sacrificial anodes may disperse and has little effect for protection.

On the other hand, consumption of sacrificial anodes set in the foundations of the Kurushima Kaikyo Bridges, at which duration of corrosion protection was expected as 50 years, was measured. By comparison between measured and expected consumption, the actual consumptions were found to be smaller than the expected values as shown in Figure 13. Also, self-potential of the whole caisson surface has reached protective level and it is considered that this stable condition reduces consumption rate of the anodes. Especially, consumption of 3P and 5P which were coated in the dry docks was small. It is assumed that the existence of the coating made whole caissons reach protective potential. Also, even for the 4A that showed the largest consumption, the actual consumption rate was about half of the design value and therefore protective duration of 100 years can be expected.

To understand the abovementioned effect quantitatively, relation between ED film thickness and protective current density was studied with using test pieces (steel plate of 410 x 510 mm). As shown in Figure 14, there is a correlation between required current density and ED film thickness. It was found that when ED film thickness is about 250 µm, current density that is 1/30 of the one required when there is no ED film can protect steel caisson from corrosion. Also, since this result was obtained from the small specimen tests and therefore electric current supplied from anodes might have affected more broader area, less current density may be required in the actual application. To confirm this, test application in broader area is being undertaken with using steel caisson without anticorrosion measures.

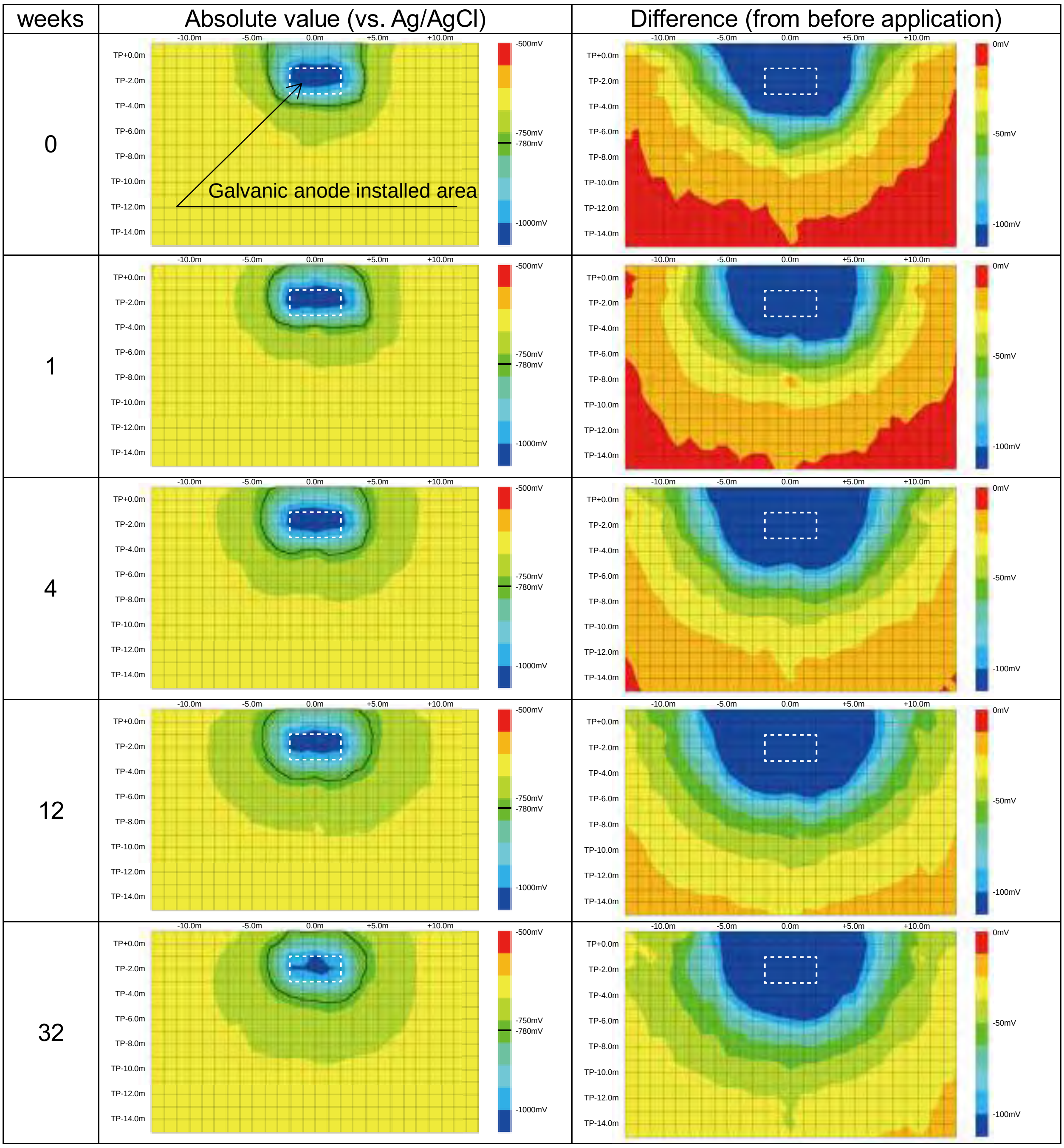



*Figure 12  Transition of distribution of self-potential*

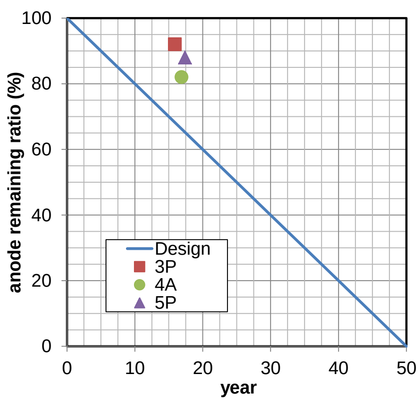



*Figure 13  Consumption of anode*

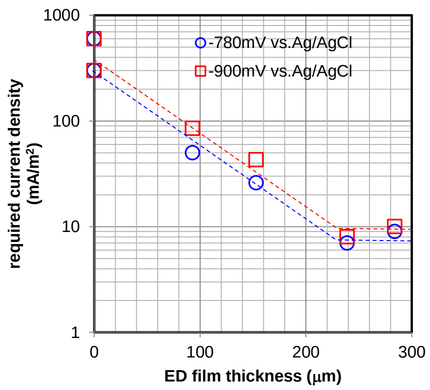



*Figure 14 Relation between thickness of ED film and required current density*

Furthermore, removal of sea creatures by applying strong electric current before ED application was tested. As a result, removal was not observed with the capacity of existing facility. However, it became clear that removal by divers becomes relatively easy after this strong current application. Required time to clean an area of 4 m$^2$ (2 m x 2 m) where strong current was applied was 14 minutes and 50 seconds. Since an area of 2.2 m$^2$ where no current was applied was cleaned in the same duration, improvement ratio of work efficiency by strong current application turned out to be 1.8.

## MORE ECONOMICAL ANTICORROSION SYSTEM

For the steel caissons of HSBs, anticorrosion measures are being undertaken from shallow foundations. For the deep foundations (deeper than 50 m), such as the Akashi Kaikyo Bridge and the Minami Bisan-Seto Bridge, they have the following issues. In order to solve them, various tests are being conducted in-situ.

1) Installation of lower support (Figure 6) is difficult

2) Complete removal of sea creatures and rust by divers is difficult because of broad surface area

3) Formation of ED film is costly and time-cosuming because of broad surface area

Although some issues remains, more economical anticorrosion measure is expected as follows based on the findings in the former sections.

| | |
|---|---|
| [Underwater zone] | ED method with thin ED film (average: 500 μm, minimum: 200 μm) by the facilities shown in Figure 15. |
| [Tidal zone] | Coating with paint applicable to wet surface (Cathodic protection by galvanic anodes to boundary to ED area) |
| [Splash zone] | Coating with paint applicable to wet surface with fluorine resin top coat |
| [Surface cleaning] | Light cleaning after strong electric current application |

Also, since the cathodic protection applied in tidal zone is considered to bring whole caisson to protective potential in the long run, it is expected that the future maintenance can be reduced to anode replacement only.

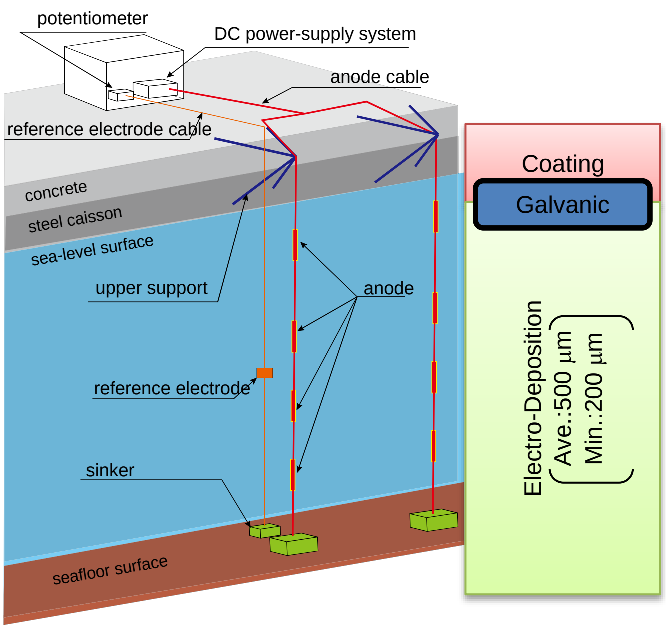



*Figure 15 More economical anticorrosion method for steel caisson*

## CONCLUSIONS

The HSBE is making various efforts to maintain the long span bridges in sound condition for more than 200 years to provide better services for customers to use the expressway safely, securely, and comfortably. In this paper, results obtained through studies of anticorrosion measures for steel caissons of underwater foundations were presented. The outcomes obtained so far are as follows.

1) If there is ED film with thickness of 250 μm, 1/30 of originally required electric current can protect steel caissons.
2) With ED film or coating, sacrificial anodes set in tidal zone may bring whole caisson to protective potential.
3) When whole caisson reaches protective potential, consumption rate of sacrificial anodes will be relaxed.
4) By the application of strong current, surface cleaning becomes easy.

In addition to the abovementioned outcomes, considering safety of diver work and depth of foundations, cathodic protection and ED method with thin ED layer for foundations shallower than 10m and deeper foundations are advantageous respectively. For splash and tidal zones, coating system with paint applicable to wet surface is applied. However, since the boundary becomes weak point, combination with cathodic protection, the main purpose of which is to protect ED film, is effective. It is expected that the results of these studies contribute to the development of more economical anticorrosion measures for underwater steel structures.

## ACKNOWLEDGEMENT

These studies had been conducted for a long duration and many colleagues other than authors have contributed. Also, for the actual tests and applications, assistance of Bridge Engineering Company Limited and IHI Corporation was indispensable. We extend special thanks to them.

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# PUSHING THE LIMITS OF CABLE STAYED BRIDGES – THE PARTIALLY EARTH ANCHORED SOLUTION

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## ABSTRACT

*Cable stayed bridges have seen a significant increase in span length over the last four decades, with the current world record Russky Bridge in Vladivostok, Russia spanning 1104m. Design and construction technologies for cable stayed bridges have been developed to the point where for spans up to around 1200m they can be competitive compared to suspension bridges. For bridges with a span requirement greater than this, a suspension bridge is likely to provide a more economical solution. A traditional cable stayed bridge will generally become uncompetitive for such spans as the very high compressive force in the deck governs the design.*

*Research has been carried out to investigate the possibilities of designing and constructing cable stayed bridges which can be competitive with spans significantly longer than 1200m. A partially earth anchored system is proposed which limits the axial compressive load in the deck and thereby significantly reduces the quantity of material required. In order to avoid the cantilever construction stage becoming a governing case, an innovative tie cable technology is proposed.*

*This paper presents a design for a cable stayed bridge with a main span of 1400m, which permits a 30% reduction in the steel deck quantities compared to a self-anchored cable stayed bridge built by traditional cantilever technology. The saving makes this bridge form competitive with a suspension bridge of the same span. The paper also show how the tension force generated in the deck of a partially earth anchored cable stay bridge has a beneficial geometrically non-linear effect in the global system of the bridge. Practical means of providing sufficient damping for stay cables with lengths in excess of 700m are also discussed.*

**Keywords:** cable stayed bridge; aerodynamics; long span; stay cable damping

## 1. INTRODUCTION

There are sites around the world where the new bridges required to improve existing transport networks call for very long spans. Often these are in very exposed environments. Such schemes require significant investments both in the construction phase and also during operation in terms of inspection and maintenance. Although suspension bridges are proven technology for such long spans, cable stayed bridges can offer significant benefits in terms of robustness (multiple load paths) and ease of maintenance.

## 2. KEY ISSUES

Cable stayed bridges with spans greater than 1200m will only be built in locations where such spans are needed, if a cable stayed bridge of such a span is technically feasible, and if a cable stayed span of that size is competitive against a suspension bridge. Gimsing [1] identified three important aspects of cable stay bridges where they will be more sensitive to span increase than suspension bridges:

- The compressive stresses in the deck due to gravity and wind increase with the span length and require substantial strengthening of the deck
- During the cantilever erection, the cable-stayed main span depends entirely on the strength and stiffness of the deck for lateral stability whereas for a suspension bridge the construction stages benefit from the pendulum effect
- The free length of the longest stay cables will be several times that of the longest hanger cables in a suspension bridge which means that problems related to individual cable vibration will be more severe

Solutions have been investigated to tackle these challenges.

The transmission of the horizontal components of the stay cable forces will be one of the decisive factors for the competitiveness of the cable stayed bridges if considering span lengths beyond the present limits. With a traditional cable stayed bridge and existing economically viable grades of steel, the plate thicknesses in the deck start to become impractical from a fabrication point of view beyond a span of around 1200m. It is, therefore, interesting to consider whether modifications of the structural system could reduce the critical compression in the deck of a cable-stayed bridge.

## 3. PARTIALLY EARTH ANCHORED CABLE STAYED BRIDGE

In a partially earth anchored system, some of the stay cables are fixed to anchor blocks at the ends of the bridge and therefore transfer forces to the ground, similar to a suspension bridge anchorage.

### 3.1 General Arrangement

A specimen design of a partially earth anchored cable stayed bridge type with a main span of 1400m has been carried out to provide a practical solution to the challenges. The span arrangement and typical deck section are shown in Figures 1 and 2.

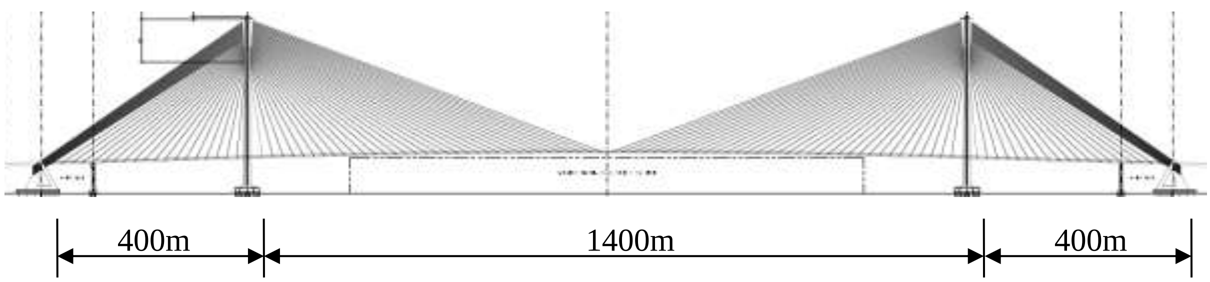


*Fig. 1: Generally arrangement of the partially earth anchored cable stayed bridge*

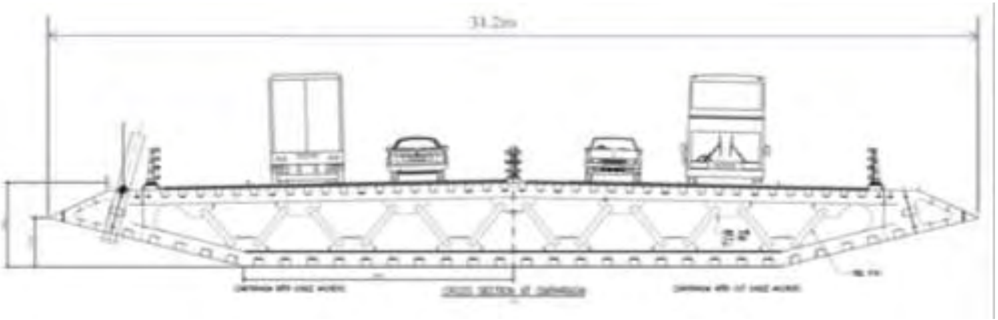

*Fig. 2: Typical deck section*

The main span is 1400m and the back spans are 400m each with an intermediate pier at 100m from the deck end. The deck is a single steel box girder carrying 2 lanes of traffic and a hard shoulder in each direction. The specimen design adopted the ground conditions, typhoon wind climate and moderate seismicity of the Incheon Bridge site in South Korea.

### 3.2 Axial Compression in Deck

The effect of anchoring some of the cables to the ground is to reduce the maximum compression in the deck due to the horizontal components of the stay cable tensions. Horizontal equilibrium of the deck is achieved not only by a compressive force at the towers but also by a tensile force at mid-span (Figure 3).

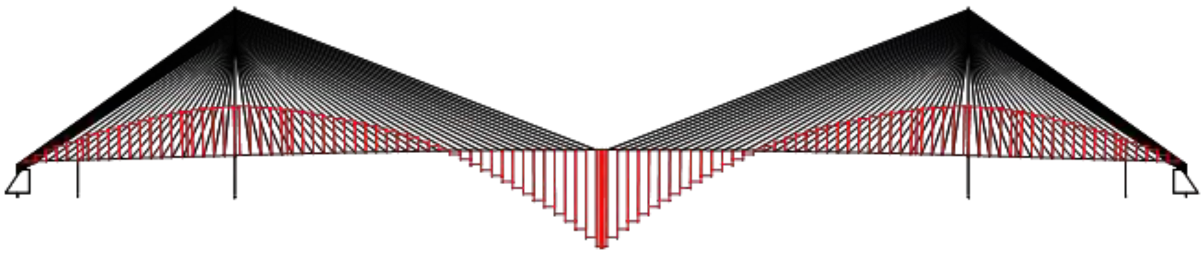

*Fig. 3: Axial force in deck of a partially earth anchored cable stayed bridge*

Compared to an earth-anchored suspension bridge of a similar scale, a partially earth anchored cable stayed bridge will have considerably smaller anchorages and a much more economical cable system. Furthermore, the erection of the self-anchored part of the deck can proceed in parallel with the construction of the anchorages which leads to potential a potentially shorter construction programme.

## 4. AERODYNAMIC EFFECTS

Wind buffeting analyses have shown that the tensile force in the mid-span has a beneficial P-delta effect which reduces the overall bending moments and displacements in the deck due to wind. This is analogous to the pendulum effect in a suspension bridge where the main cable tension has a restoring effect. The static (linear) and non-linear analysis results are compared in Figure 4.

Critical design stages include the completed bridge in-service condition and the construction stages with long deck cantilevers. The maximum cantilever erection stage is critical in terms of the aerodynamic effects including (a) wind buffeting forces; (b) aerodynamic stability (galloping and flutter) and (c) aerodynamic serviceability (vortex shedding and low wind speed buffeting).

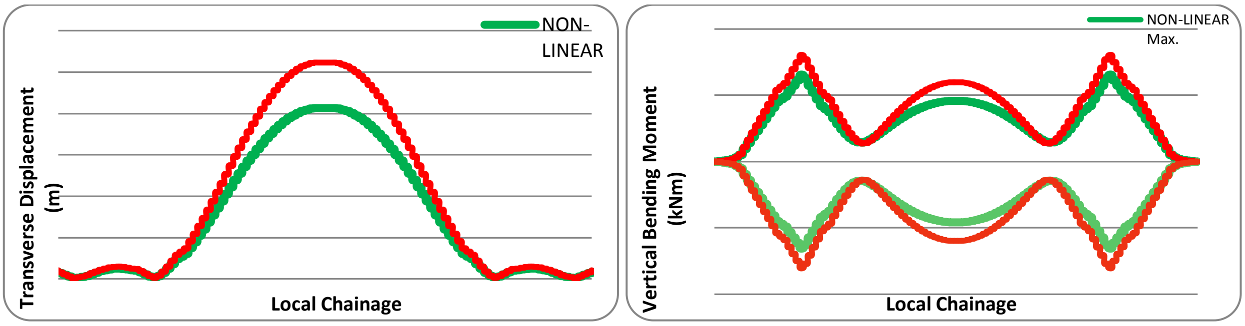


*Fig. 4: Displacement & Moment under Wind Buffeting Load Case*

To check the criticality of the construction stages, models with 1400m, 1600m and 1800m main spans were analysed and comparisons made between the in-service condition and the maximum cantilever stages. This study confirmed that the effects of high wind speed buffeting forces are a major design issue for the cantilever stage but since the in-service forces are similar for a traditionally self-anchored cable stayed bridge this does not control technical feasibility.

However, management of low wind speed vibrations to minimise loss of productivity is a major factor which must be considered. Vibrations could cause both safety problems with personnel and equipment working on the deck as well as cause difficulties with erection geometry control. To reduce the occurrence of vibrations with respect to vortex shedding, high Reynolds number sectional model wind tunnel tests would be required to optimise the cross section shape and gain confidence in the performance. Mitigation measures such as guide vanes or tuned mass dampers can be employed if necessary. Advanced technological solutions to erection geometry control based on dynamic survey measurements could reduce the impact of vibrations.

## 5. CONSTRUCTION USING TIE-CABLE SYSTEM

### 5.1 System Concept

A tie-cable construction system has been developed which introduces and maintains a mid-span tension during construction of a partially earth anchored cable stay bridge. The design objectives that led to the development of the system were:

- To find a practical and economical means to construct this form of bridge
- To reduce low-wind speed vibrations of the bridge during erection

The use of tie-cable system is illustrated conceptually in Figure 5.

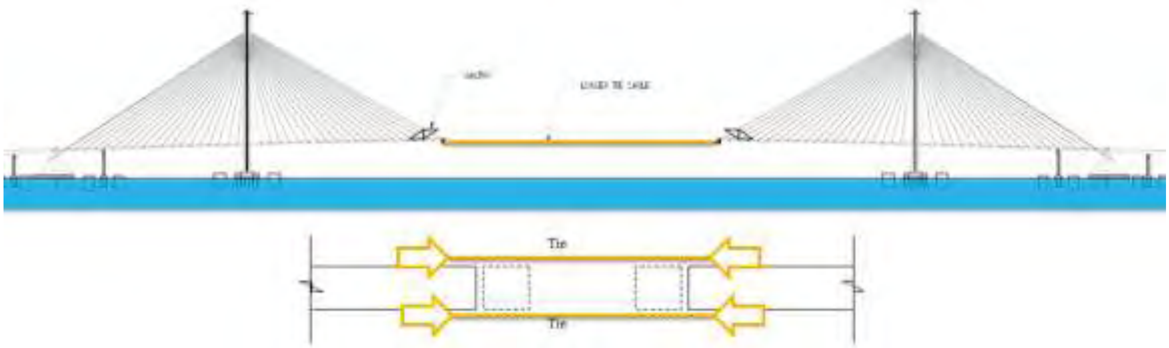


*Fig. 5: Tie-cable method for construction of partially earth anchored cable stayed bridge*

The construction concept can be defined as follows:

- A temporary longitudinal restraint connection is made between the deck and the tower
- The deck is erected by the balanced cantilever method in the traditional manner for a cable stayed bridge. The erection proceeds until the length of the cantilever is such that the axial deck forces due to wind and gravity would exceed the in-service forces.
- At this and certain subsequent stages during the erection, horizontal tie cables are introduced to connect the deck cantilevers together with a beneficial tension force
- Deck erection continues by the cantilever method using traditional erection equipment
- After mid-span closure the temporary deck / tower connection is released and the temporary tie cables are de-tensioned and removed

The temporary connection between the deck and tower (by means of additional longitudinal restraint) is necessary to resist the unbalanced axial force in deck due to stay cable stressing, tie-cable stressing and wind buffeting effects during construction.

### 5.2 Erection Sequence

Selected erection stages showing the change of axial forces in deck are shown in Figures 6 to 9 to demonstrate the effect of the tie-cable system.

The tie cables are extremely effective in achieving the design objectives set out above and offer a number of significant system benefits during construction. In particular:

- The axial compression in the deck is reduced
- The bending moments in the deck due to wind during cantilevering are reduced
- The deflections of the bridge due to wind during cantilevering are reduced

Apart from the introduction of the innovative tie-cable technology the main erection procedures for the deck follow tried and tested existing technology.

*Fig. 6: Erection stage - install tie cable and stress against towers*

*Fig. 7: Erection stage – install second tie, continue cantilevering and controlling tie forces*

*Fig. 8: Erection stage – Deck closure*

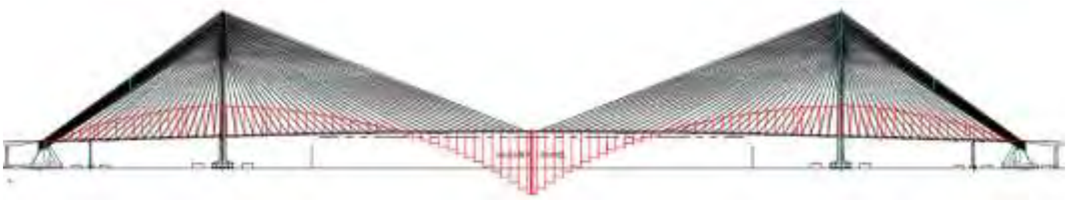

*Fig. 9: Erection stage – Remove tie and transfer tension to deck*

### 5.3 Comparison with traditional cantilevering

The use of tie-cables reduces the deck compression force during the critical construction stages. Comparisons have also been made between lateral displacements at the cantilever tips under wind loads with and without the tie-cable as shown in Table 1. The lateral displacements are significantly reduced by the use of tie-cables which provide a beneficial restoring force. A reduction in low speed wind vibrations can also be expected with the consequent potential positive impact on construction productivity.

*Table 1: Comparison of lateral displacement at cantilever tip*

| Lateral Displacement at Cantilever Tip under | Without Tie Cables (m) | With Tie Cables (m) |
|---|---|---|
| Mean Wind | 6.4 | 3.0 |
| Buffeting Wind | 6.3 | 3.6 |
| Total | 12.8 | 6.6 |

### 5.4 Steel Quantities

The estimated total steel quantities for the cases with and without tie cables are compared in the figure 10. Considering the in-service condition only, the total steel quantity required is about 29,700 tonnes (460 MPa yield strength). However, if the deck were built by cantilever construction without the tie cable system the steel quantity needs to be increased to 41,700 tonnes to overcome temporary construction stages. With the tie-cable construction method, the total steel quantity is only slightly increased to 30,600 tonnes.

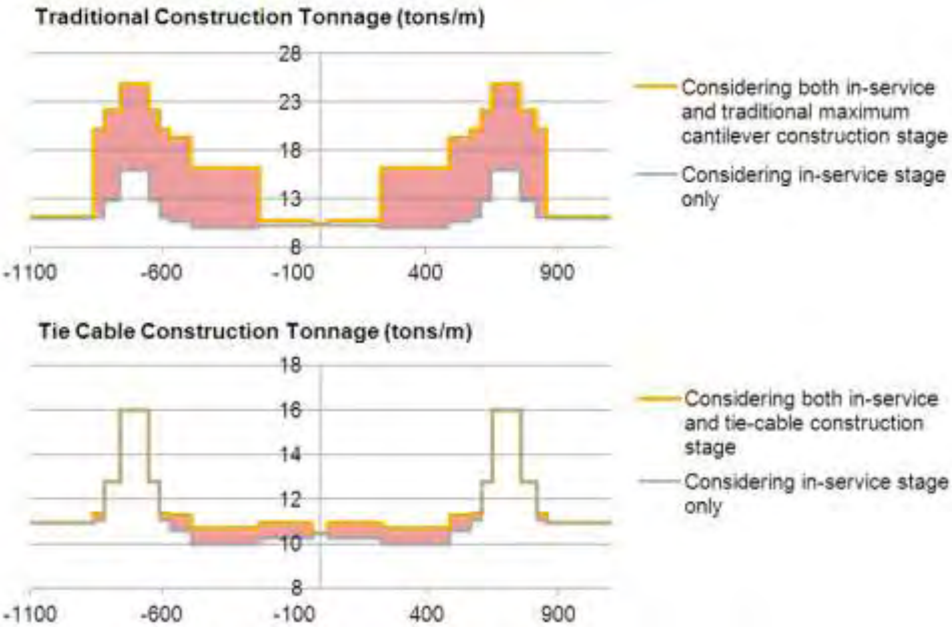


*Fig.10: Estimated steel tonnage*

# 6. STAY CABLE VIBRATION

For a 1400m main span, the longest stay cable would be about 740m. Cable vibration is key aerodynamic issue that must be addressed.

For the control of rain-wind induced vibration, all modern stay cable systems incorporate surface treatment which is typically either helical fillets or dimples to disrupt the formation of rivulets. Wind tunnel testing has shown that these are effective without excessive increase in the drag coefficient.

Even with a textured surface, a damping ratio of approximately $\delta = 4\%$ (logarithmic decrement) is still required to control vibrations. Such damping levels have been achieved on major cable stayed bridges. However, as the cable becomes longer it becomes harder to achieve the same level of damping. Arranging dampers with a sufficient ratio $x_c/L$ (where $x_c$ is the distance from the anchorage to the damper and L is the length of the cable) becomes more difficult as they would need to be a considerable height above the deck. For a given ratio $x_c/L$ there is a maximum level of damping achievable (optimal damping) which has been well established through numerical calculation and calibrated by results taken from real bridges. Attempting to add more damping at that same location will be counterproductive since it starts to act like a fixed point and is less effective at damping vibrations.

The problem of damping very long cables can be investigated by determining what value of $x_c/L$ is required to achieve a certain target level of damping. Calculations followed the formulae presented in [2].

If only one damper is provided then the required value of $x_c/L$ to achieve 4% log-dec. damping progressively increases from 0.026 at a span of 1000m to 0.054 for a 1800m span (Figure 11). As the cable length is also increasing this implies a very dramatic increase in $x_c$ from 14m at a 1000m span to 51m at 1800m span. The vertical distance between anchor and damper is 18m at the longer span which would present significant practical constraints for a traditional external damper.

It is interesting to note that although the required position of the damper at longer cable lengths is further from the anchorage, the optimal damper size is actually reducing. This means that there should be no concern over supply of the damper units.

For bridges with a span length significantly greater than 1000m the target level of damping can be achieved but traditional solutions with a single external damper fixed to the deck are unlikely to be practical, economical or aesthetically pleasing.

## 6.1 Additional Structure for Damper at Tower Side

An alternative solution would be to have dampers at both ends of the cable to avoid very large secondary structural elements to support the dampers at deck level. An investigation was made of the requirements if two sets of dampers were used – one near the deck end of the cable and one near the tower end. For simplicity it was assumed that the dampers are at equal distances from the cable ends. The required distance $x_c/L$ is approximately halved but interestingly the size of the required damper is approximately doubled.

For example, if only one damper at deck end is adopted for the longest cable of the 1400m main span bridge this damper needs to be installed at 27m away from the anchor point to achieve the required damping. Depending on the arrangement of the stay cable anchorage, this would require the damper to be at least 8m above deck level which would require a significant supporting structure taking up considerable room on the deck.

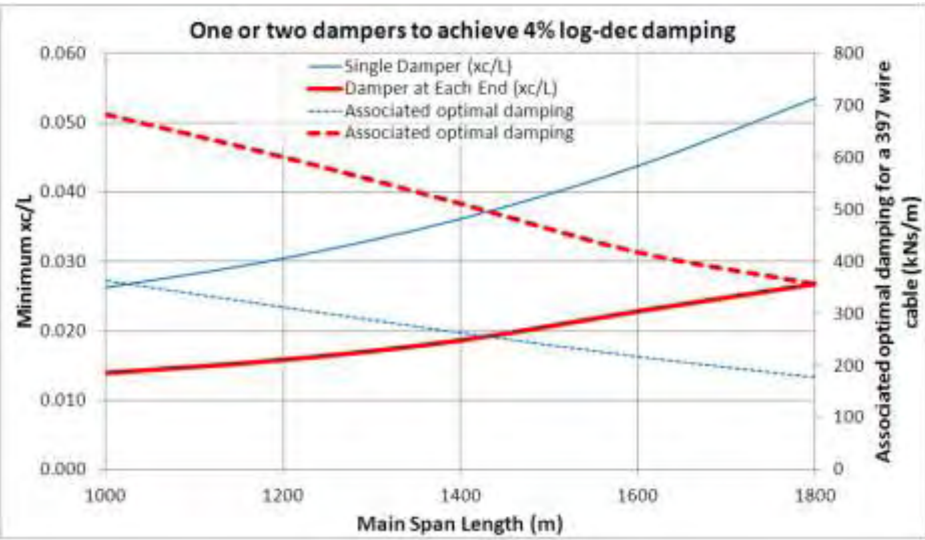


*Fig.11: One or two dampers to achieve 4% log-dec. damping*

To avoid the use of such a large structural element on deck, a second damper at tower end is proposed for cables which are longer than 500m. Provision of a damper at the tower is likely to require additional structure to achieve the necessary $x_c/L$. A solution for a 1400m span bridge is sketched in Figure 12 to illustrate the size of the additional tower top structure. An enclosed structure is introduced which allows the dampers to be located at the necessary distance from the tower anchorages. The structure encloses a series of access platforms and ladders to facilitate close inspection of the dampers. The structure is flared towards the top to provide an increasing distance between damper and anchorage for the longest cables.

*Fig.12: Additional structure at tower top to house the stay cable dampers*

The size of the additional structure is not excessive and it is more easily managed than providing additional structure at deck level because:

- The cable anchorages are close together so a single structure at upper tower can cover all of the longer cables

- Structure at deck level is close to the drivers and appears massive whereas at the top of the tower it appears more in proportion to the overall scale of the bridge.

Nevertheless, the additional structure for the tower side anchorage has an undoubted visual impact and it should be dealt with architecturally to become a deliberate aesthetic expression of the bridge which will be one of the defining features that illustrate the technical challenges the bridge has overcome.

## 7. CONCLUSIONS

Significant technical issues arise in the design and construction of ultra-long span cable stayed bridges. To date there are no cable stayed bridges with spans of 1200m or above. Although it may be technically feasible to construct a traditional self-anchored cable stayed bridge with such a span, as the span length increases the economic competitiveness compared to a suspension bridge is severely compromised by the quantity of steel in the deck necessary to resist the combined effects of wind and gravity.

A partially earth anchored cable stay bridge neatly solves this problem and leads to cable stayed bridges having the potential to be the most economical solution for a greater range of span lengths. However, unless there is a practical means to build this type of bridge the potential will not be realised. An innovative tie cable technology has been developed and studied with a specimen design of a 1400m main span partially earth anchored cable stayed bridge which makes the construction feasible and allows a 30% reduction in the steel deck quantities compared to a bridge built by traditional cantilever technology. At the same time the tie-cable reduces low wind speed vibrations which will have an important effect on reducing risks of downtime during cantilever erection.

A practical means of damping stay cables with lengths in excess of 700m has also been proposed by providing additional structure at the towers to house a second set of dampers. This solution is significantly more compact and likely to be more cost effective than extending existing technology for external dampers mounted above deck level.

With these solutions the technology is available for a dramatic increase in the maximum cable stayed span beyond today’s 1104m world record.

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# GEOTECHNICAL DESIGN EVALUATIONS FOR THE AEROELASTIC STABILITY OF CABLE STAYED BRIDGES

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## ABSTRACT

*Limiting the Tower lateral Stiffness in Cable stayed Bridges is extremely important to enhance the critical velocity for Stall Flutter condition. In this paper is examined a parametric Study from a geotechnical point of view to get the maximum limitation of the Pier lateral Stiffness in order to check how far is possible to avoid any Tower Foundation effect on Stall Flutter behavior.*

**Keywords**
Aeroelastic Stability, Cable stayed Bridges, Rocking Vibrations, Stall Flutter.

## INTRODUCTION

From the theory of long span bridges we know that critical stall flutter velocity is, (see, for example, [1]):

$$V_{0\,cr(St)} \cong 0.40\,\omega_{0\vartheta} c \tag{1}$$

where $\omega_{0\theta}$ is the fundamental torsional frequency and c is the greatest deck width. From this equation we can argue that as suspended bridges have a minor value of the fundamental torsional frequency than cable stayed bridges and so a minor critical stall flutter velocity, for equal span and deck section , in the same way focusing on cable stayed bridges a greater value of the fundamental torsional frequency can be obtained by enhancing the Pier foundation rocking stiffness.

In this paper is presented, for a fixed average cable stayed bridge spanning 350 m, in a parametric form, the variation of the Tower lateral stiffness deduced from the rocking Stiffness in dependence of the radius $r_0$ and the embedment h of the Pier foundation, for various lateral Soil shear Modulus and different total heights of the Towers.

The typical cable stayed Bridge Tower Foundation may be made by (see [2]) traditional driven piles, deeply embedded caissons or soil improvement so in this study we will focus on a base founded on improved soil or on a caisson in order to consider for the parametric evaluations the typical formulas used in vibrating machine foundations. The piled foundation will typically results a more conservative case.

## THE PARAMETRIC STUDY

### Cable stayed Bridges analytical knowledge

From the dynamic analysis of cable stayed bridges we have the following equation for the fundamental period in vertical symmetric vibrations:

$$T_{0S} =$$

$$2\pi\left\{\left[\left(\frac{L}{2H}\right)^3+\left(\frac{l}{H}\right)^3\right]\left(\mu H^2\right)\left\{\frac{1}{3\left\{\frac{E\,g}{2\sigma_g}l\frac{\left[\left(\frac{L}{2l}\right)^2-1\right]}{\left[1+a^2\left(\frac{l}{H}\right)^2\right]\left[1+\left(\frac{l}{H}\right)^2\right]}\left(\frac{l}{H}\right)^2\right\}}\right\}\left\{\frac{\left\{\frac{E\,g}{2\sigma_g}l\frac{\left[\left(\frac{L}{2l}\right)^2-1\right]}{\left[1+a^2\left(\frac{l}{H}\right)^2\right]\left[1+\left(\frac{l}{H}\right)^2\right]}\left(\frac{l}{H}\right)^2\right\}}{\left\{\frac{E\,g}{2\sigma_g}l\frac{\left[\left(\frac{L}{2l}\right)^2-1\right]}{\left[1+a^2\left(\frac{l}{H}\right)^2\right]\left[1+\left(\frac{l}{H}\right)^2\right]}\left(\frac{l}{H}\right)^2\right\}+KH}\right\}\right\}^{\frac{1}{2}} \quad (2)$$

where L is the main span, l are the lateral spans, μ is the deck mass per unit length, E is the steel elastic modulus, g are the uniformly distributed dead loads, $\sigma_g$ is the cable tension, K is the Tower Head flexural Stiffness, and a is given from equation (3):

$$a = \frac{\gamma^2 E H^2}{12{\sigma_g}^3} \quad (3)$$

where γ is the steel specific weight.

The value of $T_{0S}$ is linked the torsional vibrations natural period by equation (4):

$$\left(\frac{T_{0\varphi}}{T_{0S}}\right)^2 = \frac{I_0}{b^2\mu} \quad (4)$$

where $I_0$ is the deck section polar Moment of Inertia, and b the distance between two cable curtains typically considered to be 0.85 c. So we can calculate $\omega_{0\varphi}$ by equation (5):

$$\omega_{0\varphi} = \frac{2\,\pi}{T_{0\varphi}} \quad (5)$$

and finally we can figure out $V_{0cr(St)}$ from equation (1).

**Parametric Evaluations**

From equation (2) we note that that Stiffness K is fundamental to stabilize the deck against stall flutter condition as a great value of K entails a minor value of $T_{0S}$ and so a greater $\omega_{0\varphi}$ value.
The K value can be calculated in normal case from the rocking stiffness for an embedded foundation as reported in [3]:

$$k_r = G{r_0}^3\left\{2.5 + \frac{G_S}{G}\left(\frac{h}{h_0}\right)\left(2.5 + \frac{h^2}{3{r_0}^2}\,4.05\right)\right\} \quad (6)$$

where $r_0$ is the foundation base radius, h is the foundation embedment depth, G is the bedrock shear modulus and $G_S$ is the lateral soil shear modulus.
Hence the flexural Stiffness will be given by:

$$K = \frac{k_r}{{H_{tot}}^2} \quad (7)$$

Where $H_{tot}$ is the total height of the Tower, including the water depth.

So now we will focus on a typical steel deck with central bridge span of 350 m as reported in [1] , by varying $r_0$, h and $H_{tot}$ with $I_0$ ,calculated as Torsion Factor, is given by 2.2867 $m^4$ , G=38.46 Mpa , c= 33 m, μ/g = 0.1, g =1.2 MN/m, which gives $T_{0S}(K=0)$ =2.75 s, so getting the results plotted in the Figures from 1 to 9, paying attention to the fact that a maximum wind velocity can be quantified in 250 Km/h, that in this case from equations (1), (2),necessitates K = 1370 MN/m.

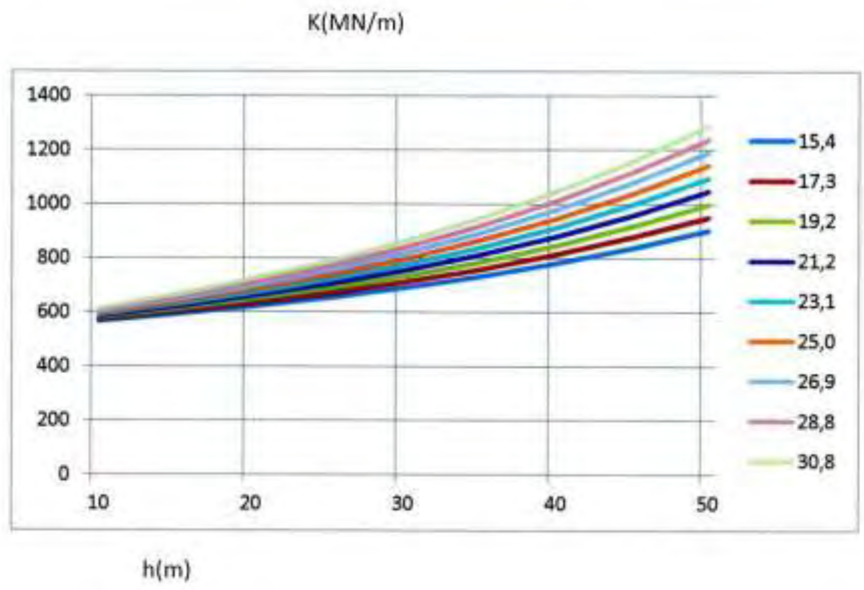


*FIG. 1: Flexural Stiffness K (h) for various lateral shear Modulus $G_S$(MPa) – Case $H_{tot}$ = 130 m ; $r_0$ = 45 m.*

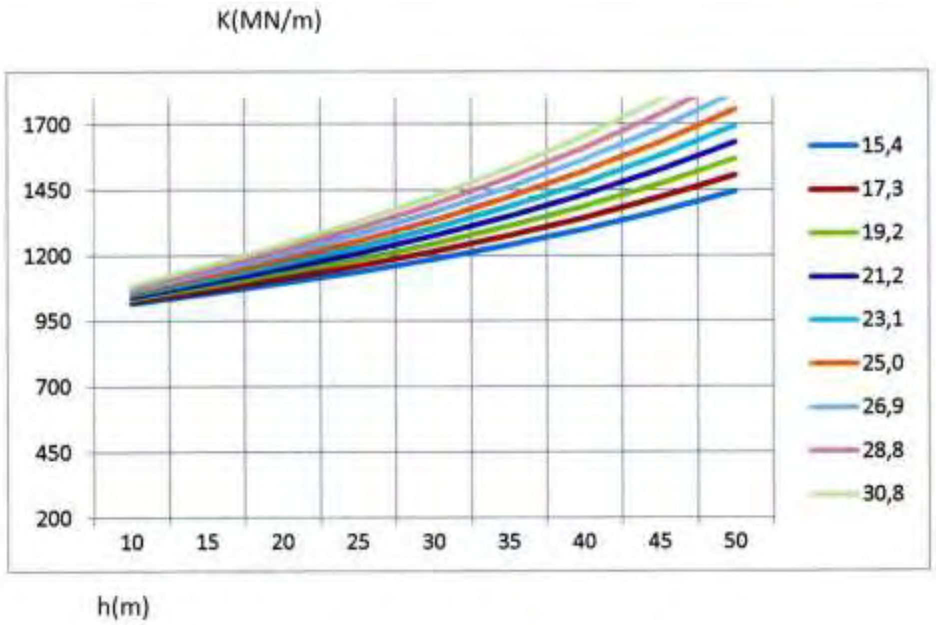


*FIG. 2: Flexural Stiffness K (h) for various lateral shear Modulus $G_S$(MPa) – Case $H_{tot}$ = 130 m ; $r_0$ = 55 m.*

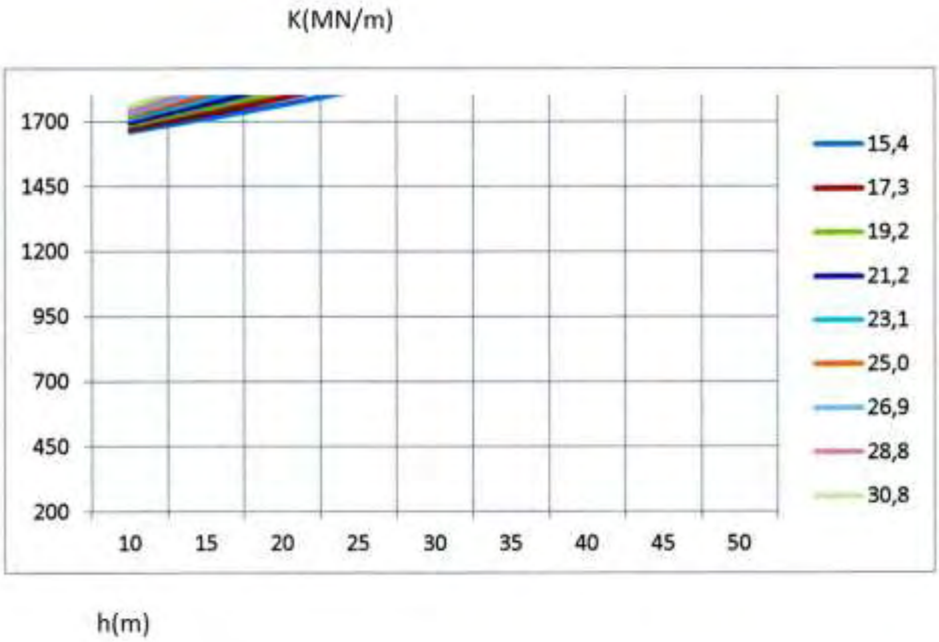


*FIG. 3: Flexural Stiffness K (h) for various lateral shear Modulus $G_S$(MPa) – Case $H_{tot}$ = 130 m ; $r_0$ = 65 m.*

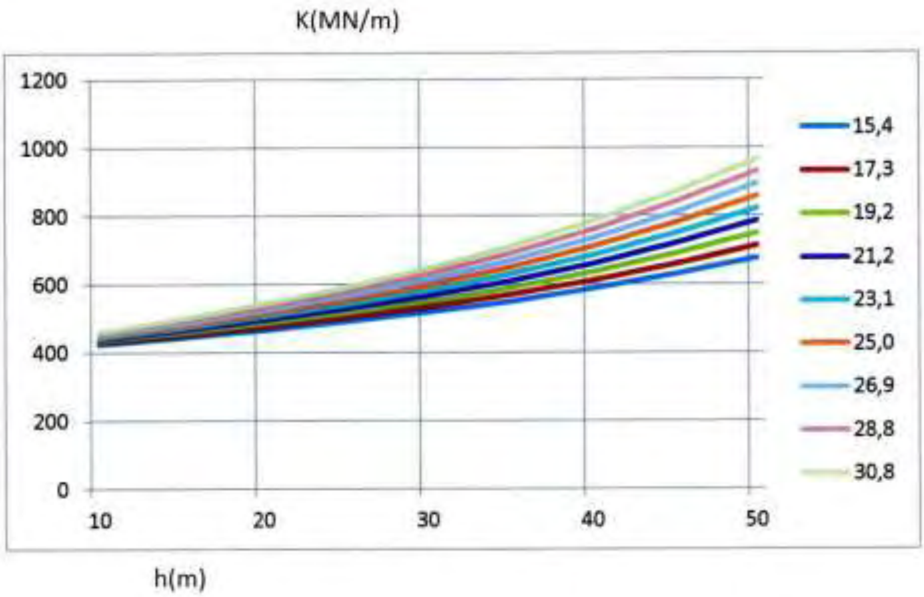


*FIG. 4: Flexural Stiffness K (h) for various lateral shear Modulus $G_S$(MPa) – Case $H_{tot}$ = 150 m ; $r_0$ = 45 m.*

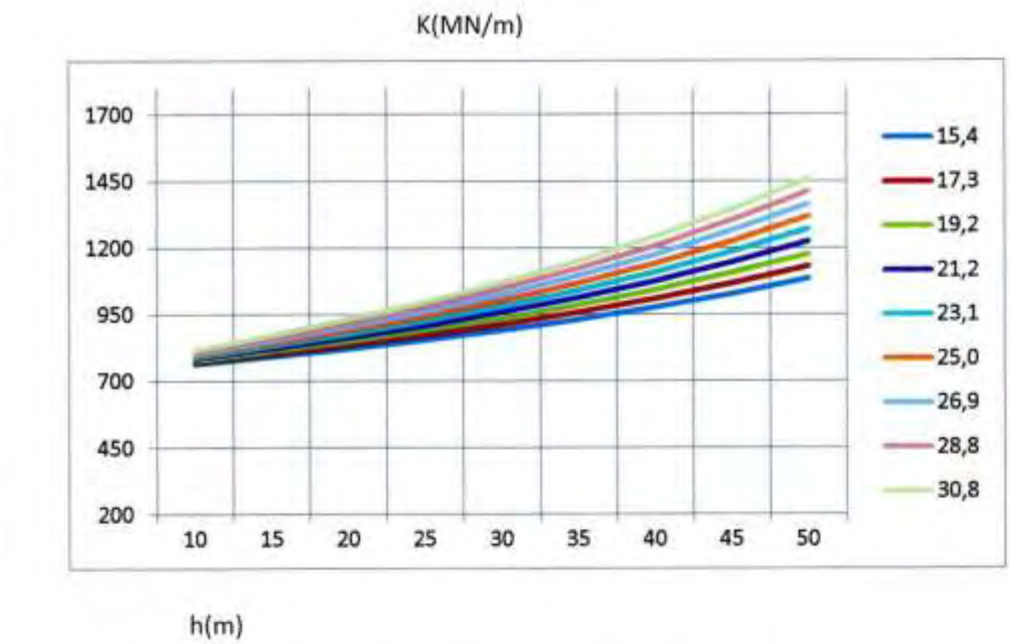


*FIG. 5: Flexural Stiffness K (h) for various lateral shear Modulus $G_S$(MPa) – Case $H_{tot}$ = 150 m ; $r_0$ = 55 m.*

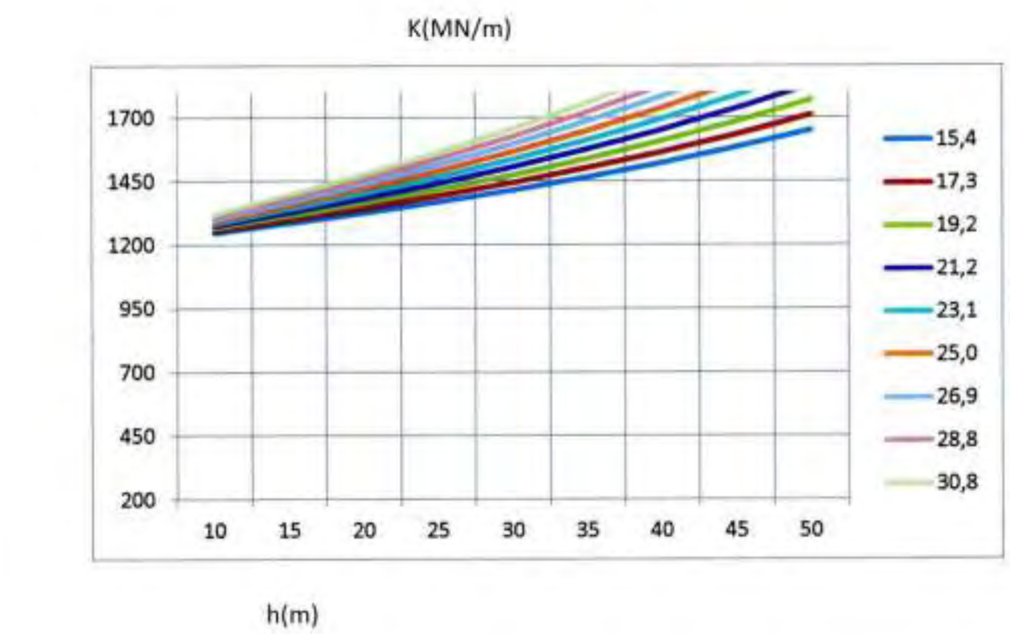


*FIG. 6: Flexural Stiffness K (h) for various lateral shear Modulus $G_S$(MPa) – Case $H_{tot}$ = 150 m ; $r_0$ = 65 m.*

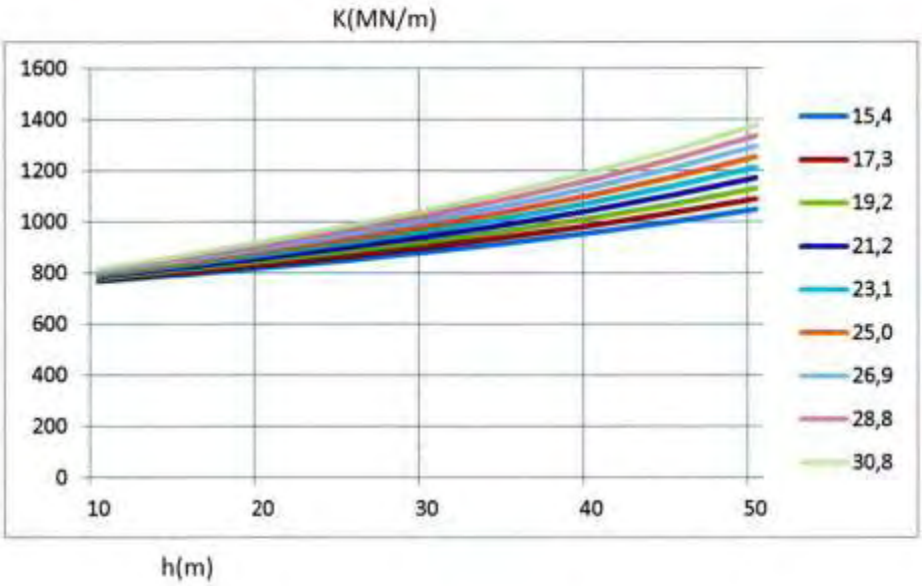


*FIG. 7: Flexural Stiffness K (h) for various lateral shear Modulus $G_S$(MPa) – Case $H_{tot}$ = 170 m ; $r_0$ = 60 m.*

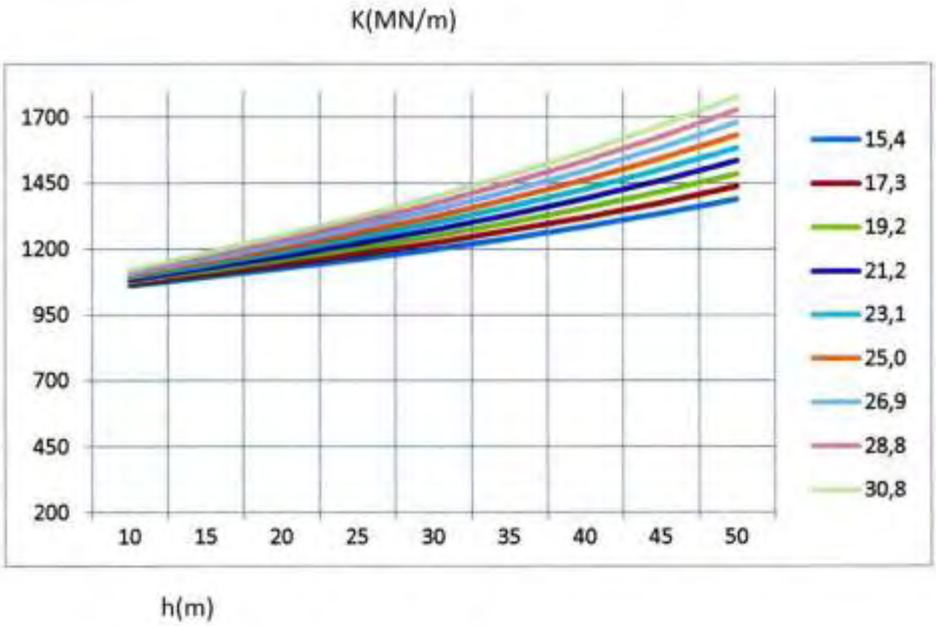


*FIG. 8: Flexural Stiffness K (h) for various lateral shear Modulus $G_S$(MPa) – Case $H_{tot}$ = 170 m ; $r_0$ = 67 m.*

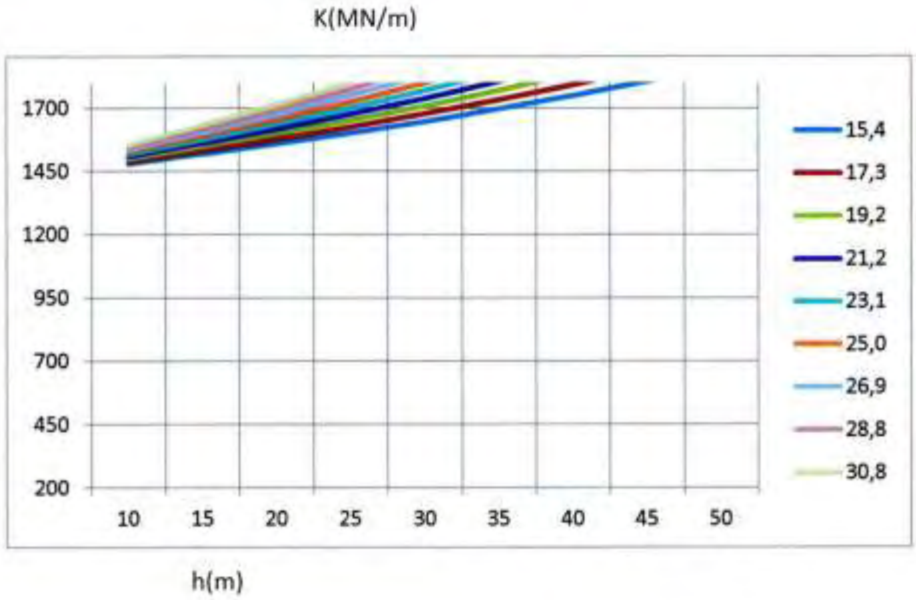


*FIG. 9: Flexural Stiffness K (h) for various lateral shear Modulus $G_S$(MPa) – Case $H_{tot}$ = 170 m ; $r_0$ = 75 m.*

## SOME COMMENTS

From Figures 1 to 3, namely for $H_{tot}$ = 130 m, it's possible to see that only a high embedment depth h even for good lateral soils gives overall stability for $r_0$ = 45 m while for $r_0$ = 55 m the overall stability generally requires from medium to high embedment depths h ranging from 30 m to 45 m and finally $r_0$ = 65 m makes safe each solution.

From Figures 4 to 6, namely for $H_{tot}$ = 150 m, it's possible to see that for $r_0$ = 45 m there is no stability at all while for $r_0$ = 55 m only high embedment depth h even for good lateral soils give overall stability and finally for $r_0$ = 65 m an embedment depth of 25 m makes safe each solution.

From Figures 7 to 9, namely for $H_{tot}$ = 170 m, it's possible to see that only a high embedment depth h even for good lateral soils gives overall stability for $r_0$ = 60 m while for $r_0$ = 67 m overall stability generally requires from medium to high embedment depth h ranging from 35 m to 50 m and finally $r_0$ = 75 m makes safe each solution.

## CONCLUSIONS

A limit value lateral stiffness K value of 1370 MN/m has been calculated for a typical steel deck, 350 m span cable stayed bridge, as an example case to judge the typical areoelastic stability of cable stayed bridges.

The lateral stiffness K has been calculated by the classical formula for circular embedded foundations subjected to rocking dynamic excitations.

A parametric analysis has been made to find out the Geotechnical Engineering decisions to attain the K limit value by varying the Pier radius and the embedment depth in various lateral soil conditions and for three Tower heights.

A reasonable solution is indeed individuated by enlarging the foundation radius $r_0$ more than deepen the embedment h, also provided that a soil improvement has been made.

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# SUBMERGED FLOATING TUNNEL IN STEEL FOR SOGNEFJORDEN

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## ABSTRACT

*Works on developing a submerged crossing of Sognefjorden were started in the early 2012. The main purpose was to create a cost effective crossing suitable to be used for very wide and deep fiords, and is a very relevant topic for infrastructures in Norway.*

*As starting point several structural geometry and material types have been studied and carefully evaluated. Crossing under sea level long distances is an engineering challenge mainly due to the large lateral forces due to currents and waves. Using steel pipes with an arc geometry means that horizontal forces will be taken as tensile/compression axial forces. Moreover, in order to improve the global buckling capacity of the arc, has been decided to build two main steel pipes and connect them with cross pipes;  this create a Vierendeel arch truss with an high global stiffness.*

*The main pipes are built up by an inner and outer steel cylinder. This solution is similar to ship design with double hull and improves the security and ductility for internal and external damage.*

*The tunnel pipes are positioned at a specific depth under sea level so that free ship navigation is guaranteed; in order to keep the level of the pipes constant will be used either tension supports cable anchored to the sea bed or pontoons rigidly connected to the pipes.*
*At the shores the solution studied is to use a method similar to the technology used for onshore process plants with incoming piping from the sea bed.*

*From a construction point of view, is possible to prefabricate the pipes in sections at shore location, then tow the elements to their final position and finally joint them.*
*For crossings wide and deep fiords a floating/submerged tunnel is evaluated to be a cost effective and environmental friendly alternative.*

## INTRODUCTION

This article describes the submerged floating tunnel that Multiconsult developed during the "Konkurransepreget dialog – Mulighetsstudie kryssing av Sognefjorden" (Norwegian Public Roads Administration, Region West, Framework for performing Competitive Dialog) [1]. During this concept phase several concepts for crossing the Sognefjorden with submerged pipe bridge were discussed. The phase ended up with a proposal for a submerged pipe bridge in steelwork. The same concept has also been studied with an alternative concrete pipe section This last solution arise several engineering challenges, especially from a constructability point of view, in fact the use of the concrete make the joint execution really complex; studies demonstrate that for short submerged pipe bridge (where only one pipe is needed to provide the requested stiffness) the concrete alternative is possible and cost-effective.

## REQUIREMENTS

The required life time for the bridge is 100 years and based on the average daily traffic it is required only to have a two lane bridge for each direction. It is also required a pedestrian lane placed in a separate pipe/tunnel.

Emergency exits from one pipe to another are needed every 250 m according to Statens vegvesen handbook 21 [2]. It is also required to provide emergency lay-by. In figure 1 is shown typical layout of the tunnel.

Sognefjorden is 3.7 km wide and 1.5 km deep at the bridge position. The navigation channel for ships shall be 400 m wide and 20 m free depth.

*Figure 1 Typical layout of the tunnel*

## THE CONCEPT

The concept consists in three steel pipes: two main pipes with two driving lanes each and one minor pipe for pedestrians. There will be traffic in one way only in each pipe. The two main pipes are built up by an inner and outer steel cylinder. The wall thickness of these two pipes will be in the range of 20 to 40 mm. To achieve adequate stiffness and avoid local buckling the plates will be stiffened by both transverse and longitudinal stiffener plates. The use of double cylinders will improve the security and ductility for internal and/or external accidents. External accidents considered are collision from ship or submarines and the internal accidents considered are explosion inside the tunnel.

Between the two main pipes will be placed a smaller pipe for pedestrian. Since this pipe is protected from any external accidents by the two main pipes, this pipe has been designed as a single tube.

All the 3 longitudinal pipes will be connected by transversal tubes spaced between 80 to 125 m. Every second transverse tube will be used as emergency exit from one main pipe to the other. As already mentioned the resulting static system is an arch Vierendeel truss that provide high global stiffness to the global structure,

The pipes have a positive buoyance of 5-10 %. Where it is possible the pipe elements have to be fixed to the sea bed using vertical cables. If this is not possible then an alternative possibility is to use concrete pontoons (with ballast) rigidly connected to the main pipes.
Figure 2 and 3 below shows the typical cross section of the bridge.

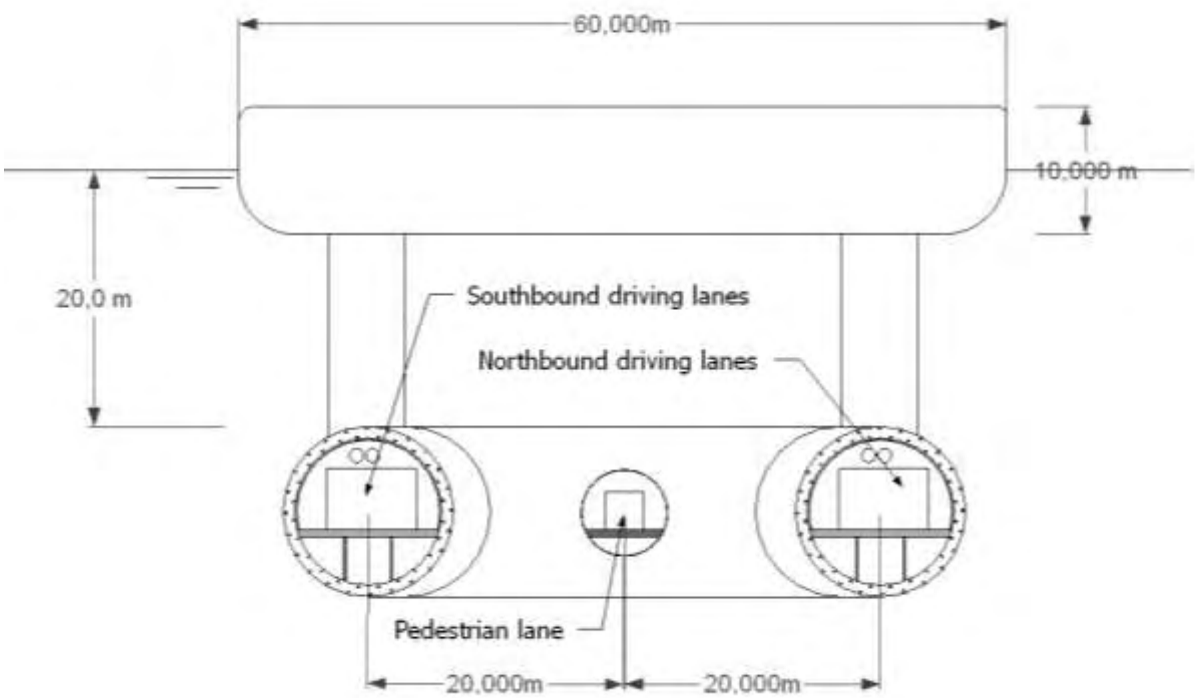


*Figure 2 Typical cross section*

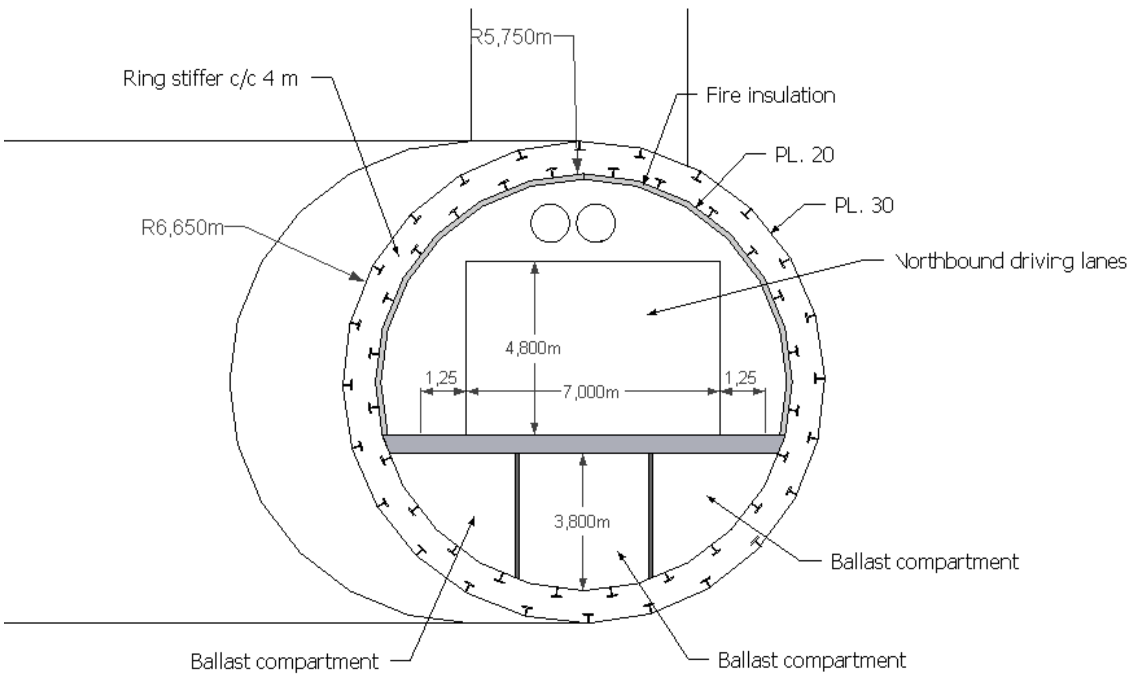


*Figure 3 Typical cross section for one pipe*

The submerged pipe bridge axis will be positioned (in the horizontal plane) on a circular arc with radius of approx. 5000 m. In the vertical plane the bridge will be horizontal at the mid part and sloping at both ends. Figure 3 shows both the horizontal and vertical alignment.

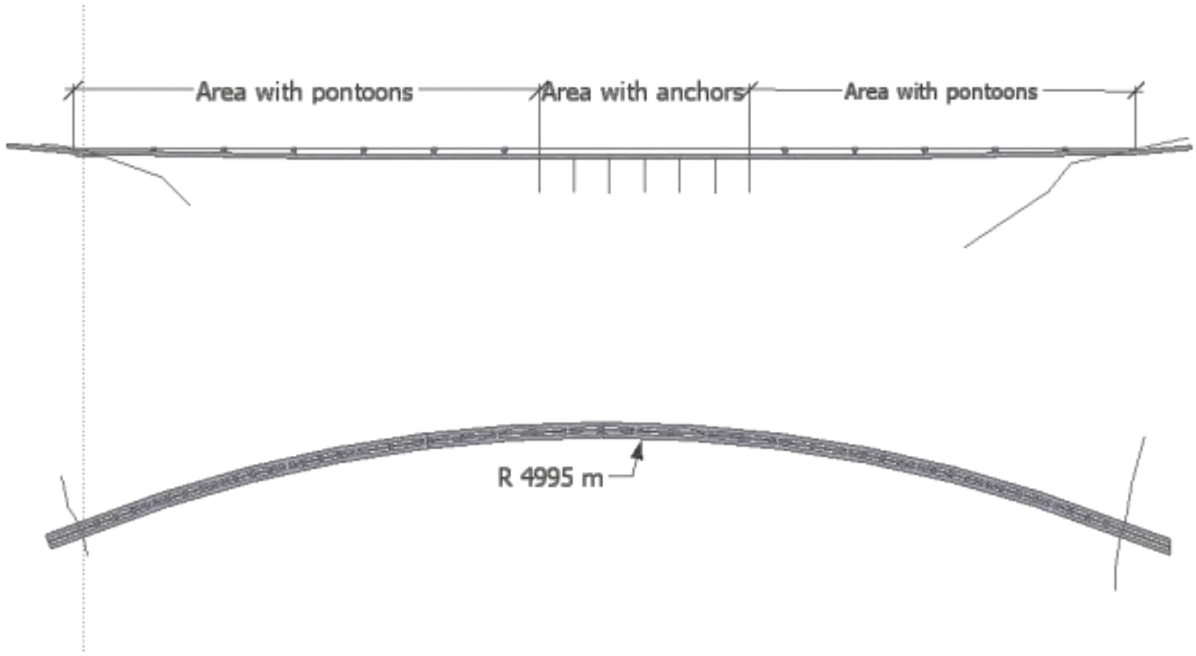


*Figure 4 Horizontal and vertical alignment*

**The main pipes**

The two cylinders will be connected by watertight bulkheads. These bulkheads will give a possibility for segmentation, and restricting water penetration in case of damage of the outer cylinder. These bulkheads will also act like ring stiffeners for the pipe. Between the ring stiffeners will be installed several longitudinal stiffeners. During the studies carried on has been discussed if it was better to use thick plates with few longitudinal stiffeners or thin plates with several stiffeners. Results demonstrate that is more efficient to use thin plates with several stiffeners, in fact this will produce less complex work with the welding process.

The ring stiffeners will be placed each 4 m. In each stiffer a manhole for maintenance and inspection will be installed. Each fifth stiffer will be without manholes. This will provide to the structure several watertight compartments, each one 20 m long. Moreover will be established access to each compartment from the area below the driving lane.

The decision of using steelwork instead of concrete means that more ballast material will be needed. The system is designed for 5-10 % positive buoyancy. To achieve this pipes have to be as small as possible. This is the reason why has been decided that the pedestrian path will be placed in a separate pipe lying between the two main pipes.

**Fire**

The pipe bridge shall resist to a fire accident of a heavy truck and so the RWS – curve shall be used for checking the fire resistance. Both steel and concrete pipe options will need additional protection against fire, using elements that resist temperature of 1350 °C.

Since the steel pipe has a double cylinder, with the outer cylinder designed to take all the external loads, in the unlikely event of a fire accident the bridge will not collapse, only the inner cylinder will result damaged.

**Foundations**

The bridge has a fixed foundation at shore on each side and a combination of anchors to the seabed and pontoons. Sognefjorden is deep with steep terrain at each shore. In this area it is not possible to use anchors at the seabed and pontoons will be the recommended solution. In the center of the fjord, the seabed surface is flat enough and that makes possible the use of anchors. Using anchors in the center will be also beneficial for the ship navigation channel.

The purpose of the anchors is to avoid the bridge to float up. The anchors will be designed as gravity base anchors. The anchors will be done in steelwork, in order to make it possible to handle them with a lifting vessel; the anchors placed by the vessel will be placed to the seabed by water ballast. When they are in the correct position then the water ballast will be replaced by heavy ballast. The heavy ballast used can be Olivine or similar products. These types of anchors have been already installed at depth of 700 to 800 m. The anchors have to be installed 6 to 12 month before installation of the bridge in order to avoid problems due to differential settlements.

The anchor rods (that connect the gravity anchor with the bridge) can be manufactured with different technologies. In particular is possible to use the following types of rods:

- Drill rods
- Normal steel pipes
- Fiber lines
- Chains (Due to the heavy self-weight it can be necessary to have buoyancy elements on the chain)

In the illustration on next page (figure 5) are shown two different type of gravity anchors with vertical rods, but also inclined rod configurations are possible.

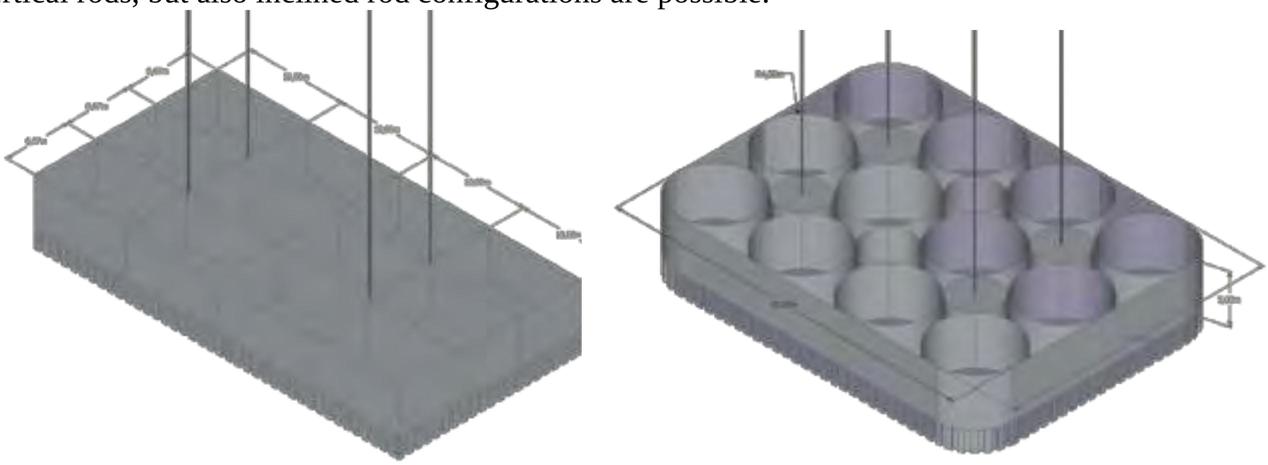

*Figure 5. Gravity base anchor with rectangular cells and circular cells.*

**Pontoons**

The submerged pipe bridge will have a positive buoyancy of about 5-10 %. For this reason the pontoons have to be ballasted in order to keep the bridge in the correct position in all the areas without gravity anchors. It is possible to use standard ballast like gravel or heavy ballast like Olivine if needed.

**Landfall / abutments**
The top of the pipe bridge will be a few meter below sea level at the shore. We recommend a pipe geometrical layout similar to what has been done for oil and gas pipes. The pipes have to be drawn into a ditch of the same bridge's width. The bottom of the pipes will be approx. 15 m below sea level, which means that the ditch has to be of about 20 to 25 m deep at the shore, depending by the topography. The ditch will be approx. 60 m wide.

We have assumed that the shores are made by rock ground. This means the blasting operation has to take place in order to realize the ditch; we recommend starting the blasting operations furthest from the sea. The ointment breakthrough against sea depends of the topography of the seabed. If the rock is steep at level -15 m then it is possible to drill the first ointment from the land side. The rest of the breakthrough has to be drilled from a barge and the charge will be placed by divers.
If the topography is not so steep all the blasting will be done from the sea side.

## STATICAL AND DYNAMIC BEHAVIOR

Loads on pipe bridges are defined in "Statens vegvesen Håndbok 185 Bruprosjektering 6.13 Rørbruer" [3]. The main loads are: Self weight, marine fouling, buoyancy, traffic load, tide water, current waves, wind on pontoons and accidental loads (fire, explosion and ship impact).
Currents are the main loads in the horizontal direction. According to [3] the currents loads can be modeled in 3 configurations: as a constant uniform load across the fjord, as an uniform load along only the central part of the fjord or as a shear current with uniform load at each half of the fjord.
A preliminary rough static calculation gives the following horizontal deflection of the bridge:

| | |
|---|---|
| Uniformly distributed load | approx. 0.5 m |
| Uniform partial distributed load mid fjord. | approx. 1.0 m |
| Shear load | approx.. 5.0 m |

Dynamic effects have not been included in this preliminary study.

The main reason for choosing an arc layout instead of a straight line for the horizontal alignment is that the horizontal loads will mainly be taken as axial loads. Because loads acting on the bridge are mainly non-radial loads, unsymmetrical and the bridge have fix ends the static analysis shows large bending moments and shear forces in the arc. A preliminary calculation shows that the stress will be moderate if the arc radius is around 5000 m. Decreasing the radius can be beneficial for bending and shear in the bridge.

The bridge is designed to act as a vierendeel girder, in order minimize buckling effects in the horizontal plane. The bridge has a buckling length of 1900 m and a slenderness of approx. 100 to 125. Has been calculated a theoretical safety factor against horizontal buckling for a linear elastic system, of approx. 15 to 25 due to uniform distributed currents load. These values are within acceptable limits. The risk of buckling (or the size of additional moments) is biggest when we have a uniformly distributed load from current across the whole fjord. It is in this case you also have the biggest axial loads. For the load case with partial uniformly distributed load only in the central part of the fjord, the axial forces are reduced by 35 %, and the bending moment and shear forces are reduced by 50%. For the load case with of shear load, the axial loads are relatively small, but the bending moments and shear forces are larger.

The vierendeel-girder structure used transfer the shear forces between the main pipes to the transverse elements. These shear forces will cause “local” bending moments both in the transverse and main pipes. The transverse pipes are placed with varying distance between each other. The transverse distance will be smaller at the ends (where shear forces are large) and bigger in the middle (where shear forces are little).

## ERECTION OF THE BRIDGE

Since most of the bridge structure is made of steelwork; it is possible to build in modules, each of them constructed in either one big yard or on several yards.

### Pipes

Modules of the pipes will be constructed in a workshop and shipped to the site. It is intended to build big modules which can float. The length of each module depends of what is optimal dimension for transportation. Each module will be equipped with preliminary watertight bulkheads for controlling water intrusion. The bulkheads will be equipped with manholes for access. We suggest that the modules have to be almost completed at the workshop with driving lanes, ballast material and other equipment. There will be some preliminary structures for make possible the assembly procedures of the modules.

### Landfall / abutments

The structure for the bridge abutment can be built of either concrete or steelwork. In both cases the structure can be prefabricated at a workshop and shipped to the site and mounted by a lifting vessel. Also in this case preliminary structures are needed for the assembly procedures.

### Erection at site

### Landfall / abutment

It will be the same solution on both sides of the fjord. A dry dock gate will be established for the dry dock. The dry dock port will be situated as close to the sea as possible. When the dry dock gate is established all the water will be pumped out of the dry dock. The bottom of the dry dock will be approx. 15 m below sea level. Landfall with all equipment needed for the connection of the bridge will be built in the dry dock. The landfall will be fixed against the rock. The dry dock gate will be opened when it is time for connecting the bridge to the landfall.

### Gravity anchors

The gravity base anchors will be installed by a lifting vessel and preliminary water ballast. Permanent ballast will be filled up through a pipe.

### Assembly of the bridge modules

Steel modules will be shipped to the site and concrete structures (pontoons) will be towed. We assume that the whole bridge will be assembled in an area with calm waters close to the bridge

site. It must be built temporary mooring points and temporary jetties. Barges, tugboats and lifting vessels are needed at site. The operations are dependent on good weather.
Pipe modules will be assembled on barges. After one new pipe module is installed, will the pipe be pushed out in the sea and pontoons will be installed. Figure 5 shows how this can be done. Temporary structures will be installed to control the vertical position and stability of the bridge. Winches will be used to control the horizontal position and movement.

During the installation the bridge will be anchored to the seabed.

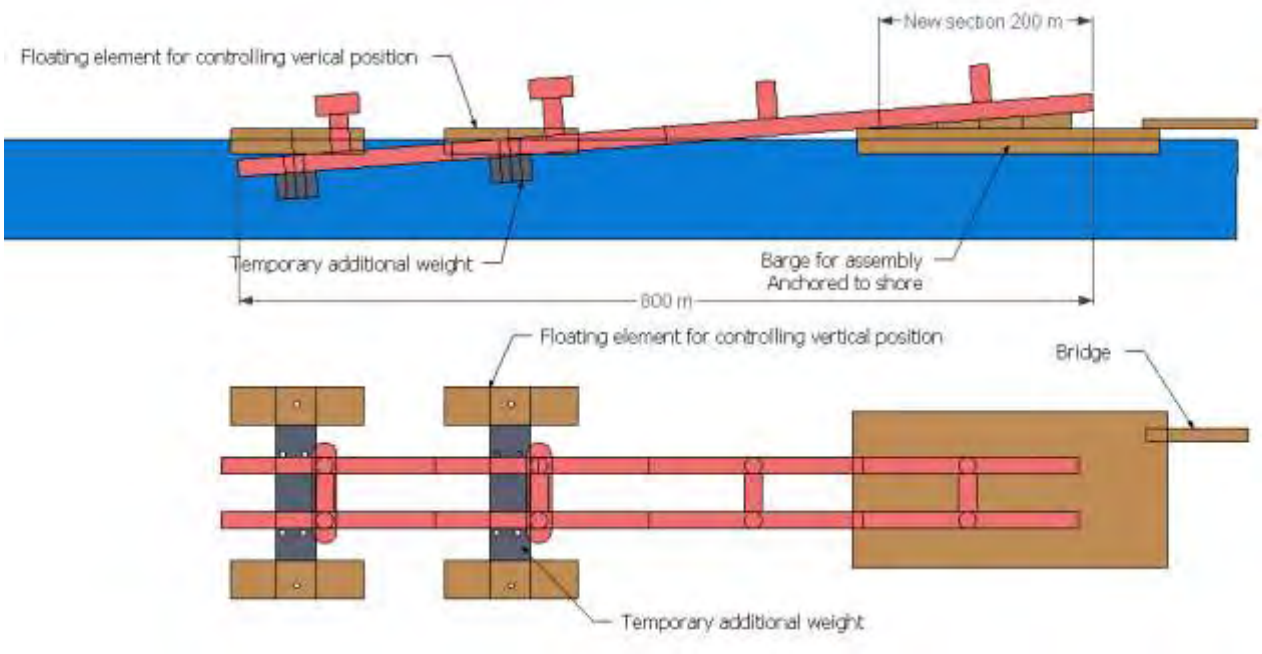


*Figure 5 Assembly of the bridge modules*

**Assembly of the bridge**
All needed components for the assembly operation will be installed on the bridge before tow out. The pipe bridge will be positioned in correct level before tow out. The complete bridge will be towed to the bridge site. For position and temporary foundation at shore we will use winches. At one side there is need for a connector. When using the connector the bridge can be winched a little bit more than necessary to get the needed space on the other side. When the bridge is in permanent position the dry dock gate will be closed and the water removed from the dry dock. Then all permanent fastening can be performed.

Total weight of the tow is about 1 mill tons. This is similar to the tow used for Troll A GBS. In that case were used 10 tug boats. Since this bridge is so long, the need of tugboat is bigger than on the Troll A GBS, especially in order to have control with the geometry and resist current loads. Figure 6 shows the marine operation in principle.

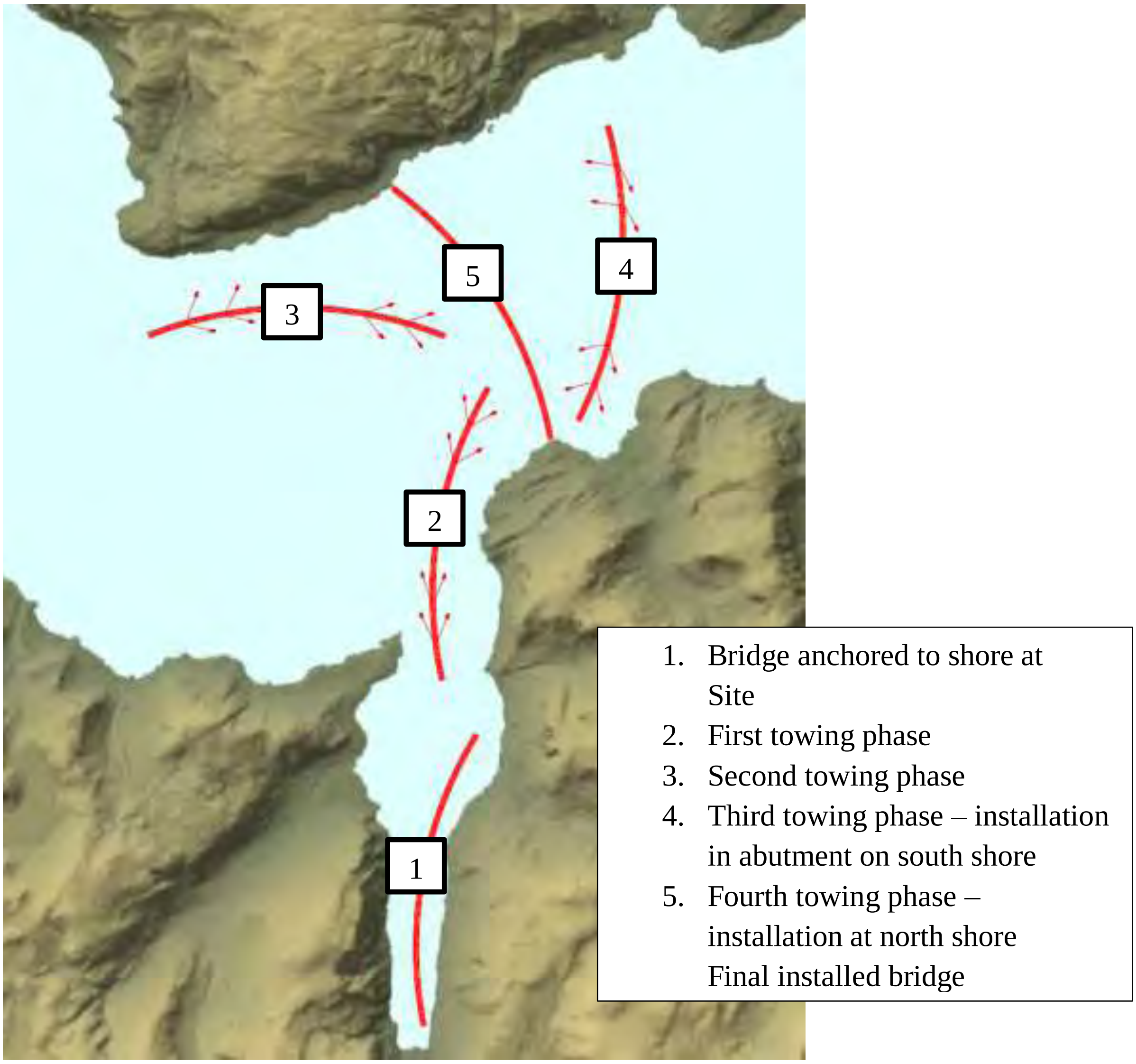


*Figure 6. Marine operation in principle*

## CONCLUSION

The study shows that it is feasible to use a submerged floating tunnel in steel for crossing the big fjords like Sognefjorden. Since all of the structure except the pontoons is submerged, the structure will have small impact on the environment.

Cost for fabrication and erection of the submerged floating tunnel has not been investigated in detail in this study. But since most of the fabrication is based on well-known principals we assume that this will be a cost effective structure for this big crossings, like Sognefjorden.

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# CHAINED FLOATING BRIDGE

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## ABSTRACT

*Crossing of wide fiords with floating bridges is a very relevant topic in Norway. A continuous, rigid floating bridge usually entails comprehensive structures. One main reason is that continuity leads to large moment and shear forces due to the bending stiffness. By introducing "hinges", the bending effects will be reduced, and the load effects will to a large extent be made up of axial forces. This will provide more cost efficient structures.*

*The process with developing a chained floating bridge began in 2010. The main purpose of this work has been to create cost effective bridges, which may be used for very long crossings.*

*A long floating bridge will be affected by lateral forces such as wind, currents and waves. By designing the bridge as a curved chain, the side forces will be taken as tensile axial forces. The bridge girder is therefore relatively slender compared to rigid structures.*

*The bridge is a modular bridge, which implies simple prefabrication and assembly. It has two main element types. The "catamaran element", which is an element comprising a bridge girder on two pontoons, and a "link element", being the intermediate girder connecting two catamaran elements.*

*The elements are connected by swivel joints, and the bridge itself has in principle no lateral rigidity. It adapts to a larger or lesser extent the form of the lateral loads. To limit lateral misalignment and to prevent folding of the bridge elements, the bridge will be tensioned at one of the abutments or stretched out in a S-shape with unilateral side anchors.*

*Most fiord crossings in Norway are protected from large ocean waves. In these crossings a chained floating bridge is a cost effective easily buildable structure, with or without side anchoring. The price per meter length of the bridge will be relatively little affected by the total bridge length.*

## INTRODUCTION

The idea of a chained floating bridge was introduced for the first time by MSc. Jan Soldal from the company Akvator AS early 2011 for crossing the Bjørnefjorden south of Bergen.

The concept was developed further during 2011 by Akvator AS and Multiconsult AS. The results of these studies were presented in the document "Ferjefri E39 – Bru over Bjørnefjorden" [1]

In parallel to the above studies for the Bjørnefjorden, during winter 2012, it was also conducted a conceptual design proposal for the E39 Sognefjorden crossing.

**DESCRIPTION OF THE CONCEPT**

The chained floating bridge is composed by several floating bridge units (Catamaran elements) assembled together by simple bridge girders (Linker elements) to a continuous long chain as shown on figure 1 below.

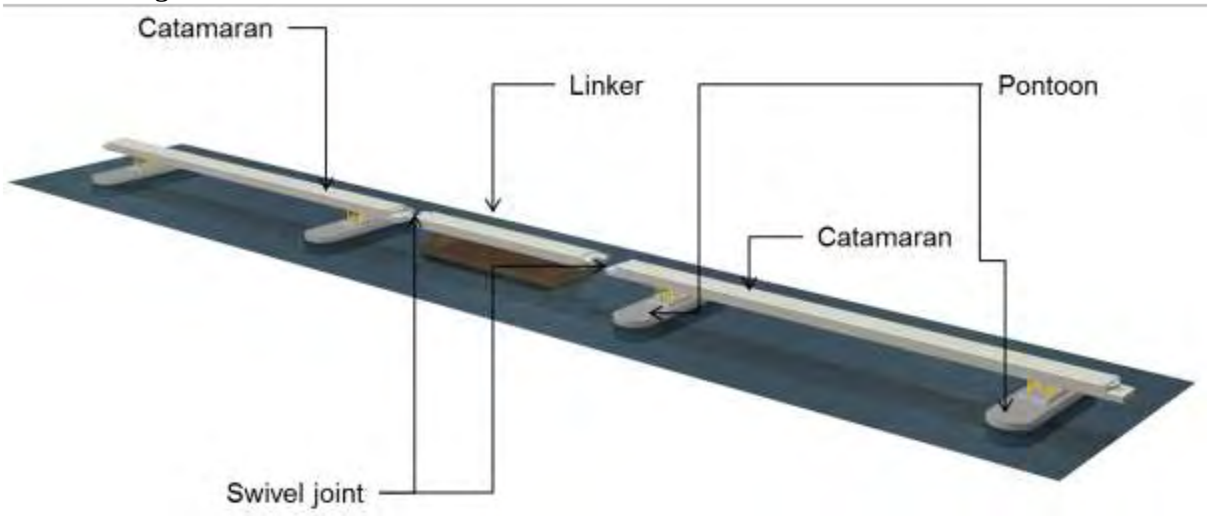



*Figure 1 Elements of a floating bridge*

The catamaran element consists of a bridge girder supported on two pontoon elements. This structure has a good stability against wind, waves and current loads.
Catamarans or multi hulls have been introduced in the modern boats design (both in leisure and sport sailing) relatively in recent times, although historically they have been used a very long time as typical boat system in both India and Oceania. The main reason is that this system provides the boats good stability against waves with a simple and cost effective construction.

The connection between catamarans and linker is made using swivel joints. The swivel joint is designed to allow small rotations around the vertical axes and transverse axes of the bridge. This means that no bending moments are transferred by the joints, and thus all the external horizontal loads (like wind, waves and currents) will be supported by the bridge through a longitudinal tension forces (similar to the behavior of a horizontal catenary).

The main principles and behavior of the chained bridge is illustrated in the following figure 2 and figure 3


*Figure 2. Static equilibrium of the chained bridge*

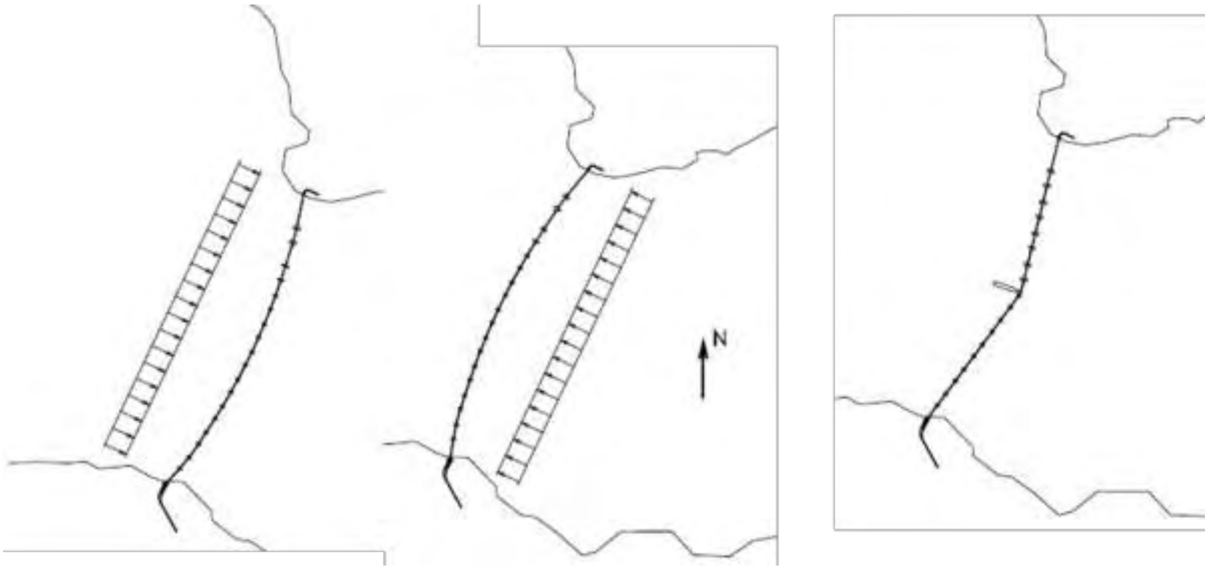



*Figure 3. Chained bridge without side anchors – shape with wind/current and ship impact.*

During the studies carried out for the chained floating bridges, two different main configurations have been considered: a system with transversal anchors and a system without anchors.

**Configuration 1 with transverse anchors**

For this configuration the bridge has got the shape of an S in the plan view. This layout allows reducing the longitudinal tensile force (by one fourth) and making a more stable structural system. In this configuration we have one fixed abutment on one side and one sliding abutment on the other side. The stability of the system is assured by a constant tension force applied (using gravity and pulley systems) at the sliding abutment position. Figure 4 below shows this configuration for crossing Bjørnefjorden.

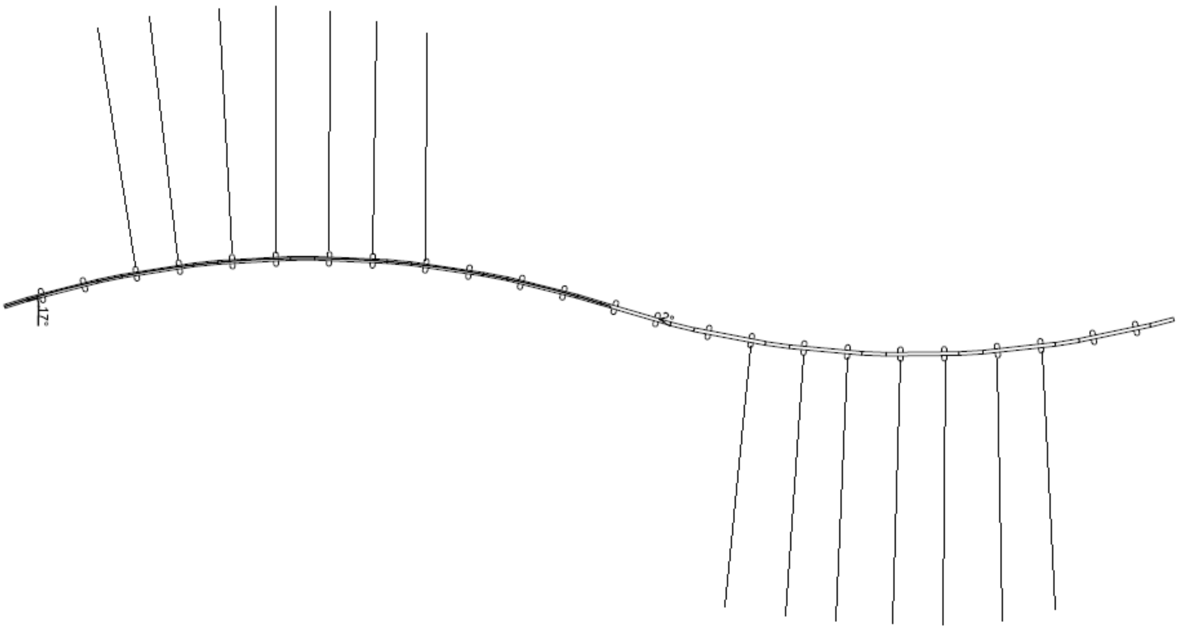


*Figure 4. Configuration 1 – geometrical definition*

We have calculated that the maximum rotation angle in each swivel point will be approximate 1° and an expected max transverse movement of about 2-4 meters. This means that the bridge has a maximum elongation of about 1.0 to 2.5 m in the longitudinal direction that has to be "absorbed" by expansion joints at the movable abutment.

**Alternative configuration 1 with fixed abutments at both ends**

In this alternative configuration, both abutments are fixed and the bridge stability is assured by a post-tensioning system anchored at the abutment position and running through all the bridge elements. In this case it is possible to use a cable joint system between catamaran and linker elements (see figure 5).

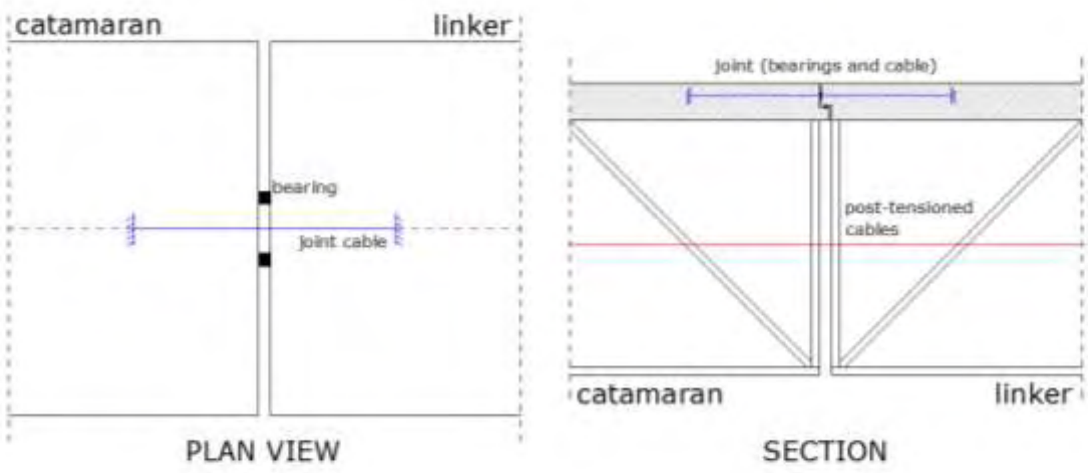



*Figure 5. Alternative configuration 1 – joint details*

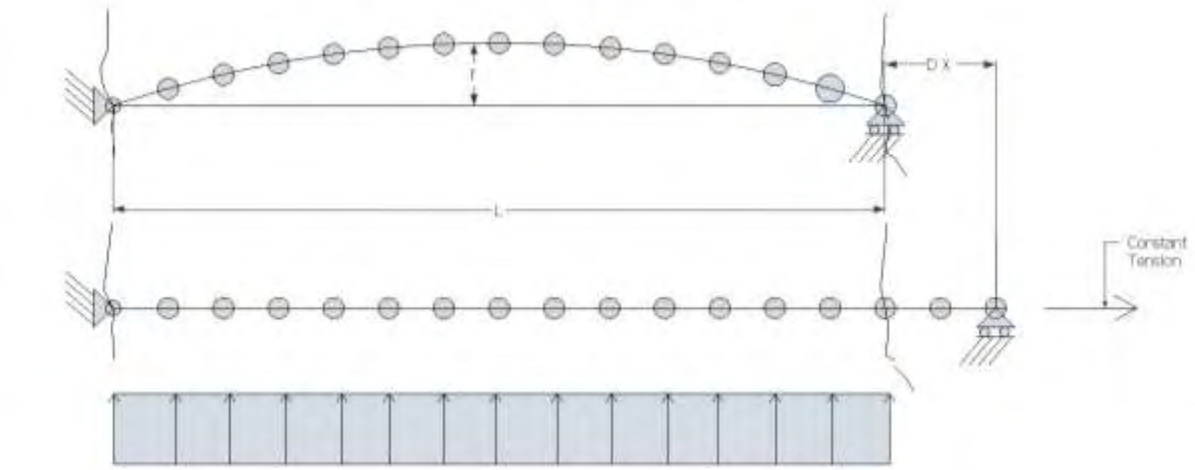
**Configuration 2 without transverse anchors**

For deep fjords, like Sognefjorden, it will be impossible to use transverse anchors. For cases like this, the bridge has to be straight and can only have a static system composed by a fixed abutment, a sliding abutment and a constant tensile force.

The static behavior of the bridge is shown in figure 6. With a maximum sag of about 6-7% of the total span of the bridge, it will be an engineering challenge to handle the elongation of the bridge at the sliding abutment.

In the case of Sognefjorden with a span of 3.7 km the elongation of the bridge will be about 40-45 m. A first solution proposed is to use standard expansion joint placed in series and supported by wheels. This will allow "absorb" the total elongation in several small standard elements.

The necessary tensile force needs to be achieved using of gravity mass, in order to always have a constant tensile force in any position the bridge can assume.




$$t \cdot f = \frac{w \cdot L^2}{8} \rightarrow T_0 = \frac{w \cdot L^2}{8 \cdot f_{max}} = \cos t \rightarrow f(w) = \frac{w \cdot L^2}{8 \cdot T_0}$$

$$f_{max} = 200\, m\, (from\ road\ profil\ regulations)$$

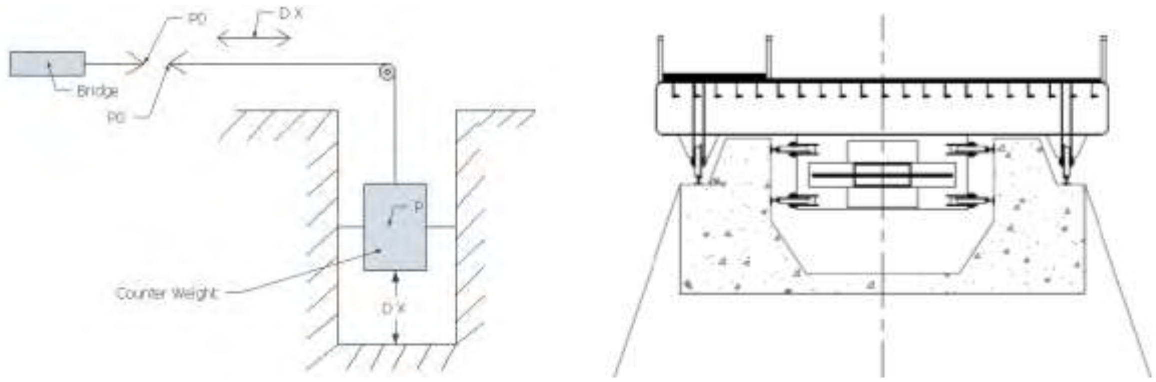



*Figure 6. Structural system of configuration 2*

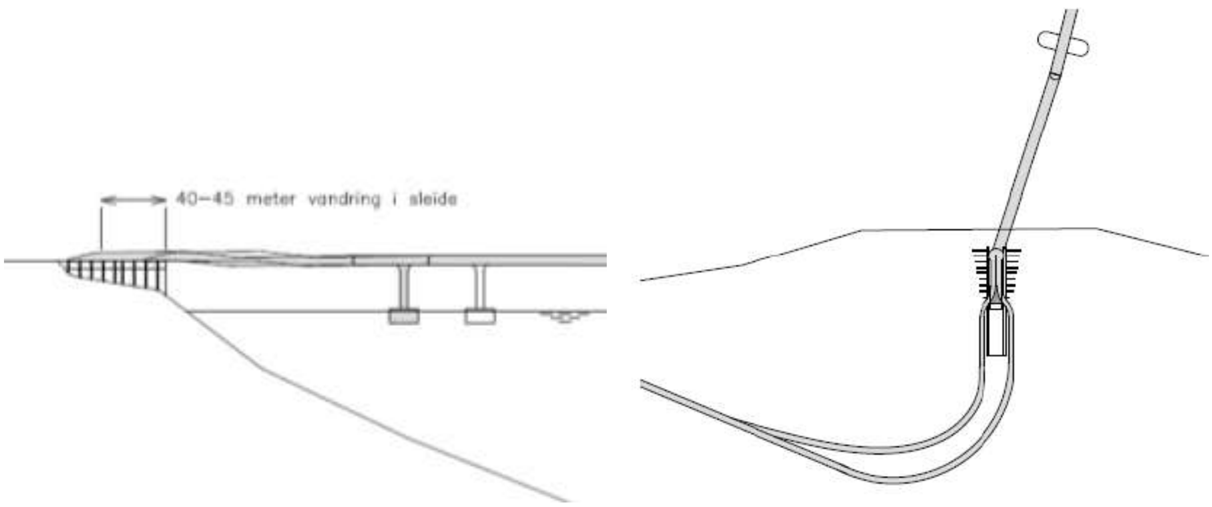



*Figure 7. Sliding abutments system details*

**Bridge deck**

With this typology of bridge it is possible to use a standard deck element, (concrete deck, steel box girder, truss girder) and the structural dimensions (with the related unit costs) are not influenced by the length of the bridge. For this reason it is a really “flexible” bridge that can have from two lanes for crossings with low traffic expectancy, to four (or more) lanes for crossings with heavier traffic. This can guarantee economic feasibility (and associated low investment cost) for crossings with low traffic and/or relative low importance.

The bridge girder, for both catamaran element and linker element, depend only by the size of each element. This allows optimized and tailored solutions for each crossing. The studies undertaken by Multiconsult shows that optimal span varies from 120 m to 400 m.

## Pontoons

Each catamaran element is supported on two pontoons. The size of the pontoons will vary in function of the span widths and the bridge height (above sea level). The pontoons construction can be made by concrete or steel cellular box. Due to corrosion problems, the need of ballast and the poor resistance against vessel collision, the steel option does not seem to be the most feasible. The concrete pontoons are made of thin walls and void cells, in order to have the needed buoyance. If needed it is possible to use water, sand or material with a higher density (like Olivine) as ballast.

An optimal solution is to have pontoons with the Eigen period higher than the wave period (generated by the wind). A typical pontoon of 70m by 20 m with a submerged height of 8-10 m, as shown in figure 8, will not suffer wave resonance phenomenon.

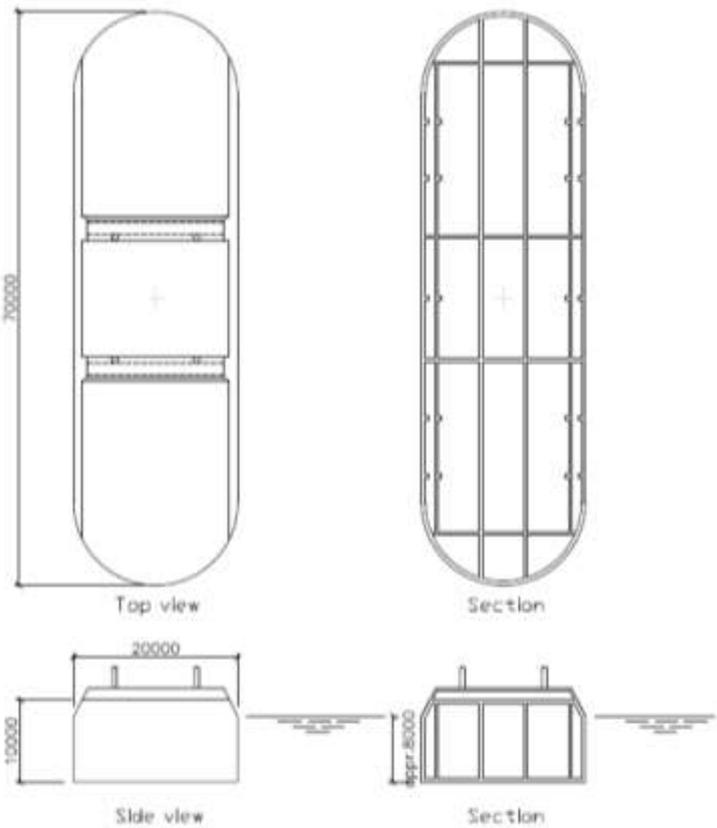



*Figure 8. Typical layout of pontoon*

The pontoons will be aligned to the transversal bridge direction. Long pontoon elements guarantee the transversal stability of the bridge (pitch moments can be carried with different water pressure over the length of the pontoon). Roll movements will not be a problem since two pontoons are connected in the catamaran elements (this means the roll moments are transformed into vertical forces in each pontoon).

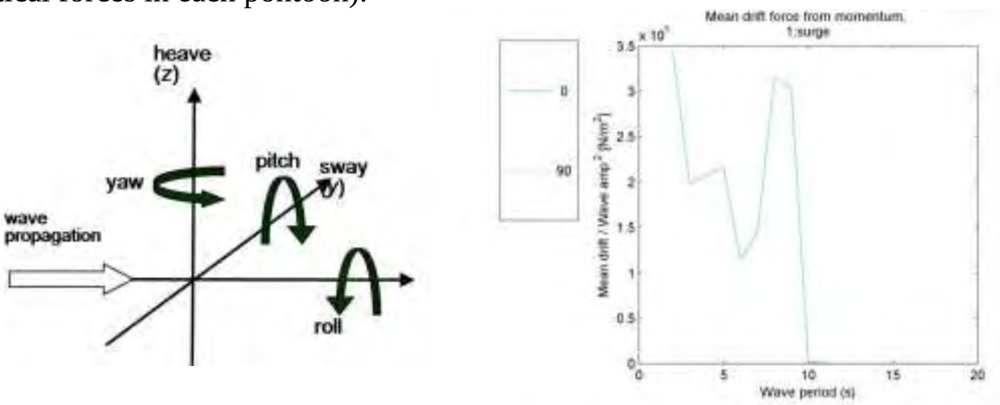



*Figure 9. Wave period and movement mode definition*

Considering safety in case of accidental situations, the design of the pontoon elements has to be done assuming that two of the outer cells can be flooded without compromising the buoyancy of the structure. Each cell can be accessed by manholes from the deck of the pontoon for maintenance, and if needed the manholes can be used to pump out water from the cells (using a portable bilge). Each cell will be equipped with alarm system for water intrusion and water level sensors (both electronic and manual systems).

**The moveable joints (swivel joints)**

The joint between catamaran and linker elements is a key part of this new bridge concept. This joint has to be able to: allow rotations along the vertical axis and the transversal axis of the bridge, transfer the tensile force, guarantee the support of the linker element over the catamaran and in addition allow the passing of vehicles on the deck.

Several solutions have been proposed in order to fulfill the above requirements, in particular a pin solution (see figure 10). This consists of a spherical bearing with blocked horizontal movements, and pre-tensioned cable with horizontal and vertical neoprene bearings (see figure 5).

Further studies need to be carried out in order to evaluate cost implication of each solution.

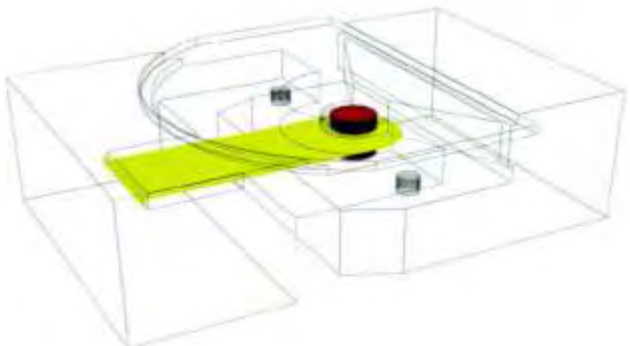


*Figure 10. Moveable joint details*

**Abutments**

In particular for the bridge configuration 2 (without any transverse anchors) there are some engineering challenges to be handled. For example the large longitudinal elongation of the bridge and the construction of the mass counterweight that assures the axial tension force needed.

As already discussed previously the only (complex) solution that allow to permit longitudinal elongations of the range of 40-45 meter is to use a system of expansion joints placed “in series”. The tension force can be produced by either gravity structures that move together with the elongation of the bridge or by series of hydraulic pistons.

For the bridge configuration 1 (with anchors on both sides) the expected elongation is relative small and there are today expansion joints with these capabilities on the marked.

## NAVIGATION CHANNEL

The size of the navigation channel will vary for each crossing site. As an example, for the Sognefjorden it is required a navigation channel that is 400 m wide and 70 m high. In general we think that for the majority of the crossings the requirement will be less demanding.

For the Sognefjorden the solution proposed was to locate the navigation channel close to the shore (on the fix abutment side) and to use a cable stayed bridge to span the required 400 meters. For other crossings, with smaller height requirements, one of the catamaran elements can be adapted to be the navigation channel. In this case the channel should be placed in the center of the bridge in order to allow an easy passage for the boats.

## CONSTRUCTION AND ASSEMBLY OF THE BRIDGE

All the components of the bridge can be constructed out-site, in special yards. The girders for both the Catamaran element and linker element can be constructed in a steel yard in modules that can be shipped to the site.

The pontoons can be constructed in a dry-dock and floated to the site. In Norway there are several dry-dock facilities at the west coast. Moreover there is a lot of experience in floating concrete structures over long distances at seas due to the needs of the oil industry.

When all the modules are shipped to site, it is possible to assemble them close to the shore protected from wind and waves. The catamaran elements can be assembled on one or several barges and then mounted on the pontoon. When all catamaran elements are finished they will be anchored to the shore. Then each catamaran element can be linked to another by a linker element (installed with a lifting vessel). Following this procedure the entire bridge can be easily assembled from the two sides of the site.

At the same time the abutments can be constructed and all the anchors can be installed at the right places and “fixed” to the sea bed.

As final construction stage the bridge will be moved using several tugboats to the exact crossing position and hooked up to the anchors and to the abutments.

## COST

The purpose of this concept is to provide a new concept for floating bridges that can have low cost when large fjords need to be crossed. With a chain behavior we can obtain a structure where the stiffness and the size of the girder is not dependent of the total length of bridge, which also means that the unit prize of the bridge will be independent by the total length.

In [1] it is made a rough cost estimate for the crossing of Bjørnefjorden that shows a unit price of NOK 1,1 million per meter.

## CONCLUSION

It is possible to make long fjord crossings by a chained floating bridge. The bridge is cost effective compared to other types of floating bridges (stiff bridges).

Chained floating bridges without transverse anchors have raised several engineering challenges that, at the moment, can be solved only with complex tailored solutions.

An anchored chained floating bridge, on the otherhand, have demonstrated to be feasible, with minor issues that are still pending further investigation, but that can be resolved with well-known technologies already used in other engineering fields (especially oil and gas industry).

## REFERENCES

1. Kværner Stord AS, Wärtsila Norway, AS Apply Leirvik AS, Advantec AS. “Ferjefri E39 – Bru over Bjørnefjorden” February 2012

# AKASHI KAIKYO BRIDGE PROJECT

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**ABSTRACT**: *Japan is mainly composed of four islands. Transportation between islands has relied on ship in olden time and affected by weather conditions. A fixed link is safer and reliable for transportation. The government had started technical investigation since 1959 in order to link Honshu and Shikoku islands. Feasibility study also carried out by the Japan Society of Civil Engineer and the final report was published in 1967. It was reported three routes of the Honshu-Shikoku Bridges were feasible to construct, but need more research for the Akashi Kaikyo Bridge. At that time the longest suspension bridge with the main span of 1298m in the world and that of 367m in Japan. The Honshu-Shikoku Bridge Authority was stablished in 1970 in order to carry out the project. Characteristics of the Akashi Kaikyo Bridge are wide strait (4km), strong tidal current (4.5m/sec), supported by sedimentary rock or semi compacted gravel, occurrence of big earthquake, strong wind speed (Design wind speed is 60 m/sec at girder), busy ship channel (1400 navigation/day), good fishery. Various technical investigation and research had been carried out for the Akashi Kaikyo Bridge and the center span length as a suspension bridge was decided to be 1990m (expanded to be 1991m due to earthquake) in consideration of cost and 1500m international wide international waterway. One of Honshu Shikoku Bridges had been completed in 1988. The longest center span length in this route was 1100m. It means that the center span length of the Akashi Kaikyo Bridge stretches almost twice. The construction of the Akashi Kaikyo Bridge started in 1988 and completed in 1998 without any fatal accident. This paper describes the way to reach the technical solution for the longest suspension bridge in the world.*

## PREFACE

The Japan consists of four major islands including Hokkaido, Honshu, Shikoku and Kyushu and additional thousands of small islands as shown in Fig.1. Total area of Japan is approximately 378,000 km$^2$. The chief natural feature of Japan is the fact that the topography is almost entirely mountainous (about 70%). As a result, cities, agricultural land and transportatation infrastructure are concentrated in very few plains along the coasts and basins in mountain areas. Another distinctive characteristic of the topography is the bow shape, extending from northeast to southwest, with the total distance of approximately 3,000km. Transportation between islands has relied on ships for a long period of time and affected by weather conditions.


*Fig. 1 Land of Japan*

People in Hokkaido, Kyushu and Shikoku earnestly wished to have their islands connected to Honshu by higaways and railways. The first achievement was the construction of Kanmon Tunnel, connecting Honshu and Kyushu: the tunnels for railways and highways were completed in 1942 and 1958, respectively. As for Hokkaido and Shikoku, the Seikan Tunnel (for railways) and one route of Honshu Shikoku Bridges (both highways and railways) were both completed in 1988. And the construction of the Akashi Kaikyo Bridge has started in 1988 after almost 30 years technical investigation. Fig.2 shows the enlargement of the suspension bridge in Japan. And Table 1 shows the world record breaking suspension bridge.

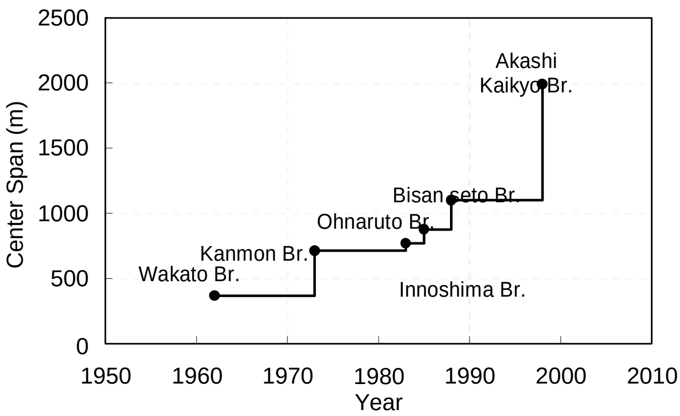


*Fig. 2 Span enlargement of suspension bridge in Japan*

*Table 1 World record breaking suspension bridge*

| Name | Country | Year | Center span(m) | Type of girder |
|---|---|---|---|---|
| Brooklyn Bridge | USA | 1883 | 486 | Truss |
| Bear Mountain Bridge | USA | 1924 | 497 | Truss |
| Benjamin Franklin Bridge | USA | 1926 | 533 | Truss |
| George Washington Bridge | USA | 1931 | 1067 | Truss |
| Golden Gate Bridge | USA | 1937 | 1280 | Truss |
| Verrazano-Narrows Bridge | USA | 1964 | 1298 | Truss |
| Humber Bridge | UK | 1981 | 1410 | Box |
| Akashi Kaikyo Bridge | Japan | 1998 | 1991 | Truss |

## SITE CONDITION OF THE AKASHI STRAIT [1]

### Topography and geology

The geological profile of the Akashi Strait is shown in Fig.3. There is a marine canyon reaching as deep as 110m and 400m in width at the center of the strait. The foundations are located on the flat seabed below sea level, ranging from 30m to 50m. The seabed under the strait consists of sand gravel, compacted sand gravel, soft rock, and granite from the top. Except on Awaji island side, the granite is located at depth of more than 150m below sea level. Therefore, relatively soft formation, compared with the other suspension bridges in Japan, was selected as the bearing layers.

### Meteorological and offshore conditions

The basic wind speed for the design (an average wind speed for the 10 minutes at 10m above sea level) is determined to be 46 m/sec by considering a return period of 150 years. Observation of wind has been carried out by 80 m high tower for 20 years. Assuming breaking waves caused by the design wind, the design wave height is determined to be approximately 9 m. Tidal current

## Earthquakes

Since Japan is a land of frequent earthquakes, an earthquake of a magnitude 8.5 which is expected to occur once in 150 years based on earthquake record within 300 km from the construction site, has been taken into account in the design of the bridge.

## Navigation

The Akashi Strait has an international sea route, which is 1,500 m wide and has a daily traffic of 1,400 ships. Therefore, careful attention should be paid to safe navigation and collision avoidance during the construction as well as after completed

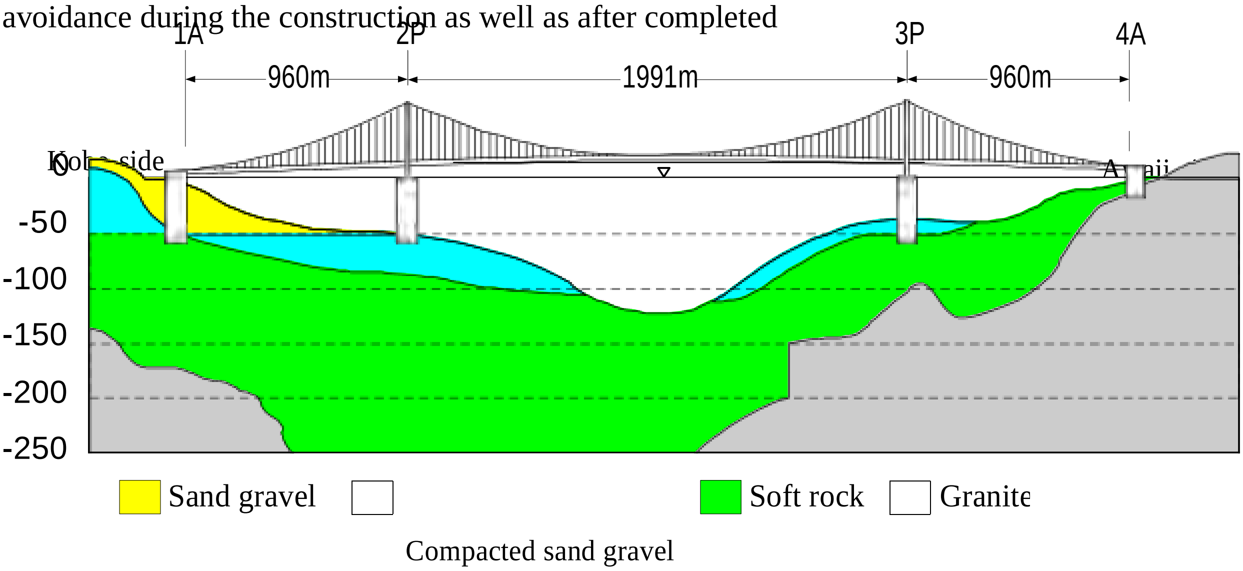


*Fig. 3 Geological profile and general view of the Akashi Kaikyo Bridge*

## DESIGN OF SUBSTRUCTURE [2]

The compacted gravelly layer was chosen for 2P and the soft rock layer was chosen for 3P. In order to investigate a ground condition, a boring of soil was performed using self-elevating platform at pier sites, because the tidal current was strong and the water depth great. Also, to seize the characteristics of the compacted sand ... n diameter, it was essential to acquire stable samples of at least 30cm in diameter as shown in Photo 1. For this reason, triple-tube sampling machine with a large diameter of 36cm was implemented.[1].

Since the supporting bed was relatively soft and the conventional sesimic design method based on firm and solid earth was not applicable, another concept for seismic design had to be established. For the foundation of the Akashi Kaikyo Bridge so enormous in scale, the concept of "dynamic mutual action" was implemented for its seismic design.


*Photo 1 30 cm Boring core of sand gravel* main structure, thus the seismic reliability was eventually verified.

In selecting an appropriate type of the foundation, large-scale field hydraulic tests, large scale excavation survey at the site, etc. were conducted to confirm the construction period, economy, scouring characteristics reliability of the construction and other factors. As a result, the following points were made clear and the Prefabricated Caisson method was chosen.

1. Grab excavation of Kobe and Akashi formation is possible.
2. By constructing riprap and mesh bags around the caisson, tidal current scouring can be prevented.
3. Prefabricated and laying-down caisson is simple and feasible, by using existing equipment and the expertise.

The top of the tower foundation is subject to an extremely large load from the tower. In addition, the foundation must remain stable under its dead load, inertial forces due to earthquakes, tidal current and wave forces. The shape and the size of the foundation were examined to enclose the tower anchor frames as well as to ensure the stability.

The resulting foundations are circular in shape, 80m for 2P and 78m for 3P in diameter and bottoms of foundations are -60m for 2P and -58m for 3P. The circular caisson section has advantages with which it forms a stable floating structure during toeing, mooring and sinking, and the sinking operation is easier since there is no orientation problem against the deviation of the current direction.

The elevation of the top of the foundation was designed to be T.P. (mean sea level of Tokyo bay) +10m in order to prevent damage to the tower due to wave actions and ship collisions.

The anchorage of the Akashi Kaikyo Bridge is classified the rigid type of spread foundation and is to support 110,000 t of the cable horizontal tension force and transmit it to the support ground. While structural dimensions above the ground for 1A and 4A are almost same, their foundations are completely different from each other because the support ground for 1A is located below 62m the ground surface and that for 4A is 12 to 21m below its ground surface. Fig. 4 shows an anchorage body and foundation of 1A [3].

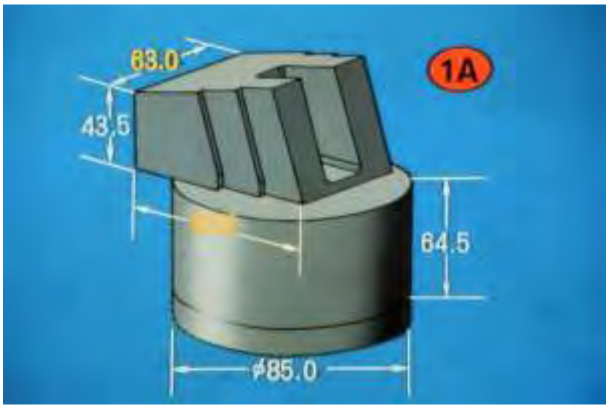


*Fig. 4 Anchorage body and foundation (1A)*

## CONSTRUCTION OF SUBSTRUCTURE [1]

Fig.5 shows the relation between the depth of seabed and speed of tidal current. It shows that pier foundation of the Akashi Kaikyo Bridge has to construct in very severe condition. Construction method is as followings.

First, the seabed was dredged into supporting bed by a huge grab-bucket excavation, and then a prefabricated steel caisson was towed to the site by a fleet of tugboats as shown in

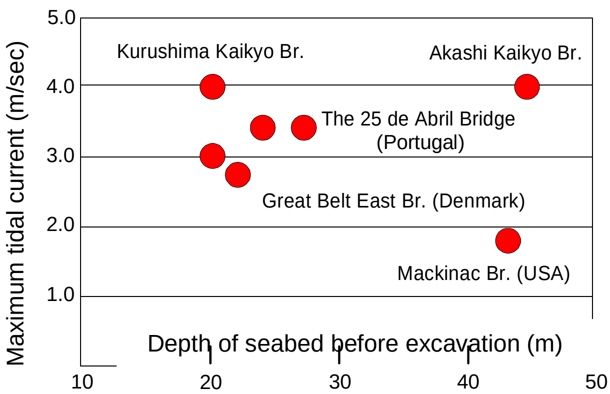


*Fig. 5 Relation between depth of seabed and speed of tidal current*

Photo 2 and installed down on the supporting bed in the water as shown in Fig.6.

*Photo 2 Towing of steel caisson to the site*

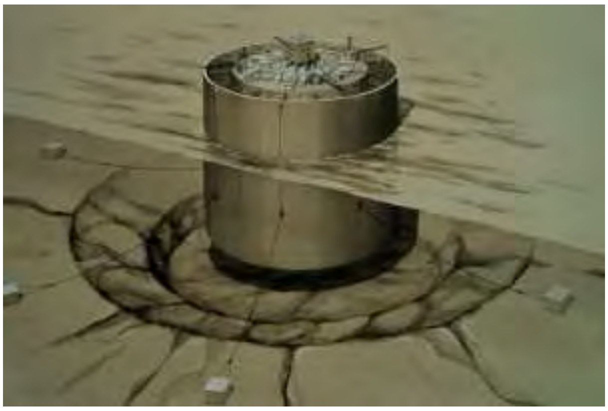

*Fig.6 Installation of prefabricated steel caisson*

The Akashi Strait with its rapid tidal current and sand gravel of the seabed is liable to scouring. The intricate vortex and acceleration flow around the caisson generate strong and complicated sheering/lifting force around the structure, thus causes scouring. Among a lot of preventive measure against scouring of maritime structure, riprap showed to be most effective in term of function, cost and maintenance, in a strong tidal current like in the Akashi Strait. According to the scale model experiment shown in Photo 3, rubble of 1 metric ton was proved to be stable enough against a tidal current of 4 m/sec. Therefore, 3 m thick riprap layer was formed around the caisson in the range of three times of caisson diameter.

*Photo 3 Scouring test with protection*

The periodical depth survey has shown that the condition is stable at large without any major evidence of scouring. Once a caisson was installed on the seabed, concrete was cast to fill the caisson.

In the Akashi Kaikyo Bridge, desegregating underwater concrete was applied. Desegregating admixture itself had been developed in Germany. At that time in Japan, though, there was no application to large scale structures. Therefore, a wide range of experiments from basic to large scale with operation were required. The essential characteristics for under-water concrete are; a higher desegregation, lasting fluidity and heat crack resistivity. For this reason, low heat generative cement mixed with desegregating admixture and super-plasticizer was used. Photo 4 shows the slump test. And it was found that enough concrete strength could get within 10m flow distance. Photo 5 shows casting underwater concreting work. On completion of underwater concrete, ordinary reinforced concrete was casted in atmosphere to form a foundation.

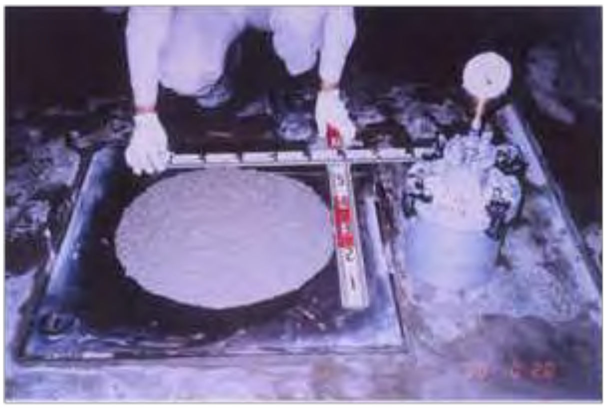

*Photo 4 Slump test*

*Photo 5 Casting of underwater concrete*

## DESIGN OF SUPERSTRUCTURE

### Tower [3]

The tower of the Akashi Kaikyo Bridge, to which the cable reaction of about 100,000 ton (1 GN) acts, is 287m of height. The steel tower vibrates easily under wind, not only during construction but after completion as well. Control of vibration, therefore, was one of the most important issues in design of the tower. Various investigations including wind tunnel tests had been conducted for many years, and control methods for the vibration of the towers were developed. In order to reduce the amplitude of the vortex induced oscillation caused by wind smaller than the design speed, Tuned Mass Damper are installed inside the tower shaft.

### Cable [1]

Main cable of a suspension bridge principally supports the dead load and live load of the bridge. The longer a center span becomes, the greater the rate of dead load increases. In the case of the Akashi Kaikyo Bridge, 90% of its main cable section bears the dead load. The decrease of dead load directly results in the reduction of steel volume as a whole and suppression of construction cost. A pre-study on the Akashi Kakyo Bridge showed that the conventional cable material of 1600MPa (160kgf/mm$^2$) would require double lines of cable on each side, totally 4 lines of main cable. This would complicate the structure as well as construction; therefore cable material of higher strength had to be developed. With use of 1800MPa (180kgf/mm$^2$) high strength wire as well as re-examination of its safety factor, the main cable of the Akashi Kakyo Bridge resulted in the single line of 1.1m diameter on each side. Fig.7 shows the history of cable strength in suspension bridges.

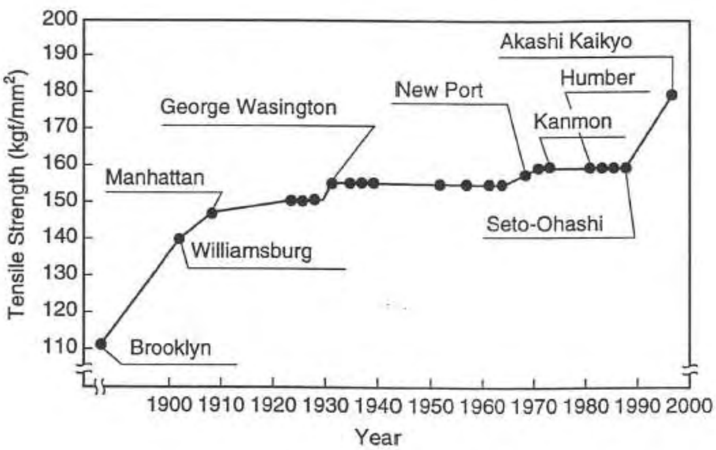


*Fig. 7 History of cable strength*

### Girder

The stiffening girder of the Akashi Kaikyo Bridge is so thin as compared to the scale of its supporting span. It is susceptible to deform and generate self-excited oscillation by wind because of its low natural frequency. The wind proof stability was also examined by the wind tunnel test which used 40m long, 3 dimensional whole-bridge scale model, in addition to the conventional independent girder as shown in Photo 6.

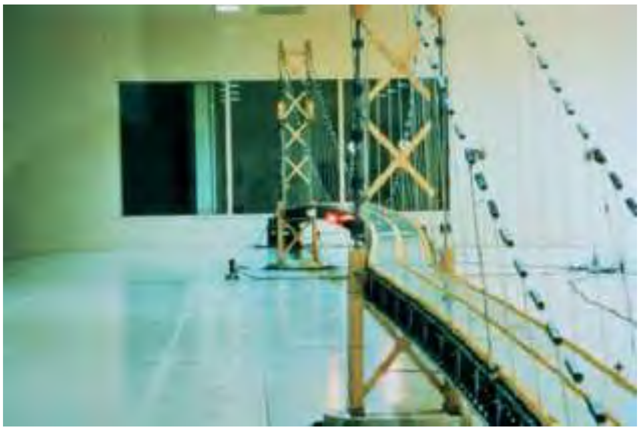

*Photo 6 Wind tunnel test for the Akashi Kaikyo Bridge (Scale: 1/100)*

## CONSTRUCTION OF SUPERSTRUCTURE

### Tower [4]

To fabricate the tower, the tower was divided into 30 blocks vertically, and each block was further divided into 3 cells, whose weight did not exceed the crane capacity of 160 ton. As the vertical accuracy, the tolerance at the top of the tower was specified to be less than 1/5000 × (tower height). To realize this accuracy, in the shop, each block had to be fabricated with the accuracy of less than 1/10,000. Then, 3 cells were temporarily assembled, and the sectional planes were ground and polished to the required flatness, using a specially made large size cutting and grinding machine. Afterward, the blocks were again separated into 3 cells and transported to the site. At the site, climbing tower crane was adopted to minimize the erection period. Photo 7 shows erection of tower.

*Photo 7 Erection of tower*

### Cable [4]

To minimize the erection period, the prefabricated parallel wire method (PS method) was applied. The composition of the cable is shown in Fig. 8. The cable erection was started by carrying a pilot rope from shore to shore as shown in Photo 8 The conventional way to carry a pilot rope was to pull a rope with floats by a tug-boat or pull a rope by tall floating crane in Japan. But this system was not applicable at busy ship channel. Then it was decided to pull a rope by the

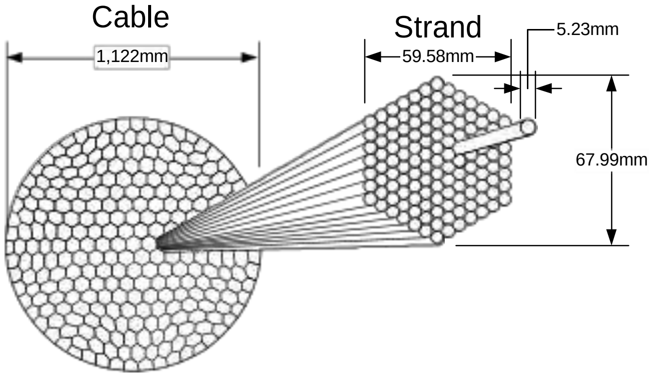


*Fig. 8 Cross section of cable*

*Photo 8 Pilot rope by a helicopter*

It made possible by use of very light polyaramid fiber rope with a diameter of 10 mm. Testing of pilot rope by a helicopter was carried out at the harbor in 1990. The pilot rope was connected with steel ropes, and was pulled for the replacement to the stronger ropes. These works were repeated, and the hauling system for cable strands was completed. Using this system, catwalk ropes were erected with floors on them. Cable strands were erected as follows: 1) strand reals were transported to the yard near anchorage, 2) each strand was pulled by a strand carrier on a hauling system along the catwalk.

Cable sag was measured and adjusted during the night when the temperature is stable. Prior to decision of corrosion protection system for the Akashi Kaikyo Bridge, cable condition of existing suspension bridges were inspected and it was found that surfaces of some cables were found to be rusted due to insufficient protection against water. Many tests and investigations were carried out, and it was found that a better corrosion protection would inevitably need to improve the environment within the cable. The basic relationship between humidity and corrosion of galvanized wire is that almost no corrosion occurs when atmosphere around the strands has a relative humidity of less than 60% as shown in Fig.9.

The dry-air injection system was firstly adopted in the main cable of the Akashi Kaikyo Bridge in the world. Fig.10 shows the schematic illustration of dry-air in a cable. Strand ropes have been used as suspenders on Japanese suspension bridges, whereas parallel wire strands were introduced to the Akashi Kaikyo Bridge.

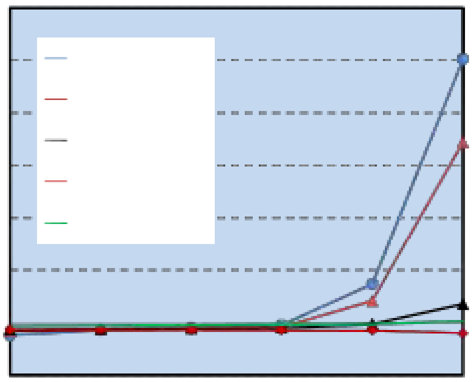

*Fig. 9 Results of critical humidity test with varying salinity*

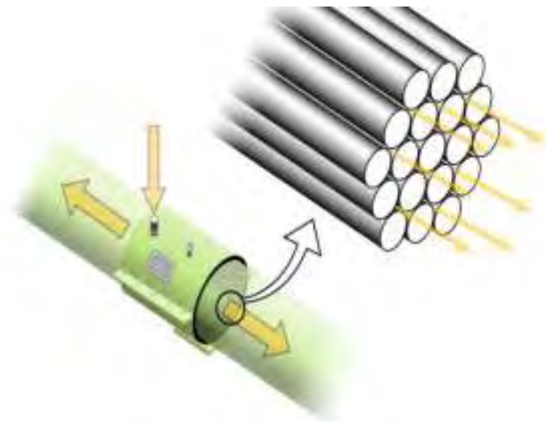

*Fig. 10 Schematic illustration of dry-air cable*

### Stiffening girder [4]

Girder erection was started by lifting large-block truss girder using a 3,500~4,000 ton floating crane at the two towers and the two anchorages as shown in Photo 9. As these areas are located outside of the navigation channel, those methods could be applied. Consequently, pre-assembled plane truss members with a length of 28m were lifted up on the deck, carried to the front of the erected girders, and connected to them.

*Photo 9 Large-block truss girder erection*

### Effect of the Southern Hyogo Earthquake [4]

On January 17, 1995, just after the cable had been erected, the Southern Hyogo Earthquake with a magnitude of 7.2 occurred. Its epicenter was only 3.5 km east from the center of the Bridge. After the earthquake, displacement of foundation by crustal movements was found as shown in Fig. 11. Then modified design of stiffening girder was absolutely required soon after the earthquake. There was almost no damage on the completed structure. Photo 10 shows the completion of the Akashi Kaikyo Bridge
.

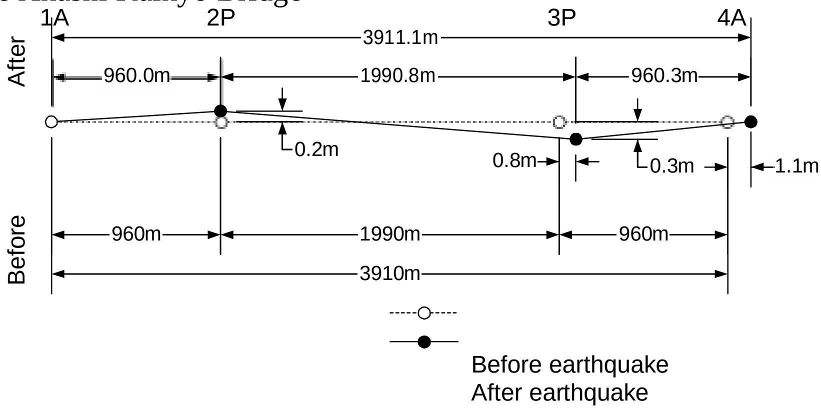


*Fig. 11 Displacement of the bridge after the earthquake*

*Photo 10 Completed Akashi Kaikyo Bridge*

## MONITORING [5]

The items of the field measurements are wind speed and its direction, earthquake motion and the bridge behavior caused by them. Strong wind data was recorded during typhoons that hit the bridge site until now as shown in Fig.12.

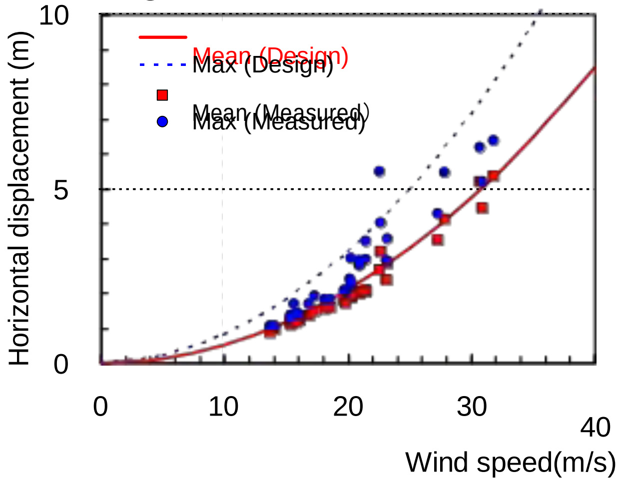


Fig.12 Relation between wind speed and horizontal displacement at the center of main span during typhoons (Design vs. Measured)

## CONCLUSION

It took almost 30 years for technical investigation and 10 years for construction of the Akashi Kaikyo Bridge. Many new technologies have been developed in order to construct the world

longest suspension bridge. It pasts 15 years after completion, there are no structural concerns and the bridge is serving as a trunk highway between Honshu and Shikoku islands.

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# SUSTAINABILITY THROUGH INNOVATION IN DESIGN AND CONSTRUCTION: SECOND PENANG BRIDGE, MALAYSIA

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## ABSTRACT

*Jambatan Kedua Ptd. Ltd. (JKSB), a wholly-owned company of the Malaysia Ministry of Finance, Incorporated (MoF Inc.) is the concessionaire for the Second Penang Bridge Project (PB2X). The bridge with estimated cost of RM4.5 billion is set to be the longest in South-East Asia with a total length of 16.9 km over water. The construction of the Second Penang Bridge commenced in November, 2008 and currently is at 93% progress and to be completed by September 2013. It faced various challenges in applying sustainability to both design, construction, operation and maintenance. The Second Penang Bridge is pioneering in Malaysia to be fully designed for seismic load for a 475year return period earthquake and a 2500 year return period earthquake with 'no collapse' criteria. This paper will further discuss on the project management, the fast-track concept and innovation in the implementation of a sustainable bridge design and construction.*

## INTRODUCTION

The Second Penang Bridge linking Batu Maung in Penang Island and Batu Kawan in Seberang Prai on the mainland when completed will improve trade efficiency and enhance logistics systems by providing better connectivity and accessibility to Penang International Airport. The bridge is aimed to alleviate the current overloaded traffic at the existing bridge and to meet the future traffic demand, apart from being one of the key elements in the development of Penang as logistics and transportation hub for the northern region of Malaysia under the Northern Corridor Economic Region (NCER) programme. Feasibility study on the project started under the 8$^{th}$ Malaysia Plan and was completed in 2002. The preliminary Environmental Impact Assessment study was undertaken for the project and approved by the Department of Environment in 2007. Initial works including soil investigation, topographic survey, dredging works and test piles begin immediately by the previous concessionaire, UEM Group Bhd.

## PROJECT DESCRIPTION

JKSB was appointed the concessionaire for PB2X in August 2008 for period of 45 years. It is responsible for the project management, design, construction, operation and maintenance.
The project alignment is depicted in Figure 1.

*Fig. 1. Project alignment*

PB2X is divided into the following packages (Fig.2). Package 1 is the works for main navigation span, substructure and foundation and Package 2 is superstructure works of approach spans. Both packages are design and built contract with horizontal split responsibility. Package 3A is the interchange at Batu Maung, Package 3B is the land expressway at Batu Kawan, 3C is the trumpet interchange at North-South expressway and 3D the toll plazas and related works. Packages 3E, 3F and 3G are the toll collection system, traffic control and surveillance system and M & E works for Package 3A and 3C respectively.

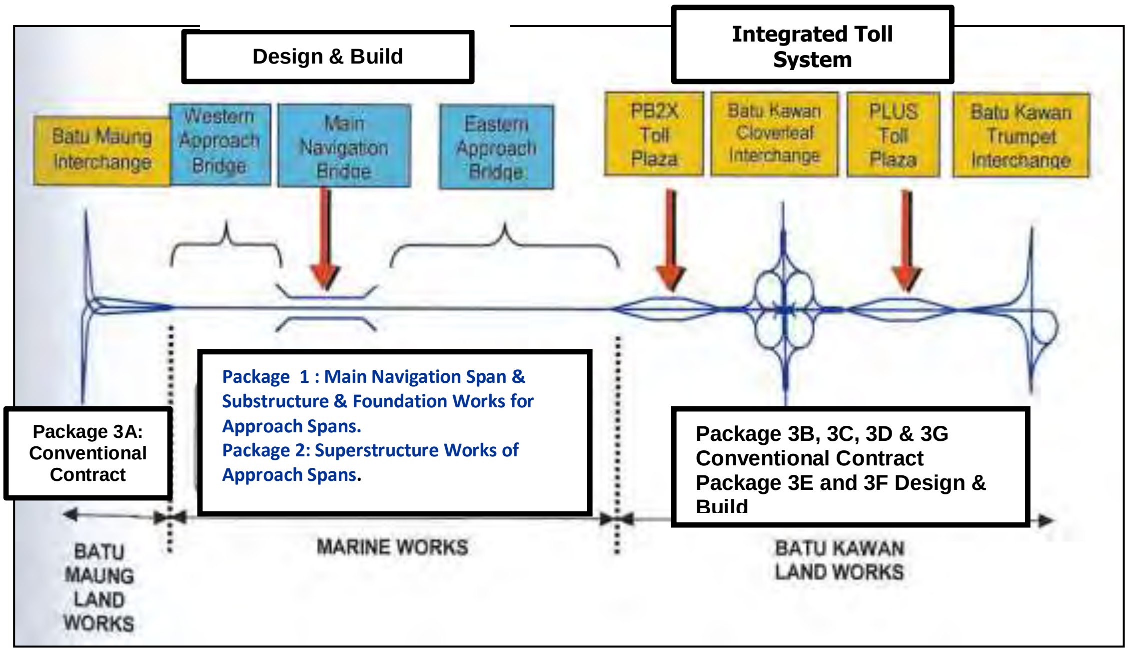


*Fig. 2. Overall work packages diagram*

## SUSTAINABLE DEVELOPMENT

JKSB has undertaken the lifecycle management of PB2X where sustainable development and green technology are key to building the future. Sustainable development is an enduring balanced approach to social progress, economic activity and environmental responsibility. The emphasis to a lowest lifecycle cost is to promote the concept of design for durability.

Durability is influenced by the following factors [1]:
- i. Design and detailing
- ii. Specification of materials used in construction
- iii. Quality of construction.

In our effort to conserve natural resources and protect the environment, high standards of environmental protection were incorporated into the project. Marine fauna were closely monitored to avoid changes to the sensitive marine environment.

**Planning Stage**

In the feasibility study, alternative alignments considered are the Northern Route, the Mid-Channel Route and Southern Route; with the Northern Route the highest IRR. However, the alignment of the Southern route was chosen as to promote socio-economic progress in the less developed south that would provide a balanced development across Penang state.

**Design Stage**

Most of the bridge sections utilized IBS and prefabricated on land to reduce the amount of time spent at sea and the risks of damaging or polluting the marine environment. The segmental box girders (SBG) were optimally design for minimum weight and lesser embodied energy by adopting higher reinforcement ratios and less but higher strength concrete [2]. The use of hybrid pre-stressing encompassing both external and internal pre-stressing increases design economy.

Corrosion protection is provided for the stay cables by using at least 3 complete nested barriers. The strands are galvanized are individually sheath inside a grease filled HDPE duct and additionally surrounded by external HDPE duct.

The steel fender system is adopted for the main Cable Stayed bridge over man-made island due to its environmental friendliness, minimal impact on water flow, cost saving and shorter construction period.

All concrete use of high performance concrete with RCPT < 800 coulombs in 56 days, concrete cover and crack width conforming to latest Eurocode requirement. Fly ash-based green cement are to be used for low temperature rises of the pile caps and piers to reduce risk of thermal cracking during concreting.

The geotechnical design of the land expressway complies with 100% primary consolidation and a settlement requirement of 50mm in 20 years. Embankments are compacted to not less than 98% of the optimum dry density using the modified Proctor test. The ground improvement scheme carried out was prefabricated vertical drains and vibro stone columns with surcharge, and piled embankment.

The bridge articulation use high density rubber bearings (HDRB) for seismic protection. HDRB use natural rubber, possess high damping properties and lower embodied energy.

All design, detailing and specification are verified by the Independent Checking Engineer (ICE) prior to construction.

**Construction Stage**

Stringent quality control in accordance to project specifications is enforced to ensure minimal maintenance. Internationally accepted best practice was adopted for the bridge construction including equipment selection and working method statements. Every two months, periodic site audit are done by the ICE.

The dredging of the 270m wide construction channel involving 14 million cubic metres of the Great Kra Flats seabed. The sludge was disposed 40km away off Pulau Kendi by barges installed with satellite tracking, trap door and draft sensor devices.

PB2X construction use repetitive steel formwork and machineries for casting of 291 nos. pile caps, piers, pylons and 8092 nos. of SBG.

A monthly environmental monitoring audit is done by and independent EIA consultant. Quarterly fisheries impact assessment for marine and fisheries resources including aquatic environment and aquaculture are also done.

**Operation and Maintenance Stage**

Structural Health Monitoring System (SHMS) shall be used for preventive maintenance to assess the state of health of the structures. The SHMS shall be develop and integrated to a Bridge Management and Maintenance System (BMS) to ensure maximum optimization of the bridge maintenance program and the reduction of maintenance costs.

## FOUNDATION AND SUBSTRUCTURE DESIGN

For the marine portion, Soil Investigation (SI) works were carried out with 205 nos. of boreholes drilled of which 50 percent were technical boreholes and 50 percent were common geological boreholes.

Bored piles of 2.0m diameter with average length of 120m were adopted for the Cable-Stayed Bridge. It enables immediate in-situ evaluation of drilled soil layers to revise foundation length due to changes in soil conditions. The bored piles have achieved a capacity of 25MN at 120m depth with 8m socketed in bedrock. Reversed Circulation Drilling (RCD) method was adopted and the total time taken is 2 weeks to complete one point for each RCD. The integrity of shaft concrete was checked using cross hole sonic login (CSL). The CSL worked with 2 pairs 60mm diameter access tube embedded in all bored piles.

The platform constructed for the bored piling works which after the piling works are lowered down together with the Steel Fender to be reused as a pile cap soffit and side formwork for the Cable-Stayed bridge pile caps. The 6m thick pile caps were cast in two layers using low heat cement and iced water pump into embedded cooling pipe system.

For the approach spans of the marine bridge, the substructures adopt a variety of piling types which are the spun piles, steel tubular piles and bored piles. Spun piles of 1.0m diameter with prefabrication length of 65m with no joints were used over most of the entire approach spans substructure. The 1.6m diameter steel tubular piles with average driven length of 80m were used in deep water areas (adjacent to the Main Navigational Span). 1.5m bored of average length 85m were used at mudflats near to mainland due to difficulty in dredging.

To minimize temporary works and in situ works, precast RC shells were used for the pile caps. The pile cap were designed to have two casting stages. The 1st layer is cast to act as a base for the installation of precast concrete shell and to act as permanent formwork for the 2nd layer pile cap construction. 518 nos. of low piers (maximum height 6m) were constructed in a single cast using one continuous set of prefabricated steel formwork from pier to crosshead. 60 nos. of high piers (>6m to 21.6m) were constructed using layers of prefabricated steel formwork.

## SEISMIC DESIGN CONSIDERATION

Second Penang Bridge area is located within the stable Sunda tectonic plate with low seismic activity level. However, this low seismic region is situated about 300-600 km from Sumatran faults which have produced earthquakes with ground motions that are felt in buildings in Georgetown, Kuala Lumpur and Singapore.

In line with the current design requirement, the Second Penang Bridge is pioneering in Malaysia to be fully designed for seismic load for a 475year return period earthquake and a 2500 year return period earthquake with 'no collapse' criteria. The seismic design was based on Design Response Spectrum from a Seismic Hazard Assessment Study conducted for the project. The seismic design criteria are as per Table 1. However, during the design review process, the ICE had highlighted that the spun piles at the approach marine bridge by Package 1 Contractor could only safely cater for the 475 year earthquake and found to be overstressed under the 2500 year earthquake event and the piles would experience section failure due to brittleness.

*Table 1: Seismic design criteria for Second Penang Bridge*

| **Return Period (yrs)** | **Peak Bedrock Acc. (PBA)** | **Peak Response Acc. (PRA)** | **Damage Performance Level** | | |
|---|---|---|---|---|---|
| | | | **Marine Bridge** | **Land Expressway** | |
| | | | | **Critical Bridge** | **Other Bridges** |
| **475** | 0.0555g | 0.1773g | Minimal damage | Minimal damage | Repairable damage |
| **2500** | 0.11g | 0.3261g | No collapse | | |

Hence, a resolution between JKSB, ICE and Package 1 designer was reached by changing the bridge articulation via introducing seismic bearing as construction was already at an advanced stage [3] Package 2 Contractor was instructed to adopt High Damping Rubber Bearings (HDRB) to replace the conventional mechanical pot bearings (see Fig. 3). HDRB has the ability to withstand large displacement in bilateral and rotational direction, durable with minimal maintenance as well as utilizing natural rubber available locally. The design was carried out by Tun Abdul Razak Research Centre (TARRC) at Brickendonbury, United Kingdom, a laboratory of the Malaysian Rubber Board (MRB).

## MARINE BRIDGE DESIGN

The superstructure of approach spans adopts SBG of 14.08m width, 4.0m length and 3.20m depth. Short line casting was selected because it does not require extensive casting facilities, special heavy lifting equipment and storage. Segments typically weighed between 69 – 100 tonnes. Early strength of 15 MPa after 10 hours is required for internal, side and cantilever formwork to be stripped. Typically on average 15 SBG are cast every day using 22 nos. of moulds. The segments are launched to the sea via barges and erected on a span-by-span using overhead self launching girder. 40 nos. complete spans of 14 segments are easily able to be completed within one month period. All segments are epoxy glued together to prevent leakage of water into the SBG.

The cable stayed bridge utilizes post tensioned concrete beam-and-slab decks. The concrete cross girders are cast in-situ with a 250mm deck slab. At the pylon the deck is built-in into the legs to provide fixed support. The semi fan layout stay cables system is designed based on parallel strand system with associated anchorages and deviation saddles of low relaxing high strength steel strands of diameter 15.7mm are arranged symmetrically in 2 planes of 18nos. of cables. The typical spacing of the cables at the edge beam and pylon is 6m and 2.525m respectively. The cables are symmetrically tensioned and anchored both at the edge beams and at the pylons. The bridge also considers the rupture of any two adjacent stay cables with a combination of 10% live load at ultimate limit state.  Corrosion protection is provided for the main tension elements by using at least 3 complete nested barriers. The strands are galvanized and individually sheath inside a grease filled HDPE duct and additionally surrounded by external HDPE duct. The erection cycle for each deck segment is typically 12 days.

## GREEN BUILDING TECHNOLOGY

The design of the Toll Plaza and Administrative Building Complex was based on requirements of 80% of IBS, 80 (Gold) for PLUS Toll Plaza and 88 (Platinum) for PB2X Toll Plaza.  The details of additional works to achieve target score of platinum are:

i. To reduce air conditioned area and maximize air conditioner set efficiency (to BEI less than 100).
ii. To reduce day lighting to less than 50% floor area
iii. To reduce internal noise level with additional insulation (to less than  dB 40)
iv. Rain water harvesting for 30% reduction in potable water consumption
v. Water efficient with no potable water for landscape
vi. Installation of metering and leak detection (FMS)
vii. To recover condensate water from air conditioners.

## CONCLUSION

The implementation of this fast-track project particularly on its construction techniques are to be exemplary and reference to other upcoming bridge constructions of its kind.  The execution of design & build concept for the major portion of the project is anticipated to produce impressive results and lead to many innovations as well as promoting a cost-effective bridge engineering and maintenance practice in Malaysia.  JKSB is committed to complete the Second Penang Bridge with the highest quality, timely delivery, within the budgeted cost and to contribute towards sustainable development.

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# PLANNING OF MODERN ROAD TUNNELS FOR THE FUTURE

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## ABSTRACT

*For the last 20 years we have seen more and more focus on tunnelling when the Authorities try to solve some of the many problems that follows from an increasing amount of traffic volume all over the world not only on the main highway networks but also in major cities. We have seen a double up of sold cars and consequently a double up of measured amount of goods to be transported on i.e. European roads within the last 10 years creating severe problems for mankind in relation to especially the environment, noise, air pollution and traffic safety.*

*By going underground with the traffic the impact on the environment and surroundings can be minimized to a reasonable level while it opens up for other possibilities on how to use the areas above and near the tunnel.*

*It is in the future inevitable that an increasing pressure on the political systems worldwide will come from its citizens, neighbours, regional and municipality authorities, environmental groups and non- government organisations (NGO) to investigate alternative solutions during planning of traditional roads in open areas. This also indicates that more and more planning activities of modern road tunnels will take place in the next decades in many countries worldwide.*

*During the last 15 years we have unfortunately experienced severe accidents in European road tunnels especially in the Alps. One major accident in the Mont Blanc Tunnel (1999, between France and Italy) caused 39 people killed from smoke when a transport caught fire in the middle of the tunnel. And lately the collapse of the tunnel roof in the Sasago Tunnel, Japan the 2nd December 2012 with 9 people killed. The Mount Blanc Tunnel accident initiated a lot of new and much more restricted legislation for planning of road tunnels in Europe among other things the appearances of the EU Directive 2004/54/EC of 29 April 2004 on minimum safety requirements for tunnels on the Trans - European Road Network (TERN).*

*The basis for all planning and design works are tunnel safety matters for the road users. Apart from that, planning of modern road tunnels is a vast variety of complex aspects to consider, and that is why it requires a systematic approach from the first phase. This paper deals with a brief introduction to a systematic approach for planning and designing of modern road tunnels considering and overcoming most challenges prepared for the future traffic situation.*

## INTRODUCTION

First of all planning of modern road tunnels is in general a combination of the following topics:

- Safety aspects
- Technical issues
- Environmental issues
- Aesthetical issues
- Economy and

- Operation and Maintenance issues

Secondly it is a complex interaction between different people with different competencies as:

- Owners (Clients, Concessionaires)
- Authorities (Politicians)
- Emergency Entities (Fire Brigade, Police, Rescue Teams, Hospitals etc.)
- Tunnel planners
- Operation and maintenance staff with experience from existing road tunnels
- Consultants, specialists within i.e. structures and installations (M&E)
- Contractors and suppliers with experience from existing road tunnels

As indicated, it requires very competent people to take the overall lead when planning of modern road tunnels takes place. As tunnel safety matters are the governing topic, tunnel planners often have a background as engineers with a combination of many years of work experience from existing tunnels in operation from an Owners organisation, where communication with many different entities and taking care of stakeholder interests are an essential competency. Planning of modern road tunnels is not a hardcore engineering job which has been misunderstood many times in the past and still is in many companies and by clients; but requires a combination of practical engineering, sound engineering judgement and heavy senior experience obtained during many years from operation of existing tunnels.

Furthermore, it is of utmost importance that the planning and design of a modern road tunnel is possible to construct by Contractors within existing technology and at the end can be approved by any authority involved. A sustainable planning and design solution that considers and covers all aspects of what is required by modern society is what professional tunnel planners are looking for to present for the decision makers. And not to forget; the tunnel must be planned for the future taking all prognosis, forecasts and developments into consideration.

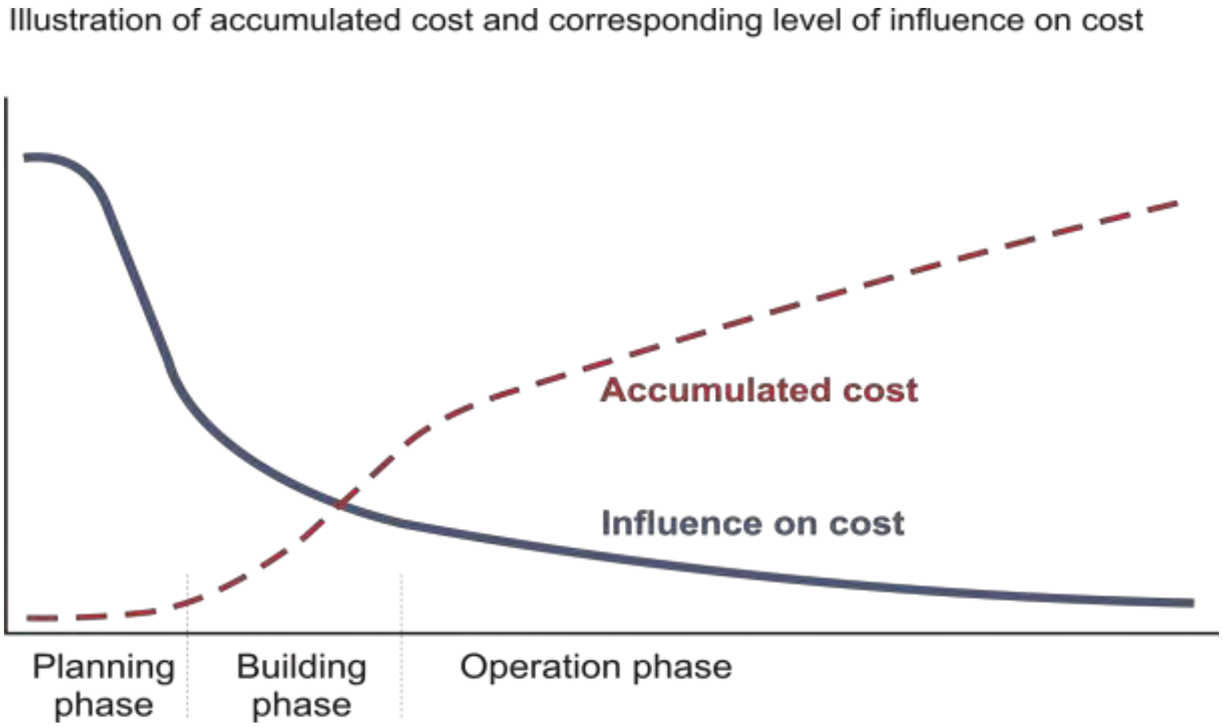



*Figure 1: PIARC model. Relations between accumulated costs and levels of influence on costs for different phases.*

Figure 1 shows the international well know relation between accumulated costs and the level of influence on costs for different phases in a tunnels lifetime coming from the World Road Association (PIARC).

As little as 5 % of future costs are used during the Planning Phase of a tunnel; but the influence on total lifetime costs is 60 – 70 %. This demonstrates the importance of taken tunnel planning seriously and into account during design. The decisions in this phase are thoroughly investigated and verified using a considerable amount of time, which comes as a big surprise to many tunnel owners. The figure also shows that a good planning has a major impact on future operation and maintenance costs and thus may reduce the total lifetime costs, if properly planned for. A wrong decision or lack of necessary investigations in the planning phase of related issues to the design, construction or operation may have a serious impact on later safety matters for the road users as well as on costs for construction and operation and maintenance during the tunnels lifetime.

## SAFETY PHILOSOPHY AND SAFETY CONCEPT

The international statistics show that the amount of incidents and accidents occurrences are less in road tunnels than on open roads. This means that the risks for road users are less in road tunnels, if planned, designed and constructed the right way, than for similar open roads having the same traffic volume and composition. However, when severe accidents occur in a tunnel the consequences can be much more serious than on an open road. This maybe due to the following:

- The road user's possibilities for escaping the tunnel and the rescue team's possibilities to get close to the place of accidents are reduced
- Accidents leading to fire in the tunnel will create severe consequences for road users and tunnel material. For the road users caught in a fire the most important danger is the development of smoke and its gasses that may prevent or make it difficult evacuating people and at the extreme lead to poisoning and suffocating.

It is thus of utmost importance in the initial phase of a road tunnel planning to define and elaborate a Safety Concept based on a Safety Philosophy.

Typically in Europe the basic Safety Philosophy behind modern road tunnel planning is:
- To establish basic rules ensuring a uniform high safety level for the users when driving in a tunnel nationwide
- To establish at least the same safety level in tunnels as on open roads

The idea behind this philosophy is to ensure that the road user will not feel any difference in traffic safety when driving on open roads or in tunnels no matter where in the country in question or between countries.

A Tunnel Safety Concept has the overall goal to plan, construct, operate and monitor the tunnel to avoid accidents. In case of any accident, catastrophe/disaster the tunnel shall be planned and designed to ensure:
- Evacuation of people in the tunnel
- Access for Fire Brigade and Rescue Teams
- Limit to a minimum the size of fatalities and injuries
- Limit to a minimum damages on structures and installations to maintain the passability for the road traffic as fast as possible after an incident or accident

- During sectioning of installations and systems for monitoring ensure that a catastrophe only has partial consequences on lighting systems, emergency telephones, ventilations systems, power supply, traffic management systems and fire fighting systems
- Availability of necessary safety measures and equipment to facilitate that persons involved in accidents or incidents can do self-rescuing and/or prevent severe consequences for the structure


*Figure 2 and 3: Safety Concept must define all aspects to avoid severe tunnel fire.*

The Tunnel Safety Concept is in many countries defined by the political system and approved by authorities involved in tunnelling issues. The Concept and its consequences are detailed outlined and described in a document, which will be the governing document for the tunnel from planning to operation of the tunnel.

The Tunnel Safety Concept document is generally based on as a minimum:

- National Legislation, Standards, Codes and Guidelines to be followed
- EU Directive 2004/54/EF of 29. April 2004 which states the minimum safety requirements to road tunnels on the Trans – European Road Network (TERN)
- International Standards, Codes and Guidelines from i.e. PIARC (World Road Association) and ITA (International Association for Tunnels and Underground Works)
- Experience/lessons learned from existing tunnels on national roads
- Standard and Codes from neighbouring countries


*Figure 4 and 5: Full Scale Emergency Exercises in road tunnels.*

The Tunnel Safety Concept document includes a detailed description of i.e.:

- Organisational set-up, staffing, skills, defined responsibilities and roles according to i.e. EU Directive
- Requirements and responsibilities for: Administrative Authority (Article 4), Tunnel Manager(Article 5), Safety Officer (Article 6) and Inspection Entity (Article 7)
- Planning Phase (i.e. safety documentation, Annex II, item 2)
- Construction Phase (i.e. ensure correct implementation of safety by Safety Group)
- Operation Phase (i.e. ensure correct Operation & Maintenance of all safety aspects)
- Authorities involved (i.e. Owner, Police, Rescue Team, Fire Brigade, Hospitals etc.)
- Emergency plans, procedures and instructions for all entities for selected scenarios
- Plan for execution of emergency exercises periodically (EU Directive, Annex II, item 5)
- Safety Analysis and Documentation (EU Directive, Annex II)
- Quantitative Risk Assessment (QRAM) and Operational Risk Assessment (ORA)

The Tunnel Safety Concept document forms the basis for requirements to structures, mechanical and electrical installations (M&E) including Intelligent Transportation Systems (ITS).

## CLASSIFICATION OF ROAD TUNNEL

As an easy, simple and well proven approach in the initial phase for planning of minimum cross section design and its basic elements, the tunnel capacity and the minimum requirements for safety equipment and installations, it is recommended to make a classification of the road tunnel. Basic data needed for tunnel classification is the Average Annual Daily Traffic (AADT) 20 years ahead from the opening date plus the total length of the tunnel. Traffic data to be used comes as outputs from Traffic Analysis and its forecasts.

A typical tunnel classification is shown in figure 6 and 7 using the Norwegian classification system from Håndbok 021 (2010). Knowing the AADT (y - axis) and the tunnel length (x – axis) the tunnel class is found to be one from A to F in the diagram. For each class follows a simple cross section design proposal.

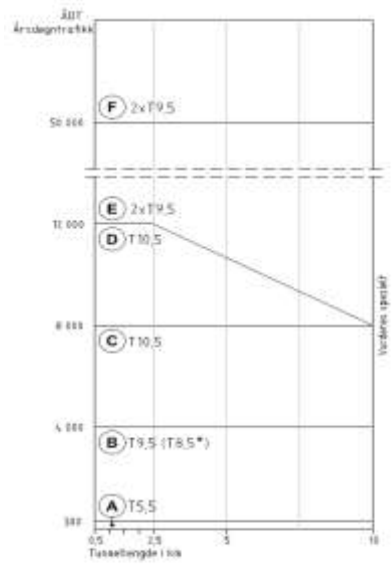



*Figure 6: Tunnel Classification (Norway)*

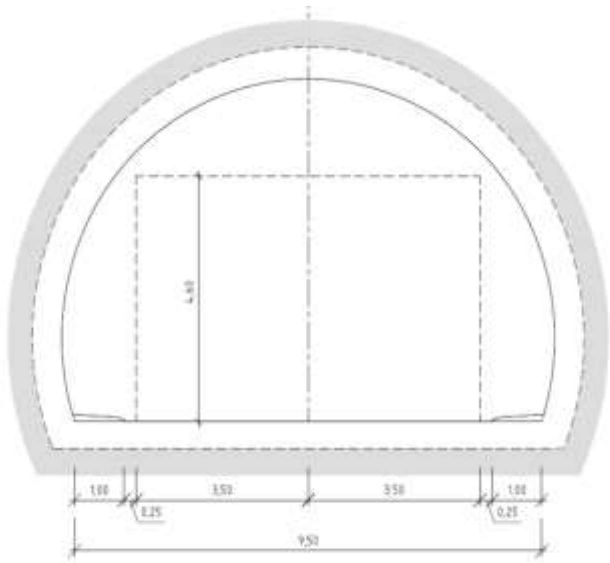
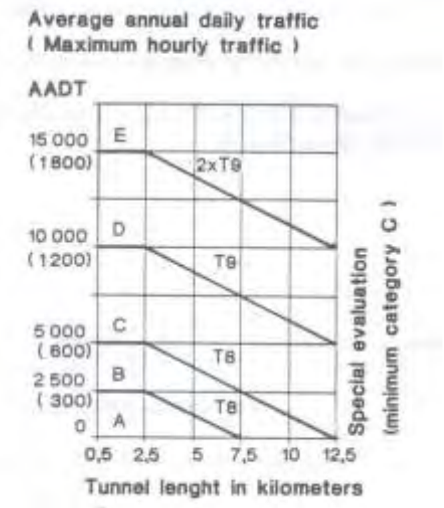


*Figure 7: Corresponding Cross Section design (E, F) for each tube.*

Knowing the tunnel class a corresponding method exists to determine the minimum safety equipment to be installed in the tunnel. In figure 8 and 9 is shown the PIARC model for tunnel classification and minimum equipment. Other classification systems to be recommended for use are the ones in British Standard (BS) and the Tunnel Standard from Sweden (TRVK and TRVR 2011).




| EQUIPMENT | TUNNEL CLASS A | B | C | D | E | REMARKS |
|---|---|---|---|---|---|---|
| Emergency evacuation lights | | ○ | ○ | ● | ● | Ca 60 m between lights |
| Fire extinguishers | ● | ● | ● | ● | ● | Class A,B : Every 250 m<br>C,D : Every 125 m<br>E : Every 60 m |
| Fire hydrant | | ○ | ○ | ○ | ○ | To be evaluated in cooperation with local fire brigade |
| Emergency telephone | ○ | ● | ● | ● | ● | Class A Every 500 m<br>C,D Every 250 m<br>E : Every 100 m |
| Emergency road signs | ● | ● | ● | ● | ● | Signs showing nearest exit / emergency exit.<br>To be installed by the side of fire extinguishers |
| Blinking red light | | ○ | ● | ● | ● | Directed by CO measurements or tunnel control staff |
| Barriers ( manual ) | ○ | ● | ● | ● | ● | Automatic barriers if AADT > 10 000 |
| Variable text signs | | | ○ | ● | ● | |
| TV servaillance | | | | ○ | ○ | To be evaluated in each case |
| Radio communications | ○ | ○ | ● | ● | ● | |
| Radio telephone | ○ | ○ | ○ | ○ | ○ | To be decided in cooperation with Telegraph office |
| Emergency power | ○ | ○ | ○ | ○ | ○ | To be evaluated in each case |
| Vehicle height control | ○ | ○ | ○ | ○ | ● | To be used in class A – D with clearance less than 4.8 m |

*Figure 8: Road Tunnel Classification (PIARC)*   *Figure 9: Minimum equipment (PIARC)*

The EU Directive also gives in tables in Annex I, item 2.19 informative summaries of minimum requirements to road tunnels.

This simple approach gives the planning engineer in an early phase an idea of how the cross section elements may look like; but not all information for making the final cross section geometry design including minimum safety equipment to be installed in the tunnel. In some countries exists a road tunnel catalogue for directly use based on the same principles as behind the tunnel classification system.

The classification system gives information on whether the tunnel shall be a bidirectional (two way traffic) or unidirectional (one way traffic) road tunnel; but does not give the planning engineer exact information on capacity and geometry with respect to road user safety on modern premises. This indicates that other factors must be considered for cross section design as i.e.:

- Traffic volume and its intensity over time during the day (congestion problems)
- Traffic composition as i.e. percentage of heavy transport, amount of dangerous goods transports in the tunnel
- The tunnel location, i.e. urban or rural area, over or under water
- The road class for the approach roads
- Tunnel length and complexity as number of exits/entrances and number of conflict points
- With or without emergency lanes
- Special road users as disabled people/handicap persons (EU Directive, Article 12)
- Scenarios for typical accidents and its consequences (requirements from safety risk analysis)
- Requirements from Emergency Entities (ventilation, escape routes, road signs/signals)
- Requirements to Operation and Maintenance activities
- Future plans for the road network the tunnel is a part of

A modern road tunnel is always for safety matters planned as a unidirectional tunnel for AADT ≥ 20.000 as a general rule and as such in full compliance with the EU Directive. In many countries it is standard that all road tunnels are designed as unidirectional with a minimum of 2 lanes in each tube based on safety, operation and maintenance matters only.

The Tunnel Safety Concept and decision on tunnel class give input to design requirements for 1) Structures, 2) Mechanical & Electrical Installations, 3) Safety equipment and monitoring systems and 4) Operation and Maintenance activities.

## CROSS SECTION GEOMETRY OF ROAD TUNNELS

Based on the tunnel classification and all the other factors to be considered as mentioned earlier, it is possible to make a draft final cross section design for a modern road tunnel with due respect to national and international codes, standards and guidelines. The final decision is always based on a technical – economical evaluation approved by the Authorities and verified by Safety Risk Analysis complying with the Safety Concept.

In brief shall be highlighted some very important cross section elements and its recommended geometry for modern road tunnel planning and design not included in the principles for classification or seen in catalogues.

The width of carriageway in tunnels is in Europe normally set to minimum 3.50 m. In a few countries is today used 3.75 – 4.00 m as standard. The edge lane is recommended 0.50 m for safety reasons. Crash barriers as New Jersey type is recommended to minimum 0.40 m in width.

Sidewalk is established along both tunnel walls as absolutely minimum of 1.00 m. For escape reasons, for placing of fixed signs and information panels, and for being able to open the SOS cabin doors comfortably during an emergency with traffic still moving, it is recommended widening the sidewalk to 1.25 – 1.50 m or to use a wide shoulder instead of 1.75 m as minimum according to recommendation from PIARC. The shoulder will serve as a place to temporarily park a normal vehicle during breakdown. PIARC recommends a shoulder width of 3.25 if a Heavy Good Vehicle (HGV) is suppose to be parked in the tunnel safely. In modern road tunnel design the shoulder is typically placed at the same level as the asphalt of the road carriageway.

Whether to design for an emergency lane or not is normally based on Safety Risk Analysis where also economical aspects are considered. The width of emergency lane varies typically from 2.50 – 3.25 m depending on national standards for open roads. PIARC recommends as minimum 3.25 m, but an emergency lane ≥ 2.50 m is always seen as a major improvement of the overall tunnel safety for the road users.

The vertical clearance above carriageway is for new road tunnels designed as minimum 5.00 m with the tendency today of 5.50 m. Sidewalks and shoulders are best design for the same vertical clearance as the carriageway even 4.50 m is seemed adequate in some countries. Room for ventilation systems, information panels, fixed road signs and signalling etc. requires mostly 1.00 - 1.50 m bringing the total vertical height up to typically minimum 6.50 - 7.00 m in the tunnel.

*Figure 10 and 11: Modern cross section design for road tunnels in Stockholm. Source: Trafikverket.*

More detailed recommendations on cross section elements and its geometry are to be found in the PIARC publications:

- Cross Section Geometry in Unidirectional Road Tunnels, Paris, 2001
- Cross Section Design for Bidirectional Road Tunnels, Paris 2004

## ALIGNMENT

Briefly shall be mentioned an important criteria to comply with due to safety reasons and very often neglected or overruled in the planning phase for economical reasons.

A longitudinal gradient above 5 % shall not be permitted in new road tunnels. In tunnels with gradient higher than 3 %, additional and reinforced measures shall be taken to enhance safety on the basis of risk analysis. All international experience shows the importance of not exceeding these limits due to especially the risk of backend collision and the increasing amount of breakdowns of HGV. These are two of the most common types of accidents/incidents in most road tunnels. All cross fall of pavements most at least be 2.00 %.

The vertical and horizontal alignment must be established for a design speed much higher than the allowable actual informed speed in the tunnel. It is commonly recommended to have an allowable speed of 85 – 90 % of the design speed. For maximum speed in road tunnels is recommended a limit of no more than 90 km/hour, which is widely accepted.

Finally it is important that the alignment of tunnel and ramps fits into the location in an aesthetical optimum way with due respect for the environment.

## SERVICE GALLERY

In tunnel types as Cut – and – Cover and Immersed Tunnels the tendency is to establish a service gallery between the two tubes. The objectives for such a service gallery are i.e.:

- Improved conditions for Operation & Maintenance (may also prolong/improve the lifetime of installations)
- Can be used as escape route in case of an emergency (not primarily escape route)

- Improved protection of mechanical and electrical installations in case of i.e. fire in the tunnel

The effective width of a service gallery is recommended to be ≥ 1.50 m from experience and for giving necessary room for working conditions.

## CONTROL AND ALARM EQUIPMENT

Modern new road tunnels are planned with a comprehensive amount of technical installations to control and monitor the tunnel and traffic conditions at all times for complying with the Safety Concept in the operation phase. The following equipment and technical installations mentioned are all installed as standard in road tunnels today and connected to the SCADA system initiating alarms to the emergency entities:

- SCADA System (Supervision, Control and Data Acquisition), overall PC based system for monitoring
- CCTV (Closed Circuit Tele Vision), Internal TV system by mounted cameras in tunnel area
- IDS (Incident Detection System), detection of stopped/slow driving vehicles/lost goods
- SOS Cabins equipped with minimum emergency telephone, fire alarm button, fire extinguisher
- Automatic fire detection systems (IDS, linear heat detection systems)
- Instruments for measuring visibility in the tunnel area (and the amount of particles)
- Instruments for measuring gas concentrations and emissions from vehicles ($CO$,$NO$,$NO_x$) in closed tunnel
- Sensors for doors, alarm boxes, fire extinguisher
- Device for checking over height vehicles before entering the tunnel

The SCADA system is the most essential part of the equipment installed in the Control Room. Furthermore, systems for management of Operation and Maintenance activities, Winter Maintenance, monitors from cameras (CCTV and IDS) and ITS systems can be managed and monitored 24 hours a day by the operational staff from the Control Room.

*Figure 12 and 13: Typical manned tunnel control room for monitoring of tunnel safety and operation activities.*

## ESCAPE ROUTES AND COMMUNICATIONS

To ensure the road user's safety at all times during the normal operation situation, and if severe incidents or accidents occur as i.e. fire in the tunnel, a number of facilities are installed to communicate with people and, if necessary, to empty the tunnel for people as fast as possible to a safe place in or outside the tunnel.



*Figure 14 and 15: Examples of escape route, signs and LED lights used in modern road tunnels.*

The following normal facilities mentioned are all standard in modern road tunnels today:

- Emergency doors/escape doors between bores (cross passages for bored tunnels)
- Emergency telephones (SOS Cabins)
- Radio transmissions
- Provisions for mobile telephone
- Communication equipment by cameras (IDS and CCTV)
- Loudspeakers (most be considered carefully due to noise level in tunnel, may not be cost-effective)
- Escape route signs
- Smoke free escape routes
- Emergency and escape route lighting (evacuation lighting)
- Variable message signs (VMS) and Information Boards

## FIRE FIGHTING AND ANTI POLLUTION EQUIPMENT

If the worst case scenario happens, a fire in the tunnel, modern road tunnels are all well equipped to prepare the road users for self rescuing up to a certain level plus a comprehensive amount of installations to be provided and used by the Fire Brigade during fire fighting. Furthermore, systems are installed to extract smoke and other dangerous emissions and provide fresh air in the tunnel during both normal operation and if a fire occurs. In the following is mentioned different types of mostly standard equipment installed in modern road tunnels:

- Hand operated fire extinguisher
- Fire hydrants
- Water reservoir or water main through the tunnel
- Fire hose coil with supply of special treated water
- Sprinklers, water mists and deluge systems (not cost-effective, only installed based on risk analysis)
- Closed drainage system
- Explosion safe pumps in tunnel basins
- Air purification systems (electrostatic filters are still not standard)
- Reversible tunnel ventilation systems as jet fans or semi/full transversal systems
- Prepared action plans for emergency staff

- Fire fighting engine

## OTHER REQUIREMENTS AND EQUIPMENT

To ensure functional safety at all times for the road users, it is important that structures, mechanical and electrical installations are protected and systems are provided to optimize the traffic flow during normal and restricted situations as i.e. periods with congestions, maintenance activities and severe incidents and accidents. The following requirements and equipment are most commonly planned for in modern road tunnels:

- Functional safety at all times
- Uninterrupted power supply (UPS systems)
- Tunnel lighting system including evacuation lighting system
- Reversible ventilation in case of fire
- Wall panels for i.e. aesthetical and safety reasons, crash barriers for impact protection
- Fire protection on roof and walls (fire board panels, special concrete)
- Traffic regulation installations (ITS systems)
- Queue warning system
- Road user information systems
- Speed limitation systems
- Reversible lanes/cross over possibilities using lane signal panels
- External electrical power supply from both tunnel entrances

*Figure 16: Reversible traffic and cross over.*

*Figure17: Preparation for cross over during night time.*

## REQUIREMENTS TO OPERATION AND MAINTENANCE ACTIVITIES

The tunnel must always be planned, design and constructed to ensure that all aspects of the safety concept are complied with during the later operation and maintenance phase, the longest phase in the tunnels lifetime. This requires implementation of a systematic approach to all operation activities and that maintenance works are properly planned for in an early phase. A typical operation and maintenance system to be planned for consists of as minimum:

- Control of functionality of all integrated systems 24 hours a day all year around
- Cleaning and tunnel washing activities
- Visual inspection systems
- Function control of equipment, monitoring and alarm systems periodically

- Systematic planned service works are optimised during execution
- System condition verification is registered for monitoring
- Emergency exercises (full scale) are planned, organised and executed systematically
- Training of operation staff continuously

PC based systems are commonly used for management of operation and maintenance activities. These systems are best integrated into the SCADA and Traffic Management systems.

*Figure 18: Tunnel washing by tunnel washing truck*

*Figure 19: Maintenance works of tunnel ventilation*

## TECHNICAL REQUIREMENTS IN GENERAL

Based on the approved Tunnel Safety Concept document a huge amount of general requirements are described in more details for all structures and installations during the planning phase for later on to be fully implemented during the design and construction phase.

For structures typical decisions on requirements are taken and further elaborated during the planning phase:

- Decide on tunnel type, shape, statically behaviour, foundation issues, joints, lining and cladding system etc.
- Decide on material to be used as concrete type and steel quality, cathodic protection etc.
- Design requirements to ramps, approaches etc. follows the main tunnel
- Design requirements to portals, technical buildings, use of staircases and elevators etc.
- Type of joints, asphalt pavements, drainage systems, basins, pumps sumps etc.

For equipment and installations to be implemented in the tunnel as mentioned earlier, decisions on requirements to be elaborated are typically based on:

- All kind of equipment and installations must be resistant to the design fire (2 hours)
- During a fire all equipment and installations must be designed to maintain necessary functionalities to minimize the risks of a disaster or a catastrophe
- All technical equipment and installations must be sectioned trough out the tunnel meaning that if a severe incident or accident occur only a part of the tunnel will be without safety functionality
- Fire in one tube must not spread to the other tunnel
- All equipment and installations requirements must be specified for functionality during normal operation, in case of fire and during severe failure of essential parts

Necessary precautions most be taken to prevent any kind of vandalism or terror attack in the initial planning phase.

The documents containing tunnel safety concept and general requirements to structures and installations create the basis for all planning activities, conceptual studies, detailed design works, construction works and later how to operate and maintain the tunnel.

*Figure 20 and 21: Technical requirements to prevent tunnel in operation from collapsing as in the Hanekleivtunnel, Norway 2006.*

## SAFETY RISK ANALYSIS

The planning works, conceptual design studies and later detailed design works must always comply with the Tunnel Safety Concept. This is verified by as a minimum:

- Performing Risk Assessment Studies and Safety Analysis (ref. EU Directive, Annex II)
- Performing Operation Risk Assessment Studies (for conceptual and detailed design plus when ever needed during the operation phase (ref.EU Directive, Article 13 and Annex II)
- Performing study of “Transportation of Dangerous Goods” through the road tunnel (EU Directive, Annex I)
- Construction Risk Analysis
- Other safety related documents as requested by the Authorities and Emergency Entities

If the design is not in compliance with the Tunnel Safety Concept the design must be revised.

## CONCLUSIONS

It must be foreseen that an increasing number of road tunnels will be constructed worldwide in the next decades trying to solve the increasing amount of problems on the environment and traffic safety that subsequently follows from an increasing amount of transportation works on our roads. It most also be foreseen that an increasing pressure to go underground with the road traffic will come from citizens, neighbours, authorities, environmental and non – government organisations especially in urban areas.

Planning of modern road tunnels is a vast variety of complex aspects to consider. It is therefore of utmost importance that a systematic approach from the initial phase is introduced. This paper has dealt with an overall general introduction on how to consider and overcome most challenges for future road tunnels.

It is to be emphasised that a well prepared Tunnel Safety Concept with subsequently technical requirements to the tunnel establishes the best tool for the planning process and management of modern road tunnels – from planning to operation phase – following the national legislation, standards, codes and guidelines at all times. A well planned road tunnel gives the owner the optimum technical and economical solution during a tunnels lifetime considering all aspects involved.

It is the experience that the EU Directive is best used as a checklist before all documents are to be completed for the approval process. The Directive will be updated within short time.

Finally, it is the experience that in spite of the fast technological development within especially mechanical and electrical installations and equipment using very advanced IT technology, the technical requirements coming from authorities and emergency entities involved in road tunnel planning and design, with due respect to road tunnel safety, is going even faster, higher and more tighten compared to the actual technological development. This will in the future require all tunnel planners to be even more creative looking for sustainable alternative solutions for overcoming all the challenges beneficial for the road tunnel users.

## SELECTED REFERENCES


(1) PIARC, Cross Section Geometry in Unidirectional Road Tunnels, Paris, 2001
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(3) Håndbok 021, Norwegian Public Road Authority, 2010
(4) TRVK and TRVR Tunnel 11, November 2011, Trafikverket, Swedish National Road Authority
(5) EU Directive 2004/54/EC of the European Parliament on minimum safety requirements for tunnels in the trans - European road network (TERN)

# DEVELOPMENT OF SHEATHED STRANDS FOR BRIDGES AND PERMANENT MOORING APPLICATIONS

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## ABSTRACT

*This paper reports about developments in the corrosion protection of spiral strand and locked coil strand with plastic sheathing. These strands can be used in all types of rope supported bridges (eg cable-stayed, suspension etc). They are also used as permanent mooring ropes for floating bridges and offshore platforms. In this application the ropes are used under water and anchor floating platforms to the seabed at depths of up to 1500 metres.*

*In both applications, corrosion protection is a primary requirement. This paper describes the historic steps in the development of corrosion protection from the use of bright wire to galvanising with Zn95/Al5 and additional sheathing and sets these into the context of the requirements of Norwegian Handbook 122 for bridge cables.*

*Sheathed spiral strands have been used for permanent offshore moorings since 1980's. However the improved corrosion protection of the sheathing is still only used occasionally for bridges and has not found its way into standards yet. Sheathed locked coil strand for bridges combines technical and commercial advantages as an alternative to paint coatings. Scaffolding, the lengthy painting process and dependency on weather conditions are eliminated. This enables shorter construction times and can help to keep costs down.*

*Project references and experiences with these tension members are discussed including the hanger cables for the iconic Lusail Bridge in Qatar and the mooring ropes for an offshore project which had the additional requirement of strakes on the sheathing to disturb water currents.*

## PRODUCT DESCRIPTIONS

Spiral and locked coil strand assemblies are prefabricated tension components for structural applications [1]. The assemblies consist of the strand and permanently attached sockets.

Spiral strands are made up using helically spun round wires in several layers (Figure 1a), usually closed in opposite directions to minimise the residual torque. Locked coil strand are a special type of spiral strand onto which covers of helically z-shaped wires in several layers are added (Figure 1b). Spiral strand and locked coil strand have been made by Bridon in diameters up to 180mm.

**(a)** **(b)**

*Figure 1: Rope constructions for structural applications, (a) Spiral strand, (b) Locked coil strand.*

Spiral strand and locked coil strand are made to the product standards EN 12385-10 [2] and, where specified, to Norwegian Handbook 122 [3]. Additionally, spiral strands are described in ASTM A475 [4] and locked coil strands in German TL Seile [5]. They are used in conjunction with structural design standards like Eurocode 3 Part 1-11 [6] and ASCE 19 [7]. Spiral strand and locked coil strand assemblies are factory made including the attachment of the sockets. The manufacturing takes place in a fully controlled environment with established methods reducing quality sensitive on-site work to a minimum. This gives the basis for a consistent and monitored quality. Factory fabricated cable assemblies arrive on-site ready for installation.

Layers with z-shaped wires lead to a higher cross sectional metallic area compared to spiral strand with round wires. Strands with more than 3 layers of z-shaped wires have a metallic cross section of minimum 88%, which is considerably higher than the value for any other wire based tension components. In architectural design, this means that strand diameters can be minimised for a design minimum breaking load. Spiral strand typically have 73% metallic cross-section. The difference in metallic cross-section leads directly into a difference in axial stiffness and if the same wire tensile grades are used there is also a difference in minimum breaking load. As round wires can be manufactured in higher tensile grades than z-shaped wires this usually makes up for the difference in terms of strength. The nominal grade of z-shaped wires is 1570N/mm$^2$. Round wires in Handbook 122 are also restricted to 1570N/mm$^2$. EN 12385-10 [2] and Eurocode 3 Part 1-11 [6] allow the use of 1770N/mm$^2$ for round wires. Spiral strands for the use in permanent offshore mooring lines use round wires up to 1960N/mm$^2$.

The axial stiffness is of major interest to bridge designers. It is the product of the elastic strand modulus and the metallic cross sectional area. The strand modulus is a reflection of the wire material modulus and of the helical wire arrangement in the strand. The strand modulus can be influenced by the manufacturer by the choice of helix and usually lies between 155kN/mm$^2$ and 170kN/mm$^2$. For structural applications, locked coil strands are often preferred over spiral strand because of superior corrosion protection, the higher axial stiffness and the better clamping capabilities.

The failing of strands submitted to fatigue loading is fundamentally different to that of conventional steel structures where almost the whole fatigue life of the structure is related to the period before the first crack occurs. In contrast to this, the first wire crack in a strand has no relation to the subsequent failure of the complete strand much later. In locked coil strand, the interlocking full lock wires avoid the popping out of a broken wire in the outer layer. Due to the spiral assembly a broken wire can carry full load again after approximately three lay lengths (~ 3 x 10 x rope diameter).

Eurocode 3 Part 1-11 [6] assigns the detail category $\Delta\sigma_c = 150N/mm^2$ to locked coil strand. When tested this usually happens with an upper load of 45% of the minimum breaking load, a variation in stress of 150N/mm² and 2 million load cycles.

The blocking compound added during spinning and the galvanizing of all wires reduces the inter-wire friction. These measures together with enhancements in steel making and wire drawing have improved the fatigue performance in the past decades. Recent test results show a remaining breaking load after the cyclic loading of more than 90% of the design breaking load.

## CORROSION PROTECTION

The major cause of deterioration of bridge cables is corrosion. Spiral strand and locked coil strand have multiple corrosion barriers; (i) Galvanising of all wires, (ii) Filling the inter wire gaps with a blocking compound, which is a corrosion inhibitor, during rope manufacture also prevents the intrusion of corrosive media into the rope in service, and (iii) an additional painting of the cables with specially approved paint systems can be used for aggressive atmospheres. In locked coil strands the z-shaped wires themselves provide an additional effective surface barrier against the penetration of corrosive media because of the interlocking of the z-shaped wires.

Wires are usually hot dip galvanized in a molten zinc bath with a minimum coat weight of 300g/m². Wires can also be galvanized with Zn95/Al5 through double dip galvanizing process for improved corrosion resistance by approximately a factor 2 to 3 compared to the equivalent coat weight of zinc. Zn95/Al5 is usually used in the two outer layers of a locked coil strand and for all wires in spiral strand. ASCE 19 [7] and Eurocode 3 [6] specify Zn95/Al5 galvanized wires as an option. This type of galvanizing is often referred to as Galfan® which is a registered trademark of the International Lead Zinc Research Organization (ILZRO). Galfan® was developed in 1980; the first application in spiral strand and locked coil strand for structural applications was in 1991.

Several paint systems for spiral strand and locked coil strand assemblies exist for structural applications. This includes single layer to three layer systems and some require special surface preparation e.g. sandblasting. Additional corrosion protection by paint is applied on manufacturing site to prevent any damage from occurring during transportation and handling. External corrosion protection by paint requires moveable scaffolding or fixed scaffolding over the entire length. This results in significant additional costs and the progress of work is dependent on weather conditions.

Polyethylene Sheathed spiral strand is a key product for long term permanent mooring applications where design lives exceed 20 years. Previous experience suggests that polyethylene sheathing of greater than 7mm is sufficiently impermeable to vapour and offers full corrosion protection for the design life of the cable.

The advantages of polyethylene sheathing of ropes for structural applications are manifold. Unlike paint systems, the sheathing is applied in the factory. The time, cost and weather dependency associated with applying a paint system are eliminated. Like paint systems, sheathed ropes can be made in different colours to suit the architectural design. The plastic sheathing meets the full design life of the rope and does not need further maintenance whereas paint systems need refinishing or to be re-applied periodically.

## SCOPE OF SUPPLY FOR LUSAIL BRIDGE

The development of a sheathed locked coil strand for structural application followed an enquiry from Samsung C&T Engineering and Construction Company to supply suspension and sheathed hanger full locked coil ropes for the Lusail bridge contract in Qatar. A Danish team of Dissing+Weitling architects in cooperation with COWI have designed two bridges which – with their clear reference to Qatar's "Q" – have become a Lusail icon (Figure 2). Two steel rings 50m above the sea serve as the pylons for this suspension structure with a main span of 200 m and a width of 28 m. The girders are carried by a main cable consist of 6 locked coil strand with a diameter of 140 mm and 43 sheathed locked coil strand with a rope diameter of 70 mm. Sheathed locked coil strand for hangers was specified to increase corrosion protection due to the atmosphere in this region.

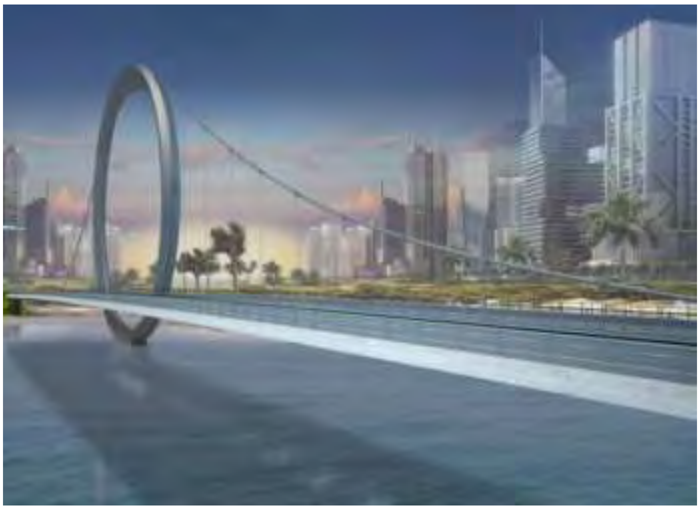


*Figure 2: Artist's impression of the proposed suspension bridge in Qatar [8].*

The requirements of sheathing such as thickness, colour, UV resistance, physical and mechanical properties was in conformance with the technical specification for the Lusail bridge contract, which required 70mm locked coil strand sheathed to a finished diameter of 82mm. The test also included pre-stress measuring and marking of the rope in advance of the sheathing process, and subsequent socketing and testing of the completed sheathed rope assembly.

## SHEATHING MATERIAL AND PROCESS

Pressure grade High Density Polyethylene (HDPE) optimized with additives was used in the sheathing trials as it offers a cost effective solution with a good combination of physical and mechanical properties. Long term exposure to sunlight can cause plastic to degrade over time, with the rate of decay accelerating over time. A compatible UV stabiliser was added to the plastic during the sheathing operation to provide good colour stability and prevent or slow down the degradation of plastic caused by in-service exposure to UV radiation and temperature. The critical properties for the grade of HDPE used in the sheathing trials are listed in Table 1.

*Table 1: Properties of the HDPE sheath.*

| Property | Value | ASTM test method |
|---|---|---|
| *Density (g/cm$^3$)* | 0.95 | D 1505 |
| *Flexural modulus (N/mm$^2$)* | 700 | D 790 |
| *Tensile elongation at break* | 600% | D 638 |
| *Hardness, shore D* | 64 – 65 | D 2240 |
| *Low temperature brittleness* | - 40˚C at 50% flexibility (min) | D 746 |

Schematic of the extrusion line arrangement as used for sheathing the locked coil strand samples is shown in Figure 3.

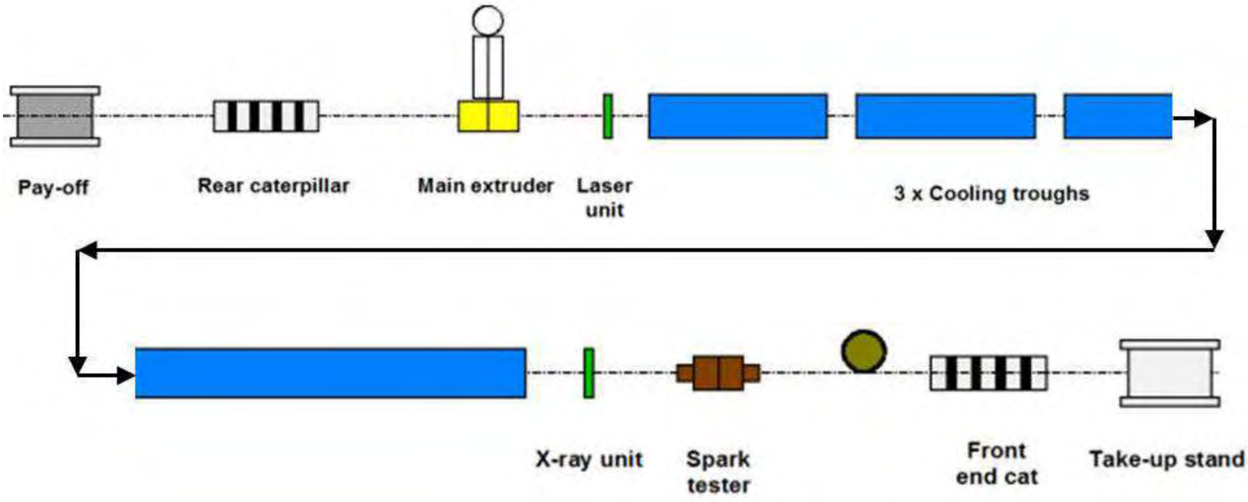



*Figure 3: Schematic of the extrusion line setup for sheathing locked coil strand.*

In the case of plastic sheathing of spiral strand or locked coil strand, molten plastic at the exit of the crosshead is drawn down on to the rope with the aid of vacuum in the crosshead as shown in Figure 4.

*Figure 4: "Tube down" of the molten plastic.*

Vacuum extrusion enables the plastic melt to fill the interstices between wires in the outer layer of the rope (Figure 5).

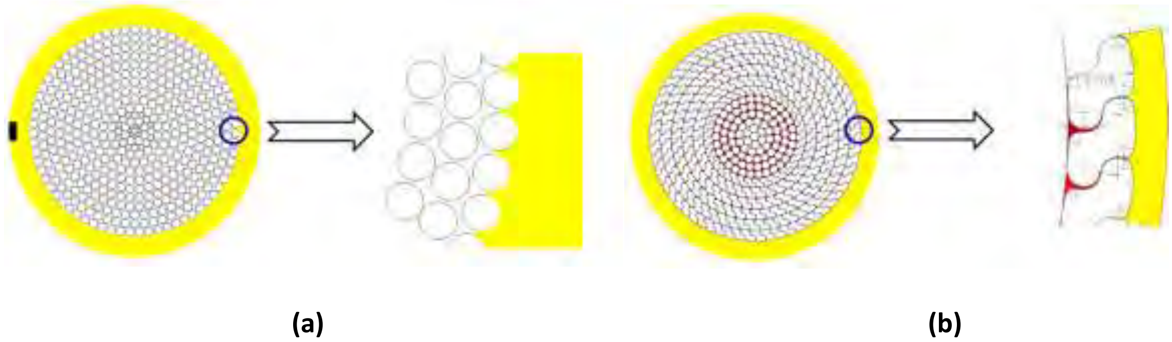



*Figure 5: Comparison of gaps between wires in the outer layer for; (a) spiral strand; and (b) locked coil strand.*

The Tube down extrusion technique, where the molten polymer melt contacts the rope outside the die, was used. This technique offered advantage of a concentric sheath around the rope irrespective of any movements in the rope during the extrusion process. During the sheathing operation, the finished diameter and sheath thickness was continuously monitored at four individual points around the circumference, ensuring accurate concentricity along the entire manufactured length. Further in-line NDT techniques were used to guarantee no pinholes or gaps are present within the sheathing. This is necessary to ensure no water ingress which could lead to subsequent corrosion.

Reference tapes on the rope surface are visually reflecting localised variation in the thickness of the sheathing in the finished rope. This aided identification of length marks on the sheathed rope. The sheathing enters the socket and the joint between the rope and socket was sealed with a heat shrink boot as an additional measure to prevent corrosion in service.

## TESTS

### Sheathing adherence

Plastic sheathing can slip relative to the rope in either of two modes: axial or helical slip. For the sheathing to develop resistance to axial slip, it must be constrained from rotation relative to the cable. The corrugations in the inside of the sheathing provide significant resistance to inhibit slip between the sheathing and the rope at any stage. However if the sheathing is not constrained from rotation, it will preferentially slip helically along the helical lay of the outer wires or shapes in the rope. The corrugations play little or no role to constrain the sheath from helical slip since they follow the valleys between the wires or shapes.

### UV test on sheathed plastics

As part of this project a UV test was completed. Ten standard tensile dumbbell samples conforming to the ASTM D638-03 [9] were machined from the plastic used for sheathing. The samples were aged in an accelerated weathering tester. This test uses a combination of heat, moisture and ultraviolet light to accelerate the degradation process in the plastic.

The radiation energy level in the chamber was set to approximately 180kLy/year to replicate one year of continuous outdoor exposure in a desert. The dumbbell samples were exposed to 20 hours light cycle at 60˚C followed by 4 hours condensation cycle at 50˚C. The test was run for a period of 500 hours.

Tensile tests were performed on virgin and UV aged samples using a Zwick Z100 tensile testing machine at a crosshead speed of 50mm/min. The average values of tensile modulus and stress at yield are reported in Table 2. As the tensile results for both virgin and aged materials were similar, it can be concluded that no degradation had occurred in the plastic as a result of exposure to UV.

*Table 2: Tensile test results for virgin and UV aged MDPE.*

| Property | Tensile modulus, N/mm$^2$ | Tensile stress at yield, N/mm$^2$ |
|---|---|---|
| *Virgin MDPE* | 700 ± 6 | 17.1 ± 1.4 |
| *UV aged MDPE* | 705 ± 5 | 17.4 ± 1.1 |

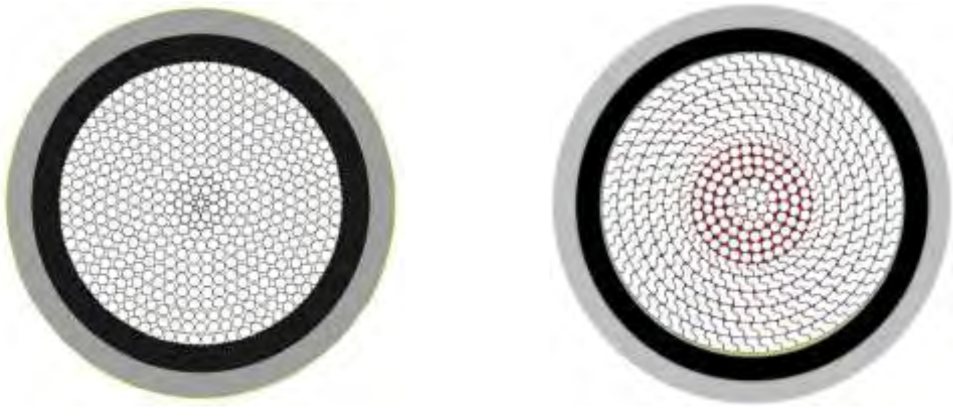
### Tests on sheathed locked coil strand

Modulus, breaking load and fatigue tests on unsheathed and sheathed locked coil strand revealed no adverse effects as a result of plastic sheathing operation.

## FUTURE DEVELOPMENTS ON SHEATHED STRANDS FOR STRUCTURAL AND MOORING APPLICATIONS

### Double sheathing of spiral strand and locked coil strand

The provision of double layers of HDPE may offer more corrosion protection and increase the longevity of the structural ropes. Sheathing with two different colours (Figure 6) also offers a visual aid to indicate areas of damage more effectively than a single sheathed product of similar thickness.



**(a)** **(b)**

*Figure 6: Cross-section of a double sheathed; a) spiral strand b) locked coil strand.*

**Sheathed spiral strand and locked coil strand with strakes**

It is known that strake profiles suppress Vortex Induced Vibrations (VIV) in cables and the benefits are known in heat exchanger tubes [10], marine risers and pipelines [11-13], bridges [14] and offshore tethered structures [15]. Conventionally on structural cables this is done by applying to the cable some segmented form of cladding provided with helical fins. Such systems are, however, inherently expensive as the cladding has to be applied and secured to the cable which involves a stop start process in the laying of the cable. When the cladding is formed of metal it can also add significantly to the weight of the cable.

Extrusion of continuous helical strakes integral (Figure 7) to the sheathing for cables such as mast stays, bridge cables and deep water mooring lines was recently demonstrated [16]. This method would offer a more cost-effective solution to the VIV problem. The extrusion process described provides a flexible approach to vary the pitch of the strakes along the length of the cable and for interrupting formation of the strakes if desired. Furthermore, laser technique to provide permanent markings along the rope length for various purposes, such as for example a longitudinal stripe of contrasting colour for cable installation purposes, markings to assist in making length measurements are described.

*Figure 7: Cross-section and isometric view sheathed spiral strand with helical strakes.*

## CONCLUSIONS

The development of the corrosion protection of spiral strands and locked coil strands for tension members in bridges and mooring applications is enhanced by sheathing methods. It has been observed that sheathed locked coil strands have the advantage to reduce the overall costs because the cost and time consumed in application of the outer corrosion protection can be done in a factory under controlled environmental and quality aspects. Sheathed spiral strands for permanent mooring lines can be used for offshore application and as well for the support of platforms of floating bridges when the water depth is too deep for conventional anchorages. Double sheathed spiral strands and locked coil strand with contrasting colour may increase the longevity of the structural strands and provide visual aid to indicate areas of damage in sheathing. Extrusion of continuous helical strakes integral to the sheathing for cables such as mast stays, bridge cables and deep water mooring lines offer a more cost-effective solution to vortex induced vibration problems.

It may be concluded that sheathed spiral strands and locked coil strand offer new opportunities for the design of bridges, where the length of the bridge or the water depth is a limiting factor.

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# TVERLANDET BRIDGE - A FOUR LANE FREE CANTILEVER BRIDGE IN NORTHERN NORWAY

By

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## ABSTRACT

*Tverlandet Bridge is under construction and will be completed in the autumn 2013. The route Rv 80 between Bodø and Fauske in the county of Nordland in northern Norway is undergoing major upgrades. Increased traffic volume, presence of rockslide areas and requirements for a general upgrade of safety and comfort makes this necessary. Tverlandet Bridge crosses the inner part of Salten Fjord and the road thus avoids a rockslide exposed area at the west side of the fjord. The new bridge is shortening the distance between Bodø and Fauske with about 2.5 km.*

## INTRODUCTION

Tverlandet and Salten Fjord is located 85 km north of the Arctic Circle.

The sea depth varies across the fjord. Towards the east the fjord is shallow with maximum water depths of approx. 3-4m. In the ship channel towards west, the seabed is located 40-45m below sea level. Abt. 200m from the west side of the fjord there is a bed rock depth of abt. 31m suitable for placing a foundation. Abt. 230m from the east side there is an islet at sea level. The varying water depth leads to a number of different foundation methods and variations in span widths to be considered. The current seabed/bedrock profile, however, is perfectly suitable for a balanced Free Cantilever Bridge (FCB) concept with three piers, combined with 3 shorter side spans having constant girder height over the shallow water to the east side. Consequently the FCB concept was selected.

The total length of the bridge is 670m. The largest span is 180m and thus used as the ship channel.

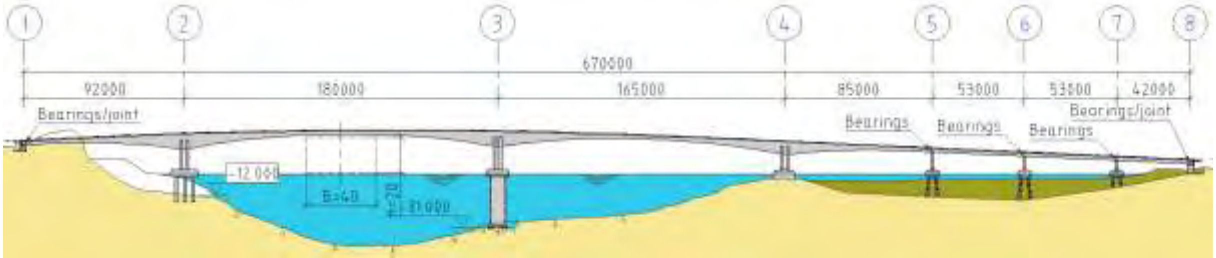


*Fig. 1 General Layout.*

The ship channel is defined to be at least 40m wide with a minimum sailing height of 20m. The Bridge is subdivided into 7 spans: 92m + 180m + 165m + 85m + 53m + 53m + 42m = 670m.

**PIERS**

All piers and abutments are founded on bedrock, either with a slab directly on bedrock or with piles through soil to bedrock.

**In axis 2** there are 12 bored piles with diameter 1.8m. The bore diameter was 2.0m and steel pipes were installed inside the bored hole. All piles are drilled to a minimum of 1.5m into bedrock. Piles are interconnected at sea level by means of a circular pile cap with a diameter of 16m. The steel tubulars were fitted with reinforcement and cast with concrete grade B35. There is one centric post-tensioned rock anchor in each pile.

*Fig. 3 Bored Piles Axis 2*

**In axis 3** the bedrock is abt. 31m below sea level. The foundation was built with use of a caisson. Abt. 1/3 of the caisson was produced in a dry dock and then floated to site for final completion. To secure stability during the floating phases there was a need for ballast in the bottom of the caisson. To ensure that the caisson was installed in the correct position, the bedrock was levelled to a near horizontal plane by means of blasting. On this surface (observing strict and tight tolerances) three small concrete blocks were placed to serve as temporary set-down points for the caisson. In the lowering phase, the caisson was ballasted with water. Stability and positioning accuracy was ensured by wires to several tug boats. The space between bedrock and bottom slab was grouted and 12 rock anchors were installed through preinstalled ducts in the caisson wall and stressed.

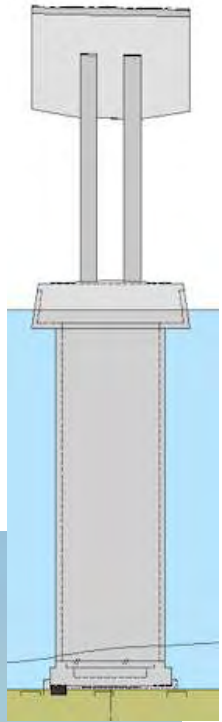

*Fig. 2 Pier Axis 3*

**In axis 4** the foundation was established using a slab on an islet in the splash zone. The slab is secured with post-tensioned rock anchors.

**In axis 5-7** the water depth is moderate, but there is a 10-15 m thick layer of soil above the bedrock. The soil do not have sufficient direct load bearing capacity, hence it is required to drive piles to bedrock. In each axis there are 14 piles. Steel tubulars having a diameter of 813mm were driven through the soil into bedrock. The tubulars acts as formwork. Reinforcement was installed and the piles were cast using concrete grade B35.

In the **FCB** part of the bridge, the pier has two twin columns in each axis with monolithic

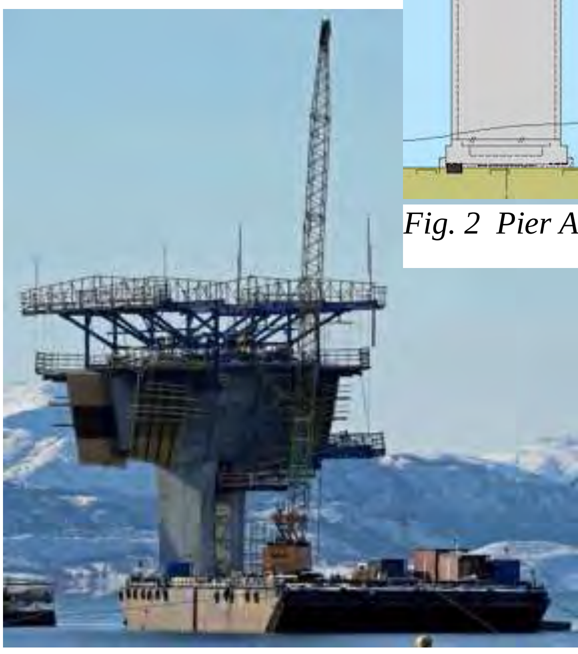

*Fig. 4 Pier at Axis 4 with FCB Form*

connection to the foundation/ pile cap and to the superstructure. The columns are wide and give adequate lateral support of the bridge. In order to accommodate the contractions and expansions in the superstructure over time, the columns are made slender and flexible in the longitudinal bridge direction. The thickness of the wall type columns is constant, while the width is variable and increasing towards the bridge girder. In axis 2 the columns have eccentric post-tensioned tendons to compensate for the permanent bending moment caused by the horizontally curved bridge girder.

In the **side spans,** i.e**.** axis 5 to 7, there are two circular columns in each axis with diameter 1.6m.

The columns are relatively short, and in order to reduce undesired strain effects due to temperature variations, shrinkage and creep in the superstructure, sliding bearings are located on the top of each column.

## SUPERSTRUCTURE

The bridge girder is a two cell box section with variable height and width. Outer walls are tilted (17° from vertical) while the centre wall is vertical. Maximum box girder height is 9.0m at the piers and minimum height is 2.6m in the middle of the main spans and in the side spans. Minimum width of bottom slab is 9.0m at the piers and maximum width is 12.8 m in the middle of the main spans and in the side spans. The bridge deck carries four lanes of traffic and a 3m wide pedestrian/bicycle lane resulting in a total deck width is 23.5m. The bridge girder is cast with concrete grade B55.

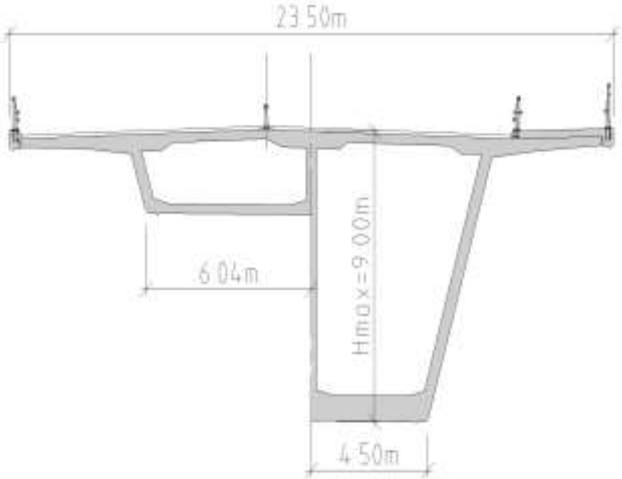

*Fig. 5 Cross-section Main Spans*

In the main spans the bridge girder is erected by means of the balanced free cantilever method using a standard form traveller system suspended at the front end of the cantilever. Following a 10-14 days cycle for the first few segments, a weekly cycle was planned for most of the remaining cantilever segments. The maximum segment length is 5.0m.

Bonded, post-tensioned tendons are the main tension element in the bridge. The longest tendons over axis 3 are 176m long, installed using the push-through method. The tendon is composed of 18 x 150mm$^2$ strands and has a breaking load of 5000kN.

*Fig. 6. Balanced Free Cantilevers Axis 4.*

Distribution of shear flow in the three webs is calculated by means of a finite element model using shell elements.

The erection of side spans was carried out with a scaffolding system supported at the permanent pile caps in axis 5-7 and using temporary supporting piled structures between axes.

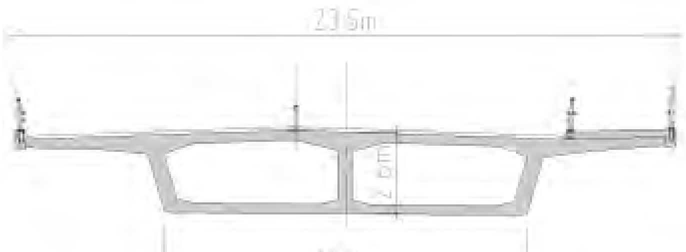


*Fig. 7 Cross-section Side Spans*

*Fig. 8 Erection of Side Span*

**BUCKLING**

Slender columns and webs in the bridge girder are investigated regarding second order effects such as buckling. The columns at axis 2 and 3 are 1.4m thick and have a height of approx. 12m (top pile cap to bottom slab of superstructure). The longitudinal deflection of the columns are calculated and second order additional bending moments are added to obtain total bending moment in the various sections of the column. Additional bending moments in the webs of the bridge girder due to second order effects are also calculated and added to obtain the total bending moment.

**DURABILITY ASPECTS**

The bridge is designed for a life-time of 100 years with an inherent requirement to avoid excessive maintenance costs. Expansion joints are located at the abutments only. Bearings are located at the abutments and at pier tops of axis 5, 6 and 7.

The high strength concrete chosen for this structure is very dense and will limit ingression of chlorides. The concrete cover varies depending on environmental impacts:

- Foundations 120mm,
- Columns 75mm
- External surfaces on bridge girder 75mm

*Fig. 9 Ice Sculpture, Arctic*

Exposed corners on the piers and bridge girder soffit are rounded, thereby reducing chloride concentrations and moisture often located near sharp corners. In addition, the bridge girder soffit is equipped with protruding drip noses near each rounded corner in order to prevent rainwater wetting the bottom slab soffit.

**WIND AND EARTHQUAKE LOADS**

Aerodynamic cross-section coefficients used in the analysis are based on Norwegian and international design codes as well as previously reported wind tunnel tests of different cross-sections.

The design wind speeds at bridge deck level (z=26m) are 32.0 m/s for the construction period and 35.6 m/s generally. Turbulence intensity in the wind field is defined by terrain category III (EN 1991-1-4).

The Bodø area has low seismic activity. The ground acceleration used in detail design of the bridge is 0.75 m/s$^2$ (R=475 years). Earthquake response is calculated with the use of modal response spectra for the lowest 100 natural eigenfrequencies and modes. The "Complete

Quadratic Combination" (CQC) method is used to establish the total structural response due to seismic loads.

Wind and traffic loads are the main contributors to bending moments in columns and foundations. Sectional forces caused by earthquake are less than for other loads in most of the structure. At a few sections seismic action gives slightly larger sectional forces than other loads.

## CONSTRUCTION SCHEDULE

Construction of Tverlandet Bridge is experienced as being slightly more time consuming than predicted in the original overall schedule. The side spans (axis 5-8) and the cantilevers from axis 4 were completed last year and the cantilevers from axis 2 and 3 started a few months ago. However, it is realistic to expect that the closure segment in the main spans between axis 2 and 3 will be concreted in August this year and that the bridge will be opened for traffic in 2013.

*Fig. 10  Side Spans and FCB Axis 4*

## ACKNOWLEDGEMENTS

Norwegian Public Roads Administration (Photos).

# CONCEPT DEVELOPMENT OF A SOGNEFJORD FLOATING BRIDGE CROSSING

by

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## ABSTRACT

*A design group consisting of the companies Aas-Jakobsen, Johs. Holt, Cowi, NGI and Skanska were in the spring of 2012 awarded by the Norwegian Public Roads Administration a contract for conceptual development of a floating bridge crossing Sognefjord in Norway. The crossing will be a part of the future E39 Fixed Link Project. At this site, the bridge will cross a strait being 3700m wide and 1250m deep. There is also a significant amount of ship traffic in this fjord, including some of the largest cruise ships in the world.*

*The development of concepts has been done in close cooperation with the Client. The chosen concept is a multiple span suspension bridge with two of the four main towers supported by floating pontoons anchored to the sea bed, see ref./1/. The span width of the suspension part will be 3x1240m. This concept will be the first of its kind in the world, and it will require experience and knowhow from long span bridge engineering and deep sea marine structural engineering to its full extent. The conceptual work has been done using a number of senior technical experts, e.g. in the areas of structural analyses, wind dynamics, wave dynamics, cable dynamics, impact dynamics, marine geotechnical engineering and marine operations.*

*The floating bridge concept will, if chosen for construction, bring about an entire new era in the field of long span bridge engineering and construction.*

*The paper outlines the work carried out in developing the concept and furthermore presents the results from the conceptual engineering in terms of technical solutions, structural behaviour and construction procedures.*

## INTRODUCTION

The E39 fixed link project is the master plan to connect the Norwegian cities Kristiansand, Stavanger, Bergen and Trondheim by means of a ferry free highway route. The E39 highway have currently altogether 8 ferry connections along the coast and the intention is to replace all these crossings with fixed connections, i.e. either by bridges or subsea tunnels. The most challenging crossing is the Sognefjord connection, since at the chosen site the fjord is 3700m wide and 1250m deep. Consequently a feasibility study was launched with the objective to demonstrate that it is feasible to cross Sognefjord with a bridge structure. If that were to be the case, then all remaining deep fjords could also be crossed by bridges. Following an initial phase of conceptual development in 2010, the conclusions were that the following concepts should be further addressed; a suspension bridge, a floating bridge, a submerged floating tunnel and a combination of the last two concepts. Late in 2011, an invitation was launched for companies to develop solutions for floating bridges, submerged floating tunnels and a combination of these.

The aim was that through a "dialogue tender phase" this should result in teams being selected to do feasibility studies of the various concepts. At that stage, in spring 2012, a team consisting of the companies Aas-Jakobsen, Johs. Holt, Cowi, NGI and Skanska were chosen to perform a feasibility study of the floating bridge concept.

During the development of the winning concept, both end and side anchored floating bridges were investigated by the team. These investigations also included one concept with ship passage near the shore area, one concept with global load transfer below sea level and one floating cable-stayed bridge.

The winning concept which was brought further on into the feasibility study is however a floating suspension bridge with three main spans of 1234m each. The feasibility study was completed in December 2012.

## CONCEPT DESCRIPTION

### General

The concept studied is a tree-span suspension bridge supported partly on floating pontoons, as shown in Fig. 1. The length of each span is 1234m and the total bridge length is 4402m including viaducts on each side of the fjord. The bridge deck carries two lanes of traffic and one pedestrian/bicycle lane. The ship clearance height is 70m. The main pontoons are anchored to the sea bed and to the shore through an anchoring system similar to that used in floating marine structures for the oil and gas industry.

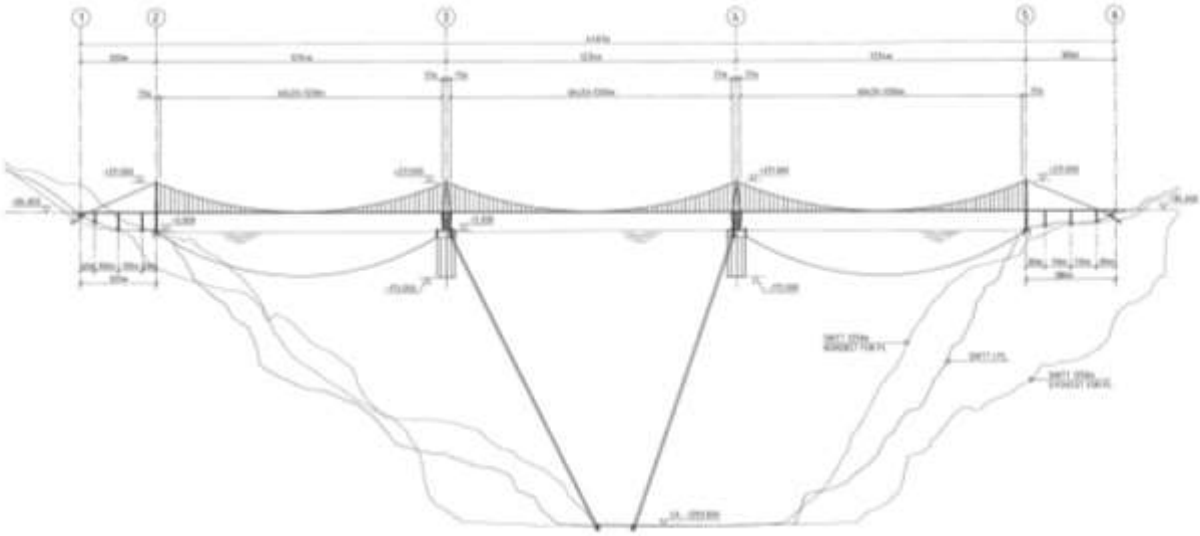

*Fig. 1: Bridge overview*

### Anchorage system

The anchorage system consists of stay cables attached to the fjord sea-bed and to the pontoons. The main part of the cables are locked coil steel spiral strands for subsea use with a breaking load of approximately 22MN, see Fig. 2. Composite materials could also be used for this part. In the top and bottom part of the cable marine chains are used in accordance with common practice.

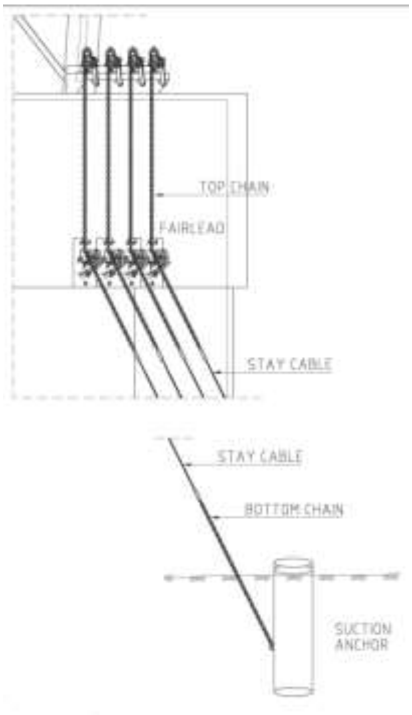


*Fig. 2: Anchorage system*

Suction anchors are located at the sea-bed where the soil surface is relatively even and where there are thick layers of clay. The dimensions of the suction anchors are calculated to be approximately 18m deep into the soil and with an outer diameter of 6m. At the top, the anchor chains are fixed to the pontoon by using specially designed fairleads attached to the pontoon. The anchor force is controlled by adjustable jacks located on the top slab of the pontoon.

The erection of the anchorage system will be in accordance with practices used for marine floating structures. This involves using onshore/dry dock production facilities, barge transportation/working platforms, offshore cranes, ROVs, temporary floating elements etc.

**Pontoons**

The floating pontoons are concrete structures with a 9 cell cylindrical configuration with diameters from approximately 35m for the centre cell to 11m for the outer ones, see Fig.3. The draft will be 175m with a freeboard of 7m. In the top 25m, the pontoon configuration is specially designed to resist and absorb ship collisions through several secondary walls and light weight aggregate concrete in-fills. The total concrete volume is 105 000m$^3$. To ensure sufficient global stability, ballast comprising 155 000m$^3$ of olivine aggregate is used.

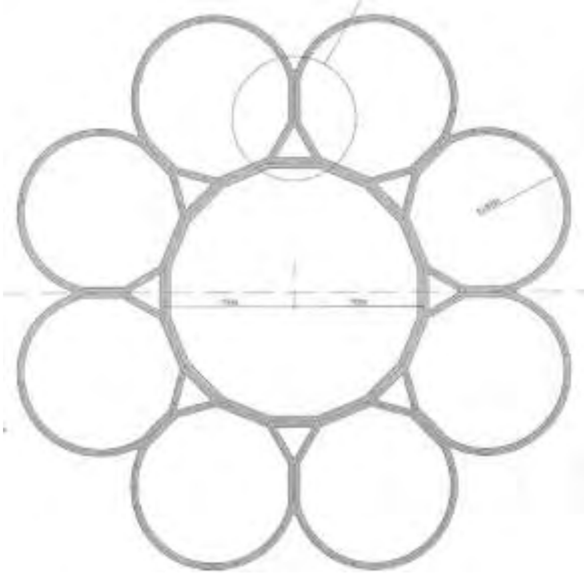

*Fig. 3: Pontoon section*

The required depth of the pontoons are governed by providing sufficient buoyancy to the bridge as well as sufficient rotational stiffness to limit tower inclination and force transfer in to the tower saddles.

The pontoons will be constructed in accordance with traditional marine GBS structures to be found for instance in the North Sea. This includes dry dock casting of base slabs and lower parts of cell walls, and remaining work being performed in a floating condition at a suitable deep water site.

### Towers

The towers on pontoons will be steel towers, chosen to obtain minimum weight. The towers are four legged in order to be able to resist large bending moments in two directions, and with a diamond shape configuration. This shape is found to be optimal since there is a need for wide leg spacing at bridge deck level to limit deformations in the top of the towers and smaller leg spacing at the base in order to limit the size of the pontoons. The steel towers are 206 m high and the overall widths vary between 32 m at the base, up to 42 m below the deck, and down to 6.7 m at the top, see Fig.4. Typical box sections are 5x5 m$^2$. The steel towers are erected section by section using floating offshore cranes for the first sections and cranes mounted on the pontoons for the remaining sections.

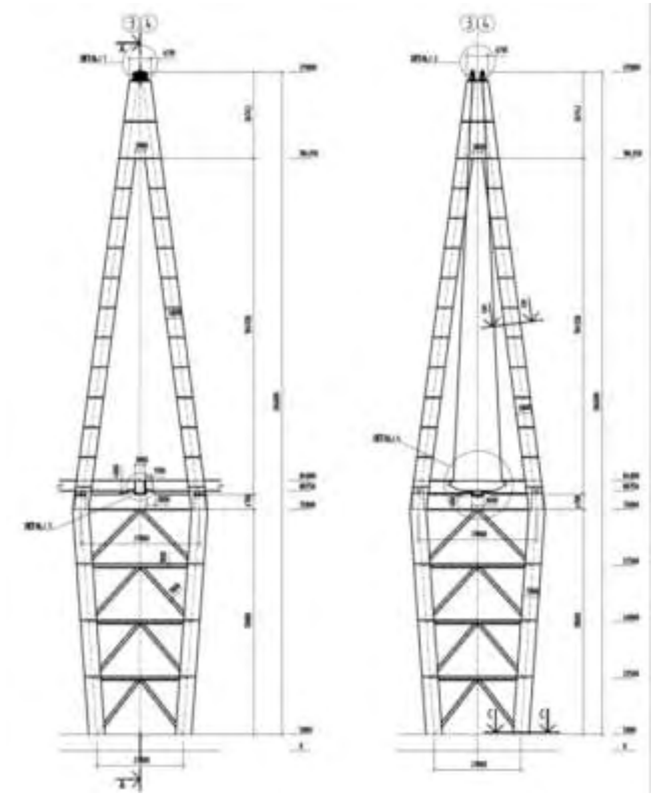

*Fig. 4: Tower layout*

The towers on shore are concrete towers with the same outer geometry as the steel towers except being two instead of four legged. They may be constructed by means of a climbing scaffolding system or by slipforming.

### Superstructure

The bridge deck is a traditional orthotropic steel box with a total width of 18.3 m and a height of 3.25 m. The transverse translation is restricted at towers by bearings. Longitudinal translation is fixed at the floating towers, making a transfer of longitudinal forces possible in the bridge beam. These forces are resisted by a system of longitudinal post-tensioned cables at the abutment and the viaduct. This system acts as a horizontal spring. This is done to avoid unintentional deflection of the entire bridge in longitudinal direction.

The main cables consist of 19 bundles with 384 wires each and with a wire diameter of 5.3mm. The wire yield strength is chosen to be 1770MPa. Aerial spinning is proposed, but prefabricated cables are also considered to be a feasible solution. The main cables are fixed by traditional rock anchorages in the abutment areas.

## DESIGN CONSIDERATION

### Loading

Critical load conditions are found to be ship collision, wind loading and second order wave loading. The loading is taken from a project specific design basis. For ship collision loading, the Client has performed risk analyses for the actual site and bridge concepts. This is based on the passage of large cruise ships. Consequently the design is performed for a ship collision event with a recurrence period of 10 000 years, specified as a ship with a displacement of 31 456 tonnes travelling at a speed of 17.7 knots.

### Analyses

A number of structural analyses have been performed involving program systems such as RM Bridge, Novaframe, USFOS, Wamit, Mimosa, Focus and FEM-design. These have been utilized to address for instance dynamic wind and waves, large deflections, ship load history and non-linearities. The global structural analysis is performed by means of RM Bridge.

### Results

The results are found to be acceptable in all structural parts of the concept and the overall conclusion has therefore been that the floating suspension bridge concept for crossing Sognefjord is feasible. There is however a need for an extensive and detailed preliminary design phase.

For the anchorage system, an important objective is to limit the global transverse deflection of the structure from wind and second order waves, as well as always ensuring that the chains/cables are in tension at all times in the operational phase. This also applies to adjacent parts of the system, such as fairleads and suction anchors. For the pontoons, the design is governed by limitation to the acceptable inclination of the towers as well as buoyance requirements in operational and damaged situation. Ship impact governs the layout of the upper part. The towers are also subjected to traditional critical loadings on a traditional suspension bridge such as wind on free standing tower as well as imposed forces from the substructure. For the bridge deck and the main cables, the system is comparable to a traditional suspension bridge regarding deflections of bridge deck and force transfer in tower saddles. In addition they are subjected to additional forces from tower deflections and inclinations transversely as well as global force transfer in longitudinal direction.

The maximum vertical deflection in the bridge deck from traffic is found to be approx. 7.5 m and the maximum tower inclination from traffic is approx. 0.5 deg. The maximum horizontal deflection from wind is found to be approx. 13.5 m and the maximum tower inclination from wind approx. 1.0 deg.

## DISCUSSION

From the study it is concluded that the structure as shown in Fig.1 and in an overall picture in Fig.5, is feasible. All results are satisfactory and realistic construction procedures have been outlined for every part. Risk analyses have been performed for both construction phase and operational phases. Some items have been identified to be of special interest for the next phase of the project. Examples of this is cost optimization, use of composite stays, special ship collision scenarios and developing better data for system damping. Further design phases also need to be accompanied by site specific soil investigations and model testing in wind tunnels and ocean laboratories/model basins.

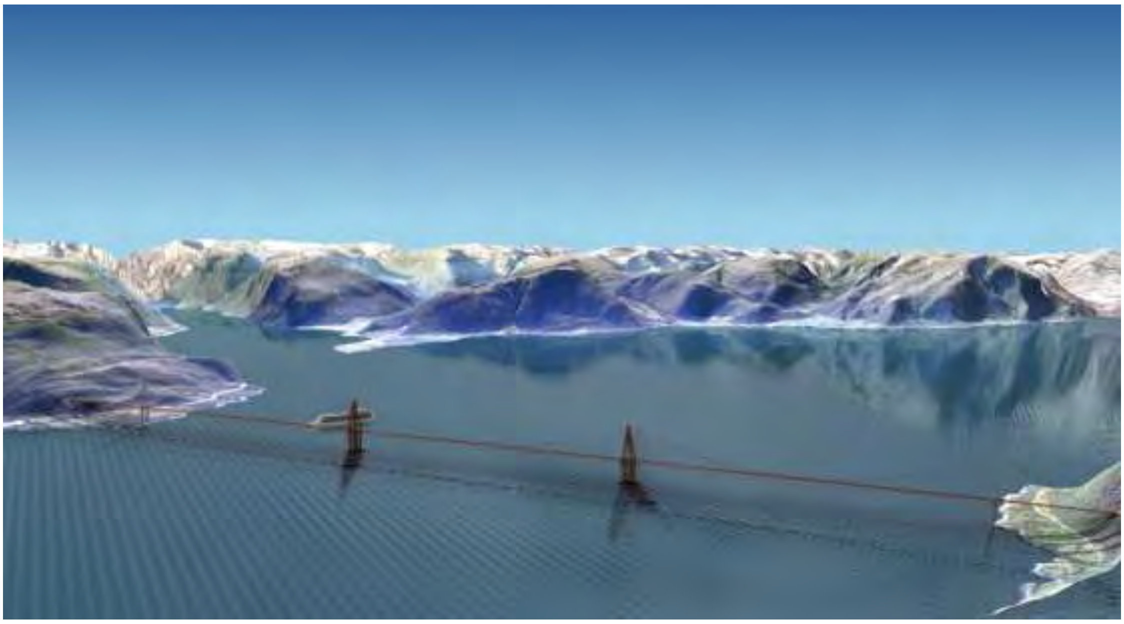

Fig. 5: 3D-plot of structure

## AKNOWLEDGEMENT

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# INNER LINING IN TRAFFIC TUNNELS

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*The standard solution for the inner lining of Norwegian tunnels is the installation of precast concrete elements. Recent incidents in existing road tunnels in Norway due to damages to the inner lining – as a consequence of frost effects, falling rock blocks or gravitational collapses – have led to a reconsideration of the existing design standards and common practise for road tunnels, as documented in the "Modern Road Tunnel Strategy Study" from the Norwegian Public Roads Administration (Statens Vegvesen). The choice of the inner lining concept is very important with respect to reliability, availability, maintenance and safety (RAMS). This is true not only for road tunnels but also for railway tunnels. Most of the newly planned railway tunnels are designed for high-speed trains, where the effects of dynamic pressure and suction loads are an important factor for the dimensioning of the inner lining. Furthermore, for such tunnels a long lifetime and a high availability (in order to face the increasing traffic intensity) are demanded. In account of this, adjustments in the design of railway tunnels are also on-going. This paper describes and compares both the Norwegian precast concrete elements concept and the cast-in-place concrete lining system – which is the standard solution for inner lining, e.g., in Switzerland – showing that the cast-in-place solution is a valuable alternative.*

## INTRODUCTION

The standard inner lining of Norwegian tunnels consists of precast concrete elements (fixed with rock bolts). Recently, some incidents occurred due to damages to these elements (because of frost effects, falling rock blocks or gravitational collapses [18]). In account of this, the Norwegian Public Roads Administration (NPRA, Statens Vegvesen) reconsidered the existing design standards for road tunnels. One of ten major research and development projects conducted by the NPRA was the sub-programme Modern Road Tunnel Strategy Study, which focused on the NPRA's strategy for tunnels and followed up issues described in two reports [19, 20]. These reports revealed the need for improved tunnel maintenance and geological documentation systems as well as increased professional expertise. Major issues, such as the design of the tunnel profile and of the tunnel lining, life cycle costs, operation, maintenance and, more in general, the harmonisation of the regulations also made up key parts of the project.

Similar considerations apply for railway tunnels. As most of the newly planned railway tunnels are for high-speed trains, the design of the inner lining is a central issue. In fact, in such tunnels the dynamic pressure and suction loads play a central role for the dimensioning of the inner lining. Furthermore, a long lifetime and (in account of the continuously increasing traffic intensity) a high availability of the tunnel are two of the major requirements for modern railway tunnels. This is because for railway tunnels too (and not only for road tunnels) adjustments in the design standards are on-going.

The present paper deals with the topic "inner lining" for both road and railway tunnels. After a description of the inner lining concepts, the paper compares the classic Norwegian lining concept (precast concrete elements) and the concept with cast-in-place concrete.

## INNER LINING CONCEPTS

### Inner lining with precast concrete elements

As already mentioned above, an inner lining consisting of precast concrete elements is the standard solution in Norway. In this case, the inner lining is non-structural and is built-in after the final rock support has been installed. The main purposes of the non-structural lining are managing of water ingress and frost protection. Furthermore, there are aesthetic requirements. As shown schematically in Figure 6a, the precast concrete elements are fixed with rock bolts (which are also used as positioning bolts for the installation of the elements). On the outside of the elements, a waterproofing membrane is placed. Between the sealing system and the rock mass there is a void, i.e. the inner lining is not in contact with the rock mass.

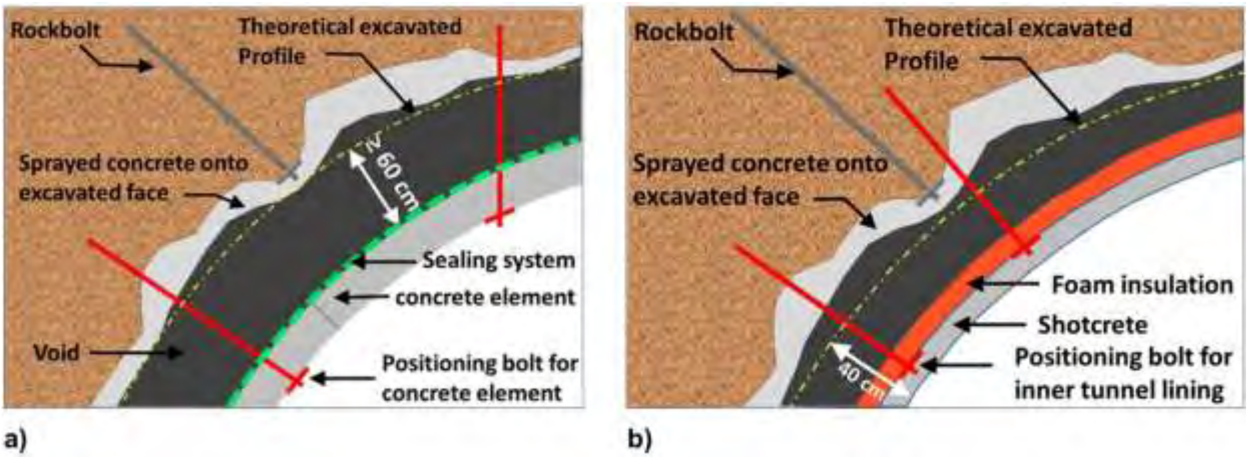


*Figure 6. (a) Inner lining with precast concrete elements; (b) foam isolation – sprayed concrete lining (after [21]).*

The temporary and final rock support of the tunnel (e.g. rock bolts and sprayed concrete as sketched in Figure 6a) is installed previously and is independent from the inner lining system. The final rock support and the rock mass surrounding the non-structural lining have to be monitored during the entire lifetime of the tunnel. However, there is generally no easy or practical way for this monitoring. As a rule, the safety inspections consist of random drillings into the concrete lining. It is obvious that such random inspections are unreliable and expensive. Therefore, the so called "Ground Penetrating Radar" has been introduced in the vault walls to map the contours of the void in a more systematic way, as this scanning technology provides satisfactory data and gives an optimal location of the apertures. Additionally, it can also be used in the vault roof to pinpoint potential rock falls [22]. In the future, detection of potential rock falls will be done by means of video cameras installed in the void behind the inner lining, thus allowing for an online supervision. However, the suitability and reliability of such a system is not proven yet.

In addition to the monitoring, regular maintenance for checking the conditions of the rock support and for removing loose rocks is necessary. The void behind the inner lining has to be at least 60 cm (Figure 6a), which is not sufficient for inspections. However, in most cases the void is bigger than theoretically required (due to systematic over profile) and inspections are possible.

**Foam isolation – sprayed concrete lining**

An alternative to the inner lining with precast concrete elements described above, is the so called "foam isolation – sprayed concrete lining" (Figure 6b). In this system, foam isolation is fixed with rock bolts (which are also used as positioning bolts during assembling) and covered with a layer of sprayed concrete. As for the system with precast concrete elements, the inner lining is non-structural and not in contact with the rock mass. The theoretical minimum dimensions of the void between the rock mass and the inner lining are smaller (40 cm instead of 60 cm).

**Inner lining with cast-in-place concrete**

The main difference between an inner lining consisting of cast-in-place concrete and the two Norwegian systems described above is the fact that the inner lining is in contact with the rock mass (or with the sprayed concrete of the temporary rock support, if any). In this way, there is no void behind the inner lining (Figure 7) and latter is a structural part of the tunnel construction which, as a rule, consists of two shells: the sprayed concrete layer of the temporary rock support (outside) and the cast-in-place concrete shell of the inner lining (inside). Between the two shells a sealing membrane is installed; both drained and undrained solutions are possible.

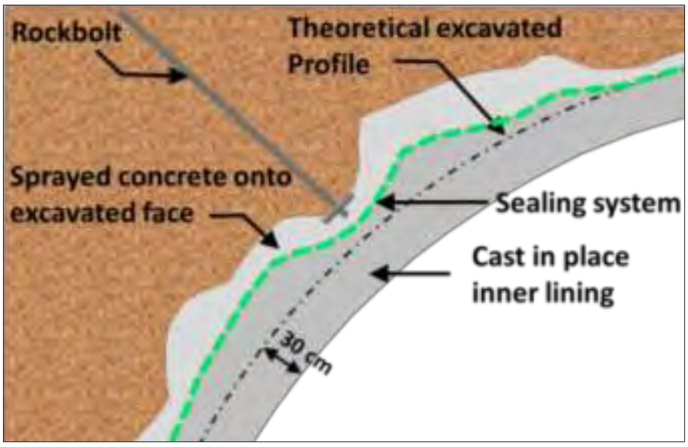


*Figure 7. Inner lining with cast-in-place concrete (double-shell lining).*

As for the Norwegian systems, the thickness of the sprayed concrete layer depends on the encountered geological conditions. For concreting reasons, the minimum theoretical thickness of the cast-in-place concrete lining is 30 cm. Usually, the inner lining is reinforced only near to cross-passages or niches. Of course, if necessary the thickness can be increased and the inner lining can be reinforced systematically. Furthermore, if required, the inner lining can be enhanced with an invert arch and closed to a ring, thus allowing the accommodation of high rock loads and/or water pressures (undrained solution).

The main purposes of an inner lining consisting of cast-in-situ concrete are manifold:
– guarantee of a lifetime of 100 years for the tunnel construction;
– reduction of the required maintenance;
– increase of the availability of the tunnel;

- provision of an internal finishing for the minimization of the air friction (this is particularly important for high-speed railway tunnels, as the internal friction influences the energy requirements);
- protection of the sealing membrane (in case of fire);
- easy mounting of the tunnel equipment (signals, handrails, ventilation, traction current, etc.);
- easy painting, which allows for a reduction of tunnel cleaning and tunnel lighting (only for road tunnels);
- accommodation of the rock pressure;
- bearing of the external water pressure (undrained solution);
- reduction of the amount of pre-grouting (in the case of an undrained solution and assuming that water inflow is acceptable during construction).

## COMPARATIVE CONSIDERATIONS

### Dimensions of the tunnel profile

The dimensions of the tunnel profile to be excavated depend, among others, on the inner lining concept (Figure 8). Applying the concept with the cast-in-place concrete inner lining, the excavation surface can be reduced. This is due to the fact that in the Norwegian system place is lost because of the void existing between inner lining and rock mass. According to Table 2, a reduction of the excavation radius of 45 cm is possible, which results in an important reduction of the muck quantity and of the corresponding transports and deposit volume. Assuming an inner radius of 5 m, the reduction in cross-section is of approximately 10 m$^2$.

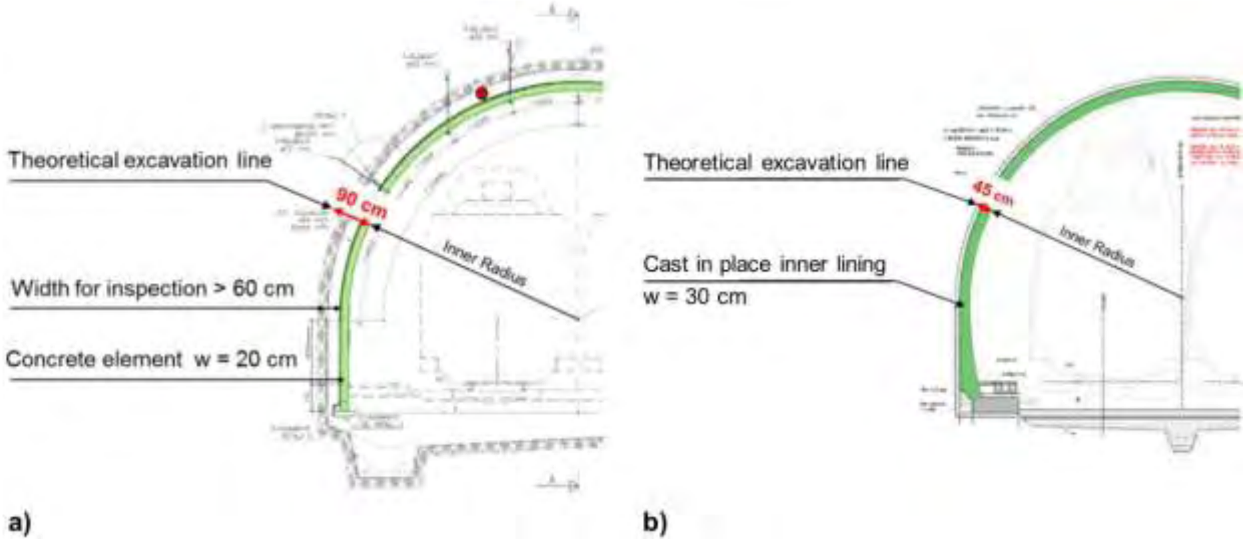


*Figure 8. Theoretical excavation line for, (a), inner lining made of precast concrete elements (after [23]) and, (b), inner lining made of cast-in-situ concrete (after [24]).*

*Table 1. Comparison of the excavation radius for the two inner lining concepts of Figure 8.*

| Concept | Precast concrete elements | Cast-in-place concrete |
|---|---|---|
| Inner lining | 20 cm | 30 cm |
| Sealing membrane | 0.3 cm | 0.3 cm |
| Inspection space | > 60 cm | -- |
| Leveling layer | 0 cm | 5 cm |
| Rock support | 10 cm | 10 cm |
| **Excavation radius** | **Inner radius + 90 cm** | **Inner radius + 45 cm** |

**Rock support**

A cast-in-place concrete inner lining is part of the final tunnel structure and has a structural action. According to this (and if designed accordingly), it can accommodate the entire or part of the rock loads in the long term. This allows for a reduction of the temporary rock support and, therefore, to a further reduction of the excavation radius compared with the concept applying precast concrete elements (with which the rock support has to accommodate the rock loads both in the short and in the long term and, therefore, has to fulfil higher requirements with respect to bearing capacity and lifetime).

**Overbreak regulations**

The considerations of above apply for the theoretical excavation radius. However, the effective dimensions of the excavated profile depend also on the amount of overbreak. The Norwegian inner lining concept assumes implicitly that a certain amount of overbreak exists. This in order to have a big enough void between inner lining and rock mass allowing for the assembling of the precast concrete elements and for inspections. On the contrary, in the case of a cast-in-place concrete inner lining more sever overbreak regulations can be applied, thus resulting in a reduction of the excavated volume. The stronger regulations are also advantageous with respect to the required concrete volume, as less overbreak means less volume to be filled with concrete.

The Swiss standards [25] define clearly the procedure for the reimbursement of the excavation volume and of the overbreak, respectively. The principle is depicted in Figure 9. A given overbreak *d* is included in the bid (this has to be declared by the contractor in its bid). The overbreak *F* is additional reimbursed (according to the unit price contained in the offer) if this is due to geological reasons. When the contractor causes itself the overbreak (e.g. as a consequence of inaccuracy of the drill holes, overloading or late installation of the rock support), this is not paid. With this regulation, the contractor is encouraged to blast as exactly as possible in order to minimize the overbreak, which is not paid and which has to be filled with cast-in-place-concrete.

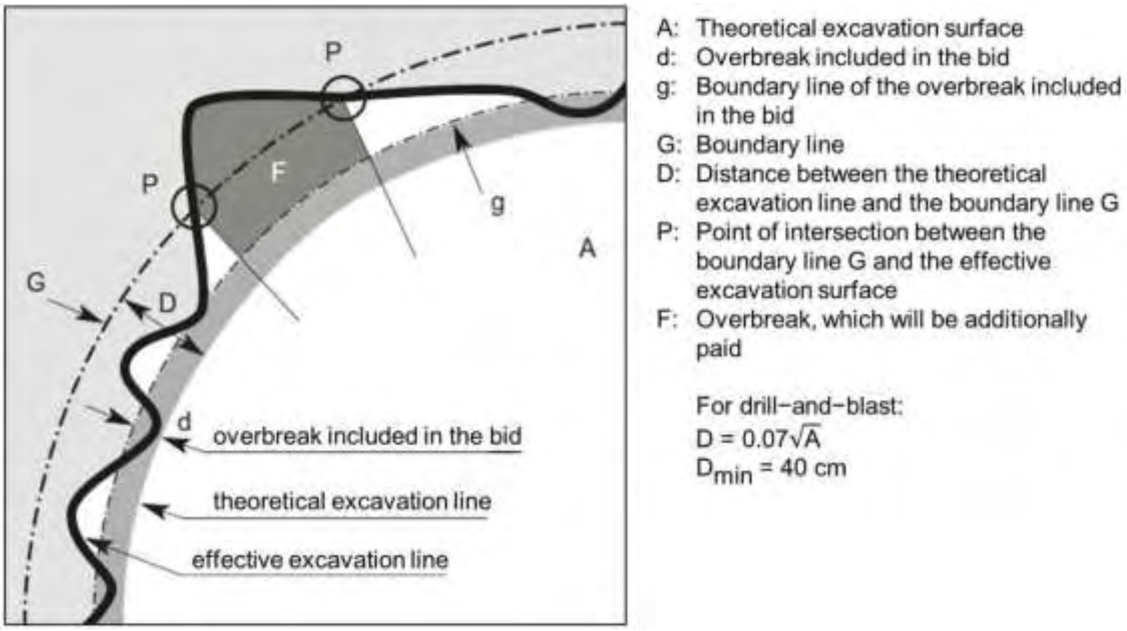


Figure 9. Overbreak regulations for drill-and-blast excavation according to the Swiss standards [25].

**Sealing**

The inner lining of a traffic tunnel is more and more planned for a functional service lifetime of at least 100 years. For this reason, the cast-in-place concrete must be protected extensively against the effects of mountain water. The degree of protection and, more in general, the design of the sealing system depend on the water pressure and on the aggressiveness of the mountain water.

The requirements concerning the waterproofing of the tunnel have to be clearly defined in the design basis. Beside the protection of the cast-in-situ concrete of the inner lining, also the operational requirements of the client as well as environmental aspects (e.g. reduction of the drainage of the rock mass) have to be considered. In the Swiss standards [26], this is handled with so called "waterproofing classes" (Table 2).

*Table 2. Waterproofing classes according to the Swiss standards [26].*

| Class | Description |
|---|---|
| 1 | **Completely dry.** No damp spots permitted on the dry side of the structure's surface. |
| 2 | **Dry to slightly moist.** Some damp spots permitted. No dripping water permitted on the dry side of the structure's surface. |
| 3 | **Moist.** Damp spots limited locally. Individual dripping spots permitted on the dry side of the structure's surface. |
| 4 | **Moist to wet.** Damp spots and dripping areas permitted. |

As a rule, the installation of the sealing system occurs in two steps. In a first step, a fleece with special fixing points (Figure 10a) is nailed on the sprayed concrete surface. In a second step, the sealing membrane is welded to the fixing points. Both the fleece and the sealing membrane are delivered in 4 m wide rolls (preassembled in the factory). For their installation a special scaffold (Figure 10b) is used. After being fixed, the 4 m wide ribbons are welded together. In areas where reinforcement has to be placed, an additional protection membrane is mounted on the inside. A big advantage of this solution is that there are no elements perforating the sealing membrane and, therefore, reliable waterproofing is easier to realise. The same does not apply for the Norwegian inner lining concept, where the positioning and fixing rock bolts of the precast concrete elements go through the sealing membrane, i.e. there are a number of points which may become leaky.

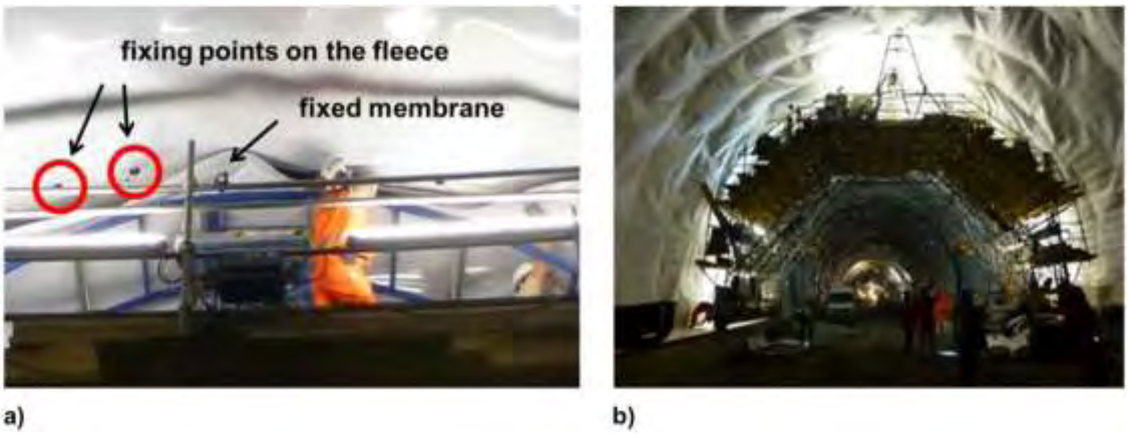


*Figure 10. Sealing membrane: (a) welding points on the fleece; (b) special scaffold for the installation.*

It has also to be mentioned that the application of a sealing membrane behind a cast-in-situ concrete inner lining requires a given evenness of the sprayed concrete surface. Figure 11 shows the corresponding acceptable geometric values. Normally, in tunnel excavated by means of drill-and-blast an additional layer of sprayed concrete (without fibres) for smoothening is necessary.

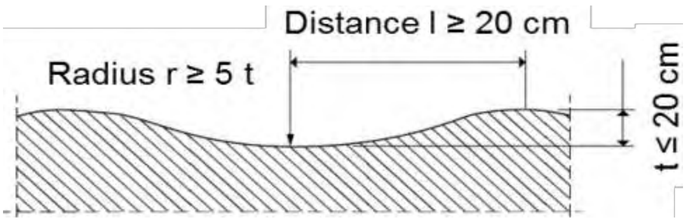


*Figure 11. Acceptable unevenness for the sealing membrane [27].*

**Concreting**

A further major difference between the Norwegian system and an inner lining made of cast-in-place concrete regards, of course, the construction method of the inner lining. In the Norwegian solution, the precast concrete elements are prefabricated in a factory, transported to the construction site and installed with the aid of the positioning bolts (Figure 6a). The mounting of the sealing membrane on the outside of the concrete elements is also part of the assembling works. On the contrary, if the inner lining consists of cast-in-place concrete, the sealing system is installed in advance in a previous working step (i.e. the sealing works and the construction works of the inner lining are not coupled, thus allowing for better quality controls). Afterwards, the inner lining is concreted in sections of 10–12 m length using a movable formwork (Figure 12). Before the formwork can be carefully stripped, the compressive strength of the concrete has to reach a minimum value defined in advance (for large tunnels, as a rule, 10 N/mm$^2$). Compliance with these requirements is verified, e.g., with the Schmidt hammer.

Figure 12. (a) Formwork; (b) steel structure of the formwork; (c) concreting (left: formwork, right; sealing membrane).

With both systems, the inner lining works can be carried out after the tunnel has been excavated or, with appropriate logistic adjustments, in parallel to the tunnel excavation. A common performance for cast-in-place concrete inner linings is of one section (10–12 m) per working day using two formworks. Due to the high degree of industrialization, careful curing and strict quality controls, the quality of the cast-in-place concrete is as good as the one of precast concrete.

**Safety and reliability**

Particularly in the case of high-speed railway lines, the loads on the tunnel walls due to the dynamic air pressure and suction as well as vertical uplift is very high. The positioning bolts used for fixing the precast concrete elements of the Norwegian system (Figure 6a) are not designed for withstanding such high pressure loads. Furthermore, due to the irregular shape of the excavated rock, these bolts have different lengths, thus leading to an asymmetric bearing (and, therefore, stress) of the entire construction. Another very important aspect is the fact that inner linings according to the Norwegian concepts (Figure 6) are not able to withstand the dynamic load resulting from a heavy rockfall (as incidents, like the one occurred in the Hanekleiv Road Tunnel [18], demonstrate).

For a cast-in-place concrete inner lining, these dynamic loads either not exist (the height of fall of a rock block is equal to zero) or do not represent a problem (as the inner lining is in contact with the rock mass, oscillations of the system are not possible). Furthermore, the rock loads are considered in the structural analysis of the inner lining. Therefore, these loads do influence neither the safety nor the reliability of the system. The safety is better also in the case of heavy road accidents, as there is no risk that parts or even the entire inner lining fall down.

**Lifetime**

As already mentioned above, cast-in-place concrete inner linings are designed for a lifetime of at least 100 years. This is a major advantage of this system, as time and cost intensive replacing works are not necessary.

**Corrosion**

Especially for road tunnels, where in winter de-icing salt is used, the risk of reinforcement corrosion is high. This risk is lower for inner lining made of cast-in-place concrete. First of all, such inner linings are, as a rule, not reinforced (i.e. the risk does not exist a priori). Furthermore, the concrete cover is higher than for precast concrete elements (for which, due to the thickness of about 20 cm, the possible concrete cover is limited). For both systems, the use of concrete with a low permeability (to prevent the penetration of salt and water) is recommended.

Corrosion can also concern the rock bolts used for the fixation of the precast concrete elements. These rock bolts can get damaged and lose their corrosion protection. After completion of the construction works, monitoring of the quality of the fixing elements is hardly possible.

**Frost**

In Switzerland, damages to the inner lining due to ice pressure are not known. This in spite of the fact that several tunnels are located in mountain areas with low temperatures during the winter (for example, the north portal of the Gotthard Road Tunnel is situated at an elevation of 1'150 m a.s.l.). The absence of such damages can be explained considering the dimensions of the elements of the sealing system installed behind the cast-in-situ concrete inner linings. The thickness of the draining layer, i.e. of the layer when water could be collected, depends on the water quantity to be drained. In the maximum case, this thickness is of approximately 1 cm. Therefore, the maximum thickness of the ice layer is also 1 cm. Due to icing, water experiences a volume increase of 9%. This results in an expansion of the water/ice layer behind the inner lining of only 0.9 mm, thus leading to a non-significant pressure acting on the inner lining.

Anyway, one method for protecting concrete against frost is the use of chemical additives to generate air voids. In this way, water inside the concrete can expand into these voids and spalling can be avoided.

**Fire**

The resistance against fire of a concrete structure depends among others on the characteristics of the concrete. Assuming the same concrete for both cases, an inner lining consisting of precast concrete elements will collapse before an inner lining made of cast-in-place concrete. This is because of a weak point of this system, i.e. the rock bolts used for fixing of the precast concrete elements, which are not able to resist the high temperatures developing already in the first minutes of a fire. It is worth mentioning here that the stability of the tunnel structure is a premise of each safety concept based upon rapid self-rescue of people.

**Fixing of installations**

Both in road and in railway tunnels a lot of installations have to be fixed to the wall. With a thickness of at least 30 cm, inner linings made of cast-in-place concrete allow the use of dowels without penetrating the waterproofing membrane (Figure 13).

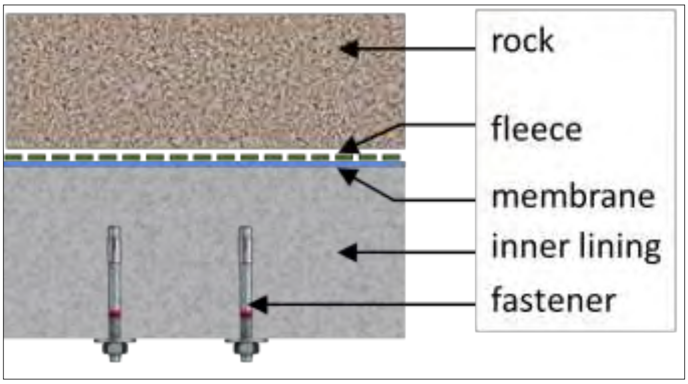


*Figure 13: Fixing of installations with dowels.*

**Costs**

One common argument against the solution with cast-in-place concrete is the price, i.e. that the investment costs are much higher than for the solution with precast concrete elements. However, recent comparisons of contractors working on the Norwegian market showed that the two solutions have comparable investment costs, provided that the costs for the temporary rock support can be reduced. As a rule, this is possible. For example, instead of expensive full grouted bolts with corrosion protection, normal friction bolts can be used. Furthermore, comparing the costs, a global evaluation is recommended. In fact, taking into account not only the investment costs but also the maintenance costs and the costs deriving from a reduced availability of the tunnel, there is a clear advantage for the solution with cast-in-place concrete.

## CLOSING REMARKS

Both inner lining concepts (precasted concrete elements and cast-in-place concrete) have advantages and disadvantages. However, the tendency for the future is towards inner linings made of cast-in-place concrete (Figure 14), which are the standard solution in Switzerland but not only). This because of the clear advantages with respect to reliability, availability, maintenance and safety (RAMS) and of the comparable construction costs.

*Figure 14: Inner lining of the Lötschberg Base Tunnel (Switzerland) [9].*

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28. Picture: Courtesy of BLS AlpTransit AG, Thun, Switzerland.

# EUROPE'S NORTHERN DIMENSION FROM A TRANSPORTATION PERSPECTIVE

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## ABSTRACT

*This paper looks at the key northern rail corridors that connect the Arctic Ocean to the Mediterranean Sea. Finland, on the northern shores of the Baltic Sea, is in traffic terms a remote outpost, with functioning fixed rail connections running only eastwards. Long subsea railway tunnels could, however, give Finland the access it needs to the rail networks of Europe. Beneath the Baltic Sea lies solid bedrock, which offers new possibilities for opening up transport connections. A key project currently being studied is a railway tunnel across the Gulf of Finland between Helsinki and Tallinn. A railway tunnel under the narrow Kvarken (Quark) stretch of the Gulf of Bothnia to Sweden would, however, be easier and safer to build as a first, trial project. This paper adds a northeastern dimension to the author's earlier article of the same title by encompassing a further traffic corridor from the Atlantic to the Pacific Ocean via Europe and Russia.*

## 1. INTRODUCTION

The author, now retired, spent 37 years in various geotechnical posts with the City of Helsinki and headed its Geotechnical Division from 1972 to 1997. The author has continued to maintain his interest in geotechnical projects over the past decade, not least those referred to here.

This paper examines rail connections via Finland to continental Europe and the other Nordic countries, and provisional connections to Russia's northeastern corridors. The construction of a railway tunnel under the Gulf of Finland and the associated geological conditions are discussed in another Finnish paper [2] to be given at the Strait Crossings 2013 symposium. A report on the transport needs of Finland's mining industry [3] was published in early spring 2013, adding to the sources available to the author of this paper.

The impact of climate change and dwindling oil reserves will force the transfer of traffic from road to rail. The receding ice in the Arctic Ocean will also open up the EU's rail connections to the north, which will mean greater use of rail corridors in the Nordic countries and in Russia. An important goal of EU transport policy is to build a network of rail corridors across the region and convert rolling stock and rail networks to ensure interoperability between different countries. Finland is still without effective rail links to other EU countries as its network has a different gauge (1524 mm), hampering connections. There is a danger that in traffic terms Finland will remain a northern outpost isolated from the other EU countries. For this reason, traffic movements across the bays of the Baltic Sea require not only car and train ferries but also subsea railway tunnels.

## 2. EUROPE'S NORTHERN RAIL CORRIDORS

A joint project by the present author and Ago Vilo, both geotechnical experts based in countries bordering the Baltic Sea, gave rise to the proposal (1999) for a Baltic Sea circular rail link of about 4,000 km (Figure 1). The route would go via Finland and the Baltic countries (Estonia, Latvia and Lithuania) and would constitute an international railway circuit linking capital cities around the Baltic.

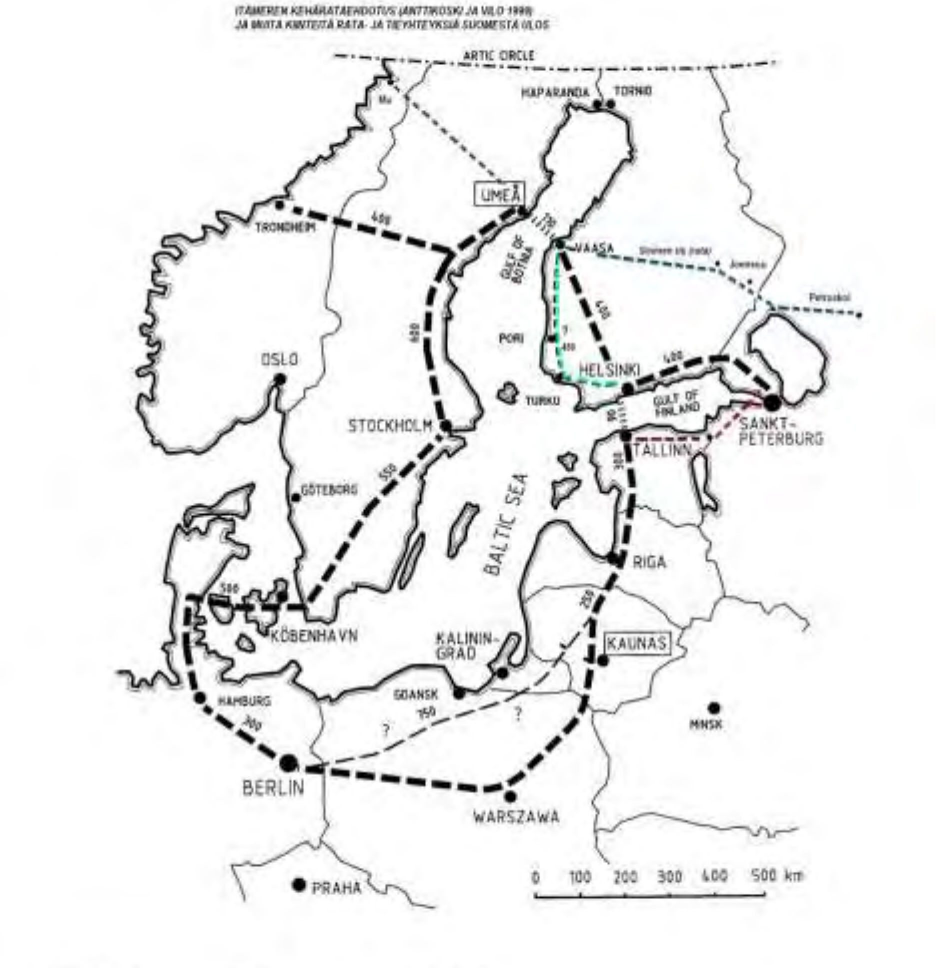


Figure 1. Outline to international fast trains route around the Baltic Sea

*Figure 1. Baltic Sea circular link proposal, 1999. Usko Anttikoski and Ago Vilo. 'Baltic Sea circular link via rock tunnels. Challenges for the 21st Century.' Proceedings of the World Tunnel Congress 99, Oslo, 31 May - 3 June 1999, vol. 2, pp 473-480 [1]. (The author has since added other rail proposals.)*

The fixed rail link from Denmark, Sweden and Norway across the Danish straits to mainland Europe was completed in 2000. This connection will be further improved when a new tunnel across the Fehmarn Belt between Denmark and Germany is completed in the 2020s. Rail services are nevertheless already running from the Nordic countries to other EU countries via Sweden, and on through the Alps as far as the Mediterranean (Figure 2).

Finland and Sweden are the EU's northernmost member states, but they have no port on the Arctic.

Led by Sweden, progress has been made in developing the Nordic countries' rail corridors. Sweden has applied for EU funding for a fast rail route to run as far as its border with Finland at Haparanda.

The construction of the Bothnian Corridor route is being followed closely by Finland, in the capacity of observer. No decisions on cross-border rail projects along Finland's western border have yet been made.

The situation in regard to the Baltic countries is also similar, as it has only been possible to plan rail projects in more recent years following the dissolution of the Soviet Union. A Rail Baltica route through the Baltic countries has nevertheless been planned and construction of this route is being supported by the EU's Transport Commissioner, Siim Kallas.

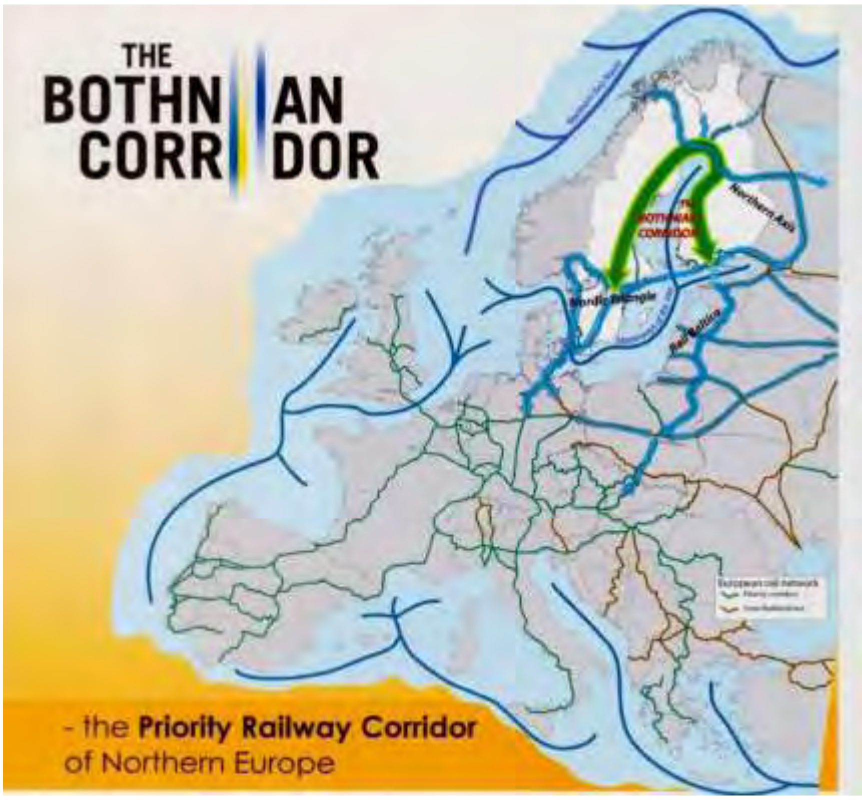


*Figure 2. The Bothnian Corridor (www.bothniancorridor.com)*

Commissioned by the City of Helsinki, a study of the bedrock for a Helsinki-Tallinn tunnel was begun in summer 2012 with the collection of existing geological data at geological institutions in both Finland and Estonia. Preliminary results will be published at the Strait Crossings 2013 symposium.

Work on the Helsinki and Tallinn twin-city development project has included completion of reports covering transport economics issues (www.euregio-heltel.org/httransplan/ (Studies)) [2]:

- 'Helsinki and Tallinn on the move. Final report of H-TTransPlan project', Ulla Tapaninen (ed.), Tallinn-Helsinki 2012

- 'Twin-city in making: Integration scenarios for Tallinn and Helsinki capital regions', Erik Terk (ed.), Tallinn 2012.

The trains using the Rail Baltica route would either be compatible with the European track gauge or would be fitted with gauge-changing equipment, allowing trains to travel to a regular timetable (Figure 3).

The EU's north-south Rail Baltica Corridor could continue via the Baltic Adriatic Corridor, thus linking the Arctic Ocean with the Mediterranean Sea (Figure 4).


*Figure 3. Rail Baltica Growth Corridor, Helsinki-Berlin. Olli Keinänen. 'Road map towards Helsinki-Tallinn transport strategy'. Presentation in Tallinn, April 2012 (http://www.rbgc.eu) [2]*


*Figure 4. Baltic Adriatic Corridor (http://www.BalticRail.com)*

# 3. EU'S NORTHERN RAIL CORRIDORS AND POSSIBLE ARCTIC PORT LOCATIONS

A report on the transport needs of the mining industry in northern Finland was completed in November 2013 [3]. The report presents rail routes to the Arctic Ocean and to Russia (Figure 5). The routes could also be used for passenger traffic.

*Figure 5. Finland's northern rail corridors (http://www.kaivosliikenne.info/) [3]*

The Finnish Transport Agency held a seminar on the needs of the mining industry and Arctic technology issues in Rovaniemi on 11-12 September 2012. The event was attended by participants from the Nordic countries and from Russia.

Mikko Niini of Aker Arctic Technology gave a presentation on the Northeast Passage and the related Arctic shipbuilding requirements (entitled 'The future of the Northeast Passage – What will it enable? Viewpoints on the discussion of Finland's link to the Arctic Ocean'). There were also presentations from a number of other northern region experts, from Norway, Sweden, Russia and Finland.

The event gave a considerable early boost to the cooperation between these countries.

Mikko Niini: The future of the Northeast Passage – What will it enable?

*Figure 6. Connecting the Arctic Ocean's Northeast Passage to Finland's rail network [3]*

## 4. RUSSIA'S NORTHERN RAIL CORRIDORS

Russia has carried out a number of studies of rail corridors between the Trans-Siberian railway and Finland, the Baltic countries and central Europe. The most important study from the Finnish perspective is the new Belkomur project (www.belkomur.com/), focusing on a new railway line in the Perm-Syktyvkar-Arkhangelsk corridor. Alternatively, the corridor could run from the Trans-Siberian railway via Kazakhstan to China and the Pacific Ocean (Figure 8). China has the international 1435 mm gauge, which means there would need to be gauge-switching where it joins the Russian 1520 mm gauge, and then again to 1435 mm for EU member states (Figure 8).


*Figure 7. The Belkomur project, part of Russia's northeast-west transport corridor*


*Figure 8. Eurasia's northern rail traffic corridors and the Belkomur rail construction project (Arkhangelsk-Syktyvkar-Perm)*

## 5. FINLAND'S RAIL CORRIDOR (BOTHNIAN CORRIDOR) AS AN INTERNATIONAL RAIL ROUTE

The Finnish rail network was constructed at a time when Finland was a Grand Duchy of the Russian Empire, and the network has retained its connection to the Russian system over its 150-year history. This is why the Finnish track gauge is still 1524 mm. The gauge in Russia and the Baltic countries is 1520 mm. Despite this small difference, Russian and Finnish trains use the same tracks, but it is uncertain whether ultrafast trains (300 km/h) of the future could also do this.

The Baltic countries have decided to build a new rail link from Tallinn to Warsaw at the EU gauge of 1435 mm. This will have implications for rail traffic between Finland and Estonia.

If Finland were to build a railway route along the shores of the Gulf of Bothnia to the European 1435 mm gauge, this would offer considerable potential for developing Finnish rail traffic to other EU countries. Such a route would allow European trains to move easily through Finland from the Baltic countries to Sweden and as far as the Arctic:

- The Rail Baltica route would continue as an EU route from the new Tallinn international transport centre to the Vimsi peninsula and across the Gulf of Finland by train ferry or tunnel to Porkkala in Kirkkonummi or, alternatively, to Pasila in Helsinki.

- The EU railway route would continue from Kirkkonummi to Vaasa via Salo, Turku and Pori.

- From Vaasa the rail link would continue across the Kvarken by train ferry or tunnel to Umeå.

- From Umeå the rail connections already exist to Norwegian ports and thus to the Atlantic and Arctic Oceans.

- The Swedish rail network would include a new connection via Haparanda to Tornio in Finland, and a new EU route would then run to the Norwegian Arctic port of Kirkenes, and possibly to the Russian Arctic port of Pechenga.

- Alternatively, the route could follow the existing Finnish network on its way to Kirkenes and Russia's northern rail network.

- The European network already connects Umeå to Sundsvall in Sweden and then to the Norwegian port of Trondheim. This allows the eastbound rail corridor from the Atlantic to traverse Finland en route to Russian Karelia and the port of Arkhangelsk and onwards to the Trans-Siberian route.

Finland's rail network could be modified to the European gauge in phases, such that European trains could run in the west of Finland while eastern parts of the country's network would accommodate the existing Finnish and Russian rolling stock.

## 6. RAILWAY TUNNELS UNDER THE BALTIC SEA

Proposals for fixed rail links from Finland across the Baltic Sea to Sweden and Estonia have been made for the past twenty years. The main railway tunnel connections proposed are across the Kvarken, across the Gulf of Finland, and on both sides of the Åland Islands (in the Archipelago Sea and the Åland Sea) [1].

All four of these are over 60 km in length and are subsea tunnels, constructed through the hard Precambrian bedrock. The lowest cost location is the Kvarken. The most demanding location is the Gulf of Finland, especially due to the soft rock on the Estonian landfall side. Long subsea railway tunnels would require extensive geological studies of the bedrock and tunnel engineering development work. This work should be done jointly by the Nordic countries (Finland, Sweden, Norway, Denmark and Estonia), as these countries have similar bedrock and possess tunnel construction technology. In Estonia and Denmark though, the hard Precambrian bedrock lies at a greater depth (more than 100 m deeper).

The biggest risks would be associated with construction of the first such long subsea railway tunnel, because of the lack of this kind of construction and operating experience in the Nordic countries.

The Kvarken tunnel would cost the least and be safest to build, and so it should be the first tunnel to be constructed, as a prototype. The experience gained could be used to good effect in the other railway tunnel projects. This would be a project of considerable international significance.

| Fixed transport connections across the Baltic Sea | | | | |
|---|---|---|---|---|
| Connection | tunnel km | surface km | altitude maximum +/- | constr. cost EUR million |
| ÅLAND ISLANDS | | | | |
| Åland Sea | | | | |
| Tunnel | 80 | 20 | -180 | 2 100 |
| Embankment (large sea bridge) | | 90 | 70 | over 5 000 |
| Railway and car ferry | (ferry 45 km) | 30 | | 300 |
| Archipelago Sea (and Åland Islands) | | | | |
| Tunnel | 80 | 60 | -160 | 2 400 |
| Embankment (large sea bridge) | | 140 | 30 | 1 300 |
| Tunnel under Åland as well | 110 | 30 | -160 | 2 900 |
| | | | | |
| GULF OF FINLAND | | | | |
| Porkkala | 70 | 12 | -220 | 2 300 |
| Pasila | 85 | 7 | -220 | 2 700 |
| | | | | |
| KVARKEN | | | | |
| Tunnel and embankment | 60 | 40 | -130 | 1 700 |
| 2 tunnels and embankment | 60+20=80 | 20 | -130 | 2 100 |

*Figure 9. Map and table. Fixed rail connections across the Baltic to Sweden and Estonia (Usko Anttikoski. Memorandum 14 September 2007, www.sgy.fi/ News / Gulf of Finland tunnel (Helsinki-Tallinn)) [1]*

## 7. TRAIN FERRIES

Traffic and transport issues for Finland and Estonia are discussed in Prime Minister's Office Publications report 7/2008 entitled 'Opportunities for Cooperation between Estonia and Finland

2008' (Jaakko Blomberg and Gunnar Okk). Pages 39-43 of the report include a discussion of the Rail Baltica plans and a rail tunnel between Helsinki and Tallinn. Regarding costs, the report states that the estimated cost of construction, in the region of EUR 3 billion, is so high that on the basis of current information it could not be financed by the private sector alone. The report's recommendations also include evaluation of the potential for a Helsinki-Tallinn train ferry service [1].

Finland no longer has a train ferry service to other EU countries, the last such service (from Turku to Sweden) being discontinued in autumn 2011. Furthermore, the former rail freight connections to Helsinki's port facilities have also been dismantled. A few years ago, however, a new rail tunnel for freight traffic from Kerava to Vuosaari Harbour was completed, though the tunnel was not designed with passenger traffic in mind.

Emission requirements for shipping (the Sulphur Directive 2015) are being tightened in the Baltic Sea and the North Sea, which is why Viking Line in autumn 2011 commissioned Finland's Turku shipyard to build a new car ferry powered by natural gas. No new train ferries have been ordered, however.

The advanced standards of Finnish shipbuilding technology are well known, and so the resources for designing and building a new train ferry should be available in Finland too. Any new train ferry would have to be designed to travel across the bays of the Baltic Sea, which vary in width from 50 to 70 km.

The train ferry concept would be something like the world's oldest vessel, Noah's Ark (Gen 6:15-16). The Ark was 150 m long, 25 m wide and 15 m high (for comparison, the M/S Star car ferry between Helsinki-Tallinn is 186 m long and 27.7 m wide, with a draught of 6.5 m and speed of 51 km/h).

The train ferry would have two or three decks. The intermediate deck could accommodate four tracks, making up a total track length of about 500 m. The lowest deck would house the technical facilities and water tanks for ballast control. The top deck would have the control centre, facilities for staff and passengers, and rescue equipment. The train deck could also be adjusted by hydraulic lifting gear if necessary.

Train ferries would have to incorporate a rapid docking capability. A new, rigid docking unit would be needed to facilitate smooth dockside track connections off the ferry and onto the rail network. This would also allow ferry-bound trains to be assembled in advance and driven straight from the fixed track onto the train ferry.

If the ferry's top speed at sea were to be 45 km/h, a sea crossing of 70 km would take about 1.5-2 hours. The entire travel time between the fixed rail networks on land at either end would then be 2-2.5 hours.

Train ferry docks could be built at existing Baltic Sea ports already equipped with a rail connection, or if necessary a new rail link could be built. Suitable ferry port locations in the Gulf of Finland on the Estonian side are the Vimsi peninsula (eastern side), Muuga Harbour, Tallinn's Old City Harbour, Paljassaare and Paldiski. On the Finnish side, suitable locations are the former dock at Vuosaari Harbour, the east side of Porkkala in Kirkkonummi, and the ports of Kantvik and Hanko.

In addition to connections across the Gulf of Finland, train ferry links could be introduced across the Kvarken and from the Åland Islands to both Finland and Sweden.

Rail links to the train ferry departure points on the mainland would be located close to the point where future rail tunnels would start. Train ferries could then help during tunnel construction work and would later function as backup connections for the tunnels.

Before any railway tunnel project is started, a train ferry link should first be set up, thereby establishing a site for the railway. In an emergency, both projects would serve as backup for each other.

## 8. RAIL CORRIDOR DEVELOPMENT: FURTHER COMMENTS

Traffic corridors traverse many different countries, which means planning must involve agreement on common standards. Such interoperability has already been achieved in international telecommunications, aviation and shipping, through agreement between the countries concerned. Harmonisation of this sort should also be sought for cross-border rail traffic.

It should be possible to use EU rolling stock in all parts of the EU, as well as in other countries that have rail traffic connections to the EU.

Finland's track gauge should be brought in line with the international 1435 mm gauge. Alternatively, an adjustable wheel gauge system could be developed on the rolling stock, allowing quick adjustment to the international or the Finnish gauge. For fast trains at least, the international gauge should be adopted.

English should be made the operating language on the railways, as is generally the case in other international activities. Flights from Europe to Asia have already been re-routed over the North Pole. Shipping will also move further north when climate change results in a shrinkage of the ice cap.

The northern regions of the Nordic countries and Russia are becoming a new focus of interest for transport and traffic.

## LITERATURE

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# PREDICTION OF WIND-INDUCED DYNAMIC RESPONSE AND FLUTTER STABILITY LIMIT OF LONG-SPAN BRIDGES USING THE FINITE ELEMENT METHOD

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**ABSTRACT**

*In this paper it is shown how unsteady aerodynamic forces can be implemented into the commercial finite element program Abaqus as an aeroelastic beam element. This new element can be used to predict buffeting response in strong winds and the flutter stability limit of bridges. The buffeting response and the flutter stability limit of the Hardanger Bridge are discussed in a comprehensive case study. It is concluded that the new aeroelastic beam element is able to capture the buffeting response and the flutter stability limit very well.*

## 1 INTRODUCTION

Dynamic response is often an important concern when designing slender steel bridges. Wind-induced dynamic response is in fact one of the most crucial issues when designing cable-supported bridges. Wind actions may cause vibrations during low mean wind velocities, buffeting vibration in strong winds as well aeroelastic instability phenomenon, such as static divergence, galloping and flutter. In this paper the main focus is on prediction of the flutter stability limit and buffeting response in strong winds. Traditionally wind-induced vibrations of cable-supported bridges have been predicted in the frequency domain, using a modal approach disregarding coupling effects among the still-air vibration modes. The aeroelastic stability limit has been estimated using wind tunnel tests with a scaled section model. In recent years it has been recognized that the buffeting response should be obtained using a multimode approach where the coupling effects are taken into account. This implies that the flutter stability limit of the combined structure and flow system will be captured by the buffeting response model.

When cable-supported bridges become longer and slenderer, it may become necessary to consider the nonlinear behaviour of the structure when predicting the dynamic response in strong winds. Nonlinearities can easily be taken into account in the time domain, and it is a clear advantage to use the degrees of freedom of the finite element model directly instead of using still-air vibration modes as generalized coordinates. This paper shows how the self-excited forces modelled by rational functions can be described in the time domain using first order differential equations. It is further shown how these forces can be modelled as additional aerodynamic degrees of freedom on Euler-Bernoulli beam elements. The element has been implemented in Abaqus as a user-defined element, and it is demonstrated that the element can be used to predict the flutter stability limit and the wind-induced response in strong winds.

# 2 THEORY

## 2.1 Modelling of self-excited forces – a review

The self-excited forces acting on a bridge deck section are commonly represented by the aerodynamic derivatives introduced in bridge engineering by Scanlan and Tomoko [1]. For a two-dimensional bridge deck section (see Figure 1), this can be expressed in matrix notation as follows:

$$\mathbf{q} = \mathbf{C}_{ae}(K)\dot{\mathbf{u}} + \mathbf{K}_{ae}(K)\mathbf{u}$$

$$\mathbf{C}_{ae}(K) = \frac{1}{2}\rho VKB\begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & P_1^* & P_5^* & BP_2^* \\ 0 & H_5^* & H_1^* & BH_2^* \\ 0 & BA_5^* & BA_1^* & B^2A_2^* \end{bmatrix}, \quad \mathbf{K}_{ae}(K) = \frac{1}{2}\rho V^2K^2\begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & P_4^* & P_6^* & BP_3^* \\ 0 & H_6^* & H_4^* & BH_3^* \\ 0 & BA_6^* & BA_4^* & B^2A_3^* \end{bmatrix} \tag{1}$$

$$\mathbf{q} = \begin{bmatrix} q_x & q_y & q_z & q_\theta \end{bmatrix}^T \quad \mathbf{u} = \begin{bmatrix} u_x & u_y & u_z & u_\theta \end{bmatrix}^T$$

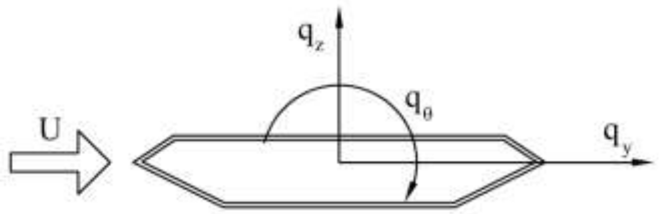



*Fig. 1: Aerodynamic forces acting on a cross section of a bridge deck. The section shown is from the Hardanger Bridge.*

Here, $V$ is the mean wind velocity; $\rho$ is the air density; $B$ is the width of the cross section; $K=B\omega/V$ is the reduced circular frequency of motion; the vector $\mathbf{u}$ contains the displacements along the girder, where $u_x$ symbolizes the longitudinal displacement, $u_y$ the transverse horizontal displacement, $u_z$ the transverse vertical displacement and $u_\Theta$ The rotation of the girder. The displacements are positive in the same direction as the forces displayed in Figure 1. $P_n^*$, $H_n^*$, $A_n^*$ $n\in\{1, 2,\ldots 6\}$ are the dimensionless aerodynamic derivatives, which are characteristic cross-sectional properties given as functions of the reduced frequency of motion. Eq. (1) is only valid for a single-frequency harmonic motion. However, by introducing the principle of superposition, Eq. (1) can be extended to any periodic or aperiodic motion by applying Fourier integral representation:

$$\mathbf{G}_\mathbf{q}(\omega) = \mathbf{F}(\omega)\mathbf{G}_\mathbf{u}(\omega)$$

$$\mathbf{F}(\omega) = \frac{1}{2}\rho V^2\begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & K^2\left(P_1^*i + P_4^*\right) & K^2\left(P_5^*i + P_6^*\right) & K^2B\left(P_2^*i + P_3^*\right) \\ 0 & K^2\left(H_5^*i + H_6^*\right) & K^2\left(H_1^*i + H_4^*\right) & K^2B\left(iH_2^* + H_3^*\right) \\ 0 & K^2B\left(A_5^*i + A_6^*\right) & K^2B\left(A_1^*i + A_4^*\right) & K^2B^2\left(A_2^*i + A_3^*\right) \end{bmatrix} \tag{2}$$

Here, $i$ is the imaginary unit, and $\mathbf{G}_\mathbf{X}(\omega)$ is the Fourier transform of $\mathbf{X}(t)$, where $\mathbf{X} \in \{\mathbf{u}, \mathbf{q}\}$, and the matrix $\mathbf{F}(\omega)$ contains the transfer functions defined in terms of the aerodynamic derivatives, which in this representation are treated as continuous functions of frequency.

The time-domain description of self-excited forces can be obtained by applying the inverse Fourier transform as shown, for instance ,by [2]. This results in the following equation:

$$\mathbf{q}(t)=\int_{-\infty}^{\infty}\mathbf{f}(t-\tau)\mathbf{u}(\tau)d\tau \tag{3}$$

Here, the matrix **f** contains the aerodynamic impulse-response functions that can be obtained by the inverse Fourier transform of the aerodynamic transfer functions defined in terms of the aerodynamic derivatives in Eq. (2). The aerodynamic derivatives are commonly known at discrete reduced frequencies and hence must be approximated with a curve fit. To be able to develop a time-domain representation of self-excited forces, the selected expression must be suitable for inverse Fourier transforming. The following expression has frequently been used in the literature, i.e., [2-5]

$$\mathbf{F}(\omega)=\frac{1}{2}\rho V^2\left(\mathbf{a}_1+\mathbf{a}_2\frac{i\omega B}{V}+\mathbf{a}_3\left(\frac{i\omega B}{V}\right)^2+\sum_{l=1}^{N-3}\mathbf{a}_{l+3}\frac{i\omega B/V}{i\omega B/V+d_l}\right) \tag{4}$$

This expression provides the following relation between the transfer function and the experimental data of the aerodynamic derivatives

$$\frac{\mathrm{Re}\left(\mathbf{F}(\omega)\right)}{\frac{1}{2}\rho V^2K^2}=\hat{V}^2\left(\mathbf{a}_1+\sum_{l=1}^{N-3}\mathbf{a}_{l+3}\frac{1}{\left[\left(d_l\hat{V}\right)^2+1\right]}\right),\quad \frac{\mathrm{Im}\left(\mathbf{F}(\omega)\right)}{\frac{1}{2}\rho V^2K^2}=\hat{V}\left(\mathbf{a}_2+\hat{V}^2\sum_{l=1}^{N-3}\mathbf{a}_{l+3}\frac{d_l}{\left[\left(d_l\hat{V}\right)^2+1\right]}\right) \tag{5}$$

Taking the inverse Fourier transform of the transfer function defined in Eq. (4) and inserting the resulting expression into Eq. (3) renders the following expression for the self-excited forces:

$$\begin{aligned}\mathbf{q}(t)&=\frac{1}{2}\rho V^2\left(\mathbf{a}_1\mathbf{u}(t)+\frac{B}{V}\mathbf{a}_2\dot{\mathbf{u}}(t)+\sum_{l=1}^{N-3}\mathbf{a}_{l+3}\int_{-\infty}^{\infty}\left(\delta(t-\tau)-\frac{d_lV}{B}H(t-\tau)\mathrm{e}^{-\frac{d_lV}{B}(t-\tau)}\right)\mathbf{u}(\tau)d\tau\right)\\ \mathbf{q}(t)&=\frac{1}{2}\rho V^2\left(\mathbf{a}_1\mathbf{u}(t)+\frac{B}{V}\mathbf{a}_2\dot{\mathbf{u}}(t)+\sum_{l=1}^{N-3}\mathbf{a}_{l+3}\left(\mathbf{u}(t)-\frac{d_lV}{B}\int_{-\infty}^{t}\mathrm{e}^{-\frac{d_lV}{B}(t-\tau)}\mathbf{u}(\tau)d\tau\right)\right)\end{aligned} \tag{6}$$

## 2.2 Finite element representation of self-excited forces

The starting point is the beam element displayed in *Fig. 2*. The element has two nodes, each with six displacement degrees of freedom. This implies that the displacements along the element are defined by the following displacement field:

$$\mathbf{u}(x,t)=\mathbf{N}(x)\mathbf{v}(t) \tag{7}$$

Here, $t$ is time; $x$ is the (local) element coordinate (see, for instance, [6]); the matrix $\mathbf{N}(x)$ contains the shape functions, and the vector $\mathbf{v}$ contains the nodal displacement degrees of freedom. The derivation of the mass, damping and stiffness matrices and the load vector, is well established and will not be dealt with here, but may be found in several textbooks, e.g., [6-8]. However, it is necessary to explain how the self-excited forces may be introduced into the finite element formulation. The self-excited nodal forces, $\mathbf{F}_{se}$, may be obtained by the principle of virtual work:

$$\delta\mathbf{v}^T\mathbf{F}_{Se}(t)=\delta\mathbf{v}^T\int_0^L\mathbf{N}^T(x)\mathbf{q}(x,t)dx \tag{8}$$

Here, $\delta\mathbf{v}$ symbolizes the virtual nodal displacements, and the vector $\mathbf{q}(x,t)$ contains the self-excited forces along the element. The self-excited force acting in each degree of freedom may then be expressed as follows:

$$\mathbf{F}_{Se}(t)=\frac{1}{2}\rho V^2\int_0^L\left(\mathbf{N}^T\mathbf{a}_1\mathbf{N}\mathbf{v}(t)+\frac{B}{V}\mathbf{N}^T\mathbf{a}_2\mathbf{N}\dot{\mathbf{v}}(t)+\sum_{l=1}^{N-3}\mathbf{N}^T\mathbf{a}_{l+3}\mathbf{N}\left(\mathbf{v}(t)-\frac{d_nV}{B}\int_{-\infty}^{t}\mathrm{e}^{-\frac{d_nV}{B}(t-\tau)}\mathbf{v}(\tau)d\tau\right)\right)dx \tag{9}$$

The expression can be rewritten as:

$$\begin{gathered}\mathbf{F}_{Se}(t)=\mathbf{A}_1\mathbf{v}(t)+\mathbf{A}_2\dot{\mathbf{v}}(t)+\mathbf{Z}(t)\\ \mathbf{A}_1=\frac{1}{2}\rho V^2\int_0^L \mathbf{N}^T\mathbf{a}_1\mathbf{N}dx,\quad \mathbf{A}_2=\frac{1}{2}\rho V^2\frac{B}{V}\int_0^L \mathbf{N}^T\mathbf{a}_2\mathbf{N}dx\\ \mathbf{Z}(t)=\frac{1}{2}\rho V^2\int_0^L\sum_{l=1}^{N-3}\mathbf{N}^T\mathbf{a}_{l+3}\mathbf{N}dx\left(\mathbf{v}(t)-\frac{d_nV}{B}\int_{-\infty}^{t}\mathrm{e}^{-\frac{d_nV}{B}(t-\tau)}\mathbf{v}(\tau)d\tau\right)\end{gathered}\tag{10}$$

The matrices $\mathbf{A}_1$ and $\mathbf{A}_2$ are referred to as aerodynamic stiffness and aerodynamic damping matrices, respectively. The vector **Z** contains the time-history-dependent terms. It is time

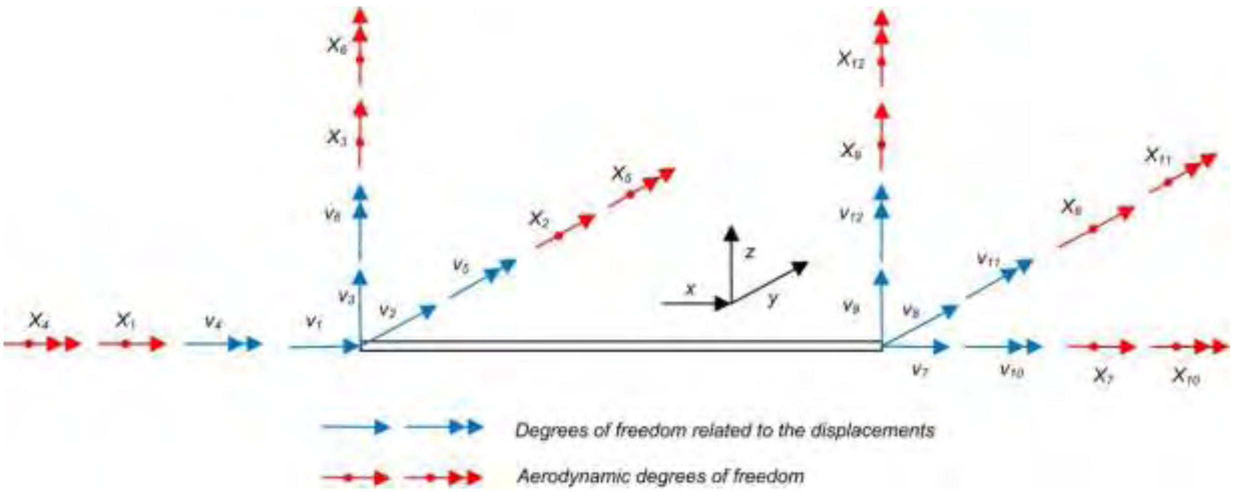



*Fig. 2: A three-dimensional aeroelastic beam element with 6 displacement degrees of freedom at each end node, in addition to the aerodynamic degrees of freedom needed for a consistent aeroelastic beam element when using one exponential filter to model the self-excited forces.*

consuming to evaluate the convolution integrals numerically. However, this can be avoided by introducing the convolution integrals as unknown in the system of equations. Considering the two-node beam element displayed in *Fig. 2*, the expression for **Z** may be rewritten as follows:

$$\begin{gathered}\mathbf{Z}=\mathbf{QX}\\ \mathbf{Q}=\begin{bmatrix}\mathbf{A}_4 & \mathbf{A}_5 & \cdots & \mathbf{A}_N\end{bmatrix}\qquad \mathbf{X}=\begin{bmatrix}\mathbf{x}_1^T & \mathbf{x}_2^T & \cdots & \mathbf{x}_{N-3}^T\end{bmatrix}^T\end{gathered}\tag{11}$$

Here, (N-3) is the number of exponential filters used in Eq. (4) to model the self-excited forces. The vector $\mathbf{x}_n$ contains convolution integrals, which can be generalized as:

$$\mathbf{x}_l=\left(\mathbf{v}(t)-\frac{d_lV}{B}\int_{-\infty}^{t}\mathrm{e}^{\left(-\frac{d_lV}{B}(t-\tau)\right)}\mathbf{v}(\tau)d\tau\right)\tag{12}$$

These types of convolution integrals also have to be evaluated when the still-air vibration modes are used as generalized coordinates. However, as shown by several authors, e.g., [4, 5, 9], these integrals can be modelled by first order differential equations, which can be obtained by taking the derivative of Eq. (12). Since the matrix related to the second derivative of the variables has to be inverted to calculate the initial second derivative of the variables, it is convenient to introduce the second derivative of Eq. (12)

$$\ddot{\mathbf{x}}_l=\ddot{\mathbf{v}}(t)-\frac{d_lV}{B}\dot{\mathbf{x}}_l\tag{13}$$

This implies that the additional equations needed, if the elements in the vector **X** are introduced as unknowns in the system of equations, can be obtained from Eq. (13). The aerodynamic degrees of freedom required to model the self-excited forces for one exponential filter are shown in *Fig. 2*. Hence, the equations of motion for a linear aeroelastic beam element may be written as:

$$\begin{bmatrix} \mathbf{M} & \mathbf{0} \\ -\mathbf{E} & \mathbf{B} \end{bmatrix} \begin{bmatrix} \ddot{\mathbf{v}} \\ \ddot{\mathbf{X}} \end{bmatrix} + \begin{bmatrix} (\mathbf{C}-\mathbf{A}_2) & \mathbf{0} \\ \mathbf{0} & \mathbf{D} \end{bmatrix} \begin{bmatrix} \dot{\mathbf{v}} \\ \dot{\mathbf{X}} \end{bmatrix} + \begin{bmatrix} (\mathbf{K}-\mathbf{A}_1) & -\mathbf{Q} \\ \mathbf{0} & \mathbf{0} \end{bmatrix} \begin{bmatrix} \mathbf{v} \\ \mathbf{X} \end{bmatrix} = \begin{bmatrix} \mathbf{P} \\ \mathbf{0} \end{bmatrix} \tag{14}$$

Here:

$$\mathbf{E} = \begin{bmatrix} \mathbf{I} \\ \mathbf{I} \\ \vdots \\ \mathbf{I} \end{bmatrix}, \quad \mathbf{B} = \begin{bmatrix} \mathbf{I} & & & \\ & \mathbf{I} & & \\ & & \ddots & \\ & & & \mathbf{I} \end{bmatrix}, \quad \mathbf{D} = \frac{V}{B} \begin{bmatrix} d_1\mathbf{I} & & & \\ & d_2\mathbf{I} & & \\ & & \ddots & \\ & & & d_{N-3}\mathbf{I} \end{bmatrix} \tag{15}$$

Here **M**, **C** and **K** symbolize the mass damping and stiffness matrices, **P** the dynamic actions and **I** the 12 by 12 identity matrix. The total number of degrees of freedom for the element is the sum of the displacement degrees of freedom and the aerodynamic degrees of freedom related to the self-excited forces displayed in *Fig. 2*, multiplied by the number of exponential filters. This implies that if the experimental data representing the self-excited forces are approximated by using two exponential filters in Eq. (4), the total number of degrees of freedom will become $N_{dof} = 12 + 12 \cdot 2 = 36$. This number of degrees of freedom may seem high, but this approach requires far less computational effort than solving the 24 convolution integrals numerically in each time step. The additional aerodynamic degrees of freedom must be taken into account when the system of equations for the global model is established. The displacements at a node that two beam elements have in common are equal. When the aerodynamic degrees of freedom displayed in *Fig. 2* and defined in Eq.(12) are considered, it is seen that if the two neighbouring elements have the same aerodynamic properties, the degrees of freedom will have the same values for both elements. This implies that the commonly applied assembly technique may be used for the aerodynamic degrees of freedom related to the self-excited forces.

### 2.3 Implementation of the aeroelastic element in Abaqus

The time domain simulations of the dynamic response presented in this paper have been performed using the commercial finite element program Abaqus. The aeroelastic element presented in the section above has been implemented as a user subroutine. The HHT-α [10] algorithm is used in Abaqus to solve the dynamic equilibrium, but since α is assumed to be zero in the calculations performed in this paper, the algorithm is equal to the well-known Newmark-beta method [11], which simplifies the implementation of the element slightly. In the Newmark-beta method the equilibrium is satisfied at time increments, and at $t_{n+1}$ the equilibrium reads [12]

$$\bar{\mathbf{M}}\ddot{\mathbf{v}}_{n+1} + \mathbf{g}(\dot{\mathbf{v}}_{n+1}, \mathbf{v}_{n+1}) = \mathbf{f}_{n+1} \tag{16}$$

Here, $\bar{\mathbf{M}}\ddot{\mathbf{v}}_{n+1}$ represents nodal forces related to the accelerations; $\mathbf{g}(\dot{\mathbf{v}}_{n+1}, \mathbf{v}_{n+1})$ represents forces related to the displacements and velocities, while $\mathbf{f}_{n+1}$ represents the dynamic actions. In Abaqus the solution is obtained by Newton iterations on the residual **r**, implying that the residual and its total derivative (Jacobian matrix) **J** must be given in the subroutine.

$$\begin{aligned} \mathbf{r} &= \mathbf{f}_{n+1} - \bar{\mathbf{M}}\ddot{\mathbf{v}}_{n+1} - \mathbf{g}(\dot{\mathbf{v}}_{n+1}, \mathbf{v}_{n+1}) \\ \mathbf{J} &= -\frac{d\mathbf{r}}{d\mathbf{V}} = -\frac{\partial \mathbf{r}}{\partial \mathbf{V}}\frac{\partial \mathbf{V}}{\partial \mathbf{V}} - \frac{\partial \mathbf{r}}{\partial \dot{\mathbf{V}}}\frac{\partial \dot{\mathbf{V}}}{\partial \mathbf{V}} - \frac{\partial \mathbf{r}}{\partial \ddot{\mathbf{V}}}\frac{\partial \ddot{\mathbf{V}}}{\partial \mathbf{V}} \end{aligned} \tag{17}$$

## 3 CASE STUDY: THE HARDANGER BRIDGE

A simplified model of the Hardanger Bridge will be considered in the case study. The Hardanger Bridge is currently under construction. When it is completed, it will have a main span of 1310m and towers that are 186m high. The distance between the two main cables is only 14.5 m, which means that it will become one of the slenderest bridges in the world. Curve fits to the aerodynamic derivatives defined in Eq. (1) are shown in *Fig. 3***.** The markers represent experimental data, while the curves represent the rational functions defined in (4), where two exponential filters have been used. A simplified model of the Hardanger Bridge has been created in Abaqus. Some of the most relevant vibration modes are shown in *Fig. 4* together with the natural frequencies and damping ratios.

### 3.1 Stability limit

The stability of an aeroelastic system can be studied by considering its eigenvalues [14-17]. The eigenvalues are in general complex $z_n=a_n+b_n$, where $a_n$ is related to the damping of the aeroelastic system, while $b_n$ is related to the frequency. This implies that when $a_n<0$, the system is stable, while it becomes unstable when $a_n>0$. Since the self-exited forces change the stiffness and damping properties of the system, the eigenvalues needs to be considered for a range of mean wind velocity. An example of how the frequencies and damping ratios change with increasing wind velocity is shown in Fig. 5. Here the imaginary and real part of the eigenvalues are represented by the frequency and the damping ratio respectively. The calculations have been performed considering seven still-air vibration modes. As can be seen from the figure, the damping for one of the modes becomes zero at about 85 m/s, which implies that the structure is unstable. The stability limit of the simplified model of the Hardanger Bridge has also been calculated using the first 200 still-air vibration modes as generalized coordinates. This provided a stability limit of $V_{CR}$=79 m/s. In Fig 6 the vertical response at the mid-span at three mean wind velocities is displayed. The bridge is subjected to impulsive loading at the mid-span. When the mean wind velocity is smaller than the critical velocity, the system clearly has positive damping. When the mean wind velocity is equal to the stability limit, the response obtained using Abaqus appears to be stationary, indicating that the damping is very close to zero. When the mean wind velocity is larger than the stability limit, the system is clearly unstable, resulting in divergent motion. This confirms that the element is capable of capturing the flutter phenomenon very well.

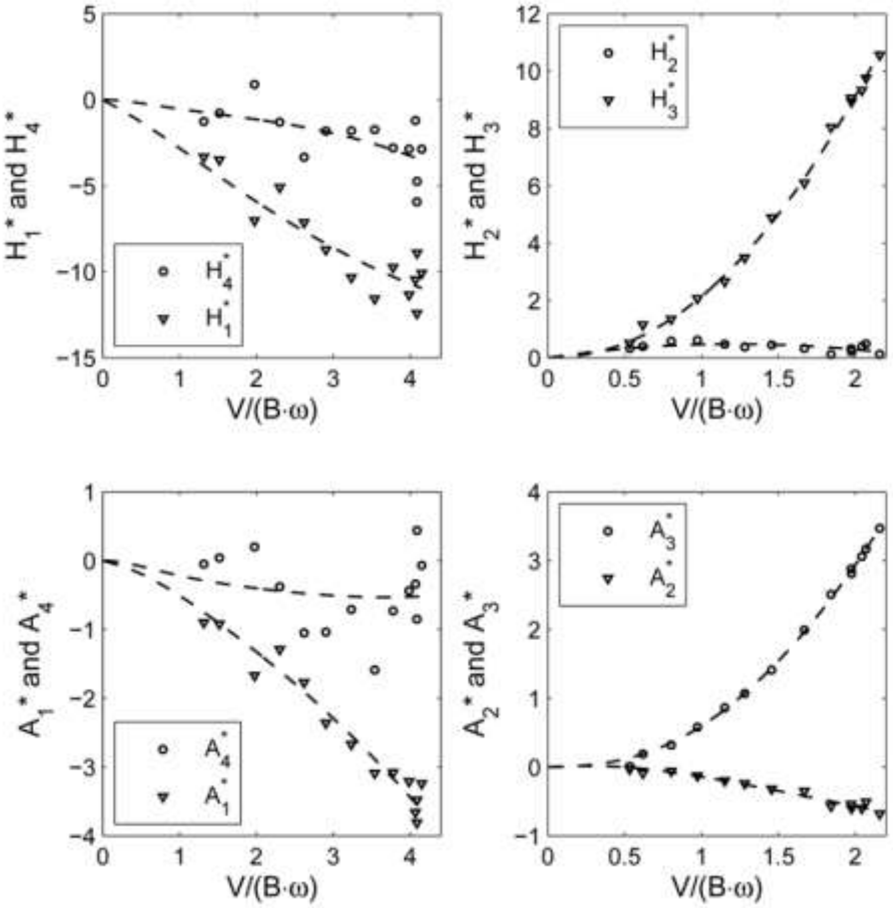
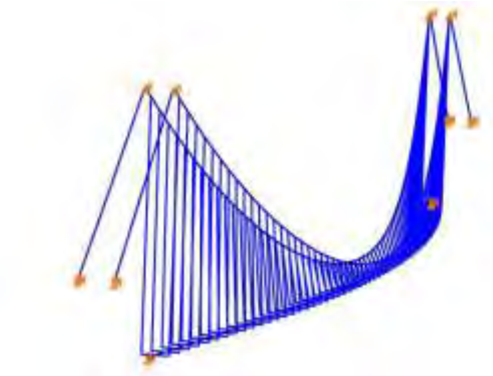
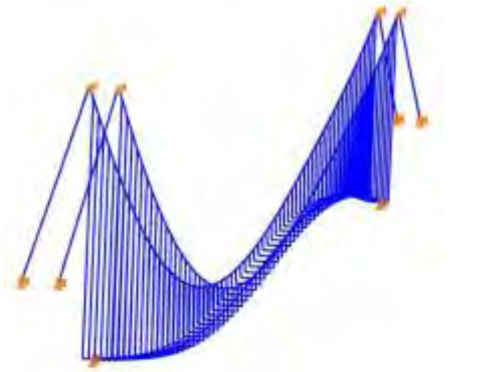


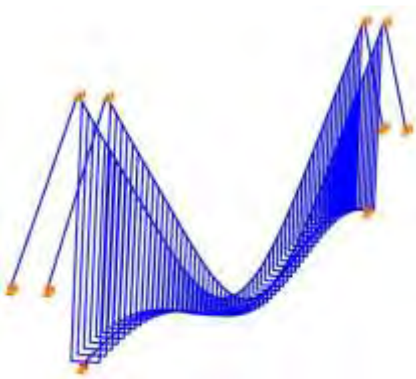

*Fig. 3: Curve fits of rational functions defined in Eq.(5) to the experimental results of the aerodynamic derivatives from [13]*



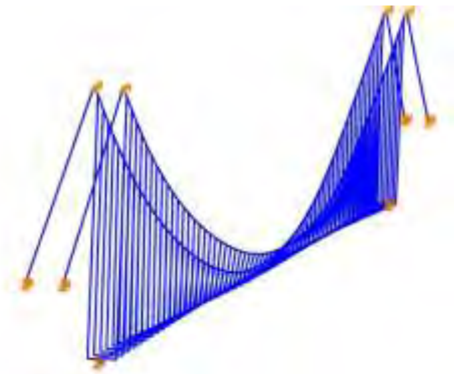
$\omega = 0.31(rad/s),\ \xi = 0.56\%$



$\omega = 0.67(rad/s),\ \xi = 0.26\%$



$\omega = 0.88(rad/s),\ \xi = 0.23\%$



$\omega = 2.38(rad/s),\ \xi = 0.45\%$

*Fig. 4: Selected vibration modes of the Hardanger Bridge.*

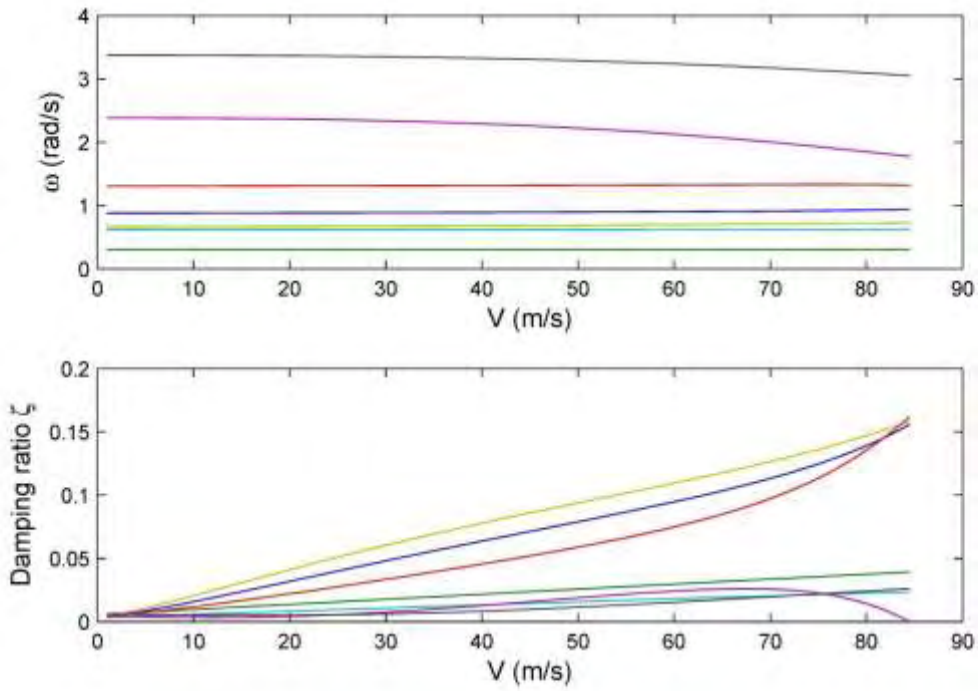



*Fig. 5: Top figure: In-wind frequencies. Bottom figure: In wind damping ratios*

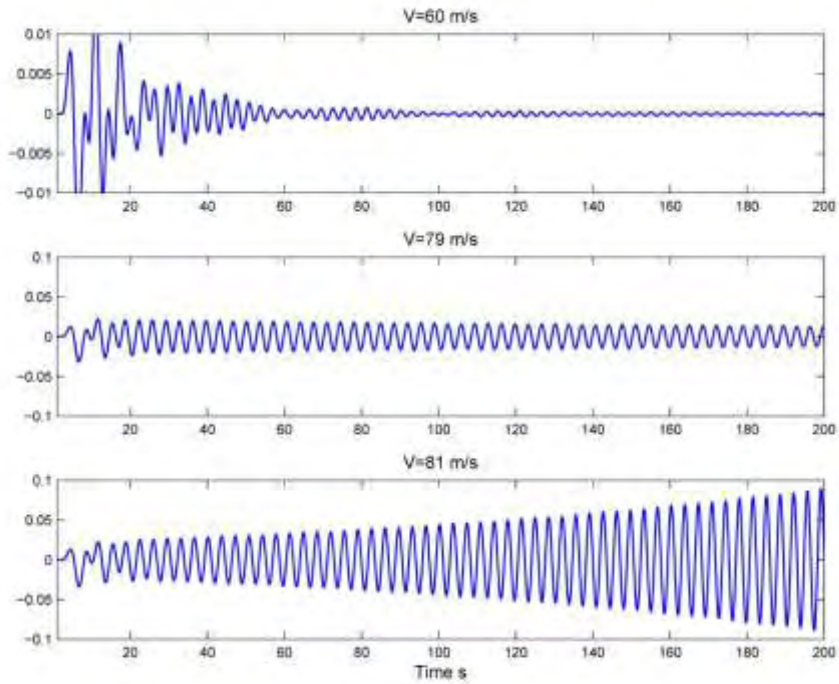



*Fig. 6. Wind-induced buffeting response at a mean wind velocity of 40 m/s*

### 3.2 Buffeting response

The cross-spectral densities of the wind field are assumed to be given by:

$$S_{uu}^{+}(\omega)=\frac{40.58Vz\kappa}{\left(1+9.74\,\omega z/V\right)^{5/3}}\exp(-1.4\frac{\Delta x\omega}{V}),\quad S_{ww}^{+}(\omega)=\frac{0.82Vz\kappa}{\left(1+0.79\,\omega z/V\right)^{5/3}}\exp(-\frac{\Delta x\omega}{V})$$
$$S_{uw}^{+}(\omega)=-\frac{2.23Vz\kappa}{\left(1+1.67\,\omega z/V\right)^{7/3}}\exp(-\frac{\Delta x\omega}{V}) \qquad (21)$$

Here, $S_{uu}^{+}(\omega)$ is the cross-spectral density of the along-wind component $u(t)$ at two points along the beam with separation $\Delta x$; likewise, $S_{ww}^{+}(\omega)$ is the cross-spectral density of the vertical across-wind component $w(t)$, while $S_{uw}^{+}(\omega)$ is the cross-spectral density of $u(t)$ and $w(t)$ at the two points. The height above ground is denoted as $z$ and is taken as 50 m, while $\kappa$ is the roughness coefficient at the site, which is assumed to be 0.0031. Time series of the fluctuating turbulence components at 65 points along the beam are obtained by Monte Carlo simulations [18-20], with a cut-off frequency of $\omega_u$=20 rad/s and $\Delta\omega$=0.0005 rad/s. The time series at point $m$ can then be obtained by

$$x_m(t)=\sqrt{2\Delta\omega}\,\mathrm{Re}\left(\sum_{l=1}^{m}\sum_{k=1}^{N}L_{ml}(\omega_k)\exp\left(i(\omega_k+\phi_{lk})\right)\right) \qquad (22)$$

Here, $L_{ml}(\omega_k)$ denotes the elements of the lower triangular matrix obtained by factorising the cross-spectral density matrix according to the relation

$$\mathbf{S}(\omega_k)=\mathbf{L}(\omega_k)\mathbf{L}^{*}(\omega_k) \qquad (23)$$

where the elements in $\mathbf{S}(\omega_k)$ represent the cross-spectral densities of the fluctuating velocity components at the 65 points along the girder. Since the main focus in the current study is modelling of the self-excited forces, the cross sectional admittance functions' influence on the response has been assumed negligible. This implies that the buffeting action at point $n$ is given by [21, 22]

$$\mathbf{q}_{Buff} = \begin{bmatrix} q_x & q_y & q_z & q_\theta \end{bmatrix}^T$$
$$\mathbf{q}_{Buff}(x_n,t) = \frac{\rho VB}{2}\begin{bmatrix} 0 & 0 \\ 2(D/B)\bar{C}_D & (D/B)C_D' - \bar{C}_L \\ 2\bar{C}_L & C_L' + (D/B)\bar{C}_D \\ 2B\bar{C}_M & BC_M' \end{bmatrix}\begin{bmatrix} u(t) \\ w(t) \end{bmatrix} \tag{24}$$

Here, $q_n$ $n \in \{x, y, z, \theta\}$ is the buffeting action in the coordinate system of the element; $D$ is the height of the girder, and $B$ symbolizes the width of the girder. $C_n$ , $n \in \{D, L, M\}$ symbolizes the drag lift and overturning moment force coefficient, where a bar represents the mean value, while a prime denotes a derivative with respect to the angle of attack.

The standard deviations of the horizontal, vertical and torsional response of 10 time domain simulations at a mean wind velocity of 40 m/s are shown in Fig.7 together with frequency domain results, where still-air vibration modes are used as generalized coordinates. As can be seen from the results, the time domain simulations correspond very well to the frequency domain results, indicating that the time-finite element introduced in this paper is able to capture the behaviour of the structure very well.

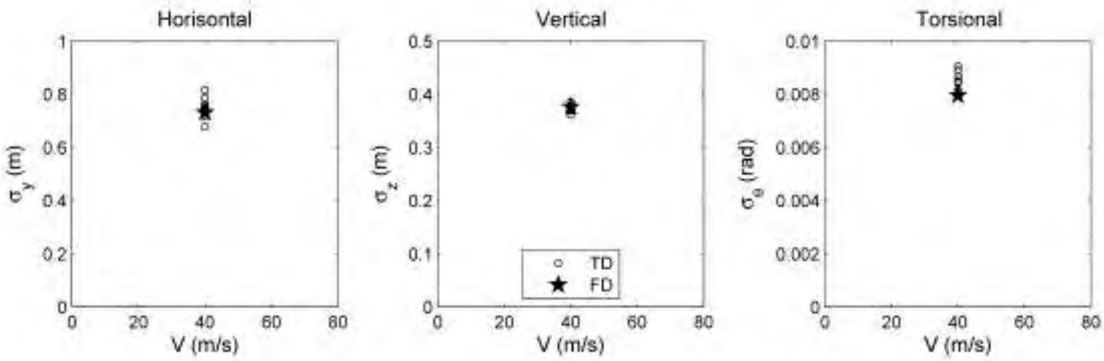



*Fig. 7: Standard deviation of the horizontal, vertical and torsional response. The star represents frequency domain results, while the circles represent time domain simulations.*

## 3 CONCLUDING REMARKS

It has been demonstrated how self-excited aerodynamic forces modelled by rational functions can be introduced into a finite element beam model in terms of aerodynamic degrees of freedom. The aeroelastic beam element has been implemented successfully in the commercial finite element program Abaqus. The flutter stability limit and the wind-induced buffeting response obtained using the aerodynamic element corresponded very well to frequency domain results, where still-air vibration modes are used as generalized coordinates. This confirms that the aerodynamic beam element developed in this paper can be used to predict the buffeting response and the flutter stability limit of long-span bridges.

## ACKNOWLEDGEMENTS

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# PREDICTION OF WAVE INDUCED DYNAMIC RESPONSE IN TIME DOMAIN USING THE FINITE ELEMENT METHOD

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**ABSTRACT**

*Dynamic response induced by wave and wind action is the most important concerns when designing long submerged floating tunnels and floating bridges at exposed locations. The response has commonly been predicted using linear power spectral density methods, but since the bridges will become longer and more flexible in future projects, structural nonlinearities may have significant influence on the dynamic behaviour. The authors are currently developing computational procedure for prediction of dynamic response of floating and submerged tunnels using time domain step by step integration. Preliminary results using the Bergsøysund Bridge as a case study are presented in this paper.*

## 1 INTRODUCTION

Wave induced dynamic response is one of many important quantities when designing slender floating bridges. The wave induced response are often assessed applying linear power spectral density techniques since the wave actions are stochastic in nature and the hydrodynamic coefficients in the system of differential equations are functions of frequency [1]. When the structures become slenderer, structural nonlinearities may become significant and have to be taken into account in the design, which implies that nonlinear methods combined with Monte Carlo simulation become feasible.

In this paper it is demonstrated how the frequency dependent added mass and damping coefficients can be converted to the time domain applying the Fourier transform. It is further shown how the corresponding memory effects can be modelled as first order differential equations and how these can be solved within a nonlinear finite element framework. Modelling of sea states as a homogeneous random field is also thoroughly discussed together with how the cross-spectral density of the wave actions can be obtained using a linear transfer functions derived assuming small amplitude waves. Having the cross-spectral density of the wave actions it is shown how Monte Carlo simulations can be used to obtain time series of the wave actions.The framework outlined above has been implemented into the commercial finite element software Abaqus. The proposed framework has been tested by calculating the dynamic response of the Bergsøysund Bridge subjected to wave action represented by system of forces and moments acting on each pontoon (see Figure 1).

*Figure 15: The Bergsøysund Bridge*

# 2 MODELLING

## 2.1 Motion induced forces

The hydrodynamic mass and damping of the study structure are frequency dependent which implies that the corresponding time domain properties will be time related. This is visualised as a memory process in the fluid-structure system in the time domain representation. For single harmonic component, exp(-iωt), with frequency ω, the motion induced part of the hydrodynamic force can be written as

$$\mathbf{q}(\omega,t) = \mathbf{M}_{hd}(\omega)\ddot{\mathbf{u}}(t) + \mathbf{C}_{hd}(\omega)\dot{\mathbf{u}}(t) + \mathbf{K}_{hd}\mathbf{u}(t) \tag{18}$$

Here, $\mathbf{M}_{hd}$ and $\mathbf{C}_{hd}$ are the frequency dependent hydrodynamic mass and potential damping matrices, $\mathbf{K}_{hd}$, is the hydrostatic restoring matrix, while $\mathbf{u}$ symbolizes the displacements of the structure. Eq. (18) is only valid for a single-frequency harmonic motion. However, by introducing the principle of superposition, Eq. (18) can in practice be extended to any periodic or aperiodic motion by applying Fourier integral representation. Hence, the motion induced forces can be expressed as follows:

$$\begin{aligned} \mathbf{G}_{\mathbf{q}}(\omega) &= \mathbf{F}_{hd}(\omega)\mathbf{G}_{\mathbf{u}}(\omega) \\ \mathbf{F}_{hd}(\omega) &= -\omega^2\mathbf{M}_{hd}(\omega) + i\omega\mathbf{C}_{hd}(\omega) + \mathbf{K}_{hd} \end{aligned} \tag{19}$$

Here, $i$ is the imaginary unit, and $\mathbf{G}_{\mathbf{X}}(\omega)$ is the Fourier transform of $\mathbf{X}(t)$, where $\mathbf{X} \in \{\mathbf{u}, \mathbf{q}\}$, and the matrix $\mathbf{F}(\omega)$ contains the hydrodynamic transfer functions defined in terms of the hydrodynamic mass, potential damping and restoring matrices, which in this representation are treated as continuous functions of frequency.

The time domain representation of the motion induced forces can be obtained applying the inverse Fourier transform to Eq. (19). This results in the following equation:

$$\mathbf{q}(t) = \int_{-\infty}^{\infty} \mathbf{f}_{hd}(t-\tau)\mathbf{u}(\tau)d\tau \tag{20}$$

Here, the matrix $\mathbf{f}$ contains the fluid-structure interaction impulse response functions defined in terms of the hydrodynamic mass, potential damping and restoring matrices. Since the hydrodynamic mass and potential damping coefficients are obtained at discrete frequencies it is convenient to curve fit the data using the following expression [2, 3]:

$$\mathbf{F}_{hd}(\omega) = \mathbf{a}_1 + \mathbf{a}_2 i\omega + \mathbf{a}_3 \left(i\omega\right)^2 + \sum_{l=1}^{N-3} \mathbf{a}_{l+3} \frac{i\omega}{i\omega + d_l} \tag{21}$$

The model coefficients can be determined by fitting the expression, either to numerical or experimental data describing the frequency dependence of the fluid-structure interaction system. In practice this can be done by using the following real expressions:

$$\mathrm{Im}(\mathbf{F}_{hd}(\omega)) = \mathbf{a}_2\omega + \sum_{l=1}^{N} \mathbf{a}_{l+3} \frac{d_l \omega}{d_l^2 + \omega^2}$$

$$\mathrm{Re}(\mathbf{F}_{hd}(\omega)) = \mathbf{a}_1 - \mathbf{a}_3\omega^2 + \sum_{l=1}^{N} \mathbf{a}_{l+3} \frac{\omega^2}{d_l^2 + \omega^2} \tag{22}$$

The elements in the matrices, $\mathbf{a}_n$ $n \in \{1\ 2\ 3\ l+3\}$, and the constants $d_l$ are determined by a least square fit to the data, i.e. the frequency dependent matrices of Eq.(2).

The hydrodynamic impulse response functions can be obtained formally by taking the Fourier transform of Eq. (21). That gives the following expression:

$$\mathbf{f}_{hd}(\tau) = \mathbf{a}_1\delta(\tau) + \mathbf{a}_2\dot{\delta}(\tau) + \mathbf{a}_3\ddot{\delta}(\tau) + \sum_{l=1}^{N-3} \mathbf{a}_{l+3}\left(\delta(\tau) - d_l\, \mathrm{e}^{-d_l\tau} H(\tau)\right) \tag{23}$$

Here, $\delta$ represents the Dirac delta function, the dots indicate time derivative, and $H$ is the Heaviside unit step function. Inserting the hydrodynamic impulse response function into Eq. (20) renders the following expression for the motion induced forces in time domain:

$$\mathbf{q}(t) = \mathbf{a}_1\mathbf{u}(t) + \mathbf{a}_2\dot{\mathbf{u}}(t) + \mathbf{a}_3\ddot{\mathbf{u}}(t) + \sum_{l=1}^{N-3} \mathbf{a}_{l+3} \int_{-\infty}^{\infty} \left(\delta(t-\tau) - d_l H(t-\tau)\, \mathrm{e}^{-d_l(t-\tau)}\right)\mathbf{u}(\tau)d\tau \tag{24}$$

The equation is readily rewritten as:

$$\mathbf{q}(t) = \mathbf{a}_1\mathbf{u}(t) + \mathbf{a}_2\dot{\mathbf{u}}(t) + \mathbf{a}_3\ddot{\mathbf{u}}(t) + \sum_{l=1}^{N-3} \mathbf{a}_{l+3} \left(\mathbf{u}(t) - d_l \int_{-\infty}^{t} \mathrm{e}^{-d_l(t-\tau)}\mathbf{u}(\tau)d\tau\right) \tag{25}$$

This time domain representation of the hydrodynamic forces is equivalent to frequency domain model given by Eq.(1). The time domain hydroelastic coefficients will be exemplified in Section 3 below.

## 2.2 Finite element representation of motion induced forces

We consider the pontoon of the floating bridge as a rigid body attached to the flexible girder of the bridge system. Then the following equation of motion applies:

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{K}\mathbf{u} + \mathbf{q}(t) = \mathbf{P}(t) \tag{26}$$

Here **M**, **C**, and **K** represent the structural mass, damping and stiffness matrices respectively. i.e. properties of the system without water (sometimes refered to as the dry system properties), while **u** symbolizes the displacements of the pontoon. The motion-induced hydrodynamic forces are denoted **q**, contain the wet system additional mass, radiational damping, and restoring, while **P** represents the wave action. From Eq.(1.8) we get:

$$\mathbf{q}(t) = \mathbf{a}_1\mathbf{u}(t) + \mathbf{a}_2\dot{\mathbf{u}}(t) + \mathbf{a}_3\ddot{\mathbf{u}}(t) + \mathbf{Z}(t)$$

$$\mathbf{Z}(t) = \sum_{l=1}^{N-3} \mathbf{a}_{l+3} \left(\mathbf{u}(t) - d_l \int_{-\infty}^{t} \mathrm{e}^{-d_l(t-\tau)}\mathbf{u}(\tau)d\tau\right) \tag{27}$$

$$\mathbf{Z} = \mathbf{Q}\mathbf{X}$$

$$\mathbf{Q} = \begin{bmatrix}\mathbf{A}_4 & \mathbf{A}_5 & \cdots & \mathbf{A}_N\end{bmatrix} \qquad \mathbf{X} = \begin{bmatrix}\mathbf{x}_1^T & \mathbf{x}_2^T & \cdots & \mathbf{x}_{N-3}^T\end{bmatrix}^T \tag{28}$$

Taking the derivative of the time history dependent term $\mathbf{x}_l$(t) gives us the following relation:

$$\dot{\mathbf{x}}_l = \mathbf{u} - d_l\mathbf{x}_l \tag{29}$$

The equation of motion can then be written as follows:

$$\begin{bmatrix}(\mathbf{M}+\mathbf{a}_3) & \mathbf{0}\\ \mathbf{0} & \mathbf{0}\end{bmatrix}\begin{bmatrix}\ddot{\mathbf{u}}\\ \ddot{\mathbf{X}}\end{bmatrix} + \begin{bmatrix}(\mathbf{C}+\mathbf{a}_2) & \mathbf{0}\\ -\mathbf{E} & \mathbf{B}\end{bmatrix}\begin{bmatrix}\dot{\mathbf{u}}\\ \dot{\mathbf{X}}\end{bmatrix} + \begin{bmatrix}(\mathbf{K}+\mathbf{a}_1) & \mathbf{Q}\\ \mathbf{0} & \mathbf{D}\end{bmatrix}\begin{bmatrix}\mathbf{u}\\ \mathbf{X}\end{bmatrix} = \begin{bmatrix}\mathbf{P}\\ \mathbf{0}\end{bmatrix} \tag{30}$$

$$\mathbf{E}=\begin{bmatrix}\mathbf{I}\\\mathbf{I}\\\vdots\\\mathbf{I}\end{bmatrix},\quad \mathbf{B}=\begin{bmatrix}\mathbf{I}&&&\\&\mathbf{I}&&\\&&\ddots&\\&&&\mathbf{I}\end{bmatrix},\quad \mathbf{D}=\begin{bmatrix}d_1\mathbf{I}&&&\\&d_2\mathbf{I}&&\\&&\ddots&\\&&&d_{N-3}\mathbf{I}\end{bmatrix} \tag{31}$$

In order to use traditional integration schemes, e.g. the Newmark's β-methods, to solve the equation of motion, it is convenient to take the derivative of Eq.(27) two times instead of ones. This provides the following equation of motion for each hydrodynamic element:

$$\begin{bmatrix}(\mathbf{M}+\mathbf{a}_3) & \mathbf{0}\\ -\mathbf{E} & \mathbf{B}\end{bmatrix}\begin{bmatrix}\ddot{\mathbf{u}}\\ \ddot{\mathbf{X}}\end{bmatrix}+\begin{bmatrix}(\mathbf{C}+\mathbf{a}_2) & \mathbf{0}\\ \mathbf{0} & \mathbf{D}\end{bmatrix}\begin{bmatrix}\dot{\mathbf{u}}\\ \dot{\mathbf{X}}\end{bmatrix}+\begin{bmatrix}(\mathbf{K}+\mathbf{a}_1) & \mathbf{Q}\\ \mathbf{0} & \mathbf{0}\end{bmatrix}\begin{bmatrix}\mathbf{u}\\ \mathbf{X}\end{bmatrix}=\begin{bmatrix}\mathbf{P}\\ \mathbf{0}\end{bmatrix} \tag{32}$$

Then applying a standard assemble procedures the global system equation can be created by adding the dry elemental matrices to the wet elemental matrices represented, in principle, by Eq.(15) above.

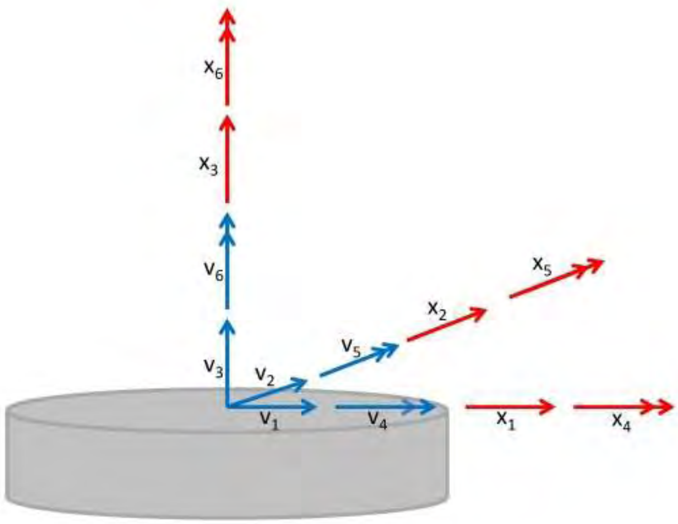



*Figure 16: Hydrodynamic element with 6 displacement (blue) and 6 hydrodynamic (red) degrees of freedom*

### 2.3 Implementation of the hydroelastic element in Abaqus

The time domain simulations of the dynamic response presented in this paper have been performed using the commercial finite element program Abaqus. The hydroelastic element presented in the section above has been implemented as a user subroutine. The HHT-α [4] algorithm is used in Abaqus to solve the dynamic equilibrium, but since α is assumed to be zero in the calculations performed in this paper, the algorithm is equal to the well-known

Newmark's β-method [5], which simplifies the implementation of the element. In the Newmark's β-method the equilibrium is satisfied at time increments, and at $t_{n+1}$ the equilibrium reads [6]

$$\bar{\mathbf{M}}\ddot{\mathbf{v}}_{n+1}+\mathbf{g}(\dot{\mathbf{v}}_{n+1},\mathbf{v}_{n+1})=\mathbf{f}_{n+1} \tag{33}$$

Here, $\bar{\mathbf{M}}\ddot{\mathbf{v}}_{n+1}$ represents nodal inertia forces, $\mathbf{g}(\dot{\mathbf{v}}_{n+1},\mathbf{v}_{n+1})$ represents forces related to the displacements and velocities, while $\mathbf{f}_{n+1}$ represents the dynamic actions. In Abaqus the solution is obtained by Newton iterations on the residual **r**, implying that the residual and its total derivative **J** (Jacobian matrix) must be given in the subroutine, i.e.:

$$\mathbf{r} = \mathbf{f}_{n+1} - \bar{\mathbf{M}}\ddot{\mathbf{v}}_{n+1} - \mathbf{g}\left(\dot{\mathbf{v}}_{n+1}, \mathbf{v}_{n+1}\right)$$
$$\mathbf{J} = -\frac{d\mathbf{r}}{d\mathbf{V}} = -\frac{\partial \mathbf{r}}{\partial \mathbf{V}}\frac{\partial \mathbf{V}}{\partial \mathbf{V}} - \frac{\partial \mathbf{r}}{\partial \dot{\mathbf{V}}}\frac{\partial \dot{\mathbf{V}}}{\partial \mathbf{V}} - \frac{\partial \mathbf{r}}{\partial \ddot{\mathbf{V}}}\frac{\partial \ddot{\mathbf{V}}}{\partial \mathbf{V}} \tag{34}$$

### 2.3 Wave actions

Wind generated sea waves are commonly approximated as a locally homogeneous random field for engineering purposes. The sea surface, modelled as a locally homogeneous random field, can then be expressed mathematically by the following Riemann-Stieltjes integral:

$$\eta(\boldsymbol{x},t) = \int e^{i(\boldsymbol{\kappa}\cdot\boldsymbol{x}-\omega t)} d\boldsymbol{Z}_{\eta}(\boldsymbol{\kappa},\omega) \tag{35}$$

Here the surface elevation $\eta$ is a scalar quantity given as function of location $\boldsymbol{x} = \{x \ y\}$ in space (mean sea surface) and time $t, \boldsymbol{\kappa} = \{\kappa_x \ \kappa_y\}$ is wave number vector, $\omega$ is frequency, and $Z_\eta$ is a spectral process with independent increments. For stationary and homogeneous random field the spectral process is related to the wave spectral distribution/density as:

$$E\left[d\boldsymbol{Z}_r(\boldsymbol{\kappa},\omega)\, d\boldsymbol{Z}_s^{\mathrm{T}*}(\boldsymbol{\kappa},\omega)\right] = d\boldsymbol{G}_{rs}(\boldsymbol{\kappa},\omega) = \boldsymbol{S}_{rs}(\boldsymbol{\kappa},\omega)\, d\boldsymbol{\kappa}\, d\omega \tag{36}$$

Here, the indices *r* and *s* to two points in time and space, $\boldsymbol{G}_{rs}$ denotes the spectral distribution and $\boldsymbol{S}_{\eta_r\eta_s}$ the corresponding spectral density, $E[\cdot]$ symbolises the mathematical expectation, the superscript T means transpose and the asterisk complex conjugate. A Gaussian random field is an important special case of the above model, defined as a random field described in terms of Gaussian probability functions, which is completely described by its power spectral density (provided that the mean value is zero), and hence, through the Wiener-Khinchin theorem, by its two-point correlation function.

For small amplitude water waves (Airy waves) the wave number and frequency is related through the dispersion relation given as:

$$\omega^2 = g\kappa \tanh(\kappa h) \tag{37}$$

Here, $g$ is acceleration of gravity, $h$ is the water depth and

$$\boldsymbol{\kappa} = \{\kappa_x \ \kappa_y\} = \kappa\{\cos\beta \ \ \sin\beta\} \tag{38}$$

Equation (1) reveals a direct method for Monte Carlo type simulation of Gaussian waves fields provided that the wave spectral density is given (see Eq.(2)). A possible simplified model for the spectral process is:

$$d\boldsymbol{Z}_\eta(\boldsymbol{\kappa},\omega) \sim \delta\boldsymbol{Z}_\eta(\boldsymbol{\kappa},\omega) = \sqrt{2\boldsymbol{S}_\eta(\boldsymbol{\kappa},\omega)\,\delta\boldsymbol{\kappa}\,\delta\omega}\, e^{ir} \tag{39}$$

where, $r$ denotes independent uniformly distributed random numbers between 0 and $2\pi$, and the infinitesimal quantities have been approximated as finite.

The model outlined above reveals that the wave spectral density can be expressed as a function of frequency and direction (see Eq. (37) and (38)). Traditionally, the directional wave spectral density is written as follows:

$$S_\eta^{(d)}(\omega,\theta) = S_\eta(\omega) D(\omega,\theta) \tag{40}$$

where, $S_\eta(\omega)$ is the so-called one-dimensional wave spectral density and $D(\omega,\theta)$ is the directional distribution, sometimes called the directional function. Then the cross spectral density of the wave elevation can be expressed as follows:

$$S_{rs}(\omega) = S_\eta(\omega)\int_{-\pi}^{\pi} D(\omega,\theta)\exp\left\{-i\frac{sign(\omega)\kappa(\omega)}{g}(\Delta x cos(\theta) + \Delta y sin(\theta))\right\}d\theta \tag{41}$$

where, $\Delta x = x_r - x_s$ and $\Delta y = y_r - y_s$, and $\kappa(\omega)$ denotes the wave number as a function of frequency (see Eq.(3.3)). The complex coherency is subsequently given as:

$$Coh_{rs}(\omega) = \int_{-\pi}^{\pi} D(\omega,\theta)\exp\left\{-i\frac{sign(\omega)\kappa(\omega)}{g}(\Delta x cos(\theta)+\Delta y sin(\theta))\right\}d\theta \tag{42}$$

Assuming that the waves are long crested implies that the directional distribution can be approximated as Dirac delta function. Then it follows from the above expression that the coherency is equal to one. Applying Eq. (42) the complex coherency as well as the lagged coherency can be expressed in closed form for well-behaved analytical directional distributions (see below) assuming deep water conditions.

## 3 CASE STUDY: THE BERGSØYSUND BRIDGE

The Bergsøysund Bridge is a pontoon bridge crossing Bergsøysundet between the islands of Aspøya and Bergsøya in Gjemnes in the Møre og Romsdal, Norway. The bridge is 931 meters long and the longest span between pontoons is 106 meters. The wave induced dynamic behaviour of the bridge in extreme sea state will be considered below.

### 3.1 Added mass, radiational damping and restoring force

The added hydrodynamic mass, radiational damping and restoring matrices for the pontoons have been obtained using the software Wadam [7] The relation between the transfer function defined in Eq. (21) and the added hydrodynamic mass, radiational damping and restoring matrices can be modelled as:

$$\begin{gathered}\mathbf{M}_{hd}(\omega) = -\frac{\mathrm{Re}(\mathbf{F}_{hd}(\omega)-\mathbf{a}_1)}{\omega^2} = \mathbf{a}_3 - \sum_{l=1}^{N}\mathbf{a}_{l+3}\frac{1}{d_l^2+\omega^2} \\ \mathbf{C}_{hd}(\omega) = \frac{\mathrm{Im}(\mathbf{F}_{hd}(\omega))}{\omega} = \mathbf{a}_2 + \sum_{l=1}^{N}\mathbf{a}_{l+3}\frac{d_l}{d_l^2+\omega^2}, \qquad \mathbf{K}_{hd} = \mathbf{a}_1\end{gathered} \tag{43}$$

If the decay parameters $d_l$ is selected before the least squares fit to the data is performed, the problem becomes linear, which reveals that the solution can be obtained without any iterations. The optimal fit to the data has, hence, been obtained by trying a range of $d_l$ coefficients and performing linear least squares fit to the added mass and damping functions obtained from Wadam. The selected curve fit for the motion in the $x$ and $z$ direction is displayed in Figure 17 and *Figure 18*. As can be seen, the expression suggested in Eq. (21) provides a fair least squares fit to the data for both the added mass and damping coefficients.

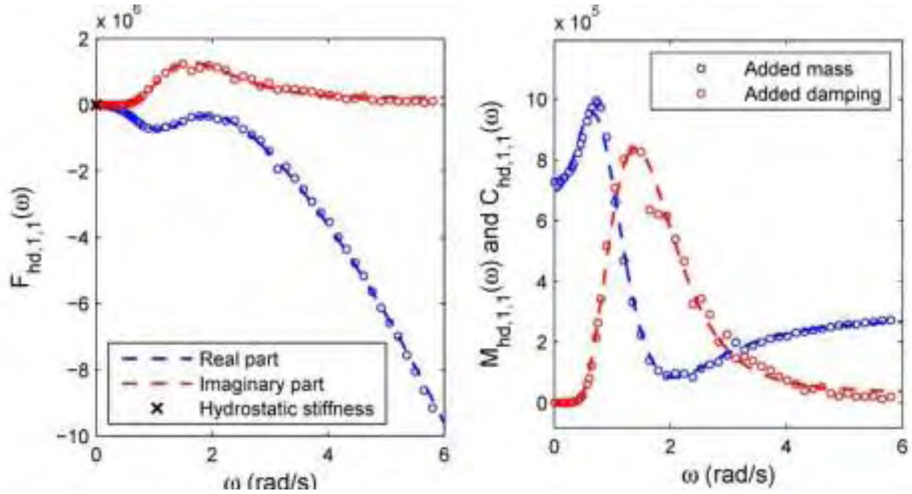



Figure 17: Left figure: Transfer functions for motion in x direction. Right figure: Added mass and damping coefficients for motion in the *x* direction

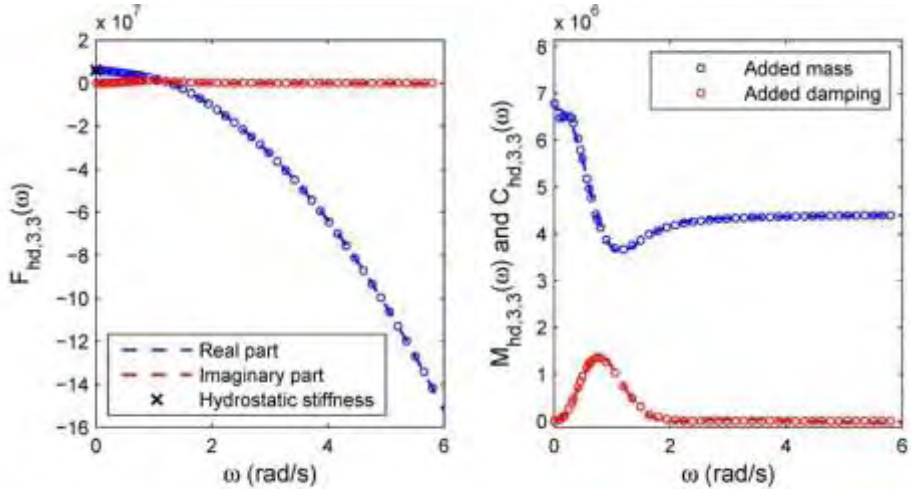


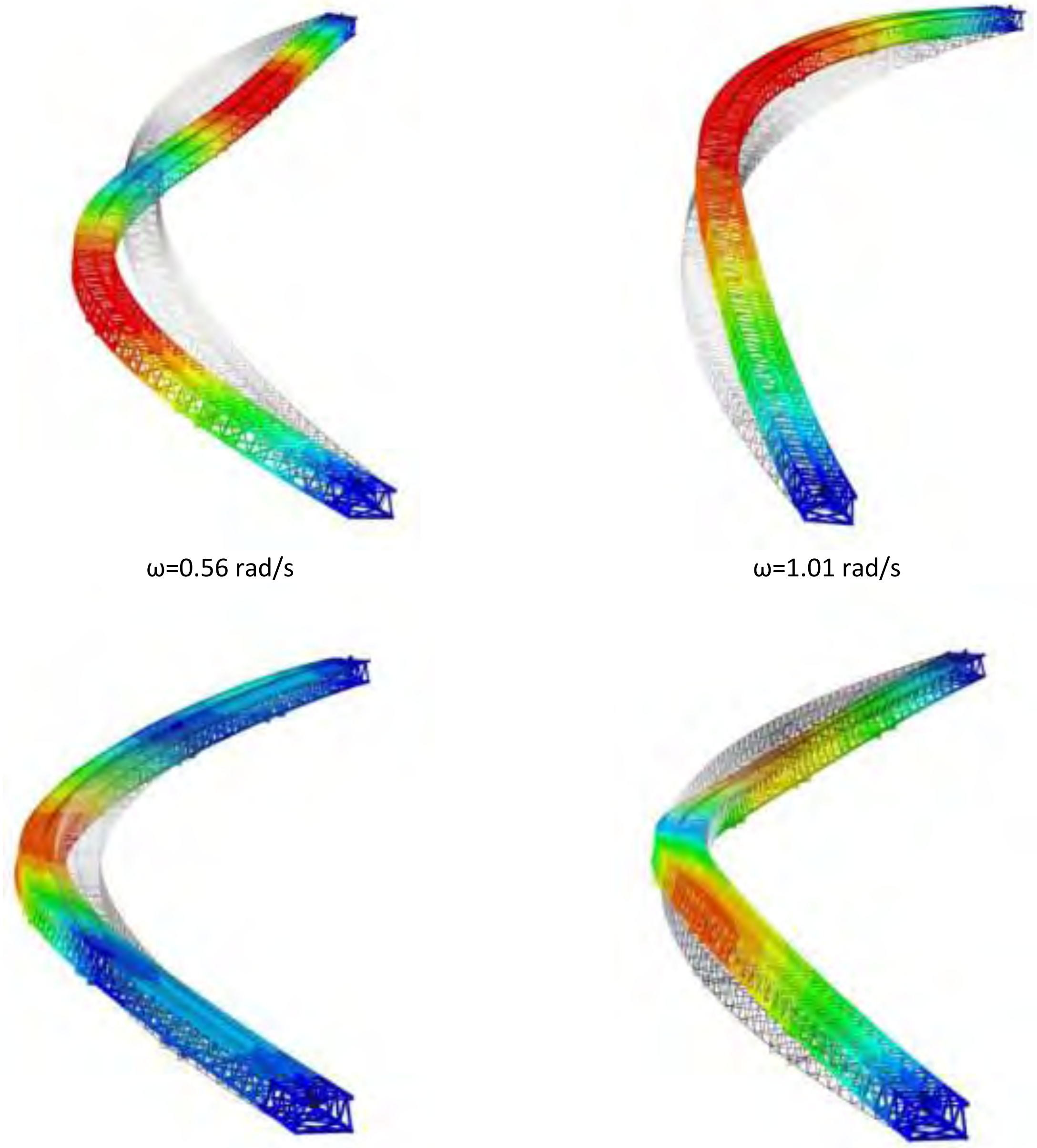

*Figure 18: Left figure: Transfer functions for motion in z direction. Right figure: Added mass and damping coefficients for motion in the z direction*

### 3.2 Finite element model

A finite element model of the bridge has been created in Abaqus. To calculate the natural frequencies taking into account the frequency dependent hydrodynamic mass, the following eigenvalue problem can be considered:

$$\left[(\mathbf{K}+\mathbf{K}_{hd})-\omega_n^2(\mathbf{M}+\mathbf{M}_{hd}(\omega_n))\right]\mathbf{\Phi}_n=\mathbf{0} \tag{44}$$

As can be seen the eigenvalue problem contains frequency dependent added mass matrix . Hence, the natural frequencies presented in this paper have been obtained by solving the eigenvalue problem in an iterative manner. A converged solution is generally achieved after three iterations. The four first vibration modes are displayed in *Figure 19*.



ω=0.56 rad/s ω=1.01 rad/s

ω=1.07 rad/s ω=1.07 rad/s

*Figure 19. Selected vibration modes.*

### 3.2 Wave actions

The one dimensional wave spectral density applied is the so-called ITTC spectrum. The sea state described is characterised by the following parameters: modal period of wave spectral density, $T_p$ = 10 s, standard deviation of wave amplitude, $\sigma_\eta$ = 0.99 m, corresponding to significant wave height of 4 m. We would like to stress that this sea state should not be considered as valid for the Bergsøysund, and is included here only for illustration purposes.

The directional distribution is commonly characterised by a bell shaped function centred around the mean wave direction. The distribution applied in the current case is the so-called *cos-2s* distribution. This distribution has the following closed form expression for zero mean wave direction:

$$D(\theta, s) = \frac{\Gamma(s+1)}{2\sqrt{\pi}\,\Gamma(s+1/2)} \cos^{2s}\left(\frac{\theta}{2}\right) \tag{45}$$

A directional wave spectral density is illustrated in *Figure 20* together with the coherency and a realization of the sea surface.

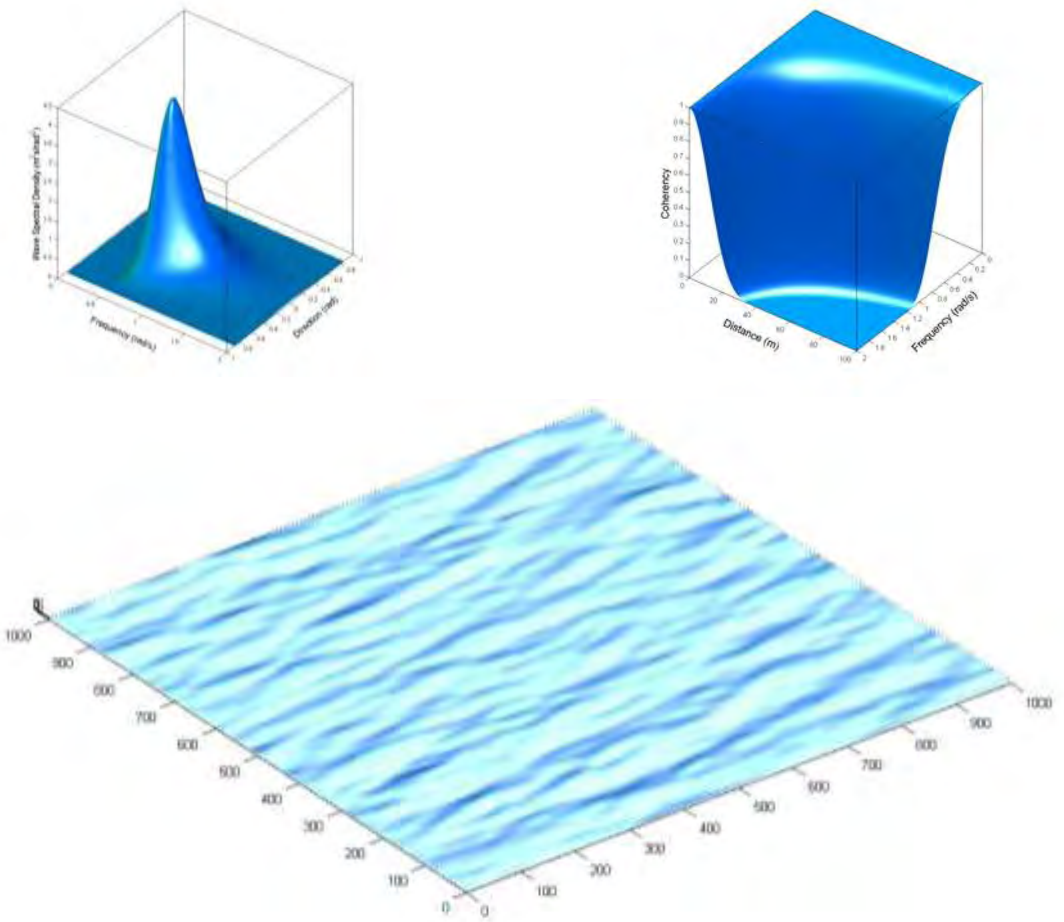



*Figure 20: Spatial variability of wind generated sea waves. The directional wave spectral density is shown in the top left figure. The coherency is shown in the top right figure while a snap shot of long crested water waves covering 1 by 1 km sea surface is shown in the bottom figure*.

The wave forces acting on our study bridge are generated by extending the simulation procedure outlined above. It is assumed that the pontoon behaves like a rigid body and the wave action can be expressed in terms of the three force components along with the three moment components, termed: surge, sway, heave, roll, pitch and yaw, respectively. These force components are modelled in terms of transfer functions obtained by Wadam [7]. The transfer functions derived for the current case study are displayed in *Figure 21*. Analysis of generated time series describing the wave action reveal that the correlation between the heave forces on neighbouring pontoons are small while the apparent correlation between forces on pontoons further away seems to be insignificant on the average.

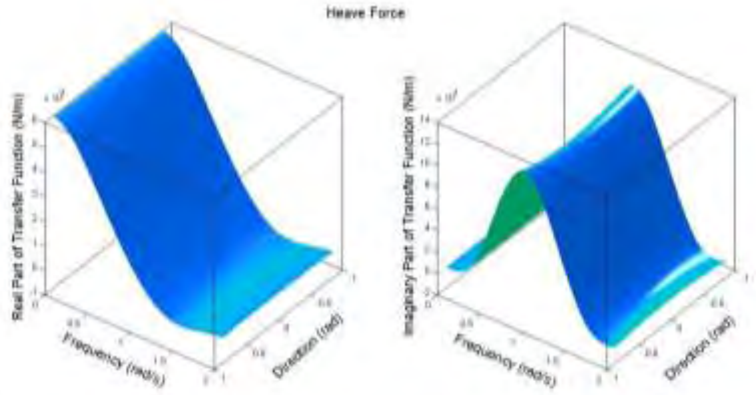



*Figure 21: Transfer function for the lift force of one pontoon obtained by Wadam [7]. Left figure: Real part. Right figure: Imaginary part.*

### 3.3 Wave induced dynamic response

The wave induced dynamic response is obtained by Newmark's β-method applying β=1/4 and γ=1/2. The vertical and horizontal response at the mid-span is presented in *Figure 22* obtained using the above mentioned sea state as a basis for the wave-induced excitation acting on the pontoons.

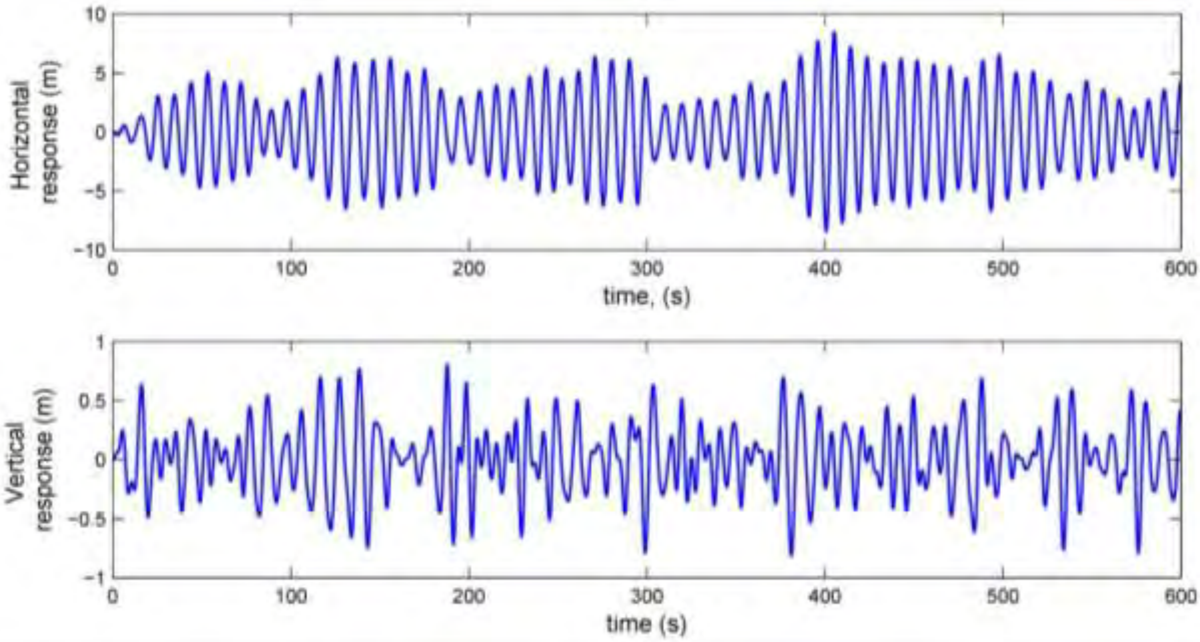



*Figure 22: Calculated horizontal and vertical dynamic response at the mid-span of the Bergsøysund Bridge*

As can be seen from the figure, the maximum horizontal response is about 8 meters, while the maximum vertical response is about 0.8 meters, which is found in reasonable agreement with the above described sea state.

## 4 CONCLUDING REMARKS

The authors are currently developing computational methods for prediction of dynamic response of floating and submerged tunnels based on step by step integration in the time domain. Preliminary results, where the Bergsøysund Bridge is used in a case study, are presented above. It is concluded that the added hydrodynamic mass and radiational damping can be modelled with sufficient accuracy using first order differential equations and, furthermore, that the dynamic response of the study bridge can be assessed in an efficient manner using the approach suggested in this paper.

## ACKNOWLEDGEMENTS

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# ULTRA-LONG UNDERSEA TUNNELS

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## ABSTRACT

*Strait crossings are important for flexible trade and communication. If the strait is wide then ship traffic has been the alternative. To date, the maximum length of an undersea railway tunnel is approximately 50km. There are a number of reasons for this length limitation including the transport of workers and materials on slow construction trains which impact on the construction time, and the cooling and ventilation of the tunnels both during the construction and operational phases. The potential of a long construction periods – in any form of project – may also cause difficulties in financing of the scheme.*

*The engineer and developer Allan Sharp found an easy way of overcoming the length limitation, by simply imposing vertical shafts on the route of the sub-sea tunnel. In this layout it is possible to have several construction fronts, shorter construction time and thereby easier financing. The tunnel itself is known technology, and so is the vertical shaft, from the offshore oil and gas industry.*

*It is the objective of the paper to describe how these two technologies are joined to present a valuable contribution to major strait crossings.*

## INTRODUCTION

Compared with their under-land counterparts, the construction and operation of undersea tunnels which are used for transportation purposes have their own unique challenges and constraints – not least for the longer tunnels.

Even for the trans-alpine tunnels which have been built or are under construction, the fact that the tunnel is located under a land mass allows intermediate access points, e.g. shafts and adits, to be positioned along the length of the route to facilitate multiple faces for the construction of the tunnel, and emergency egress points for the evacuation of passengers in the event of an incident. Such a layout would be difficult for an undersea tunnel, unless it were to pass below a series of islands – either natural or man-made, and in deep sea conditions this in unlikely to be the case. A concept has been identified, however, that provides such a solution for the longest of tunnels that are currently being considered, and even for deep ocean locations, and with minimal impact to the environment.

The Channel Tunnel between UK and France is the longest undersea crossing in the world and provides a link for the high speed rail network in the UK and that of France and mainland Europe – especially when the AlpTransit project is completed. The Storebaelt and Oresund crossings provide such rail connections in Scandinavia, and the planned Fehmarnbaelt will complete the linkage. The latter two tunnels of course incorporate both rail and roadways. The longer tunnels are generally required for rail only, and rail transport is seen as the sustainable solution to long-distance transport.

Outside Europe, high speed rail lines are being implemented all around the world and operational systems exist in China, Japan, Korea and Taiwan. The possibility of forming a North-East Asia rail network by connecting the rail lines in the four countries is impeded by the geography of the region. The rail links would require the construction of a number of deep sea tunnels, all of which would be considerably longer than have ever been built to date. Nevertheless, it is understood that the Chinese are currently considering a tunnel between mainland China and Taiwan, and the link between Japan and South Korea has been studied for many years[1]. Other than the political issues, a practical means of building these ultra-long undersea tunnels would still remain the major obstacle.

Although a number of islands exist in the 230km of sea separating South Korea and Japan, any undersea section would be considerably longer than any similar tunnel built to date. Fewer islands exist in the Taiwan Straits and the undersea section is likely to be of the order of 150km if such a link were to be built. The depth of the sea in the Taiwan Straits reaches 70m, so building an artificial island would be a major undertaking and would undoubtedly meet with considerable environmental objection.

A solution for constructing such major tunnels may, however, already exist – by utilising the proven technology that has been used for the past 30 years in the oil and gas industry. The concept makes use of the gravity structures that have been constructed in areas such as the North Sea between UK and Scandinavia to form the required intermediate islands. These structures would act as the staging areas for the construction of the tunnels, and in the long term would provide emergency access points and could also potentially provide a power source for the operations of the railway.

## CHALLENGES IN CONSTRUCTION

The longest undersea rail tunnels in the world are the Channel Tunnel between UK and France, and the Seikan Tunnel in Japan. Although the overall length of the Channel Tunnel is 50km from portal to portal, the undersea portion is 38km in length. The tunnelling sites were located close to the coastlines, and the longest of the tunnel drives was 22km. The Seikan Tunnel is 53.8km in total length, but only 23.3km is undersea. Again the undersea portion was excavated from both shorelines, so the longest of the tunnel drives was of the order of 12km.

One of the major considerations with the construction logistics of tunnels is accessibility, and the delivery of materials and personnel to the working face, together with the efficient removal of spoil. The locomotives that are used to for these purposes generally operate at speeds of between 15-20kph. Indeed in the confines of a tunnel under construction it would be unsafe to operate locomotives at a speed which would be hazardous to the workers within the tunnel. Additionally, the faster the locomotive speed, then the greater the power they require, and if diesel powered, the greater the heat generated and the exhaust emissions created. As such, the size of the ventilation ducts to the tunnel face increases in size. Overhead line power could be provided, but there are associated safety considerations with such a choice. But overall, the longer the tunnel then the more ventilation is required throughout its length to maintain the necessary air quality. A critical point will ultimately be reached where the tunnel diameter is dictated by the size of the temporary ventilation ducts, and not for the final operational requirements – which is an additional cost for the project.

For the Channel Tunnel, a high performance temporary track was installed which allowed the locomotives to travel at a maximum permissible speed of 30kph. Notwithstanding this, towards the end of the tunnel drives, when the length of the tunnel exceeded 20km, then it took the workforce almost an hour to reach the TBM, and a similar duration at the end of the shift to exit the tunnel. Labour costs quickly become a major factor when 20% of each working day is spent travelling to the tunnel face. Ultra-long tunnels would compound these logistics.

The total length of the Gotthard Base Tunnel of the AlpTransit Project is 57km. If this tunnel had been constructed from each portal, then the longest drives would have been comparable with those used for the Channel Tunnel. However, it was considered that the most cost-effective way to construct the AlpTransit Project was to use multiple drives, via a number of intermediate shafts and access adits the deepest of which was 800m deep. With the use of these shafts and adits the longest of the tunnel drives is 14km which, although a considerable distance, is still reasonable in terms of logistics. Such intermediate facilities would have been difficult to provide if the tunnel was located under the sea.

## TUNNEL OPERATIONS

As previously stated it is assumed that any ultra-long tunnel, either under-land or sub-sea, would be used for rail travel only. The operational issues discussed below therefore relate to the safe and efficient running of a high speed rail system within such a tunnel, and are generally based on the design requirements for the Channel Tunnel[2]. However, should the tunnel be designed for a highway instead, then the issues relating to air quality, drainage, and particularly fire and life safety would still remain valid.

### Temperature

Electrical power used by locomotives in overcoming aerodynamics and rolling resistances degrades into heat. Within a tunnel environment this is supplemented by heat from power losses, train air conditioning systems and tunnel services. The average level of heat generated in the Channel Tunnel has been calculated at 80MW, with only 10MW conducted away through rock strata and tunnel ventilation.

Without cooling, air temperatures could reach 50°C after 2-3 months – and this is in the temperate climate of Northern Europe. Thus a cooling system is therefore needed to maintain air temperatures below 25°C. This is provided by water chilled to 3°C which is circulated through 400mm pipes mounted on the walls of the running tunnels. Longer tunnels would require larger pipes, as the water temperature could not be lowered any further.

### Air Quality

Maintaining air quality during the operations of the tunnel is a fundamental design issue for the safety of the users of the system. Generally fresh air is pushed through the running tunnels by the movement of trains. However, if a central services tunnel is required then other means of ventilation is needed. For the Channel Tunnel there are two ventilation systems within the tunnels: the normal ventilation system (NVS), and the supplementary ventilation system (SVS), used during emergency conditions.

The NVS provides fresh air to the service tunnel from fans located at each of the shorelines. The capacity of these fans and the provision of air-control devices in the cross-passages ensure that the air in the service tunnel is separated from the running tunnels and is kept pressurised at all times.

The emergency SVS provides smoke control and purging of the running tunnels in the event of a fire. Again, the fans are located on each of the shorelines. The SVS fans for the Channel Tunnel supply 300m³/sec of air directly into the running tunnels, sufficient to induce longitudinal airflows around stationary trains.

Obviously larger or additional fans could be provided to supply the necessary ventilation systems for longer tunnels. However, the relationship between tunnel length and fan capacity is not linear, and the complexity of the ventilation design increases with the length of the tunnel as there are likely to be more trains within the tunnel. The provision of intermediate ventilation installations would greatly benefit the efficiency of the air quality systems. This is clearly not practical with undersea tunnels unless an intermediate connection to surface can be provided.

### Drainage

To reduce hydrostatic loads on the tunnel linings of the Channel Tunnel, small groundwater flows are permitted through the joints of the segmental concrete linings. The seepage water is collected in a carrier drain located in the invert of each of the tunnels, and transferred to two pump stations located 5km and 15km from the shorelines.

The collected seepage water is then pumped back to the mainland via four pipes, each 400mm in diameter, which are located in the service tunnel. The space provisions for these pipes has a significant impact on the diameter of the service tunnel.

### Fire and Life Safety

One of the major considerations in the design of any transportation tunnel is the provision of a place of safety in the event of a major incident such as a fire occurring in the tunnel. This is usually accommodated by the provision of cross passages or escape routes to allow passengers to vacate the incident tunnel. Any new tunnel would be designed in full accordance with international safety requirements irrespective of length of tunnel. However, for an ultra-long tunnel, a major issue comes with access for emergency services and evacuation of injured people if the incident occurred a long way from the shoreline. The international standards do not give precise guidelines on the time required for the emergency services to reach the scene of the incident, or to attend to casualties. For facilities such as the Channel Tunnel, a dedicated response crew is employed so that any incident can be responded to immediately. However, the incident could still occur 25km away from where the response crew is based, and the time for them to reach the scene would be longer than for an accident occurring in an urban environment. For an ultra-long tunnel, the time could be considerably longer and potentially unacceptable in terms of safety.

## BENEFITS OF INTERMEDIATE STAGING AND ACCESS POINTS

It is considered that the construction of any ultra-long tunnel would benefit from the provision of intermediate staging points along the route in order that the tunnel can be excavated via a number of faces. The overall construction schedule is usually shortened considerably with multiple drives. Also, the travel time for the construction crews along the developing tunnel is kept to reasonable times which would reduce the labour costs of the project. Such intermediate access points have been provided for in the AlpTransit project, but would seem to be impractical for undersea tunnels unless islands are formed along the alignment.

The overall flexibility of the construction programme would also be enhanced. Any delays on one face would be compensated for by continued work on the opposite face and the meeting point would be adjusted accordingly. It would be expected that a variety of ground conditions would be encountered during the construction of a long tunnel. If such a tunnel were to be constructed from a single face, then the TBM would need to be designed to cope with all the different types of ground conditions. The efficiency of the TBM and production rates would be likely to suffer as the “one size fits all” machine is not specifically tailored to the conditions at a particular location. By dividing the overall tunnel into a number of smaller sections, it may be possible to build the tunnel with TBMs that are specifically designed to the ground conditions along that section.

Any transportation tunnel which will accommodate people would also benefit by the inclusion of intermediate access point along the route to compartmentalize the overall scheme into a number of discrete and manageable sections. Thus separate cooling and ventilation plant can be provided for each of the compartments. An intermediate fresh air intake station would greatly increase the efficiency of the ventilation system. More importantly, evacuation from the tunnel in the event of any major fire can be undertaken in a reasonable time, thus improving the overall safety performance.

For many undersea tunnels the sea depth would generally be in excess of 100 metres, which would make the formation of man-made islands impossible. Other options should therefore be investigated, and these are discussed below.

## OFF-SHORE GRAVITY STRUCTURES

There are 50 major offshore concrete structures serving the oil and gas industry. Most of them sit on the sea bottom, connecting the reservoir far underneath the sea bed to a production facility on top of the platform. Some of them also store oil.

The typical construction scheme is to use a suitable dry dock for the lower part, and then commence construction afloat. The topside facilities may be built simultaneously, and the two, topside and platform, may be coupled inshore and towed to location. Installation may be as simple as adding water, and will typically take only a few days.

This method of construction favours deep fiords, particularly if the topside weight is large. Fig.1 shows the tow out of the Troll A platform, displacing 1 million tons, with its 22,000 tons topside 150m above sea level.

Fig.2 shows the typical construction process: the mono-tower platform Draugen is used for illustration. This platform is operated by Norske Shell, and it has performed very well since the installation in 1993.

In fact all the concrete platforms perform well, which may seem surprising, considering the harsh environment some of them sits in, as exemplified in Fig.3. Modern platforms have longevity of more than 200 years, exceeding by far the production span of the oil or gas reservoir. The first concrete platform was installed in 1973, 40 years ago, and many platforms have surpassed their design lives but still in good shape.

*Fig.1. Troll A during tow out*

*Fig.3. The harsh environment of the North Sea*

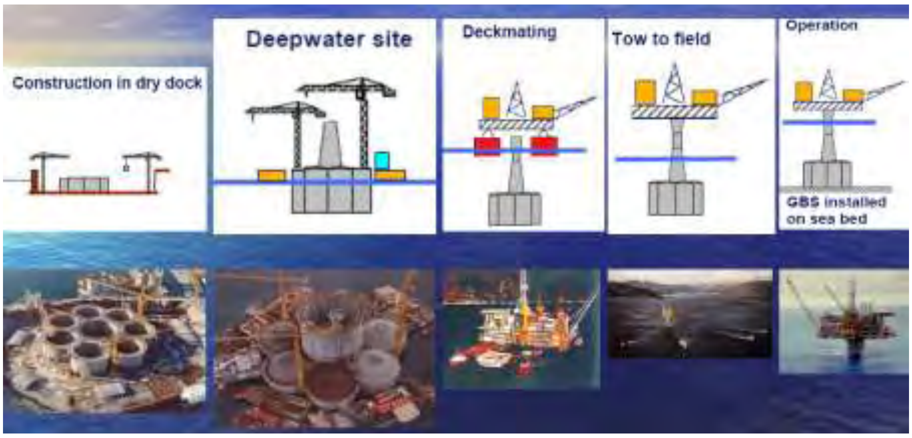


*Fig.2. Construction of a gravity based concrete platform*

The state of platforms is described in reference[3].

The 50th concrete platform was installed last summer, east of Sakhalin Island in Russia. It is operated by ExxonMobil. It has been designed to resist sheet ice. The 51st, also to be operated by ExxonMobil, is now under construction in Canada, and shall resist icebergs. In general earthquake is considered and designed for, as well as waves and other environmental loads.

Typically the platform has a connection to something. It may be risers for oil or gas, it may be drilling conductors. The typical platform has drill conductors penetrating the bottom of the structure, and into the sea bed. There are often many of these, but they are not so large. Measures to prevent inflow of water are taken; sometimes the conductors have their own water filled shaft.

The Draugen platform, Fig.4, is in its simplicity quite suitable as "shaft" to the ultra-long undersea tunnel. The existing Draugen platform sits in 253m water depth, on soft soil. So size can be considerably reduced, and the base suited to the soil conditions at hand. The scheme for making the connection between the platform and the sea bed is described in the following section.

## SECURING THE STRUCTURE INTO THE SEA BED

The condition of the seabed at each site of landing of a concrete structure is the major determinant of the method by which a seabed-to-structure interface is selected and installed. Seabed could be of rock with little or no soil cover, or of medium to deep soil cover down to a stratum suitable for supporting tunnel construction. A seabed of deep topsoil is considered to be the greatest technical challenge so the proposed solution to this case is discussed hereafter.

*Fig.4. The Draugen Platform*

A template structure of steel shall be landed over pre-installed posts and set onto the seabed. The structure will act as guide and stabilizer for a caisson of reinforced concrete, integrated tool rail and integrated pipework for supply to the cutting edge of jetting muds and sealing grouts. The caisson shall be set into the soil through the template and, by a combination of jetting (drilling muds supplied to cutting edge, returns through casing annulus) and dredging of the inner catchment soil, the caisson shall reach the stratum for tunnel construction.

On reaching the bedrock or stratum of (relative) impermeability, a train of under-reaming tools shall be mounted on the rail – said tool being an adaptation of the Mitsui Aqua-Header™ with appropriate supply of drilling muds and cuttings removal. Controlled and progressive lowering of the caisson into the socket created by the under-reaming train will be the scenario for the cementing operation. Cement at several volumes greater than the socket will be pumped into the socket to envelope the caisson edge. On setting, the cement will provide a barrier to water ingress. Methods under patent application allow testing and remedial operations.

After the caisson edge is secured to water ingress, the template structure and piles are removed and the platform is landed over the caisson i.e. the caisson stub is in the centre of the shaft of the platform. The caisson is then tied-back to surface by the sequential placement of sections of reinforced concrete matching that of the caisson thus allowing intermediate cementing and testing. The result shall be a shaft within the platform but not mechanically tied to the platform. The bore of the shaft can be pumped dry with the only influx being that caused by the permeability of the formation. Construction of the dry shaft to the tunnelling horizon may now commence.

On reaching the depth for tunnel construction, a cavern is excavated out of the formation and secured by shotcrete. The cavern would be similar in size to that constructed for the undersea crossover cavern for the Channel. This would be large enough to allow assembly of modular TBMs which are then launched in the manner known to the industry.

On completion of all tunnel-boring operations, work would commence on converting the shaft within ground to a series of supply (manpower lifts and ventilation) shafts which can seal against catastrophic failure of the platform and related structures.

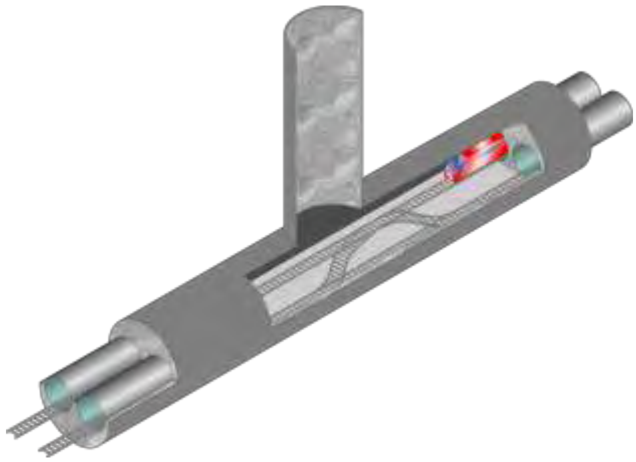

*Fig. 5. Final Configuration of Launch Cavern*

## CONSTRUCTION SCHEDULES

A comparative study has been undertaken to investigate the potential benefits in terms of construction schedules when using the intermediate staging areas provided by the gravity structures as compared with a conventional approach for an undersea tunnel by driving from each portal. The example taken for the study was the mainland China to Taiwan Tunnel, which has an undersea length of 150km. i.e. each tunnel drive would be approximately 75km from each portal.

The basic assumptions were as follows:

- Procurement, delivery and assembly of TBMs is 18 months;
- Average production rate for each TBM is 120m per week;
- Installation of mechanical and electrical systems can be done concurrently with TBM drive;
- Installation of rail track is 1000m per week, and undertaken after completion of all tunnelling activities;
- Testing and commissioning: six months following completion of installation of track.

Based on the above, a 150km long undersea tunnel excavated from each portal would be open for revenue after a period of 15.5 years.

The bulk of this time is the tunnelling itself: 75km at 120m per week would take a period of 625 weeks, i.e. 12 years. And 120m per week assumes no major breakdown of the TBM, which is highly unlikely when excavating 75km.

For the alternative, it has been assumed that 4 gravity structures are installed along the route, thus dividing the overall tunnel into five sections each 30km in length. Based on previous experience, each of these structures would take approximately 18 months to construct and install in place on the sea bed. A further 12 months has been allowed for the excavation of the shaft at the base of each structure and the development of an undersea cavern to allow tunnelling to commence.

The overall construction programme is shown in Fig.6 below.

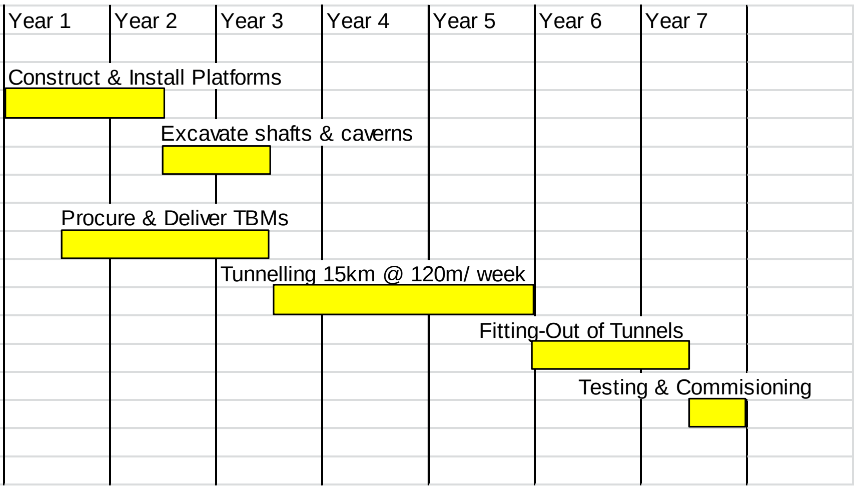


*Fig: 6: Construction Programme using Gravity Structures*

For this option the tunnel would be open after 7 years, i.e. 8.5 years quicker than the conventional method. The major time saving is obviously in the tunnel excavation – 2.5 years compared with 12.

What is most interesting to note is that if such a construction method were to be adopted, say installing gravity structures at 25-30km centres along the length of the alignment, then the tunnel would be complete, portal to portal, irrespective of the tunnel length after a period of 5 years. The long-lead item would then become the installation of the rail track, which could only be undertaken from each portal.

Ultra-long undersea tunnels could therefore become a reality.

## STRAITS AND BEYOND

The Oxford English Dictionary definition of a straits is a “narrow passage of water connecting two large bodies of water”. A strait, however, could still be several kilometres in width, and the section of the Gulf of Finland between Helsinki and Tallinn which is a distance of approximately 60km could fall into this category. Also worthy of consideration is the Gulf of Bothnia under which Helsinki and Stockholm (and thus St Petersburg, Copenhagen, Hamburg) would be linked. Certainly the use of at least one intermediate staging point along this route, which is part of Rail Baltica between Helsinki and Berlin on one of the Trans European Transport Networks (TEN-T), would be of huge benefit in making this concept a reality.

A number of longer routes have been identified in Asia where fixed links are desired. As stated previously, a link between Japan and Korea has been the subject of a number of studies extending many years. The shortest route for this link would require an undersea tunnel of 128km in length, but with a maximum water depth of 220m for this particular route. Shallower water depths can be identified, but the associated tunnel lengths are greater.

More recently a tunnel linking South Korea and China has been discussed, the route of which across the Yellow Sea from near Incheon in Korea to Shandong region in China would require a tunnel in excess of 300km to be formed under the sea. It is noted that there are no islands in the Yellow Sea to simplify the construction logistics. However, the use of gravity structures would be an obvious solution.

As discussed in the section above, a tunnel across the Taiwan Straits to link mainland China with Taiwan would be approximately 150km in length. The depth of the sea is approximately 70m, which is well within the range of the gravity structures described in this paper. The use of four gravity structures to form intermediate staging points along the route would allow the tunnel to be divided into five sections, each 30km in length. The longest tunnel drive would then be 15km, which is similar to the longest drive on the Gotthard Base Tunnel. It is acknowledged that any platform located in the Taiwan Straits would need to be designed to resist major seismic loadings and the adverse weather conditions encountered during typhoons. However, the fact that these types of structures have been built in the harsh environment of the North Sea, and are still operational after 30 years demonstrates that these challenges can be met.

It is recognised that the world is becoming more reliant on wind power, with off-shore wind being the main contributor to this form of renewable energy. The conversion of the gravity structure from a construction staging area into a power source, with both wind turbines and solar panels, would be of considerable benefit for the rail operations, and provide financial benefits to the owner for the lifetime of the railway.

The platforms could also provide accommodation for emergency crews or for maintenance staff for these ultra-long tunnels, rather than having to access the tunnels for the portals. This would have major benefits for the O&M costs for the overall facility.

## CONCLUSIONS

The Straits Crossing Symposiums are held to identify the technologies that can be employed to create fixed links which obviate the barriers created by straits and sounds to the provision of rail and road transport links around the world.

It is recognised that high speed rail lines are being considered as a more sustainable option than air travel. The next stage of the evolution of the fixed links would be the construction of major undersea tunnels that would eclipse the Channel Tunnel in terms of length and complexity.

It can be demonstrated that a viable method for constructing these ultra-long undersea tunnels already exists, and the method proposed has major benefits for both the construction logistics and the long-term operations of the rail link.

The use of technology developed by the oil and gas industry would seem to have major benefits in providing sustainable solutions to some of the world's transport requirements.

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# THE FEHMARNBELT TUNNEL CROSSING: THE WORLD LARGEST IMT

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## ABSTRACT

*The Danish Parliament decided in 2011 to have Femern A/S develop a Plan Approval Design and a Tender Design for the immersed tunnel as the preferred solution for the Fixed Link across Fehmarnbelt. The decision was based on a Conceptual Design prepared by the Ramboll-Arup-TEC JV and the design group has since then been developing the design further in cooperation with the Owner Femern A/S.*

*The Fehmarnbelt Tunnel connecting Germany and Scandinavia accommodates a four lane motorway and a double track railway. The tunnel is of unique scale and will set new records in terms of dimensions with a subsea length of 18 km, a width of 42 m and foundation depths reaching more than 40 m below sea level. The Fehmarnbelt tunnel will consequently be by far the longest tunnel and one of the deepest tunnels of this type ever built.*

*The tunnel civil works is planned to be tendered in 2013 and will include four major contracts: two tunnel contracts, one dredging and reclamation contract and one contract for the land works.*

*The prequalification process started in autumn 2012 and the planning approval processes are underway in both Denmark and Germany. The tenders for the civil works will, due to the complexity of the Project, be conducted by means of the competitive dialogue procedure. After the dialogue phase the contractors shall submit binding bids with prices specified. Signing of the four civil works contracts is planned for summer 2015.*

*The paper will focus on some of the technical challenges posed by the extraordinary length of the tunnel, such as the provision of mechanical and electrical services and construction tolerances.*

## FEHMARNBELT LINK

The selection of a tunnel instead of the originally preferred bridge option was based on the thorough comparison of a tunnel vs. a bridge solution and has gained a certain notoriety within the tunnelling fraternity. This has almost overshadowed the record-breaking nature of the project itself. At a length of 18 km, this will be almost three times as long as the longest immersed tube tunnel (IMT) ever built (*Figure 23*), dwarfing its sibling, the Øresund tunnel.

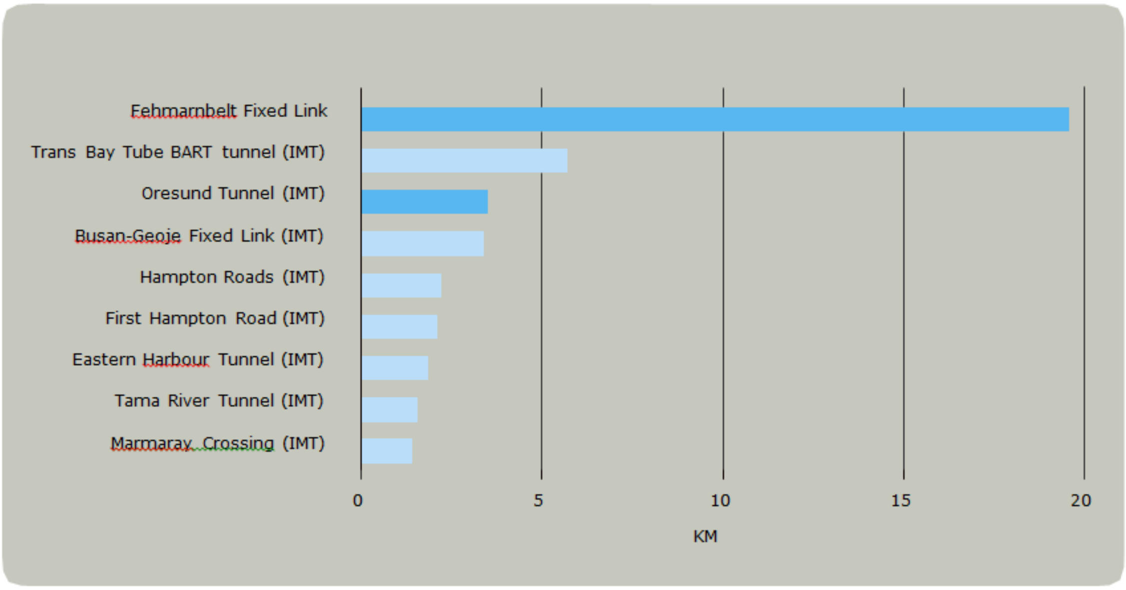


*Figure 23: The World´s Longest Immersed Tunnels*

Since the signing of the Treaty in September 2008 by the Danish and the German Ministers of Transport, there has been a huge amount of activity on the studies of various options for the crossing and the design and the preparation for the planning approval process for the link across the belt between Lolland (Denmark) and Fehmarn (Germany). *Figure 24*


*Figure 24: Map of Fehmarn Belt Crossing*

The Fixed Link will supplement the existing ferry connection with a four lane motorway and a double track railway and runs on an almost straight line just east of the existing ferry harbours. The total length of the overall project is about 25 km when one includes the connections to the existing roads and railway. With a speed of 110 km per hour, this will offer motorists a journey time of approximately 10 minutes through the tunnel while for train passengers, the journey will take seven minutes from coast to coast.

While Denmark will finance the link and is responsible for the construction and future operation, this is very much an international collaboration and the planning processes are underway in both Denmark and Germany. A Danish state owned organization, Femern A/S, has been established to deliver the project and in April 2009 the Rambøll-Arup-TEC Joint Venture was selected for the design of the tunnel alternative. This article will recap on the salient elements of the design and the programme of the future work. Those interested in the story of the competition between the bridge and the tunnel options can find it described elsewhere (ref.[1]).

## GEOLOGY

The first geotechnical site investigations began in 1995 and continued in a programme with different focuses as the project developed. The main components of the programme were as follows:

1. Geophysical investigations to characterise the spatial distribution of deposits
2. Boreholes and CPTUs
3. Advanced laboratory testing
4. Geophysical investigations focussed on detailed knowledge for the planning and approval process (both for geotechnical reasons and also cultural heritage and unexploded ordnance).
5. A high resolution seabed elevation model
6. Large scale tests on the folded Palaeogene clay

This is described in more detail in Kammer et al (ref.[2]).

From these investigations a 3D model of the geology was compiled. The geology of the seabed in this strait between Lolland and Fehmarn consists of a mixture of glacial till, highly plastic paleogene clays, clay, meltwater sand, sand till and gravel. The tunnel is mainly located in these Quaternary deposits and Tertiary clays which overlie Cretaceous Chalk (*Figure 25*). However, one of most interesting strata for the designers is the folded Palaeogene Clay. This is a high to very high plasticity clay, often with significant smectite content. Ice pressure in the Quaternary period heavily disturbed the upper 10 to 30 m of this strata, creating giant folds. Since this strata has been heavily overconsolidated during its history, one concern is its swelling potential during the unloading caused by the excavation of the trench. Results from the large scale tests suggest that the clay actually behaves more like a lightly overconsolidated material.

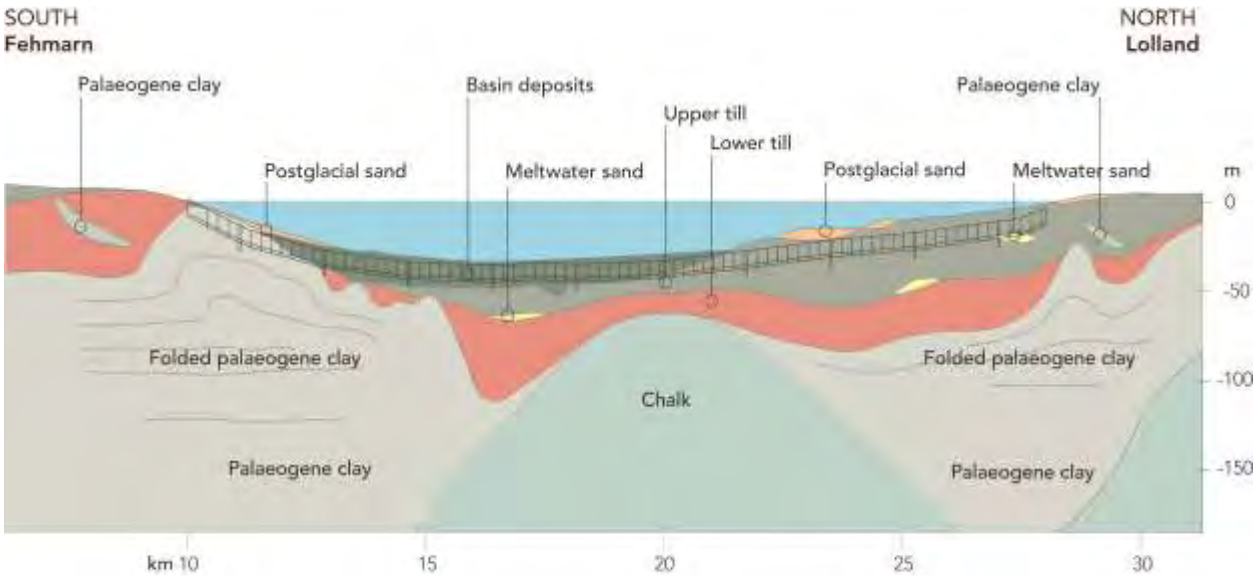


*Figure 25: Geological Profile*

## ENVIRONMENTAL IMPACT

Along with cost efficiency (in terms of both capital and whole-life costings) and the provision of both a road and rail crossing, minimizing the environmental impact was a key design criterion for the fixed link. The IMT lies below the seabed and there will be no intermediate artificial islands in between the Danish and German shore, unlike the Øresund crossing. This leaves the Fehmarn Belt untouched during the operational lifetime of the project. Hence this IMT option has minimized the environmental impact in the following ways:

i. There is no visual impact on the Fehmarn Belt, preserving valuable nature and the environment,
ii. There are no restrictions to ship traffic after completion of the tunnel project and there is no risk for a ship collision (the IMT is designed to withstand impact from falling anchors and a sinking or stranded ship)
iii. There is no adverse impact on the major bird migration routes in the area.

Nor does the design of the tunnel have any permanent effects on the marine environment. The reclaimed land areas near both shores protrude into the sea less than the existing breakwaters of the ferry harbours, ensuring that the water exchange between the Baltic Sea and the North Sea through the Fehmarn Belt is unaffected. Of course, the dredging of the trench for the IMT has an impact on the environment during the construction period but fortunately it has been concluded that the marine flora and fauna in this area are relatively insensitive to this impact. The dredging operation will move more than 15 million cubic meters of soil. Mechanical dredging by Backhoe Dredgers, Grab Dredgers or Trailing Suction Hopper Dredgers, in the deepest layers, has been foreseen because spillage is less compared to hydraulic dredging. Thereby the impact on the water quality is reduced.

## BORED VS IMMERSED TUBE TUNNEL

With the ever-increasing capability of TBMs, it is no surprise that the question of the best tunneling method surfaced during the design period. A thorough study was made of the TBM option, taking as a prerequisite the requirement that this option had to offer the same functionality as the IMT. The base case for the bored tunnel was a triple tunnel bore solution, featuring a single 15.2m internal diameter bore containing twin rail lines, and a pair of road tubes, each 14.2m internal diameter, and containing a two-lane carriageway and an emergency lane. The safety concept for the IMT envisages cross-passages at a spacing of about 110 m. In the case of the TBM tunnel, the refuges in the invert provided the place of safety with a roadway there too for access for the emergency services.

Controlling the stability of the face would have been a challenge in any case, given the large diameters and the variable soft ground. The experience on the Storebaelt tunnel gave ample evidence of that, even with smaller diameter TBMs. Inevitably, on such long drives there is also the heightened risk of a major mechanical failure or a blockage of the cutterhead, for example, by boulders from the till. At depths of more than 50 m below the surface of the sea, interventions into the cutterhead to make repairs would have been extremely difficult and this presented a serious programme risk to the project. A major concern was the handling of the palaeogene clay spoil from what were assumed to be slurry TBMs. Disposal of large quantities of this very soft slurry presented a significant challenge. The combination of these risks and other factors led the project team to select an immersed tube as the preferred tunnel type.

## IMMERSED TUBE TUNNEL

The Conceptual Design envisages an IMT consisting of standard elements, of which there are 79 in total, all with an identical form and layout, and special elements, which house mechanical and electrical (M&E) plant rooms, as described later. Each of these standard elements is approximately 217 m long, 42 m wide and 9 m high. One element weighs around 76,000 tons. The elements consist of 9 segments, 24 m long separated by contraction joints with a rubber-metal water-stop. The reinforced concrete segments will be cast individually and then joined together and pre-stressed to form one unit, which can be floated and transported after the elements have been sealed by bulkheads at either end. In the middle of every element a niche is placed above the road to allow space for the ventilation jet fans. Special rubber seals (e.g. GINA and Omega profiles) between the elements ensure a watertight connection. The final layout and number of elements will be fixed by the winning contractor team during the detailing of the project, under a design and build contract.

The production of so many and such large IMT elements present a significant logistical challenge. At this moment, it is envisaged that the elements will be produced in a purpose-built production facility at Rødbyhavn in Lolland, which may be similar to the one used for the Øresund IMT. This minimizes the influence of the weather on the production process. To meet the programme, the standard elements will be produced in parallel on 8 production lines. Roughly every 1½ weeks an element will be delivered for transport and immersion over a period of more than 2½ years.

The tunnel contains two road tubes, two rail tubes, and a central gallery, which is divided into three levels. The lower level houses the drainage pipes and water supply lines for hydrants and the fire protection system. The middle level is at the same height as the road and can be used both for maintenance personnel as well as a safety zone. The upper level is a service gallery where electrical panels are placed and utility lines run from the special elements to the operating systems in the tunnel.

## TECHNICAL INNOVATIONS

There are many aspects worthy of discussion but, in the limited space available here, a couple of the more unusual ones will be described.

### Special elements in the IMT

In addition to the standard elements, the immersed tunnel will have a total of 10 special elements that are installed at regular intervals (approximately every 1.8 km) between the standard elements. Each of these special elements is 39 m long, 47 m wide and 13 m high (*Figure 26*)

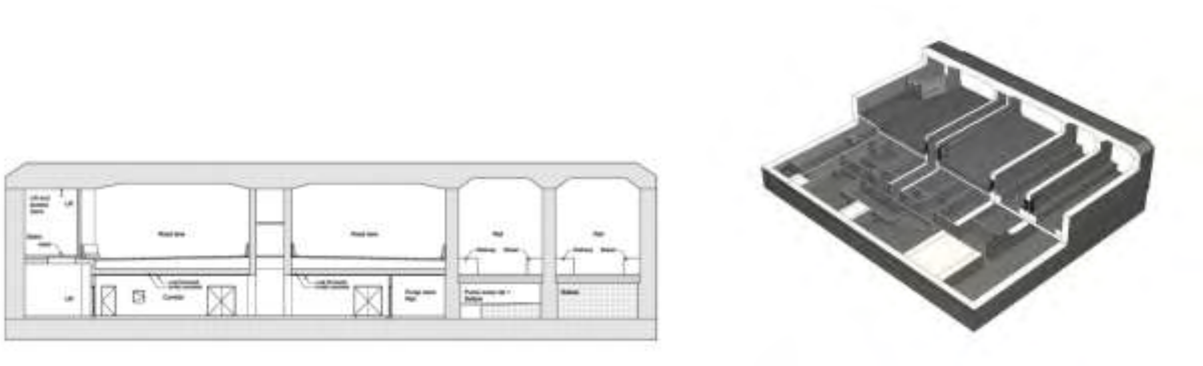

*Figure 26: Cross Section and 3D View of Special Element*

The elements contain two levels: the road/rail deck level and the lower installations level. The installations level offers space for all necessary utility facilities and the mechanical and electrical tunnel operating systems like;

- Sumps (for both road and rail tubes),
- Pump rooms (for sumps and the fire suppression system and hydrants),
- Power supply,
- Gaseous suppression,
- Ventilation,
- Drainage system,
- Heating and cooling,
- Other technical rooms.

The number of special elements derives from the need to transform the grid power supply to a lower voltage every 1800 m. In addition, the special elements provide a number of other advantages, such as:

- Parking access in a layby for maintenance vehicles from Denmark,
- Transformers which can be replaced from road level,
- Access to the mechanical and electrical equipment without interfering with traffic,
- A transverse underpass with access to the longitudinal gallery, each road tube and rail tube.

In the area of Fehmarn Belt it is possible to find dry docks which provide sufficient draught (12.5 to 13 m) and fulfill the requirements necessary to accommodate the outer dimensions of the special elements.

There are two principal construction methods that are thought to be suitable for production of the special elements: the concrete structure and the sandwich structure.

**Concrete Structure**
The production process for the special element as a concrete structure comprises the casting of the elements in an existing dry dock. The dock floor is below sea level allowing the element to float when the basin is filled after construction of the elements. After opening of the dock the element is towed to an existing quay location for final finishing works before it is towed out to the site where it will be immersed in the tunnel trench. If the special element arrives at the immersion location before the immersion date it is temporarily moored near the site.

**Sandwich Structure**
The sandwich structure for the special elements would be a double walled steel box structure which would be filled with concrete. The construction process can be split into two different sub-processes, namely (one) the construction of the steel box and (two) the filling process of the box with concrete. The waterproofing of the element is provided by the steel outer casing of the sandwich structure.

The steel casing of the sandwich structure can be fabricated anywhere in the world. However, there is a balance between construction costs and transportation costs. The dimensions of the casing are within the limits of modern bulk carriers which make transport over long distances technically feasible. The largest production facilities for ships are currently situated in the Far East, in China and South Korea. As an alternative, the steel casing can also be constructed or connected at ship yards close to the project location, the construction costs may be higher but the transportation costs will be less.

**Ventilation design and safety**
The ventilation is designed as a longitudinal system, which significantly reduces the cross-section needed in comparison to a design with transverse ventilation. The longitudinal ventilation concept also eliminates any possible need of an intermediate ventilation island, which in turn reduces the navigational risk in the strait, the environmental impact and the cost of the project. The concept was developed by careful consideration of actual number of vehicles expected in the opening year (which is relatively low), the expected traffic growth in the coming years and the reduction in car emissions over the coming years, due to the improvement in technology. Sophisticated ventilation modelling has demonstrated that the system is capable of keeping conditions in the tunnel below internationally recognized threshold values throughout the whole

lifetime of the tunnel under normal operating conditions. Actually the tunnel is self-ventilating by piston effect as long as traffic is free flowing.

In the event of a fire in the road tunnel, the traffic behind the fire will stop but the vehicles in front of the fire will continue to drive out of the tunnel, driving considerably faster than the flowing smoke layer *(Figure 5)*. Upstream of the fire the tunnel will be kept smoke free by maintaining a minimum critical air velocity with the fans, thereby preventing back-layering of smoke. The people in the stopped vehicles upstream of the fire may evacuate into the adjacent central gallery and the other road tube, via escape doors spaced at intervals of about 100 m. The short distance between the escape doors improves the capability for self-rescue.

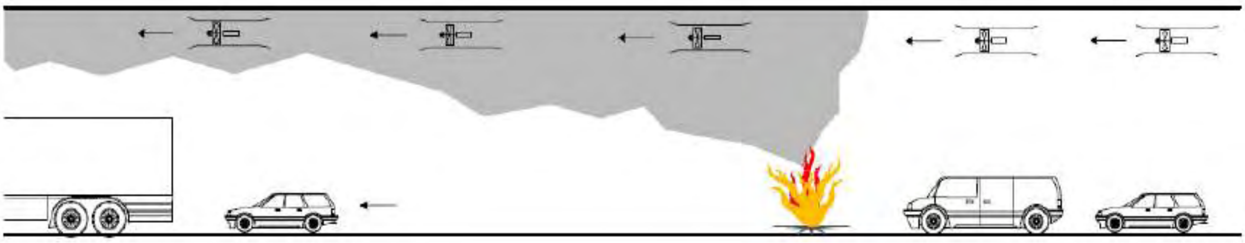

*Figure 27: Illustration of Ventilation Concept in Fire Situation*

The tunnel roof and walls are provided with a passive fire protection system. To further provide asset protection and improve safety in general the installation of a deluge system is foreseen. The system shall furthermore ensure that possible fire incidents will be kept under control until the arrival of the emergency services.

Both the normal and emergency operations will be controlled 24 hours a day by the Danish portal control room and, if necessary, a secondary control room remote from the primary facility. Other safety provisions such as the emergency lane, an automatic incident detection system and CCTV coverage throughout the tunnel will further improve life safety making the tunnel safer than an ordinary motorway.

The driver's perception and comfort have been analyzed in great detail and in the Conceptual Design moving light images as well as a system of coloured light portals have been introduced as an option. The images to be projected on the tunnel walls and the light portals shall help maintain driver awareness and interest and develop a sense of progression through the tunnel during the 10 minute journey (*Figure 6*).
*Figure* ***28***.

*Figure 28: Illustration of Moving Light Images*

**PROGRAMME**

Currently the design team is finalizing the Conceptual Design and the Plan Approval documentation. The intention is to issue the tender documents for the design and build contracts in summer 2013. Final approval by the Danish and German authorities is expected at the turn of 2014-15. The tendering process involves a two phase competitive dialogue process. Construction is scheduled to commence 2015 and the tunnel to open for traffic late 2021.

Work on this DKK 40 billion (2008 prices) mega-project comprises four major civil works contracts, as follows (ref.[[3]]):

1. Tunnel Dredging and Reclamation.
   The contractor will dredge an 18 km long and 12-15 m deep trench in Fehmarn Belt to contain the tunnel. The excavation material of the trench, approximately 15 million cubic meters, will be used for new land areas, primarily off the coast of Lolland and to a lesser extent, Fehmarn. The contract also includes other major dredging tasks, such as the harbour basin, which will be part of the site for production of the tunnel elements in Rødbyhavn.
2. Tunnel North.
   The contractor will be responsible for the Northern half of the immersed tunnel and thus half of the production site for the tunnel elements in Rødbyhavn. The contractor shall produce, transport, immerse and backfill half of the tunnel elements, which will comprise of approximately 40 standard and five special elements. This corresponds to around 9 km of the tunnel.
3. Tunnel South.
   This contract mirrors and matches the tasks contained in the contract for the Northern section of the immersed tunnel. The tunnel is divided into equal parts in order to attract as many bids as possible and so encourage competition in both price and method.

4. Tunnel Portal and Ramps.

   The contractor will build the two tunnel portals and the cut-and-cover tunnels on land, i.e. the tunnel sections between the IMT and the ramps in both Germany and Denmark. The contract also covers the ramps and connections to the existing traffic facilities on land as well as all finishing works such as road surface, step barriers and wall cladding throughout the tunnel.

In addition to these four contracts, Femern A/S will invite tenders relating to railway construction and technical installations in general at a later stage. Additional contracts for the "Advanced Activities" covering the enabling works will be released soon.

## CONCLUDING REMARKS

When the first feasibility study was done in 1999, few people would have given an immersed tunnel option much chance in comparison with a bridge solution. A series of major design innovations reversed this situation, notably the shift to longitudinal ventilation and the use of special elements to house the M&E plant rooms. Moreover the bridge team had to face stricter requirement on navigational safety and availability of the Link which in turn had a negative influence on their project costs.

Now the project team stands on the verge of commencing the tendering process for what will be the world's longest immersed tube tunnel and a mega project which will complete an important link in the European transport network.

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# CAN WE ESTIMATE THE IMPACTS OF FIXED LINKS USING CURRENT TRANSPORT MODELS?

*Experiences from concept evaluations of fixed links on E39*

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## ABSTRACT

*Transport analysis and impact evaluations are important input for decisions about infrastructure projects. The impacts on transport from fjord crossing tunnels or bridges are the foundation for the cost benefit analysis, and also the basis for estimating the income from toll collection.*

*Based on experiences from concept evaluations of several fixed link projects on E39, and an ongoing overall analysis, we question the results from transport analysis made by the official tools for such analysis; the regional transport model which estimates the demand for trips below 10 km, the national transport model, for trips of 10 km or more and the freight transport model. Both the national transport model and the freight transport model are integrated in the regional transport model in the net assignment stage.*

*We will demonstrate strengths and weaknesses in the transport models by showing contra intuitive or questionable results using the model as it is.*

*The following questions arose as the initial results from the transport model were presented: Are the transport models able to capture immediate as well as long term impacts? How would different assumptions about the monetary costs on these projects affect the forecasted demand and the cost benefit analysis? Are there other and wider ranges of impacts, if the analysis covers the total coastal highway as a whole, compared to evaluating impacts of each fixed link project individually? Do we have enough data to include transport effects of wider impacts of the fixed link projects?*

*We had to deal with these questions in the concept evaluations carried out for the various fixed links project and in the current overall evaluation. We would like to suggest improvements in the analysis tools and emphasize requirements for knowledge about impacts of fixed links projects.*

## 1. INTRODUCTION

### *1.1 What is a concept evaluation?*

Evaluation of Concepts has been a mandatory introductory part of planning activities of infrastructure solutions in Norway since 2005. Introducing concept evaluations should alleviate problems with poor cost estimates and lift the concept choice to a superior level before making decisions about public investments. Expected costs of 750 MNOK should from 2011 trigger a concept evaluation phase. The former threshold level determined in 2005 was 500 MNOK.

### *1.2 The role of transport models in Concept evaluations*

Within the transport sector, several projects have been evaluated concept wise. Essential information for the actual choice of concept is how the travelers would change their travel pattern as a consequence of the evaluated concepts. These changes cannot be observed before the project is built, and thus need to be rendered probable by estimation done in transport models.

In Norway the Regional Transport Model (RTM), which is a transport model system covering all of Norway has been built up over 12 years. RTM is funded by the transport authorities, initially to be the transport analysis model intended to go into the processes of developing the national transport plans, but in later years also prepared to enter into other (or almost all) transport analysis processes. The model developing work also has included making input data available; transport network and land use situation, for the current situation and prognosis. The software used to implement the model is CUBE, distributed by Citilabs, allowing the development of a most user friendly and intuitive design. The calculation design and the accompanying input data is freely and easily available making it possible for a wide range of professionals to make use of the model system.

### *1.3 Earlier evaluation of the transport models*

The Norwegian Public Roads Administration (NPRA) carried out interviews among their employees in order to get feedback about the concept evaluation phase [1]. The discussion in the report concerns the activities in, and results from concept evaluations. The use of transport models as part of the process is briefly mentioned. The over all view is that the transport models are complex, challenging to use both practically and because it is time consuming though it was also mentioned that is used to be even worse with previous model systems. The model design in The Regional Transport Model (RTM) is claimed to perform better for evaluating corridors and road stretches compared to city analysis. The detailing level in the transport models is said to be too overwhelming for this strategic level, making the transport model calculation time consuming making boundaries for the progress of the analysis.

The report by NPRA also refers to a previous report made by the Transport Economic institute about Concept evaluation and impact analysis [2]. This has the viewpoint that it is not necessary (should not be) to construct specific models for this superior planning level, and recommends that as a principal rule the existing tools should be used (page 92). Amongst them is the RTM.

Both these reports emphasize the use of transport models as part of the analysis, but are not going into details in evaluating the RTM and its applicability for the various analysis. In 2009 an evaluation of the transport model was carried out [3], albeit almost four years ago, but is the most recent review of the RTM. The evaluation even dealt with two ferry replacement projects (page 45, Krifast and Skålavegen, both located on branch roads to E39), and certainly supports the use of RTM for such projects from the comparison between calculated increase in traffic and traffic counts.

All in all, based on previous model evaluations, there has been no indication that transport models in general or the RTM has had or have irregular problems regarding ferry replacement projects.

### *1.4 Concept evaluations of fixed links projects on E39 as an experience basis*

In later years the RTM has been used for several analysis of ferry replacement project. Rambøll and SINTEF have cooperated on doing some of these, and have gained experience with using RTM that we want to share. We use the experience to discuss the qualities of the current RTM and suggest improvements with the model and the model operation.

In the next chapter we present the main features of the RTM. Chapter three presents the concept evaluations of interest along E39. Chapter four deals with various aspects of the RTM and how it was used in the concept evaluations. The last chapter gives recommendations about improvements in the model and the application of it.

# 1 THE REGIONAL TRANSPORT MODEL

## *2.1 RTM design with various sub models*

The Regional Transport Model (RTM) is assembled by several sub models; which are dynamic regarding the demand, and fixed matrices; expressing fixed demand (see Table 3). The two major parts of the model consist of the national transport model (NTM), for long distance personal trips, and Tramod_by, for shorter personal trips. Whether long trips are represented by fixed matrices, for instance for a basic situation or it is is run for each scenario, is optional and this depends on the analysis.

Freight traffic is temporarily given by a fixed matrix which is estimated from a truck survey and continuous traffic counts; isolating long vehicles from these and assuming that they are trucks. This matrix will soon be replaced by a matrix originating from the national freight transport model.

The school model produces trips to and from schools on all levels from elementary schools to universities.

The airport-trips-model is giving the ground part of the flight trips; trips to and from airports. Only the 12 largest airports are included in the model. The matrices are fixed, and include car and public transport.

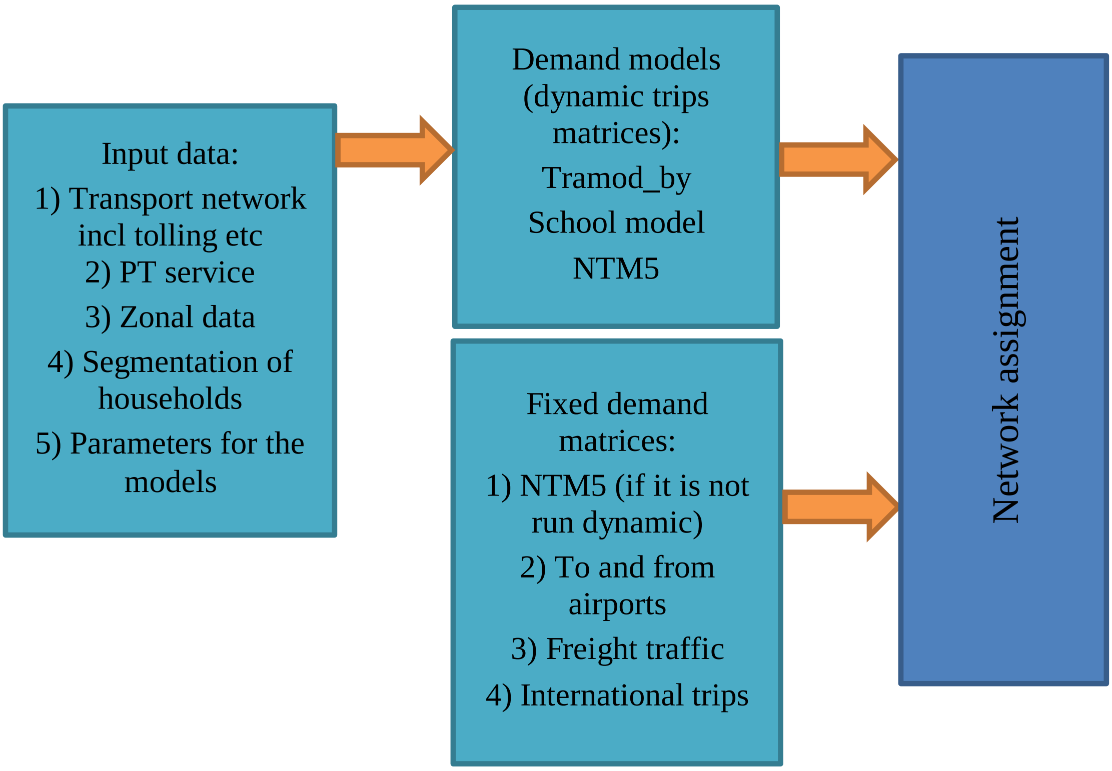


*Figure 29: The construction of the Norwegian Regional Transport Model with sub models and fixed matrices*

*Table 3: Trip categories in the RTM*

| Model | Trips | Purpose | Mode |
|---|---|---|---|
| *NTM*<br>**National transport model** | Private trips<br>≥ 100 km | Work<br>Visit<br>Leisure<br>Other | Car<br>Bus<br>Train<br>Boat<br>Plain |
| *Tramod_by*<br>**The demand model in RTM** | Private trips<br><100 km | To and from work<br>At work<br>Leisure<br>Private (incl. shopping)<br>Accompanying other | Car driver<br>Car passenger<br>Public transport (bus, train, boat)<br>Cycle<br>Walk |
| *School* | Schools, college and universities | To and from school | Bus<br>Public transport<br>Soft modes |
| **Fixed matrix:**<br>***International* trips** | Trips to and from Sweden | | Car<br>Bus<br>Train<br>(Truck) Freight |
| **Fixed matrix:**<br>**To and from *airports*** | Trips to/from 12 airports | | Car<br>Public transport |
| **Fixed matrix:**<br>**Freight** | Trips by trucks | | |
| **Not in the model system** | Foreigners in Norway<br>Local commercial trips | Weekend trips<br>Vacation trips<br>Stroll trips | Other modes than defined by Tramod_by |

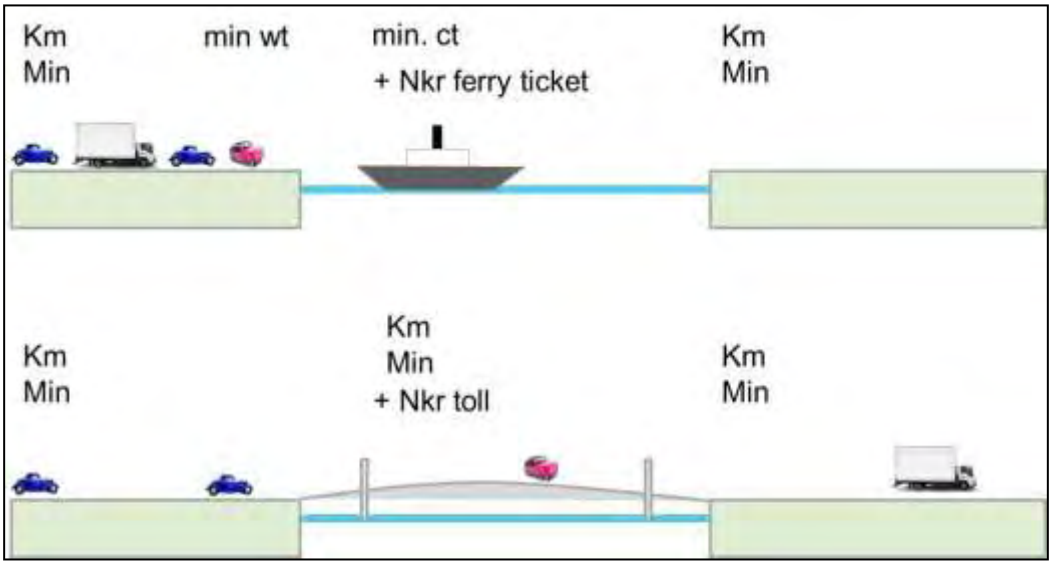


*Figure 30: Cost components used to represent ferry connections and fixed links replacing ferries*

### *2.2 How are ferries represented in the RTM*

Figure 30 illustrates how ferries and fixed links, in this case a bridge, are represented in the RTM.

Ferries are part of the transport infrastructure for cars and public transport. Ferry passengers are defined as public transport travelers if they are on board a bus link in the ferries, but also if they use soft modes, walk or bike, to the quay. The ferry parts of a car trip include waiting time and crossing time. The ferry ticket is given for the cars and the car drivers and car passengers separately. An overall factor states the discount level. The waiting time is set according to the headway, assuming uniform arrival distribution. Heavy vehicles have the same cost as passenger vehicles on the ferry.

Discounts are not included for car traffic in NTM. Also in the net assignment phase, in which the total traffic is distributed on the transport network, prices are assumed to be full; without discount.

Costs are multiplied with a discount factor, one for ferry costs and one for toll costs. This means that the discount factor expresses the share of passengers paying full price, thus the name in it self is misleading. Table 4 shows discount factors found in a travel survey on one ferry across the Romsdalsfjord from 2003. The left column (a) is used as a foundation for parameters in the model, as shown in Table 5. The average payment is however shown in the right column in Table 4, and reflects the experienced cost in a better way.

*Table 4: Discount factors found in a travel survey on a ferry connection between Molde and Vestnes from 2003*

| | **Discount factor a** | **Traveler covers costs themselves b** | **Discount factor regarding that some trips are paid for a*b** |
|---|---|---|---|
| Holiday or weekend trip | 0.83 | 0.80 | 0.66 |
| To/from work /school | 0.62 | 0.41 | 0.26 |
| Shopping | 0.83 | 0.75 | 0.62 |
| At work | 0.67 | 0.08 | 0.05 |
| Weekly commuter | 0.75 | 0.61 | 0.46 |
| Other | 0.81 | 0.61 | 0.49 |
| All trips | 0.72 | 0.39 | 0.28 |
| **All trips except for at work** | **0.77** | **0.63** | **0.48** |

*Table 5: Discount factors in the Regional Transport Model*

| | **Ferry car+car driver** | **Ferry Car passenger** | **Toll road Car driver** | **Toll road Car passenger** |
|---|---|---|---|---|
| To/from work | 0,6 | 0,83 | 0,75 | - |
| At work | 1,0 | | | |
| Leisure time | 0,7 | 0,9 | 0,8 | 0,9 |
| Follow others | 0,7 | 0,9 | 0,8 | 0,9 |
| Private | 0,7 | 0,9 | 0,8 | 0,9 |

### *2.3 How are bridges and tunnels represented in the RTM?*

Fixed links are represented by an ordinary link, with a distance, a capacity and a speed limit, often with toll costs on it. As the model for the time being is not taking vertical curvature into account, bridges and tunnels are represented the same way in the model. I reality a bridge would

mean a straighter road, because a tunnel across a fjord in most cases would have to go quite deep, given the current technology.
There are three main differences between ferries and fixed links in the RTM.

1. Waiting time
2. Crossing time
3. Distance

With the ferry the waiting time and crossing time is set previous to the calculation. It is not affected by the demand, queuing or knowledge to the timetable. Crossing the fjord on a ferry means that the vehicles are not driving and thus the distance contribution is zero from the ferry trip.

In analysis it is also required to have a formal political decision about tolling before including it to the transport analysis, unless the purpose of the analysis is to estimate the income from toll collection. This means that the tolling is left out of the concept evaluation in many cases, but would be present in most cases if the project is of such interest that it would be part of the national transport plan.

### *2.4 How is the reaction pattern to changes in the infrastructure in the RTM?*

A test of the elasticities in RTM was carried out and is documented as part of the demand model documentation [4, page 150]. The test only includes public transport and not ferry trips.

Several studies have dealt with price elasticity of various transport modes. The perhaps most comprehensive Norwegian survey was undertaken by Odeck and Bråthen [5] of 19 Norwegian toll projects. They found a short term elasticity for trunk roads from -0,24 to -0,75. Short term elasticity was defined as the effect of tolls on travel demand within one year of toll change. Traffic calculations in the regional transport model show a higher decrease in traffic demand than found in Odeck and Bråthen [5]. This is especially evident for lower tolls (see Table 6 and Table 7).

*Table 6: Traffic calculations in regional transport model (RTM), [vehicle trips per day]*

| Toll station | 0 kr | 10 kr | 20 kr | 30 kr | 40 kr | 50 kr |
|---|---|---|---|---|---|---|
| **E134 Damåsen** | 10115 | 9117 | 8719 | 8235 | 7819 | 7462 |
| **E134 Gammel trase** | 1908 | 1605 | 1410 | 1200 | 1061 | 925 |
| **Fv 286 Grossvoldveien** | 1023 | 877 | 795 | 701 | 639 | 573 |
| **E134 Saggrenda** | 5478 | 5216 | 5084 | 4925 | 4817 | 4712 |
| **SUM** | 18524 | 16814 | 16007 | 15060 | 14336 | 13673 |

*Table 7: Traffic calculations using price elasticity =0,5 (arc elasticity), [vehicle trips per day]*

| Toll station | 0 kr | 10 kr | 20 kr | 30 kr | 40 kr | 50 kr |
|---|---|---|---|---|---|---|
| **E134 Damåsen** | 10115 | 9731 | 9362 | 9021 | 8701 | 8402 |
| **E134 Gammel trase** | 1908 | 1836 | 1766 | 1701 | 1641 | 1585 |
| **Fv 286 Grossvoldveien** | 1023 | 944 | 872 | 811 | 759 | 713 |
| **E134 Saggrenda** | 5478 | 5201 | 4938 | 4577 | 4146 | 3681 |
| **SUM** | 18524 | 17713 | 16939 | 16110 | 15247 | 14438 |

Around 2007 the new RTM was tested unofficially on several projects. The feedback from these tests was, among other things, that the model generated too little traffic on ferries. These tests were sadly not documented. An impression that the model was too elastic for disbursements was established even if this was not proven or documented. One of the parameter in the RTM controls the length distribution of trips, and it was assumed that the trips probably went further if a ferry crossing was part of the trip, and so this parameter was altered as a test. Because this parameter applies to all trips and not solely ferry trips, the test was not successful. All trips became longer in the model giving unrealistic results in other parts of the model, especially in city centers.

The impression that the model is too dismissive of trips that imply direct costs, including ferry trips and tolls in general, has grown stronger since it was first suggest in 2007.

It was not until last year we started looking at the discount factor which might be responsible for the situation.

### *2.5 How about wider impacts represented in the RTM?*

Investments in infrastructure, like fixed link projects might cause spillover effects, in a collective term named wider impacts. This is additional impacts on top of initial changes in the travel pattern, but caused by the new infrastructure.

A study carried out by Lian et al [6] of 102 major road projects opening between 1993 and 2005 showed a small effect on population growth, but no effect on employment, income levels commuting or industrial growth, from road investments.

Last year COWI presented a review of the treatment of wider impacts in impact assessments in the transport sector [7]. They define "wider impacts" as impacts from transport infrastructure projects that are not already part of the methods used by the transport model and cost benefit analysis. The study calls attention to impacts in the labor marked. Large shifts in the transport service, like a fixed link as replacement for a ferry, could cause radical changes in the labor marked and added value for business and industry.

An approach based on the same principles, but tied to a specific case, was carried out in Heum et al [8]. They assessed the impacts for labor markets and the added value of a ferry-free E39 from Nordfjord to Kristiansund (the current ferry crossings in Møre and Romsdal county, which is the northern part of the western Norway). Heum et al [8] made an analysis with a similar approach but for the southern parts of the western Norway. In both studies they estimate the added value, measured in increased total income due to increased profits of higher productivity.

The definition from COWI: "all impacts not already in the model" would thus cover effects like population growth, labor market expansion and increased productivity. We could include these in the impacts analysis if we knew that there not any risk of counting the effects twice.

Population growth is represented in the RTM by demographic prognosis which are not affected by infrastructure improvements. If we find a method that gave probable size of such effects, we did not have to worry about counting this effect twice in the transport analysis.

Labor marked expansions would probably work as illustrated in Figure 31. Reduced transport cost across a fjord would imply changes in trip frequencies, destination choice and mode choice. This is part of the transport model already, and some of these changes are short term while other changes are more long term. It is often assumed that all adjustments to new infrastructure have happened after about a year. This is illustrated in Figure 31 by shorter distances, wider labor marked and more trips on the new fixed link than on the ferry it replaced. The higher productivity could also attract more investments, expand existing or arrange for new business and widen the jobmarked more, increasing the number of trips on the fixed link beyond the initial increase. The additional increase is wider impacts according to the definition from COWI, because this effect is not already accounted for in the current transport models.

An example on a situation very likely to arise along E39 can illustrate the effect. A factory is located on one side of a ferry connection, and is producing furniture. It is located close to the sea because of the transport by boat of intermediate goods to and goods from the factory. On the other side of the ferry connection is a smaller town. Easier access to the factory helps recruit more workers, increase productivity and increase the traffic on the fixed link.

In the transport model land use data (or zonal data) is given exogenously. The number of residents, students, workplaces within categories, parking policy etc. is given for each basic district. This goes into a basic year calculation, used to calibrate and validate the model. Prognoses for the development of settlement pattern taking trends about birth rates, death rates, migration pattern and immigration pattern into account are made by Statistics Norway and publicly available. Prognosis for development in workplaces is not prepared the same way.

Up until now the transport analyses actually have been carried out without prognoses for changes in the employment structure. This means that the current labor market structure is input also in prognosis for trips to and from work and at work. The number of trips is decided by the number of workers, thus there will be an increase in trips if there is an increase in residents in an area, but the trips are attracted to places where the current companies are located. Where the trips are attracted is decided by the generalized cost distribution from where the people are resided to the location of the workplace. An infrastructure project might alter the generalized costs and thus cause changes in the trip attraction. This means that we might not be able to separate between impacts already taken care of in the model and the so called wider effects. This can only be done by getting estimates about new workplaces due to the infrastructure investment, and then we probably need an alternative zero (basic scenario) to compare this to, to measure these wider impacts.

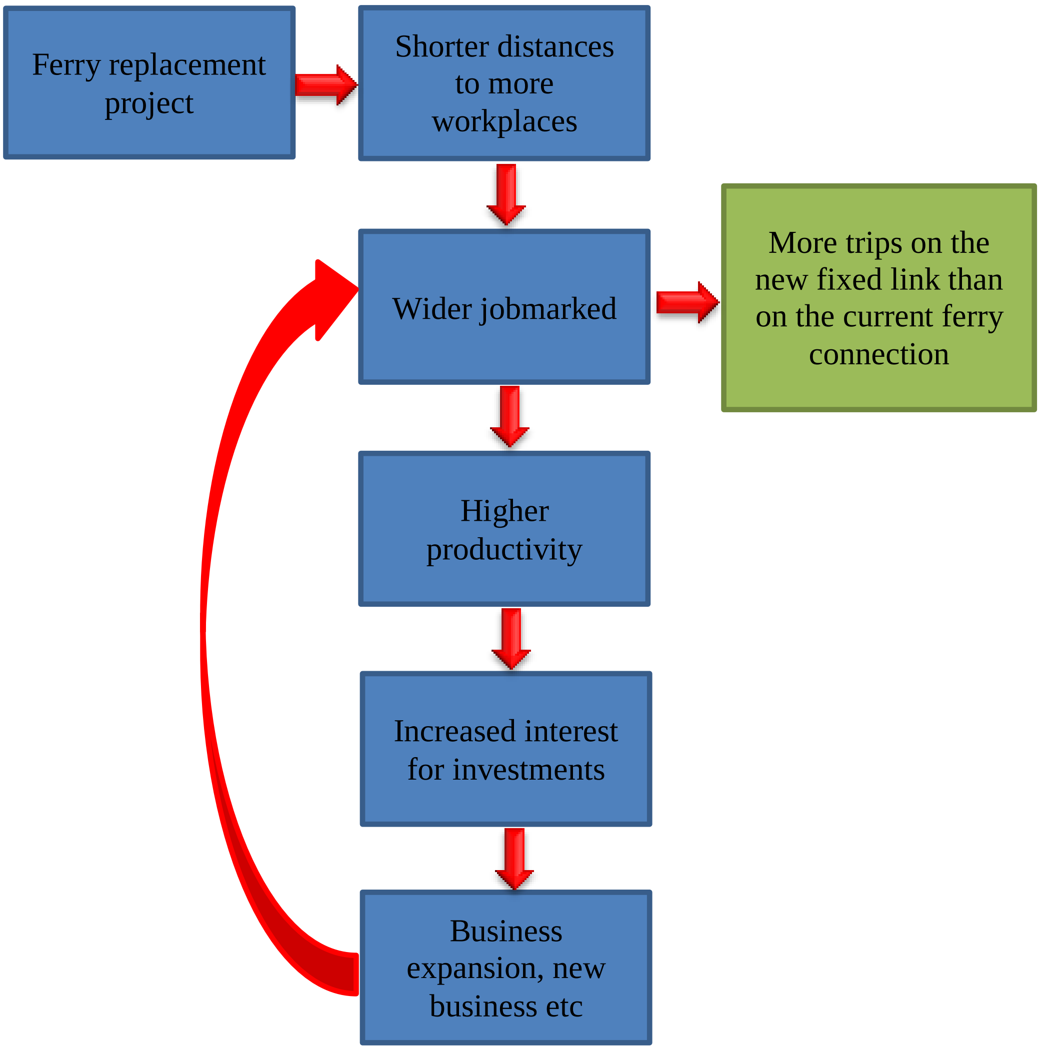


*Figure 31: Impacts on the labor marked from ferry replacement projects*

## 3. EVALUATIONS OF NORWEGIAN FERRY REPLACEMENT PROJECTS

### *3.1 Concept evaluations along E39*

The coastal highway E39 has been object of many concept evaluations carried out recently. These are shown in Figure 32 with green circles around the cities and blue around road stretches. The three on top included four ferry replacement projects. The concept evaluations for Aksdal-Bergen (2011) and Boknafjorden (2007) also included ferry replacement projects. All of these used the RTM as analysis tool. The concept evaluation for Boknafjorden was done almost eight years ago, so comments in the report about the model from that analysis are regarded as less interesting than the other four.

As a supplement to the three concept evaluations Bergsøya-Valsøya, Ålesund-Bergsøya and Skei - Ålesund, a superior concept evaluation covering Valsøya-Skei was carried out as well.


*Figure 32: Concept evaluations carried out along the coastal highway E39*

The ferry connections are shown to the left in

Figure **33**. The crossing time on each fjord is given in the map to the right in the same figure. In the right map Voldafjord is not mentioned, and this is because alterations in the road network have made another route choice, around Kvivsveien which opened in September 2012, more convenient for through-traffic.


*Figure 33: Fjords with ferries on the coastal highway E39*

### *3.2 Ferry replacement projects in recent concept evaluations*

Four concept evaluations carried out in 2011 included ferry replacement projects. All of them used the RTM. The assumptions made in the transport analysis are given in Table 8.

#### *3.2.1 Representation of ferry traffic in the RTM*

As row three in the table shows, the traffic on the ferry is well represented in the model. Still there have been concerns that on some ferry connections, especially those in proximity to larger cities, the traffic in the model is lower than is should be.

#### *3.2.2 Tolls on the fixed link*

Even if it is highly likely that the fixed link will imply a toll collection around the amount of the current ferry ticket price, most of the fixed links are analyzed in the RTM without tolls. This is a distinction between how the calculations are done for projects that enter into the National Transport Plan (NTP) and in these concept evaluations.

This naturally overestimates the traffic on the link. The explanation for this practice is that toll collection not should be assumed on links in transport analysis unless it is decided by the national assembly. In the early planning process, like concept evaluations, this has not been discussed by the authorities at all, and thus tolls are not assumed. Sensitivity analyses testing the effects of toll collection on fixed links are requested, if the time frame allows it. In Table 8, one of concept evaluations included sensitivity tests with toll collection, while the others did not.

Not using toll collection on the fixed links in the transport analyses has also been justified by the view that toll collection is too discouraging.

*Table 8: Facts about the concept evaluations carried out with a recent version of RTM and which included one or more ferry replacement projects*

| | **Bergsøya – Valsøya** | **Ålesund – Bergsøya** | **Skei – Ålesund** | **Aksdal – Bergen** |
|---|---|---|---|---|
| Fjord | Halsafjord | Romsdalsfjord | Storfjord and Nordfjord | Bjørnafjord |
| Year of Concept evaluation | 2011 | 2011 | 2011 | 2011 |
| | | | | 2018-42 |
| Observed traffic/ Estimated traffic on ferry (2010) | 835/1100 | 1917/1800 | 1548/1300 (Solevågen – Festøya) 1235/1400 (Anda-Lote) | 2406/ 2361 |
| Toll calculations on fixed link | No | sensitivity analysis | No | No |
| Calcuation of waiting time | headway/2 | headway/2 | headway/2 | headway/2 |
| Calculation in National Model | Yes | Yes | No | Yes |
| Freight traffic | Calibrated matrix | Calibrated matrix | Calibrated matrix | Fixed matrices |
| Annual traffic growth factors | used different growth rates (high, medium, low) | used different growth rates (high, medium, low) | 2 % | SSB |

### *3.2.3 Assumptions regarding the national Transport Model and influential area*

The National Transport Model (NTM) produces private trips of 100 km or more. If it is expected that the project affects the trip generation, the trip pattern (origin destination pattern) or the mode choice for long trips, the national transport model should be used to estimate the demand changes. If not, the analysis can use fixed matrices (one for each mode) for the basic situation, allowing only for changes in the route choice.

As we can see from Table 8 calculations with the NTM was carried out in three of the concept evaluations.

An ongoing project uses RTM and NTM (and even the freight transport model) to study effects of ferry free coastal highway all the way from Trondheim to Kristiansand. The effects on NTM traffic can be viewed in Figure 34. Ferry free connection can imply reduction in travel times using the E39 all the way of 7-9 hours. The reduction causes huge effects for long trips; less traffic chooses inner routes and more traffic chooses the coastal highway E39. This effect is only possible to reveal if we assume that the influential area is wide enough.

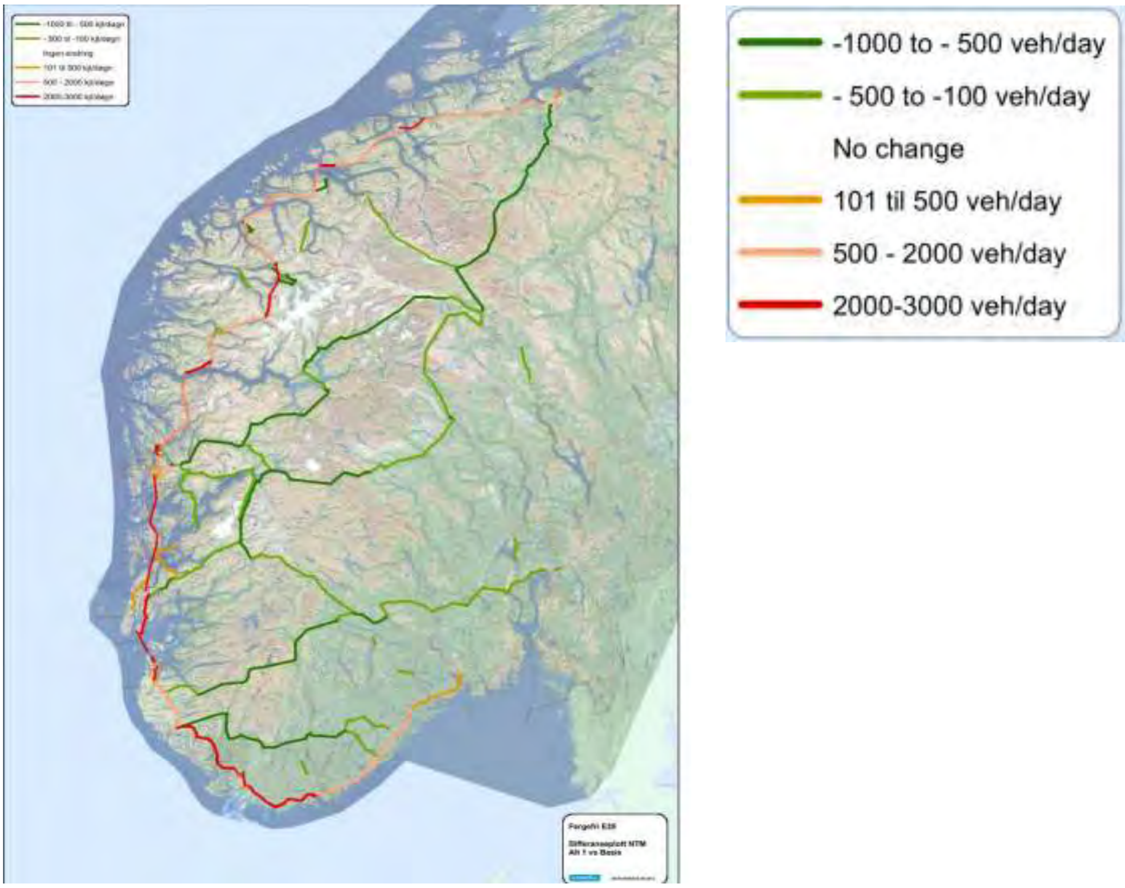


*Figure 34: Change in travel route and demand for long trips Scenario year 2020. The coastal highway ferry free (red = increase in travel demand, green=decrease in travel demand*

### *3.2.4 Assumptions regarding freight traffic*

The national freight transport model was not prepared for use as an integrated part of RTM as the concept evaluations was carried out. Consistency checks against counts indicated that the fixed matrix for freight traffic was not good enough to represent the real traffic in the north western part. We then made a new freight matrix for the main arterial roads based on counts as a calibrated matrix. The concept evaluation for Bergen –Aksdal used a fixed share based on the other traffic.

### *3.2.5 Prognoses*

Statistics Norway prepares prognoses for the population growth, taking migration, immigration, birth rate and death rate into consideration. Prognoses exist at regular intervals until 2060. Also it is possible to use the current transport situation and use growth factors prepared for EFFEKT (the cost benefit analysis tool developed by NPRA). In Table 8 the three concept evaluations to the left in the table used fixed growth factors while the Bergen-Aksdal concept evaluation has used prognoses from Statistics Norway.

The difference between using fixed growth factors and prognoses from SSB is that the last-mentioned allows for the trend that people move to centers. This is illustrated in Figure 35.

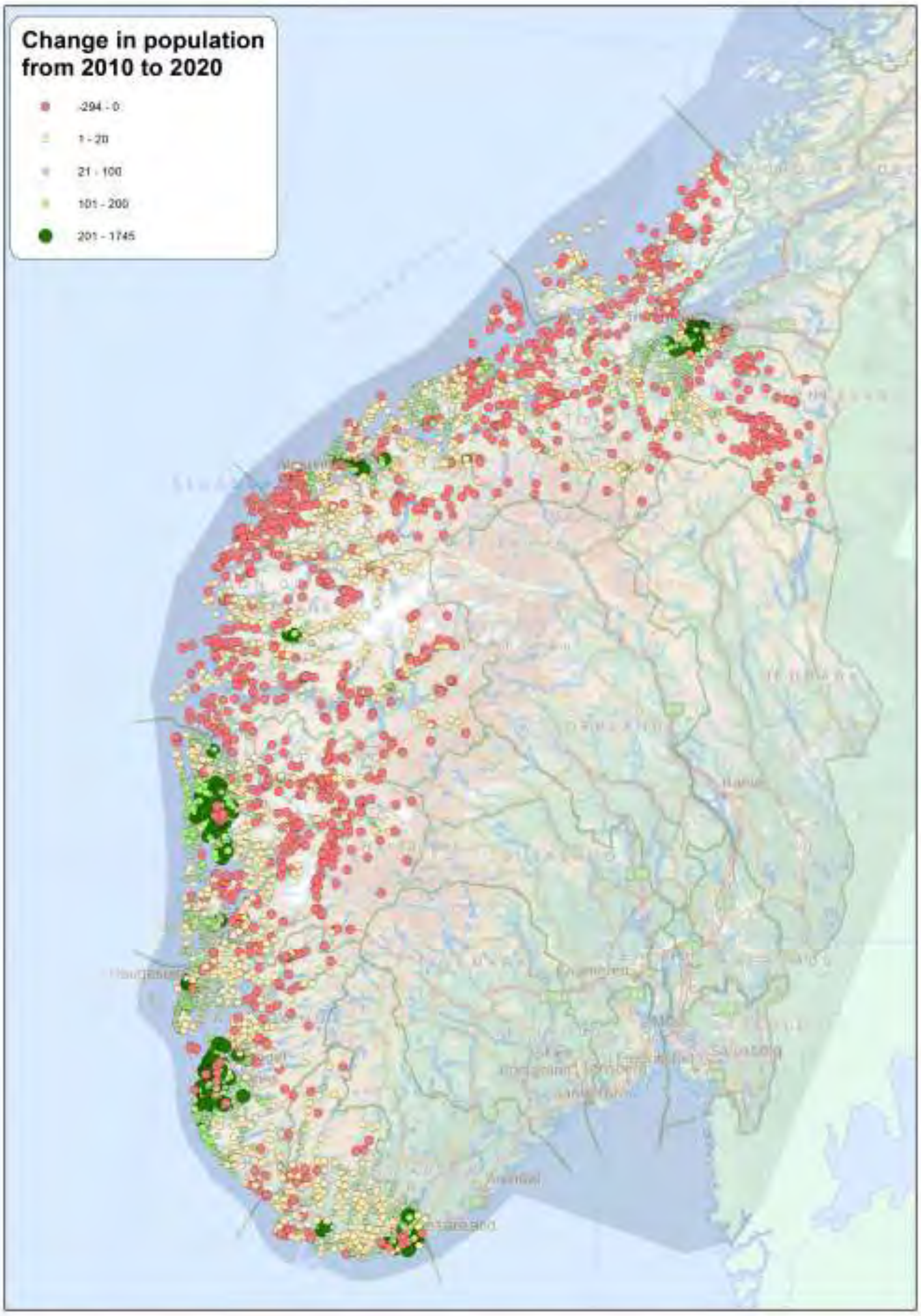


*Figure 35: Population growth from 2010 to 2020, showing the centralization trend (red =reduction, green =growth)*

For the ferry replacement projects the difference in method is probably not crucial, but the centralization trend probably gives over all reductions in the traffic on the ferry crossings.

### *3.3 How was the modeling of the ferry replacement projects received by the quality assessment process?*

The Concept evaluations carried out have all been objects of quality assessments. The team that assessed the concept evaluations were given the opportunity to assess the model used and how.

The three concept evaluations regarding projects in Møre and Romsdal was assessed together. Terramar and Oslo Economics [9] had the following comment: "The method used is assessed as suitable. The method has been used for each road stretch and not for the whole distance. That could have been done to reveal if all kinds of measures for the corridor have been identified."

The concept evaluation for Aksdal – Bergen got no comments about the transport analysis [10].

The comment from the assessment of the concept evaluation of Boknafjorden was: "Our experience is that the models give reasonably realistic results when used to assess the impacts of measures in the transport system" [11].

To sum up, not one of the teams doing quality assessment were negative to using the transport model or the assumptions made for the specific analysis.

## 4. CONCERNS ABOUT USING THE TRANSPORT MODEL

In addition to elements with the model system related to the concept evaluations, we also want to draw attention to other aspects with the transport model, which can be challenging in the current model. These are:

- Lack of validation data
- Description of the ferry trips
- Division between Tramod and NTM5
- Modeling commercial traffic

### *4.1 Lack of validation data*

Many analysts have experienced and reported that some ferry connections get too little traffic in the basic situation. The reason has not been studied in detail, but the discount factor, the lack commercial traffic or the distance distribution may be plausible explanations.

Other suggestions have been that people who are used to being dependent on a ferry connection has a higher willingness to pay than the general population. Also is it is said that a ferry replacement project can be seen as an investment in the future for the residents in the area, since the fjord crossing will be free of charge eventually when the fixed link is paid for.

### *4.2 Modeling ferries*

The description of ferry trips in the model seems inappropriate. That one description fits all the ferries is unlikely. It should be possible to specify each ferry crossing to fit the real service as good as possible. For some ferry connection, as an example, it should be possible to include capacity in the model calculation to represent the waiting time in a more realistic way.

### *4.3 Division between Tramod and NTM5*

The short distance model Tramod produces trips shorter than 100 km. Longer trips are modeled by the NTM. Ferry trips imply time use on the ferry but not distance traveled by car, which controls which model should be used. When the ferry is replaced with a fixed link, the fjord crossing imply that the border for Tramod and NTM changes, and moves closer to the ferry connection. This is not realistic. It is not realistic that people travel shorter due to a fixed link, it is more likely that the reaction goes the other way.

### *4.4 Modeling commercial traffic*

Freight traffic is included in a fixed matrix in the model, but this contains only trucks. Other commercial traffic, made by craftsman or home care local delivery services or local distribution of goods is not part of the demand model in RTM. If the vehicle is smaller than a truck; like a van or smaller, the vehicle is not part of the freight matrix, and thus not represented in the RTM at all. This can become a substantial number of vehicles on some ferry connections.

## 5. HOW COULD WE IMPROVE THE TRANSPORT ANALYSIS OF FERRY REPLACEMENT PROJECTS?

### *5.1 Including wider impacts*

Transport infrastructure is an imperative for daily life and business participation. Getting a fixed link across a fjord imply an improvement with around the clock possibility to cross the fjord and a huge increase in the capacity and reliability. We can perhaps catch some of the transport related effects in the transport model, but probably not all of them.

We will expect to find all the previous traffic from a ferry on a new fixed link. Changes in destination choice are generally the other most prevailing behavioral effect in the transport model of a fixed link. Changes in route choice are rarely an alternative. The consumer surplus calculation will consist of savings in time use for all travelers on the fixed link, due to no waiting time, and faster crossing time. The longer driving distance on the fixed link is a secondary effect and amount to a smaller cost than the earnings from reduction in time use.

Major improvements in the infrastructure might give considerable impacts for residents and business close to the fjord, beyond those that normally are included in analyses of changes in the transport pattern. In the KVU some those effects are, as an experiment included in the analyses. The impacts are however only concerning wider labor marked. This is an effect which is difficult to implement in the transport analysis. If this effect is put on top the already estimated effects from redistribution of trips to and from work, we risk double counting. New business, increased activities in existing businesses given in number of work places, and moving pattern among residents, as a direct impact of a fixed link, would be better variables to include in the analyses.

### *5.2 Surveys*

A contribution to improving the model would be to do surveys on ferry connections and on fixed links already built. The data collection could also include expansions in the work marked, number of residents and other adjustments made by people and business to adapt to a fixed link instead of a ferry connection.

Current surveys are to a certain extent accommodated to be compared with transport models. Especially for the traffic not in the current model, local commercial distribution, home care services and craftsmen should be separated from the purpose "at service" in the model, to help estimate the number of trips not estimated in the model.

Since the national travel survey has too few data with ferry trips, the sample sizes should have been extended in municipalities or counties were ferries are an important part of the transport infrastructure.

### *5.3 Description of the ferry trip*

It seems unlikely that the rough way the ferry crossing is represented in the model, reflects how people perceive a trip with the ferry. Waiting times in real traffic are controlled by the capacity of the ferry and are expected to vary by trip purpose, while the model uses half of the headway to represent waiting time. The estimation process of Tramod_by reported (Rekdal et al., 2012) that the national travel survey used in the estimation, consisted of too few trips on ferries, and demonstrated the need for more data about the perception of ferry trips. Representing ferry trips in a better way was claimed to be the second most important improvement with the demand model, after city related challenges.

An impression that the model is too elastic regarding expenses has been introduced in discussions. This is not documented anywhere, and none of the ferry connections in the concept evaluation in this paper has shown any weaknesses like that for the present situation. This could be because the traffic on the ferry was calibrated as part of the preparation of the model.

That the concept evaluations were done without assuming tolls on the fixed links, are likely to overestimate the traffic on the fixed link, and it seems unfair to compare a free fixed link to a ferry trip which include ticket costs.

We think that the reason why the fixed links are assumed without costs is related to the impression about the elasticities. Another plausible explanation is that the concept evaluation process is rather new and that the recommendations about analyzing tolls on fixed links has not been developed yet.

### *5.4 Documentation of the RTM*

Talking to analysis at the NPRA, they recognize the problem with too little traffic on some ferry connections. Part of the problem might be that the discount factor was not calibrated to each ferry connection, and that the expenses were too high in the model compared to real life.

The documentation about setting the discount factor could also have been more informative. An improvement in the model would imply to open for a differentiation in defining the discount factor for each ferry connection individually.

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# A 3700 M SINGLE SPAN SUSPENSION BRIDGE

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**ABSTRACT**

*The Norwegian Public Roads Administration (NPRA) is doing a feasibility study of crossing the Sognefjord with a single span suspension bridge. The main span considered is 3700 m. This is the longest possible single span suspension bridge along the Costal Route E39, and the Sognefjord is therefore chosen as a case study. Suspension bridges longer than 1624 m are not yet built using the design with a single steel-box girder. A new configuration of the suspension bridge is therefore necessary to cross the wide fjords along E39. This study considers different configurations of split box girders. To find representative dimensions and eigen-frequencies a model has been built up using the FEM-program Abaqus. One of the main challenges is aerodynamic instabilities, and the first step is to find a structural concept for spans longer than 1600 m suitable for a low traffic volume.*

## INTRODUCTION

The Coastal Highway Route E39 is a project taking on an investigation of eliminating all ferries along the western corridor of Norway. The project is divided into four main components with the subproject *fjord crossings* being one of them. Being 3.7 km wide and 1200 m deep, the Sognefjord is considered to be one of the most challenging fjords to cross, both in terms of technology and cost. It is therefore chosen as a reference fjord for a feasibility study. Feasibility studies are carried out on floating bridge, submerged floating tunnel and suspension bridge. This paper comprises a suspension bridge spanning the entire fjord.

## BRIDGE CONFIGURATION

The bridge proposed crosses the Sognefjord in Norway with a single span of 3700 m. It is designed to carry two traffic lanes, each with a width of 5.5 m and two 3.0 m wide walkways. However, the walkways will also be used for maintenance work.

*Figure 36: Elevation of the bridge.*

Each of the box girders (see *Figure 37*) are considered wide enough to carry two lanes of traffic if one box girder need to be closed in case of emergency or major maintenance work.

**Stiffening girder**

The stiffening girder has the form of two wedge shaped steel boxes, connected with cross-beams at the hangers every 30 m, see *Figure 37*.

*Figure 37: Cross Section of box girders.*

The width of the steel boxes is 12.9 m and the depth is 2.5 m. The steel boxes have a centre distance of 20 m. In general the steel grade used in the box girder is S355N. Thickness of the outer steel plates of the box is 8 and 12 mm, with a deck plate of 14 mm. The thickness of the trapezoidal stiffeners inside the box girder is 6 mm. The depth of the cross beams are mainly 2.5 m and the width 1.5 m, with 20 mm outer steel plates and 6 mm stiffeners inside. The first five cross beams close to the towers have a width of 2.5 m due to high shear forces from wind load.

The shape of the stiffening girder in this feasibility study is based on experience with similar Norwegian wedge shaped box girders for more than three decades. The shape may later be somewhat altered in order to improve the aerodynamic properties, but stiffness and mass of the box-girders and thus the dynamic properties of the bridge will not be significantly changed.

**Main cables and hangers**

Erection of the main cables may be performed either by the method of aerial spinning or by parallel wire strands (PWS). In this feasibility study the nominal tensile strength of the wires is chosen as 1770 MPa, giving each main cable a steel area of 1.15 $m^2$. This is the maximum tensile strength currently allowed according to Norwegian regulations, but an increase in tensile strength will probably have a positive effect on the dynamic properties of the bridge. Horizontal distance between the main cables is 31.1 m, and the sag is 370 m.

Fixing of the main cables at each side of the fjord may be performed by a rock anchoring system consisting of a splay chamber, a force transmission block and an anchorage chamber.

Hangers may be performed as full-locked coil ropes, i.e. several layers of Z-shaped wires stranded in alternating directions on top of a core of round wires. Nominal diameter of the hanger rope is calculated to be 100 mm.

**Towers**

The towers in this feasibility study are made of concrete cast in situ. Structurally they are traditional frame structures consisting of two columns and five transverse beams. The columns have a small inclination hinting an A-shaped tower. Both cross section of the columns and the transverse beams have the shape of a hollow rectangle. The wall thickness varies from 1 m up to 1.2 m, while the outer dimensions of the columns change gradually from 32 m x 15 m at foundation to 10 m x 10 m at the top.

## DESIGN BASIS

Traffic loads according to Eurocode are defined for bridges with load lengths up to 200 m only, although the Norwegian appendix expands the application to 500 m. Further expansion of the load length seems unreasonable when comparing the load magnitude in Eurocode and common practice on previous suspension bridges in Norway. Furthermore, the load factor for dead-load of 1.35 (as defined in Eurocode) is considered to be very high for a steel structure like this where you are in position to control the dead-load accurately.

In this feasibility study the traffic loads used in the design calculations for the main cables are limited to 9 kN/m per traffic lane and 2 kN/m per walkway, adding up to a total of 22 kN/m. The load factors used in the design of the main cables for dead-load and traffic loads are 1.2 and 1.3, respectively. For the box girders and hangers the traffic loads and load factors are in accordance with Eurocode.

Ordinary SLS and ULS design is based on 50 year return period with a characteristic mean wind speed $V = 44.3$ m/s normal to the bridge length axis at the elevation of the stiffening girder, which is approximately 80 meters above sea level. Eurocode does not specify a wind climate beyond 200 m, so the design of the towers is based on an extrapolation of the Eurocode description up to 450 m.

The wind load analyses are based on a wind tunnel test with a section model performed on a shape-wise similar cross section with a similar frequency ratio [2]. The results from this test are scaled to fit dimensions of the suspension bridge dealt with in this feasibility study.

## WIND ANALYSES

### Static analysis

Static wind analysis follows a procedure using a FEM-model with static wind load applied on the stiffening girder according to $C_D$, $C_L$ and $C_M$ from the wind tunnel tests reported in [2]. Table 1 shows the static response of the stiffening girder at centre of span.

*Table 9: Static response,* $\boldsymbol{V = 44.3\ m/s}$

| $x/L$ | Y [m] | Z [m] | α [°] |
|---|---|---|---|
| 0.25 | 11.2 | 0.30 | 0.84 |
| 0.5 | 9.98 | 0.30 | 0.97 |

Y: Horizontal deflection [m]
Z: Vertical deflection [m]
α: Torsional rotation [degrees]
Positive values; alongwind; lifting; windward side lifted.

The critical wind speed for static divergence can be calculated using the following expression [1]:

$$V_{kr} = 2\pi b n_\theta \sqrt{\frac{2m_\theta}{\rho b^4 C_m'}}$$

In the wind tunnel test $C_M'$ is negative at angle of attack equal to zero. If $C_M'$ had been linear, static divergence could not occur. For this cross section $C_M'$ decreases until the angle of attack is 4 degrees, then it starts to rise rapidly. Using $C_M'$ of angle of attack between 4 and 8 degrees the following critical wind speed is obtained:

$$V_{cr} = 2\pi \cdot 12{,}9 \cdot 0{,}0869 \sqrt{\frac{2 \cdot 6{,}47 \cdot 10^6}{1{,}25 \cdot 12{,}9^4 \cdot 1{,}88}} = 99\ m/s$$

This is also calculated in the FEM-program Abaqus updating the load coefficients corresponding to the angle of attack as the wind speed increases. The results correspond well with the expression for static divergence (see Figure 38).

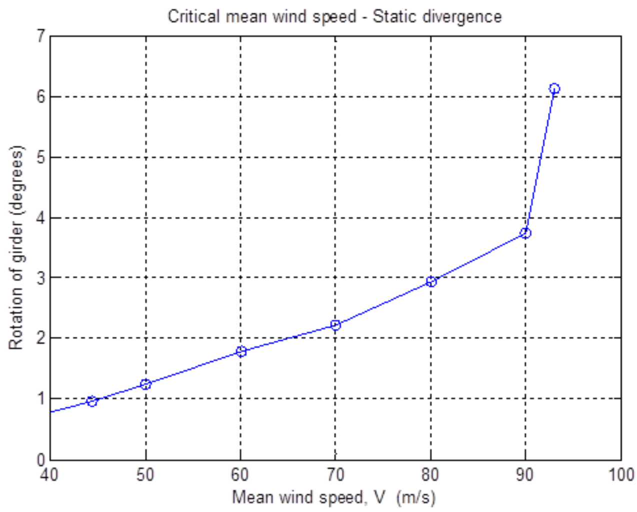


*Figure 38: Static divergence*

**Dynamic analysis**

Three individual eigenmode calculations are carried out using:

i) The computer programme Alvsat
ii) The software NovaFrame
iii) The FEM-software Abaqus

The different eigenmode calculations give almost equal results despite the different theoretical approaches applied in the programs. The first 6 eigenperiods are:

First horizontal symmetric: T = 36.5 s
First horizontal asymmetric: T = 23.0 s

First vertical symmetric: T = 12.9 s
First vertical asymmetric: T = 16.4 s

First torsion symmetric: T = 10.1 s
First torsion asymmetric: T = 12.3 s

The frequency ratios of the first torsional/vertical symmetric and asymmetric modes are 1.28 and 1.33, respectively.

Three buffeting analyses are carried out using three different programs – Alvsat, NovaFrame and multi-modal response calculation using eigen-frequencies from Abaqus. The multi-modal response calculation is done by a MatLab script developed by Prof. Einar N. Strømmen, based on the theory described in [1]. Frequency domain analyses are based on Davenport's approach [3, 4] using 18 eigenmodes. The dynamic response gives fairly similar results in the vertical and horizontal direction while there are some deviations in rotation. This is most likely due to different definitions of the spectrum and co-spectrum.

*Table 10: Standard deviation ($\boldsymbol{\sigma}$) of the dynamic response at $\boldsymbol{V = 44.3\ m/s}$ using the multi-mode approach.*

| $x/L$ | Y | Z | α |
|---|---|---|---|
| 0.25 | 1.79 | 0.71 | 0.30 |
| 0.5 | 2.18 | 0.79 | 0.38 |

Y: Horizontal deflection [m]
Z: Vertical deflection [m]
α: Torsional rotation [degrees]

## WIND STABILITY LIMIT

Two individual flutter analyses are carried out:

i) Two-mode flutter instability calculation
ii) Multi-modal response calculation

Flutter velocity is identified as the lowest wind speed at which one of the in-wind modes has zero damping thus causing "infinite" response. Different combinations of modes are evaluated. The critical wind speed (flutter) is estimated as 125 m/s, which is calculated with flutter derivatives (FDs) obtained from the buffeting response data of a section model. The four eigenmodes activated in flutter are shown in *Figure 39*, where it is seen that the asymmetric modes are shape-wise perfectly equal.

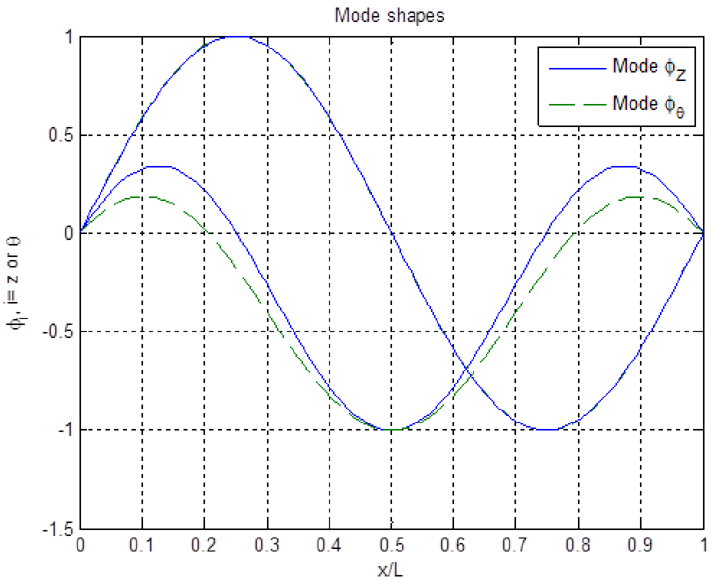


*Figure 39: Eigenmodes that contribute to flutter instability*

The asymmetric modes are the first ones to cause flutter. This is shown by plotting the frequency response function. *Figure 40* shows the normalized frequency response for the two asymmetric modes. At the left hand side on *Figure 40* the eigen-frequency of each of the two modes are visible as two peaks. As the (normalized reduced) wind speed, $V_r$, increases, aerodynamic damping and stiffness reduction is introduced causing the system behaviour to change. With the (normalized reduced) wind speed increasing even more, the aerodynamic damping causes the in-wind eigen-frequencies to change so that the two modes merge into one frequency. This causes the system response to approach infinity. This instability is seen on the right hand side of *Figure 40* where the reduced wind speed is approximately $V_r = 23$, corresponding to a wind speed equal to $125\ \mathrm{m/s}$.

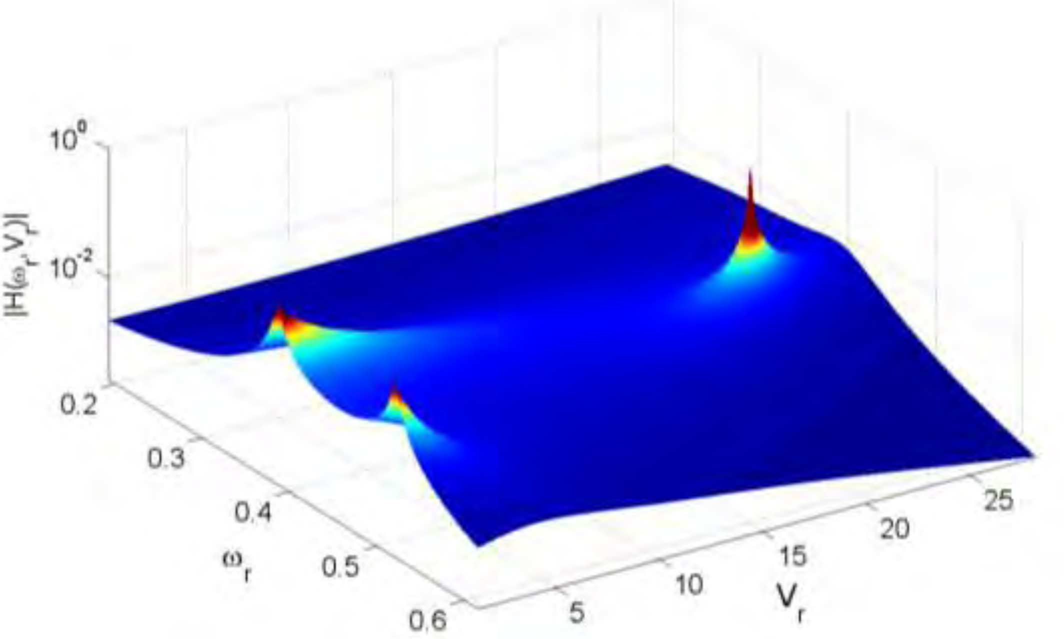


*Figure 40: Normalized frequency response plot.*

The multi-modal response calculation is similar to an ordinary frequency domain buffeting analysis, but it includes a complete description of the motion-dependent stiffness, damping and coupling terms. The calculation includes 18 modes, 3 symmetric and 3 asymmetric modes in each direction – horizontal, vertical and rotation. As *Figure 41* shows the box girder becomes unstable at a mean wind speed of about 125 m/s.

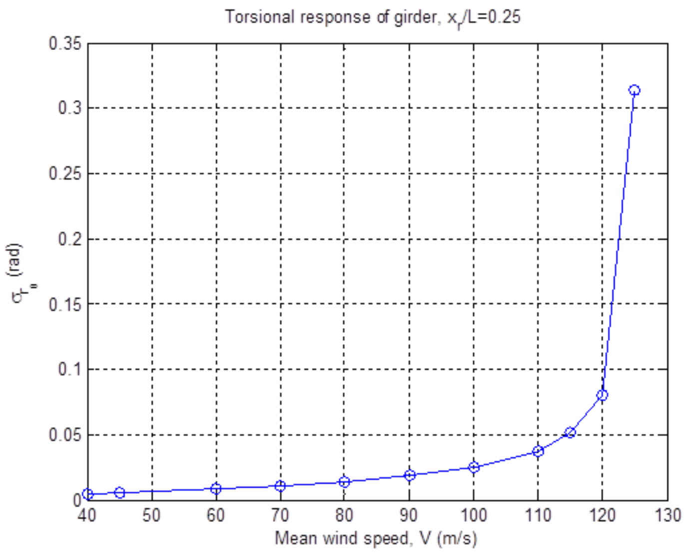


*Figure 41: Standard deviation of torsional response as mean wind speed increases.*

## CONCLUDING REMARKS

The flutter instability limit is calculated in frequency domain based on wind tunnel test reported in [2]. The calculations show an instability limit of approximately 125 m/s. For the phenomenon of static divergence an instability limit is calculated to approximately 100 m/s.

Static divergence occurs before flutter, and this indicates that the flutter calculations may be inaccurate. Although the wind tunnel tests used has a shape-wise similar cross-section and equal frequency ratio, the absolute frequency for torsion reported in [2] is much higher compared to the feasibility study reported herein. It is therefore necessary to perform extensive wind tunnel tests to establish an accurate wind loading model including shape factors and flutter derivatives.

However, this cross-section indicates promising behaviour for high wind velocities.

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# HOW TO CROSS THE 7500 M WIDE BOKNAFJORD?

*A concept study of a five-span suspension bridge, supported on TLP-Moored floaters*

by

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## ABSTRACT

*A combined hydrodynamic and structural analysis was performed in order to assess the global response of a 7500m meter long, tension leg moored, floating suspension bridge in an offshore environment. A structural model taking large displacements into account was established in RM BRIDGE [11] in order to assess the stiffness properties and eigenmodes of the bridge structure, including contributions of the tension legs and added hydrodynamic mass. The RM BRIDGE model was used to calibrate structural stiffness of the hydrodynamic model in ANSYS AQWA [10]. Fully coupled hydrodynamic time domain calculations have been performed. The results provide insight into the behavior of the bridge when exposed to first and second order wave loading, irregular wind and current.*

## INTRODUCTION

The Norwegian Public Roads Administration (NPRA) has started the work to study if it is technologically feasible to replace ferry crossings of the 8 widest fjords on the west coast of Norway by fixed links. The fjords are wide and deep. The environmental loads as wind and waves might be a challenging task. New engineering challenges combining offshore and bridge experiences are necessary to introduce.

This paper discusses a floating bridge concept where a multi-span suspension bridge is placed on tension leg platforms (TLPs). This concept comes to use where the shores are too far apart to use a regular suspension bridge (>3km), and the waters are too deep to place common foundations (>200m). TLPs have been used offshore for many decades in hard climate conditions, and have the advantage of not being prone to heave, roll or pitch. This reduces the challenges associated with tidal variations, and 1th order wind and wave response. The lateral flexibility (sway, surge and yaw) is depending on the stiffness of the vertical tethers and the bridge superstucture in the horizontal plane. The construction is made stiff enough laterally to secure that the movements related to wind, currents and waves are kept under control, but is flexible in order to ensure that temperature movements are let through.

Boknafjorden is the widest of the E39 fjord crossings, and the one that is most exposed to the wind and waves from the west. This crossing was chosen as the basecase for the study. The shortest distance is approximately 7.5 km in the area where it is applicable for E39 to go in the future.  At this location, the fjord is more than 500 meters deep without islets or shoals in the fjord.

The objective of the work carried out is to verify whether it was possible to create satisfying numerical models with existing computer programs to analyze this type of large floating bridges and to verify if the bridge concept in itself is feasible. Two numerical models have been established – one structural model with a refined beam/cable element mesh and one hydrodynamic model.
The feasibility study of the bridge consisted of an assessment of the transverse stability of the floating bridge, the dynamic response of the bridge and mooring system to environmental loads and the characteristic motion behavior. The geometrical details of the floating bridge, the process of defining and calibrating the numerical models, the analyses results and discussions and recommendations for future work are presented in this paper.

The length of each main span is 1420 meters and the four middle towers are supported on semi-submerged TLPs and moored to the bed of the fjord. The water depth at position of the TLPs varies from 450-550m The two side towers of the bridge near to the shore are traditional bridge towers fixed to the seabed, see *Figure 45*.

The bridge deck consists of an orthotropic steel box similar to the recently built Hardanger bridge. Each of the TLPs has a height of 288 meter, including a draft of 68.3 meter. All TLPs are constructed of steel.

*Figure 42: The suspension bridge placed on a Tension Leg Platform*

*Figure 43: Orthotropic steel box in deck*

*Figure 44: The 5 span suspension bridge crossing the Boknafjorden.*

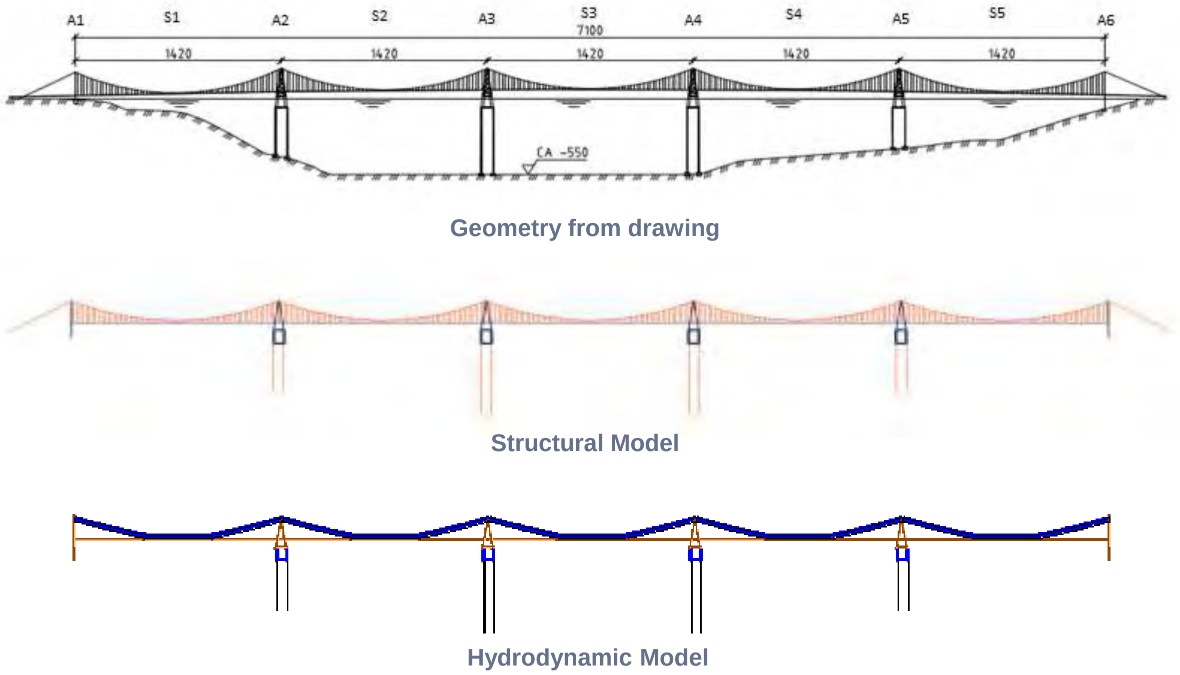


*Figure 45: Overview of bridge drawing and the two nummerical models.*

## STRUCTURAL MODEL

A full 3D structural numerical model has been established in analysis software RM BRIDGE [11] based on beam theory, covering non-linear effects as cable element formulation (according to catenary theory), geometric stiffness and large deformations. The bridge deck and TLPs are modeled using beam elements. Suspension and hanger cables as well as the tension legs are modeled using cable elements. All elements have been assigned correct geometric properties which are the basis for calculation of the corresponding numerical properties. The total buoyancy is 130% of the total structural weight, in order to obtain the necessary tension in the tethers. The buoyancy is applied upwards at the bottom of the TLP. Buoyancy and structural weight are labeled as permanent loads.

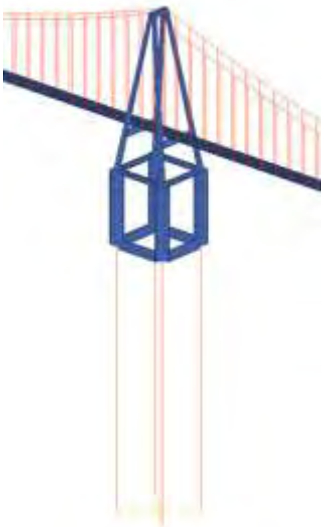

*Figure 46: Model of a TLP.*

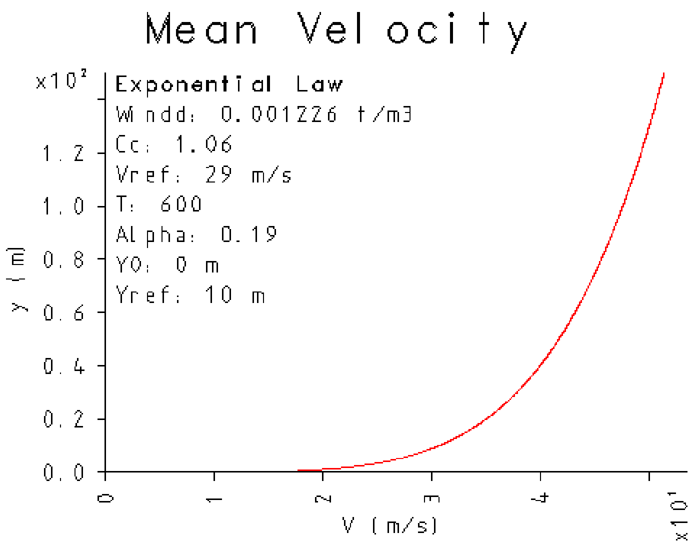


*Figure 47: Wind profile data generated within RM BRIDGE.*

Initially, the correct loaded geometry of the suspension bridges are based on the permanent loads established by shortening the main and hanger cable system using an optimization tool within RM BRIDGE to obtain correct stress-free cable lengths. The permanent loads together with the response due to environmental load or traffic load are included to set up the total correct geometric stiffness.

In the current feasibility study the structural model has been used for following studies:
- Static lateral wind to estimate the maximum lateral deformation, see *Figure 47*.
- Static traffic load using one unfavourable asymmetric load position, see *Figure 48*.
- Eigenvalue analysis.

The traffic load taken from [4] is applied simplified by adding the distributed part (3.0 kN/$m^2$) into the 6 lanes, totally 54.0 kN per m bridge. This distributed load is applied to the CoG of the bridge deck.

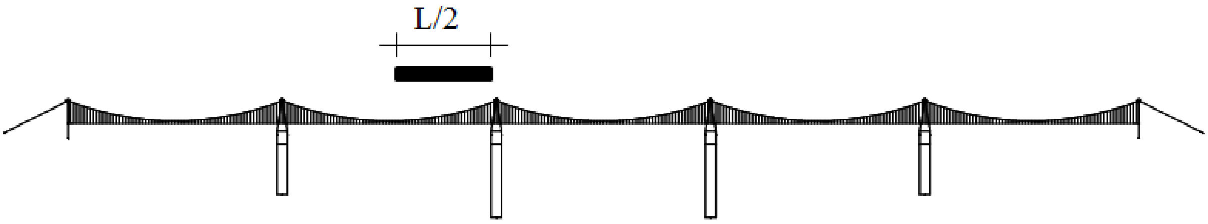


*Figure 48: Asymmetric position of the traffic load in the second span according to [4].*

The main intension of the structural model in this initial stage of the feasibility study is to provide the stiffness and the mass distribution for input into the hydrodynamic model.

## HYDRODYNAMIC MODEL

The hydrodynamic model consists of rigid structural components connected with flexible joints representing the stiffness of the component. The hydrodynamic loads on the submerged part of the TLPs are taken care of by a panel model combined with a Morison formulation.

### Software
The hydrodynamic model has been made in ANSYS AQWA Suite [10] with Coupled Cable Dynamics. AQWA uses a 3 dimensional diffraction radiation panel method. The mean wave drift forces are calculated using Newman 1967 [8] while the time varying components are taken care of by the Quadratic Transfer Functions, Pinkster 1980 [9]. Both sum and difference frequencies are taken into account. Effects of viscous drag and added mass on slender members are calculated using Morison loading.

### Panel model
The submerged part of each TLP is discretized by 3821 panels, the rest of the TLP is modeled by tubular elements which do not contribute to the diffraction-radiation calculation. The wall thickness and density of the tubular members are defined such that the generated total mass and mass distribution are according to the specified weight budget. The panel model is illustrated in *Figure 49*. Buoyancy is directly calculated from the panel model.

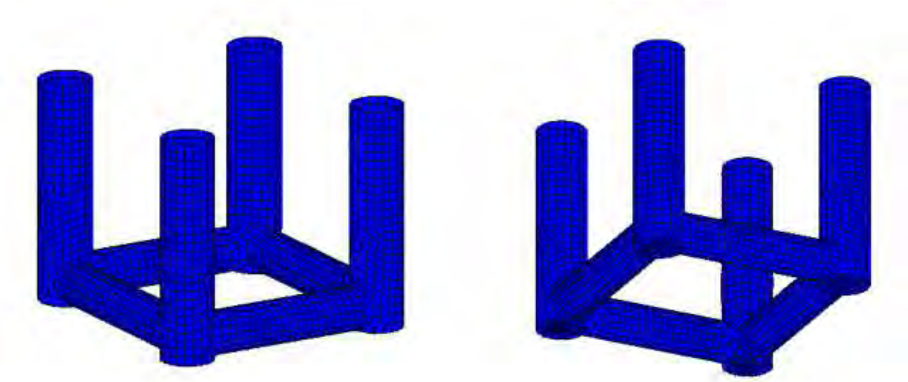

*Figure 49: Panel model of the sub-merged part of the TLP.*

**Viscous drag**

The contribution of viscous drag on the submerged part of the TLPs has been included with Morison elements. The hydrodynamic added mass term in the Morison equation has been defined as a negligible value since the added mass contribution is handled directly in the panel model. The tension legs also act as Morison elements, where both the drag and added mass contribution are taken into account. Drag and added mass coefficients have been defined according to DNV-RP-C205, ref [5].

**Bridge Deck and Suspension Cables**

Since all structural members are regarded as fully ridged in ANSYS AQWA, the effects of structural stiffness need to be taken into account by discretizing the bridge deck into sections, connected by hinges. The structural stiffness of each section is included as rotational stiffness in each hinge and based on the structural analysis model.

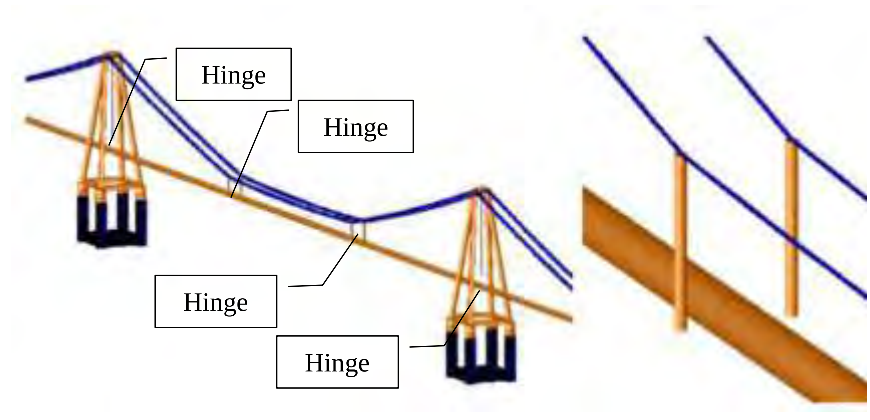


*Figure 50: Illustration of the hydrodynamic model*

The suspension cables are modeled as catenaries with the correct mass per unit length and axial stiffness. The hangers are discretized into 2x2 vertical supports per span and a vertical cable at each TLP. The vertical supports between the suspension cables and bridge deck can only transfer axial forces, they are free to rotate around all axis and can therefore not transfer moments. The model is illustrated in Figure 9. The influence of structural damping has been taken into account in a simplified way assuming that the damping is frequency independent. The damping is defined as 0.7% of the critical damping of the first eigen mode in transverse direction of one span. This damping value is derived from studies of the Hardanger bridge.

**Tension legs**

The tension legs of the TLPs are included as slender beams. Moments, transverse motions and stresses are calculated at each time step. The overview of the model is depicted in *Figure 45*.

# VALIDATION

The main purpose of the hydrodynamic analysis model is to represent the global motion behavior of the bridge in a correct way. The more local structural response was not considered to be important in this first phase of the project. In order to verify if the global structural response of the hydrodynamic model is suitable for the intended global motion analysis, a comparison study has been made between the structural and the hydrodynamic model. The structural response of the hydrodynamic model has been validated in a static and a dynamic way.

## Static verification

Static loads have been applied to the bridge in transverse direction. The loads are applied as line loads in the structural model and discretized loads at the CoG of the sections in the hydrodynamic model. The deflections of both models in transverse direction have been compared in order to assess the transverse stiffness, see *Figure 51* and *Figure 52*.

The behavior of the bridge is depicted below for the smallest transverse load and the largest transverse load. The abscissa axis represents the longitudinal direction of the bridge and the ordinate axis the transverse deformation. The response of the structural model is depicted in red and the blue line represents the hydrodynamic model. It can be seen that the bridge deck is deflecting in transverse direction between the TLPs because of the applied transverse loads. The transverse deflections of the TLPs are very similar in both models. The transverse motions of the bridge deck and suspension cables are smaller in the hydrodynamic model than in the structural model. This implies that the discretized bridge deck and cable system is stiffer in the hydrodynamic model.

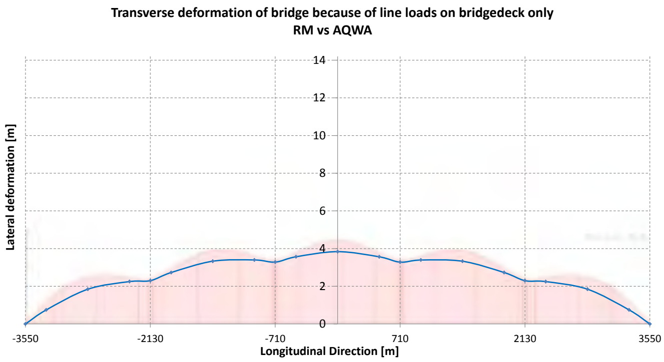


*Figure 51: Line loads on bridge deck only*

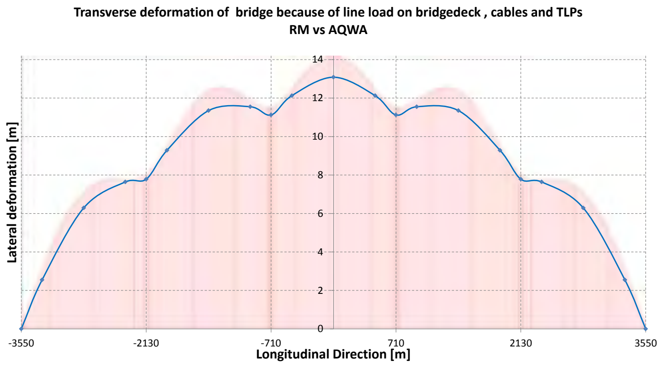


*Figure 52: Line loads on the complete model*

## Dynamic validation

The dynamic response of the two models has been checked by a natural frequency analysis of the first four eigenmodes in the horizontal plane. The validated eigenmodes are depicted in *Figure 53* to *Figure 56* below.

*Figure 53: Eigenmode no. 1*

*Figure 54: Eigenmode no. 2*

*Figure 55: Eigenmode no. 3*

*Figure 56: Eigenmode no. 4*

The comparison of the natural frequencies is presented in the table below.

| **Mode** | **Structural model** | **Hydrodynamic model** | **Difference** | |
|---|---|---|---|---|
| [-] | [s] | [s] | [%] | [s] |
| 1 | 83.3 | 83.5 | 0 % | -0.2 |
| 2 | 68.0 | 67.3 | 1 % | 0.7 |
| 3 | 57.8 | 55.2 | 4 % | 2.6 |
| 4 | 54.3 | 49.4 | 9 % | 4.9 |

From the dynamic validation, it can be seen that the hydrodynamic model is stiffer for the higher eigenmodes. In these modes, the structural response of the bridge deck and the cable system become more and more important. The behavior is therefore similar to the behavior observed in the static analyses.

## ENVIRONMENTAL LOADS

As described in the introduction, location of the bridge in the Boknafjord is directly exposed to the offshore environment. So both ocean wind and wave loadings are taken into account for the analysis.

### Waves

3 wave spectra have been considered at this analysis stage. 2 wave spectra have been based on recommendations from DNV-RP-C205 [5] and one spectrum on data supplied by the Norwegian Meteorological Institute. The spectra are described in the table below,

| **Spectrum** | **1** | **2** | **3** | **Unit** |
|---|---|---|---|---|
| Type | Jonswap | Jonswap | Torsethaugen | [-] |
| $H_S$ | 12 | 12 | 8.2 | [m] |
| $T_P$ | 12.2 | 15.9 | 13.4 | [s] |
| g | 3.3 | 3.3 | - | [-] |

Spectrum 3 is a double peak spectrum representing contribution from both the wind driven and swell driven wave system. It is expected that especially the contributions of waves with larger periods will be important for the response of the bridge due to the low natural frequencies. Wave spectrum 3 is considered to be the most realistic of the analyzed wave systems and will be used for presentation of the results throughout the rest of this paper.

**Wind**
Time varying irregular wind has been included in the analyses with aim to analyze the response of the bridge to the coupled effect of irregular wind and waves. The included wind spectrum is the Ochi&Shin wind spectrum which is specifically meant for offshore environments. There is much energy present at the low frequencies in this spectrum as there is little disturbance of the wind in the offshore conditions. The wind spectrum has a mean wind speed of 29 m/s at a reference height of 10 meters, see *Figure 47*.

**Current**
In addition a current strength of 0.2 m/s has been included into the analyses, the current is assumed to act in the same direction as the wave system.

## FEASABILITY STUDY

The feasibility study consists of analyses in both the structural analysis model and the hydrodynamic analysis model. In the structural model, responses due to asymmetric traffic loads and due to lateral static wind have been carried out. In the hydrodynamic model, fully coupled time domain calculations have been carried out in order to assess the motion behavior of the bridge. The results are presented in this section.

### Asymmetric traffic loads

*Figure 57: Vertical deformation due to asymmetric traffic load.*

The maximum calculated vertical deformation due to asymmetric traffic load at one span is 15.7m. The maximum longitudinal tilt angle is 0.046 degrees and occurs in axis A3. The tension legs have a significant stabilization effect agains tilting.

### Lateral static wind
The maximum calculated lateral deformation due to static wind, ref. *Figure 47*, is 33.9m.

### Time domain calculations
The analyses have a duration of 1 hour in order to limit the analysis time at this phase of the project. The magnitude of the presented results should therefore be interpreted with care. The typical motion behavior and frequency domain presentation of the results are however considered to be correct.
The investigated results focus on an assessment of the transverse stability of the floating bridge, the dynamic response of the bridge and mooring system to environmental loads and the characteristic motion behavior. Wave spectrum 3 will be used for the presentation of the results in this section.
From the analyses it is seen that the TLPs provide a stable basis for the bridge. Relatively small motions are observed for the TLPs, both the transverse translations and the rotations. In order to illustrate this, results of the time domain calculations are presented in *Table 11* to *Table 13*. See *Figure 45* for the positions of the vertical axis.

*Table 11: TLP - motions in transverse direction due to spectrum 3*

| Vertical Axis | Max | Min | Average | Unit |
|---|---|---|---|---|
| A2 | 14.34 | 2.49 | 7.94 | m |
| A3 | 19.49 | 3.56 | 11.65 | m |
| A4 | 19.50 | 3.57 | 11.66 | m |
| A5 | 14.34 | 2.50 | 7.94 | m |

*Table 12: TLP - Rotations about longitudinal axis due to spectrum 3*

| Vertical Axis | Max | Min | Average | Unit |
|---|---|---|---|---|
| A2 | 0.2 | -0.3 | 0.0 | º |
| A3 | 0.4 | -0.9 | -0.1 | º |
| A4 | 0.4 | -0.9 | -0.1 | º |
| A5 | 0.2 | -0.3 | 0.0 | º |

*Table 13: Bridge Deck - motions in transverse direction at center of span due to spectrum 3*

| Center of span | Max | Min | Average | Unit |
|---|---|---|---|---|
| 1 | 12.56 | 0.0 | 5.51 | m |
| 2 | 22.25 | 1.51 | 11.12 | m |
| 3 | 24.04 | 3.05 | 13.05 | m |
| 4 | 22.27 | 1.56 | 11.14 | m |
| 5 | 12.57 | 0.0 | 5.49 | m |

It can be seen that the TLPs show only small rotations about the longitudinal axis of the bridge. The transverse motions of the bridge are also considered to be relatively small compared to the total length of the bridge.

In order to get a better understanding of the motion behavior of the TLPs and the bridge deck, Power Spectral Density (PSD) plots have been made from the time signals. In Figure 58 and *Figure 59* the PSD plots of the Wave and Wind Spectra are compared to the PSD plot of the transverse motion of the TLPs at bridge axis A4 and A5, see *Figure 45*.
From these plots, it can be seen that the transverse motions of the TLPs show a strong low frequency response and a weaker wave frequency response. The low frequency response is very probable dominated by the excitation due to the wind spectrum and partly due to slowly varying wave drift forces.
There is a clear difference between the response of the TLPs at axis A4 and A5 with respect to the amount of energy present in the low frequency response. Axis A4 is positioned more in the middle of the bridge; the Tension Legs are therefore longer and the floater is further away from the bottom fixed towers at the ends of the bridge. This TLP therefore experiences less stiffness in transverse direction. The response to the first order wave excitation is however very similar.

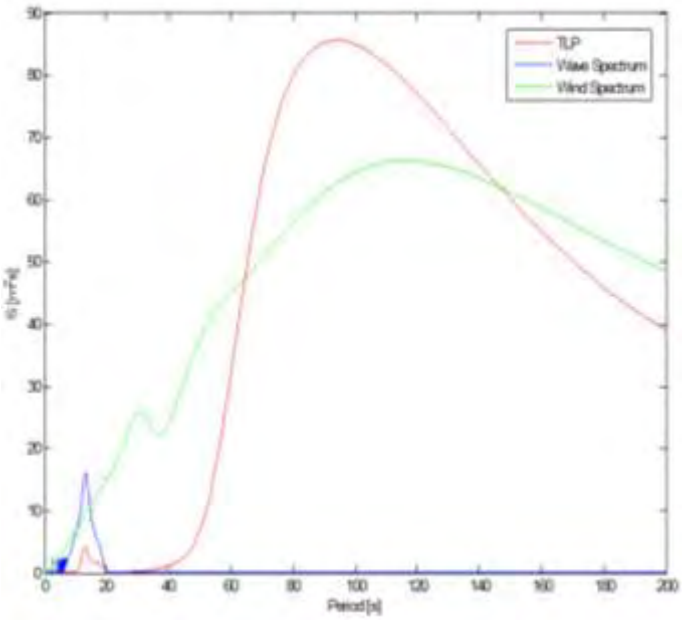


*Figure 58: PSD at axis 4*

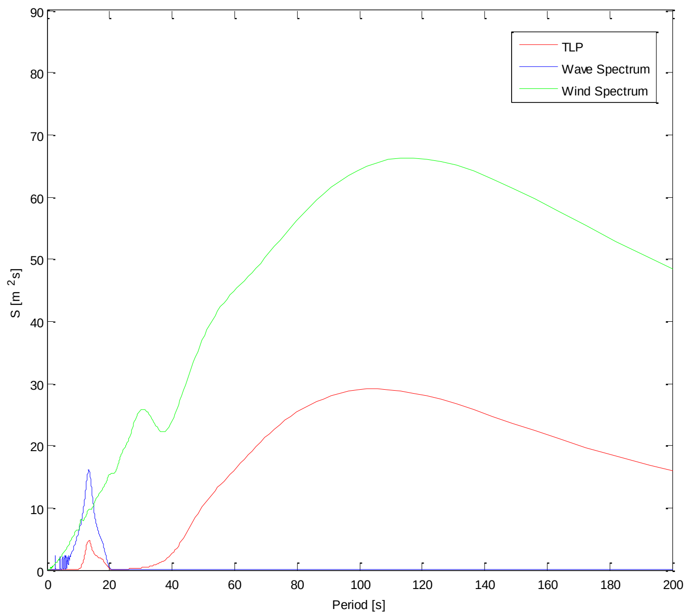


*Figure 59: PSD at axis 5*

A similar trend can be observed in *Figure 60* and *Figure 61*. These figures represent the PSD plots of the transverse motions of the bridge deck at the middle of span 3 and 5 - see *Figure 45* - respectively. The low frequency response -but also the first order wave response- is stronger at the middle of the bridge than at the sides. Remarkable is however that the first order wave response is strongly contributing to the overall motion behavior. At span 5 the peak value of the first order wave response is even larger than the response due to wind forces.

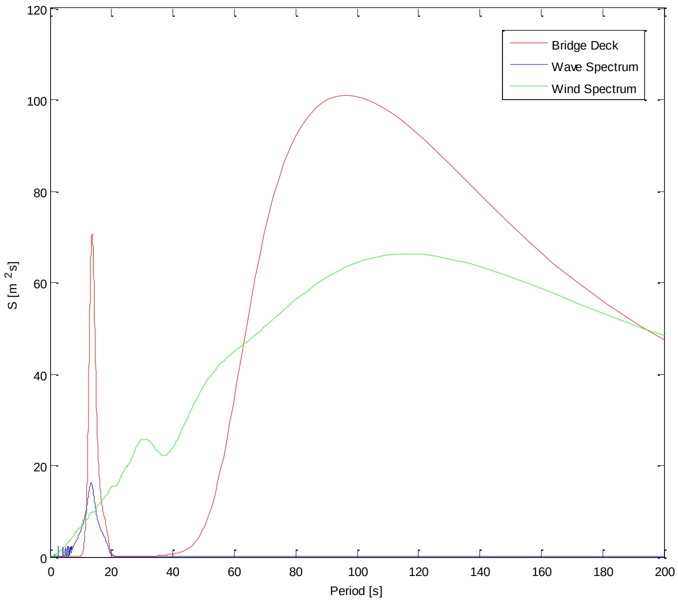


*Figure 60: PSD at middle span 3*

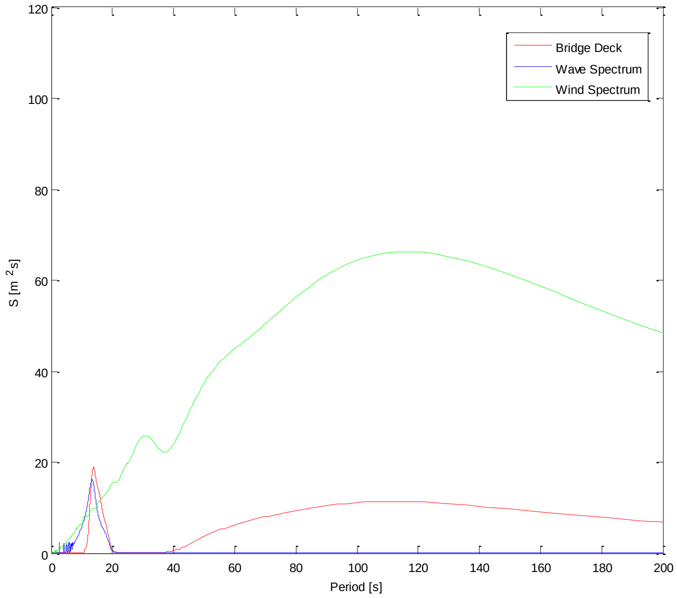


*Figure 61: PSD at middle span 5*

It seems therefore that the response of the bridge deck in transverse direction is an important mechanism in the overall motion behavior. When evaluating the response in the tension legs connecting the TLPs to the seabed, it can be seen that the response is strongly dominated by the first order wave response. This response is of course initiated by direct excitation of the TLP by the wave system, but also strongly by the swinging motion of the bridge deck and cable system in transverse direction. It is observed that the combination of these two principles often leads very low tension in the leeward tension legs, with the risk of total loss of axial tension. This especially happens for the TLPs in axis A3 and A4. This is illustrated in *Figure 62* and *Figure 63*.

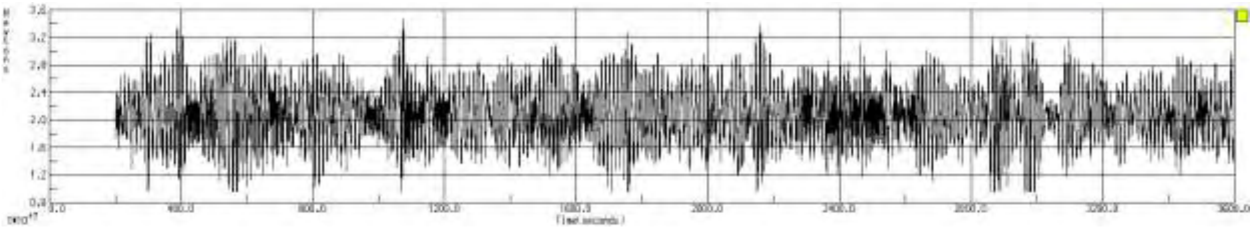

*Figure 62: Tension Leg Tension - Axis 4 - Windward Side*

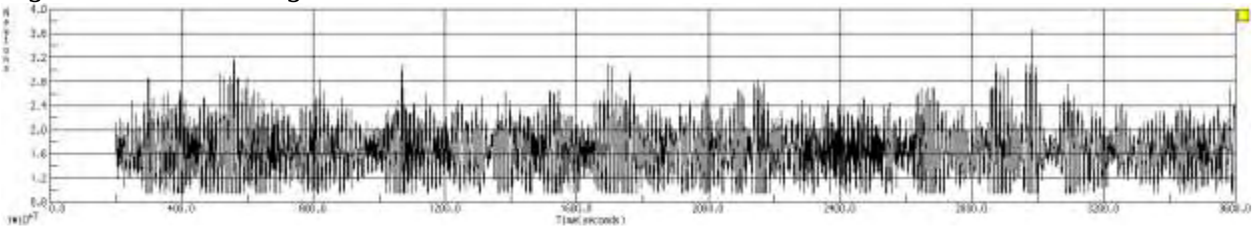

*Figure 63: Tension Leg Tension - Axis 5 - Leeward Side*

Frequency analysis of the tension in the tension legs shows that the response is strongly dominated by the frequencies corresponding to first order wave excitation. Even though there seems to be risk for a loss in tension it is not observed that this would lead to collapse in the analyses, in fact the TLP motions are very limited as can be seen in *Table 11* and *Table 12*. Slack tension legs are however a serious risk for the mooring system and need further consideration.

The tension in the tension legs and therefore also the stability of the bridge in about the longitudinal axis is strongly influenced by the transverse motion of the bridge deck. This influence seems to be stronger than expected on forehand. As earlier indicated in this paper, the aim with the hydrodynamic model was to represent the global motion behavior in a satisfying way. The stiffness of the global bridge is therefore validated with the structural model, but the more local modes as the transverse motion of the bridge deck were not the main focus. It is observed from the validation cases that the bridge deck and cable system in the hydrodynamic model are stiffer in transverse direction than in the structural model. The natural periods for this specific mode are:

- Hydrodynamic model 13.4 seconds
- Structural model 16.9 seconds

It can be seen that the hydrodynamic model is stiffer than the structural model in this important mode. This mode furthermore coincides with the expected peak periods of the wave spectrum. This is probably one of the reasons that a relatively strong response of this mode is observed in the analyses.

## CONCLUSIONS AND FUTURE WORK

A feasibility study of a TLP supported suspension bridge has been carried out by combining both a hydrodynamic and a structural model providing understanding of the responses of large floating bridges. The concept of a TLP supported suspension bridge seems, from the carried out analyses, feasible. The TLPs provide a very stable platform for the bridge deck and suspension cables and relatively small motions are observed for a water depth of approximately 500 meters. Important, however, will be to consider the response of the bridge deck and cable system more closely. In this specific analysis the eigenmode of the bridge deck and cable system in transverse

direction turns out to be important for the response of the floating bridge. It can therefore be beneficial to explore the possibilities to import the wave excitation from the hydrodynamic model to the structural model in order to be able to represent more local structural responses more accurately.
Besides the need to more accurately model this specific motion behavior, it should also be considered how to increase the natural period of this specific mode so that it is further away from the wave excitation period.

## ACKNOWLEDGEMENTS

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# THE NON-LINEAR DYNAMIC RESPONSE OF SUBMERGED FLOATING TUNNELS TO EARTHQUAKE AND SEAQUAKE EXCITATION

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## ABSTRACT

*The dynamic behavior of Submerged Floating Tunnels (SFTs or Archimedes Bridges) has caught the interest of this research group, in the past decade, with reference to different sources of excitation, namely earthquakes, sea waves and currents. Both structural and excitation models have been developed to the aim of capturing the essential features of the dynamic behavior of the SFTs under complex excitation mechanisms, i.e. 3D multiple-support seismic excitation or vortex induced vibration.*

*With reference to seismic actions, special attention has been devoted, in recent times, to some aspects of structural and excitation models; first of all, it had been noted, in previous studies, that anchoring bars which are normally located at the tunnel ends usually undergo large stress concentration due to the oscillations of the tunnel in the transversal plane. These stresses can largely exceed the yield value: this aspect, if properly controlled, can be regarded as an option for dissipating part of the energy transferred to the structure by the seismic motion. In this light, inelastic behavior of the anchoring elements, which are slender bars with hollow section in the proposed design approach, has been recently introduced in the dynamical modeling of the SFT, allowing for a detailed study of the inelastic deformations imposed by extreme seismic actions.*
*As a second improvement in the modeling approach, the hydrodynamic pressure due to the water-transmitted seismic vibration from the seabed (named as seaquake) is introduced in addition to the to the motion of the structural supports. The hydrodynamic excitation is derived from the velocity potential deduced from wave equation. Morison equation is then adopted to simulate the seaquake force on the moving tunnel body. The first results show how the seaquake mechanism can lead to a significant increase in the internal forces both in the tunnel and in the anchor bars, the effectiveness of the control system in the mitigation of dangerous structural responses and highlight the need of further investigations.*

**Keywords:** *offshore structures, submerged floating tunnels, nonlinear behavior, multiple-support seismic excitation, hydrodynamic excitation (seaquake)*

## INTRODUCTION

SFTs (Submerged Floating Tunnels), also known as Archimedes Bridges, serve as a promising alternative to cross sea-straits, lakes and waterways in general [1]. Compared to the traditional immersed tunnels, SFTs can take advantage of the water buoyancy; in addition, a SFT can be installed in very deep waters, where other crossing solutions cannot be applied. In the last decades, attention has been devoted to SFTs by researchers and engineers of many countries around the world, such as Norway, The Netherlands, Italy, China and Japan. This was also due to a prize-winning proposal for a 5.3 km SFT between Calabria and Sicily across the 350 m deep Messina strait in Italy [2].

In past research some fields were covered by this and other groups [2-4] to give a thorough view on the structural behavior of SFTs under dynamic loadings, such as seismic excitation and hydrodynamic loads due to waves and vortex-induced excitation [4,5]. In this paper the latest development in the research are presented, with reference to an application example; these developments are focused on a more refined modeling of both the structural system, whose inelastic behavior is, for the first time, fully accounted for, and the seismic excitation, which is introduced also in terms of hydrodynamic effects.

## MODELLING ISSUES IN THE SEISMIC ANALYSIS OF SFTs

In this paper, some issues related to the structural and load modeling for SFTs under strong seismic motion are addressed.

### The example SFT and its structural modeling

A 3D finite element (FE) model of the SFT, as proposed for the Messina Strait crossing, has been developed within the ANSYS software environment [6]. Figure 1 shows the SFT's front and section views. The tunnel has a circular composite steel-concrete section and is connected to the seabed by means of slender steel beams having hollow circular section and of pile foundations. The dead load of the tunnel (per length) is 1200 kN/m. Live loads and accidental loads are responsible for a 50% increase. The net buoyancy, equal to the buoyancy minus the tunnel weight, is 455 kN/m. The SFT is designed according to the concept that no water ingress in the tunnel is allowed, so that no partial-flooding loading condition is assumed.
In Tables 1 and 2, the geometrical and mechanical properties of the tunnel and anchor bars sections are listed. In past research work, the anchoring system (Fig. 1b) was modeled by two equivalent bars having the same total area and moment of inertia [2,3] of the two actual bars. In this model, the anchoring system is simulated, in a more realistic fashion, by means of two bars for each side.

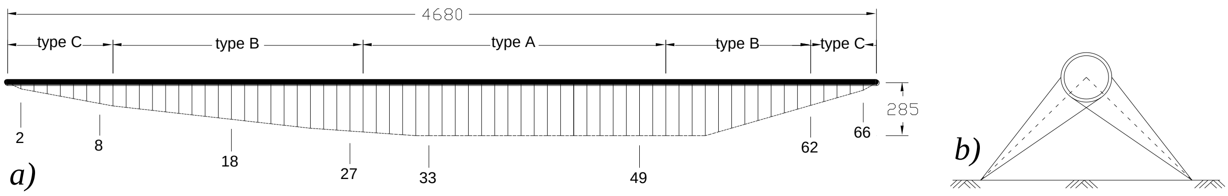



*Figure 1 – SFT, Front View (a), Section View (b)*

*Table 14 – Geometrical and Mechanical Characteristics of Anchoring Bars*

| Section A | $R_{out}$(m) | $R_{in}$(m) | Area($m^2$) | I($m^4$) |
|---|---|---|---|---|
| A | 0.933 | 0.871 | 0.348 | 0.139 |
| B | 0.975 | 0.91 | 0.385 | 0.171 |
| C | 1.029 | 0.961 | 0.425 | 0.211 |

*$R_{out}$ is the out radius, $R_{in}$ is the inner radius, and I is the initial moment.

*Table 2 – Geometrical and Mechanical Characteristics of Tunnel*

| $D_{out}$(m) | $D_{in}$(m) | Area($m^2$) | I($m^4$) | W(kN/m) |
|---|---|---|---|---|
| 15.95 | 13.95 | 58.24 | 16370 | 1200 |

*$D_{out}$ is the out diameter, $D_{in}$ is the inner diameter, and W is the weight of the tunnel.

The model, depicted in Figure 2, accounts for soil-structure interaction, geometrical and material non-linearities as well as 3D multiple-support seismic excitation applied at anchor foundation points and tunnel end supports. Linear mechanical behavior for soil is assumed. Soil-structure interaction is represented by three elastic springs and dashpots, located at each end of the anchor bars and at the shore abutments. The stiffness coefficient K and damping coefficient C of these elements are given in Table 3. In transient analyses, Rayleigh damping has been selected at this initial stage; the introduction of a transverse non linear hydrodynamic damping force has been tested, showing negligible effect.

*Table 3 – Coefficients of the Soil-Structure Interaction: anchor bars and shore*

| Direction | K (kN/m) | C (kNs/m) |
|---|---|---|
| Horizontal | 2.87e6 | 2.66e4 |
| Vertical | 1.72e7 | 1.14e5 |

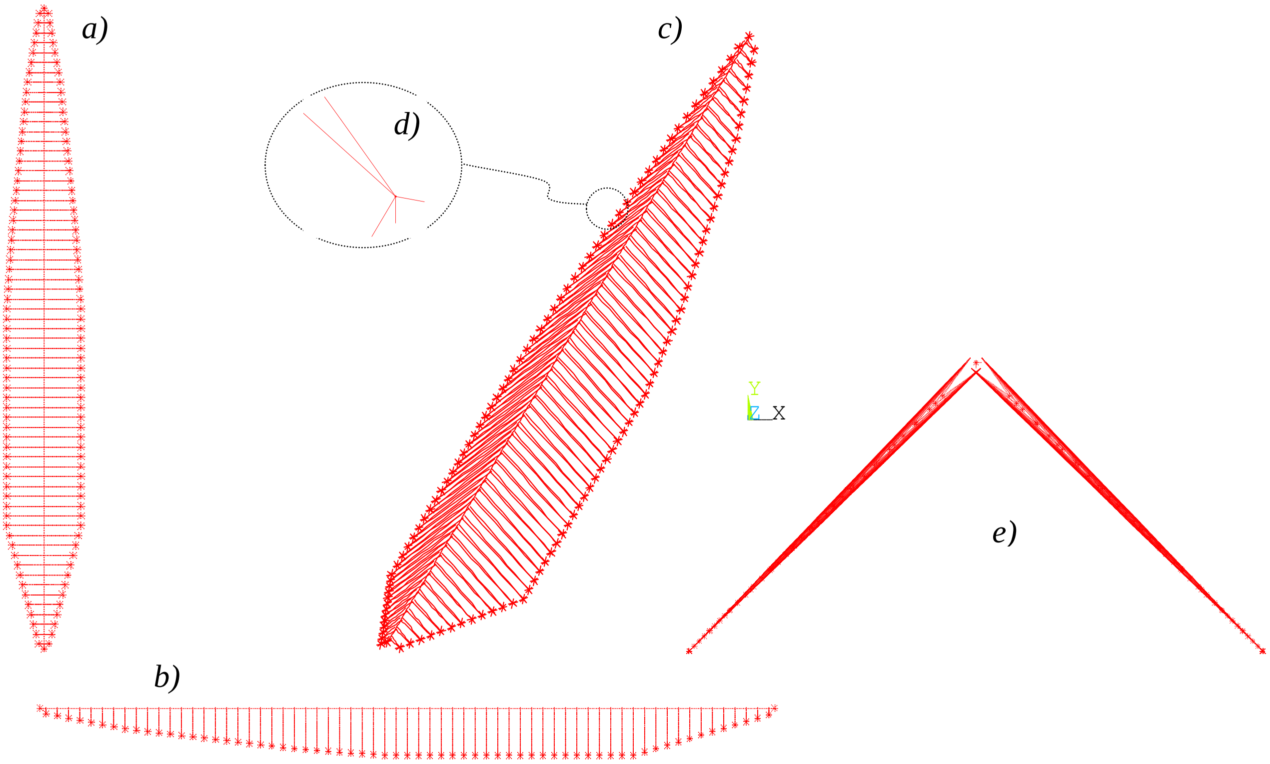



*Figure 2 - 3-D model in ANSYS with different views*
*(a) bottom view, (b) front view, (c) oblique view, (d) SSI detail, (e) side view*

Non-linear material behavior has been introduced for the first time [6] both for the anchoring bars and for the longitudinal end-restraint system. The comparison of the SFT behavior have been made considering both elastic and inelastic material for the anchor bars. These two options have been identified in the remaining of the paper as ‘elastic’ and ‘inelastic’. The steel elastic modulus is fixed to 2.06e5 MPa, the yielding stress to 2.10e8 N/m$^2$, and, when the inelastic behavior is implemented, the ratio between the post- and the pre-yielding stiffness to 0.02.
The shore connections must satisfy two functional requirements: first they must ensure water tightness to the joint between the floating tunnel and the onshore approaching tunnel. In structural term, the joint should be designed to allow free longitudinal expansion and contraction to accommodate relative displacements between the two shores, with no vertical and transversal relative movements. The main problem for the shore connection is represented by longitudinal

seismic actions; in fact, if one end is left axially free, large forces are imposed to the fixed end, also resulting in high axial forces in the tunnel section, which can be harmful for water tightness. In this work, elastic-plastic devices have been introduced as longitudinal end-restraint elements, both for limiting the forces transmitted and for dissipating some of the kinetic energy developed in the tunnel. In the configuration here considered, both ends are equipped with the devices; at one end the elastic-plastic element will be put in series with a shock-absorber, allowing slow axial movements but behaving as a rigid link when seismic actions occur.
The stiffness of the devices is derived from the yielding displacement and yielding force which are fixed respectively to 10 cm and 5% of the tunnel weight.

## Modal shapes and frequencies

In Table 4 the natural frequencies of the first six normal modes of the linearized 3D model are given. Figure 3 depicts a mode shape of SFT 3D model.

*Table 4 – Natural frequencies of the linearized 3-D model*

| Modes | Description | Natural frequencies (Hz) |
|---|---|---|
| 1) | Tunnel first longitudinal vibration mode | 0.196 |
| 2) | Tunnel second longitudinal vibration mode | 0.391 |
| 3) | Tunnel first transversal vibration mode | 0.580 |
| 4) | Tunnel first vertical vibration mode | 0.580 |
| 5) | Tunnel second transversal vibration mode | 0.585 |
| 6) | Tunnel second vertical vibration mode | 0.585 |

## Modelling of the Ground Motion

Because of the distance between the anchoring sections, the seismic input time histories should be different at each of them. The artificial ground motion is thus considered as a space-time stochastic model based on the well known Kanai-Tajimi power spectral density (PSD) modified by Ruiz and Penzien ($\omega_g = 10\,rad\,s^{-1}, \varsigma_g = 0.4, \omega_f = 1.0\,rad\,s^{-1}, \varsigma_f = 0.6$). The coherency model of Luco and Wong [6] is adopted for the spatial variation of the ground motion. A peak ground acceleration of 0.64 g is introduced according to previous studies on the seismic hazard in the Messina Strait. Artificial generation of the ground time histories was performed according to the numerical procedure proposed in [8]; an example accelerogram is given in Figure 4.

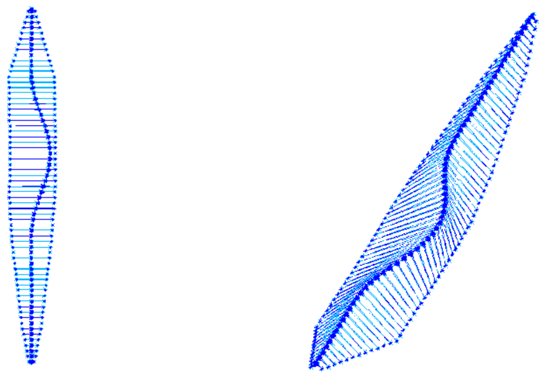


*Figure 3 – Mode shape 3, tunnel first transversal mode.*

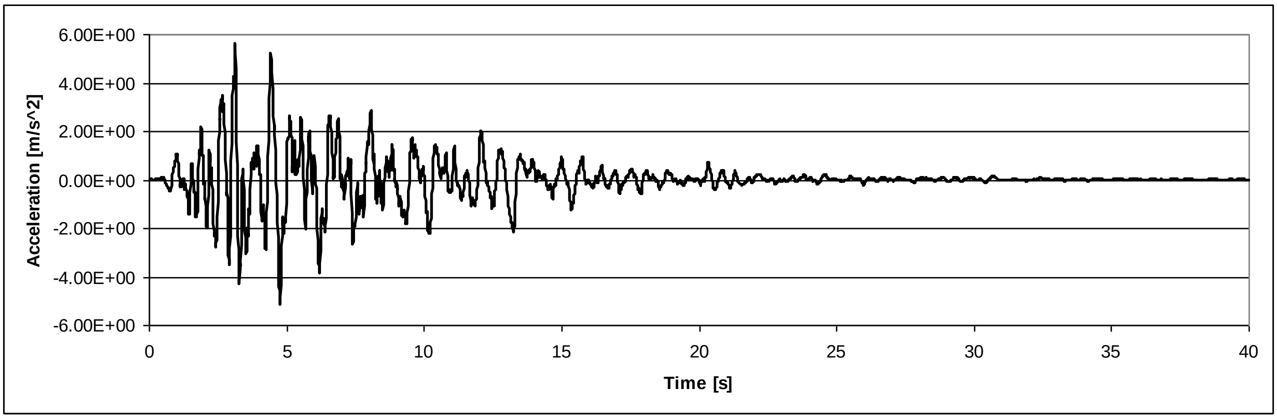



*Figure 4 – Example of artificial accelerogram*

**Modeling of the seaquake effect**

The so-called seaquake effect is due to the hydrodynamic pressure associated to the water vibration, at the depth of the SFT tunnel, induced from the vertical seabed motion.
From a literature review, there is limited research effort focusing on the response of SFTs subject to the seaquake forces [9,10], while more extensive studies have been devoted to the case of large floating structure [11]. On the basis of the latter paper development, the water motion is here derived by studying the vertical propagation of compression waves according to the following assumptions; 1) the hydrodynamic pressure related to incident waves is considered, 2) the sea water is inviscid, incompressible and irrotational, 3) the seaquake's velocity at the seabed is the same as the vertical ground velocity at this point without time lag.
Accordingly, the velocity potential is deduced from the 1-D wave equation [11] considering compatibility with the earthquake motion of the seabed as a boundary condition. The other boundary condition imposed is the free-surface one at a zero depth. From the potential the fluid particle vertical kinematics is than derived; subsequently, the force due to the seaquake on the moving SFT body in the oscillatory flow is modeled by using the classical Morison equation, upon computation of the relative velocity of the SFT with respect to the water.

**RESPONSE OF THE SFT BEHAVIOUR UNDER SEISMIC EXCITATION**

The results of the transient dynamic analyses are presented in the following sub-sections. The seismic input consists, at this initial stage, of a single realization characterized by three components which are applied to each anchor bar foundation and the tunnel end abutments.

**Anchor bars response**

Linear elastic and elastic-plastic behavior of the anchor bars is first compared. Figure 5 compares typical strain-stress relationship at the mid-section of two bars, one closest to the shore, the second one at the tunnel mid-span. The former shows high stress level when the linear characteristic is implemented, while the inelastic model shows large hysteretic cycles; compatibility with local available ductility has to be checked. Instead, the bar located at the middle of the tunnel shows much more limited ductility demand.

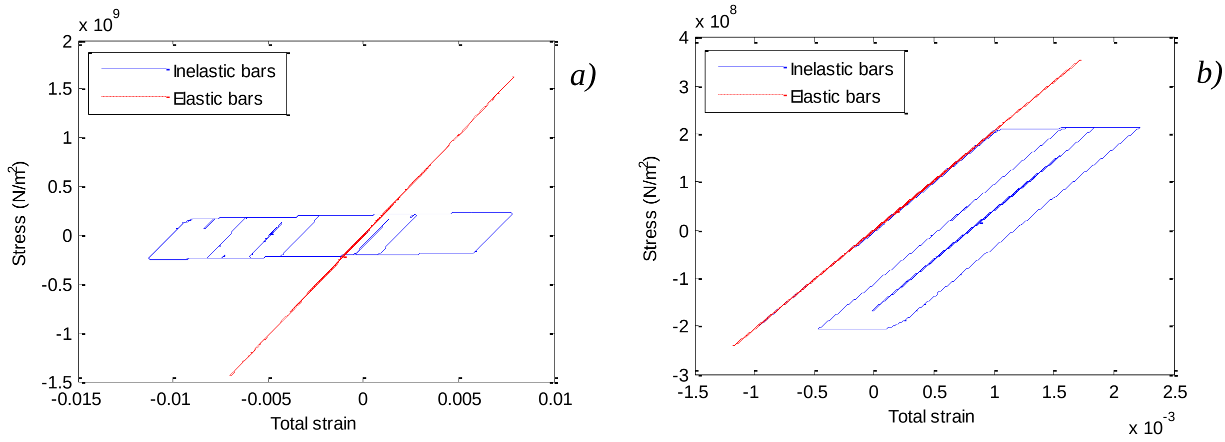



*Figure 5 – Typical strain-stress relationship for anchoring bars (a) bar close to the shore, (b) bar at tunnel mid-span.*

## Response of longitudinal restraint devices

Several fruitful applications of passive control systems, and their semi-active development also, to long span structures, such as cable-stayed and suspension bridges under seismic noise, seem promising when SFTs are considered [12-15].

The role played by the passive control system at the shores is twofold: first, to dissipate part of the seismic energy injected into the SFT structural system during the earthquake motion. The main result of this step is also to attempt to concentrate the inelastic deformation at specific and selected positions of the whole structure. The second aim of the control system is to decouple the structural motion from the shores one, shifting the structural response to lower frequencies and, consequently, far from the higher frequencies characterizing the earthquake excitation.

The expected consequence is to allow internal forces and stresses reductions at cost of larger structural displacements. Figure 6 reports the typical relative displacement-force path of the device at a tunnel end, showing reasonable ductility demand and a limited influence of the anchor bars behavior on the longitudinal motion of the tunnel.

## Tunnel response

Results in terms of vertical (y), longitudinal (z) and transversal (x) absolute displacements together with bending stress (top-position in the tunnel section) are herein reported. In figure 7 the displacement components at section 4 (from the left – see figure 1) are reported, showing the influence of inelastic behavior, especially in relation to the vertical motion, where a significant shift towards the sea bottom can be observed.

In figure 8 the time-histories of the stress on top of the tunnel section is shown, for section 1 (closest to the shore), section 4, quarter span and mid-span sections, often showing a mitigation of the tunnel stress due to yielding of the anchor bars. At section 4, however, the drift observed in figure 7a causes a large increase in the average stress, this being a phenomenon deserving careful attention in view of the tightness requirements. Note that a significant (downward) drift in the vertical tunnel displacement can be detected also in the elastic case [6], due to the geometrical effects resulting from large transverse oscillations of the anchor bars; inelastic behavior, however, can make the structure more prone to the phenomenon.

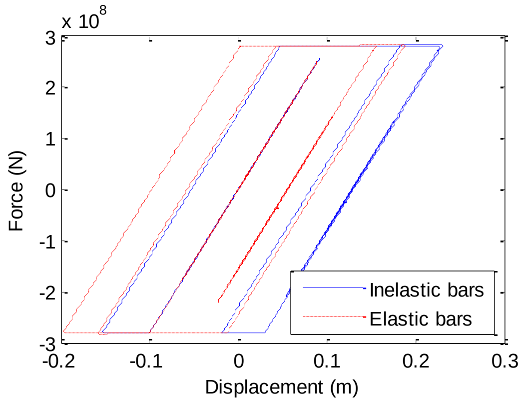



*Figure 6 – Typical non-linear behavior of elastic-plastic end restraints.*

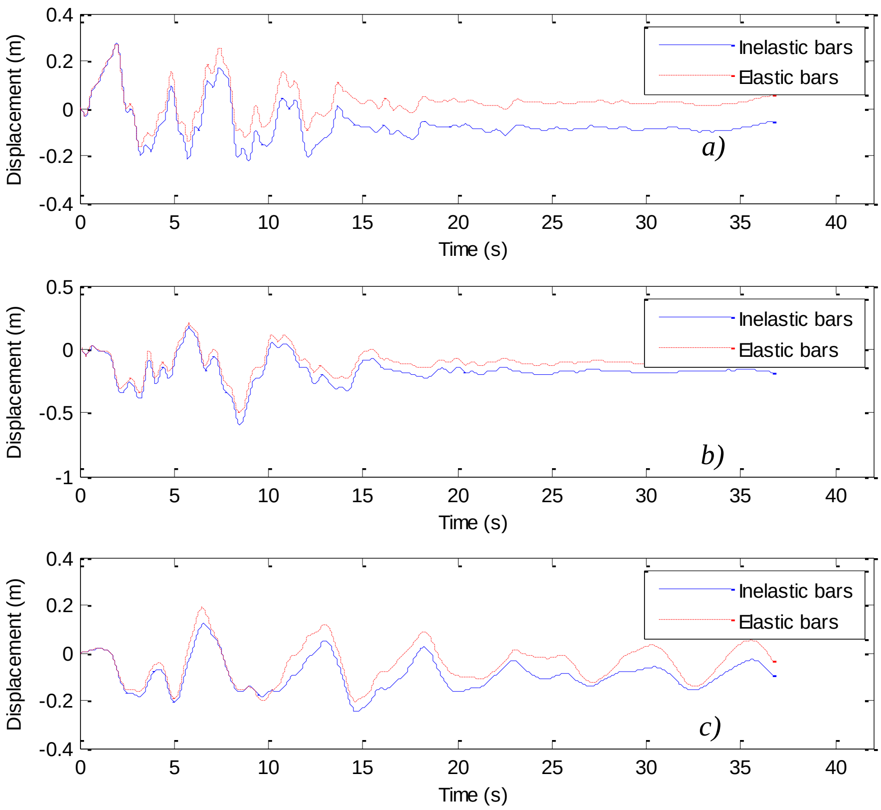
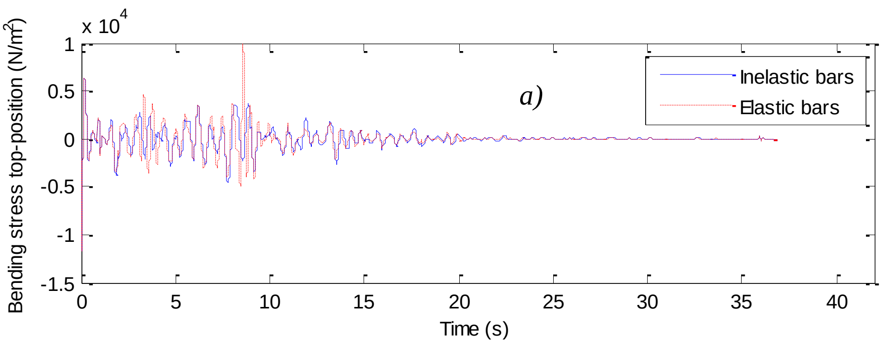



*Figure 7 – Tunnel response at section 4: (a) vertical displacement time-history, (b) transversal, (c) longitudinal.*



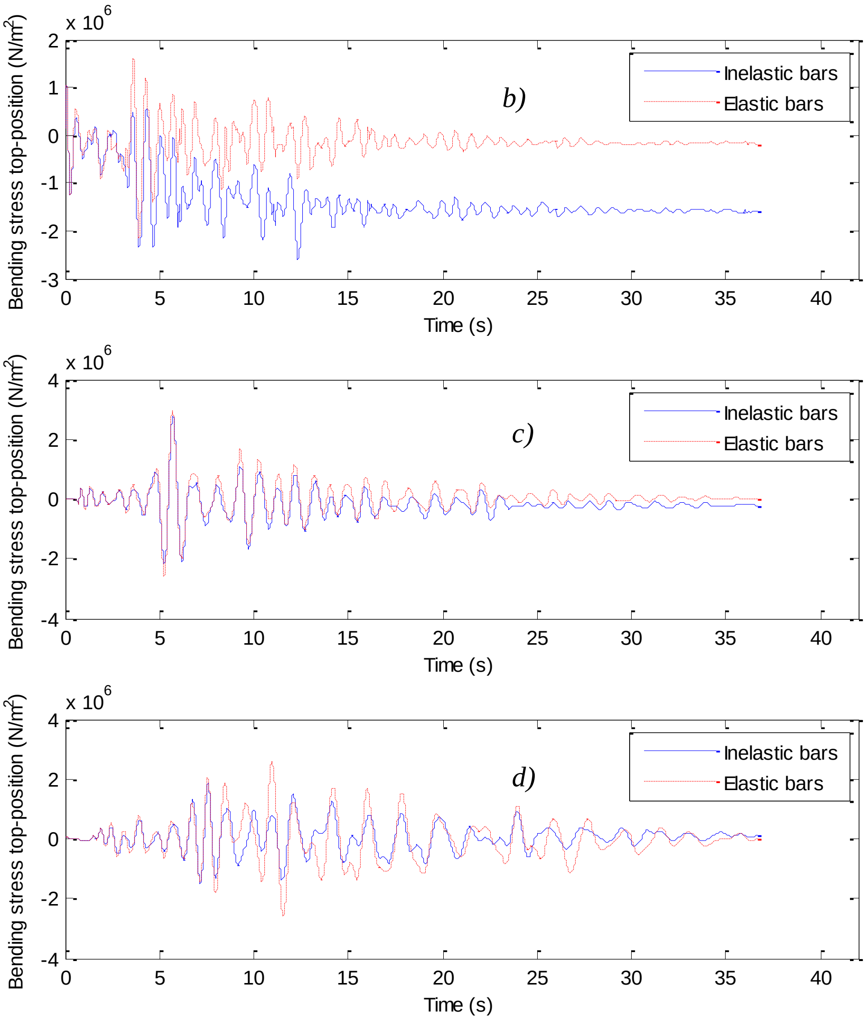
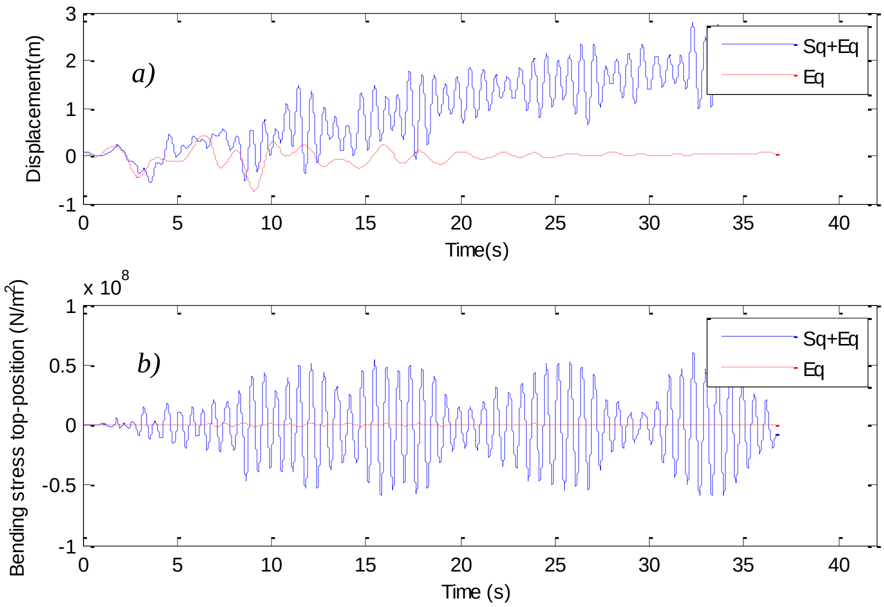



*Figure 8. Bending stress on top of the tunnel section. (a) closest to the shore, (b) section 4, (c) quarter span, (d) mid-span.*



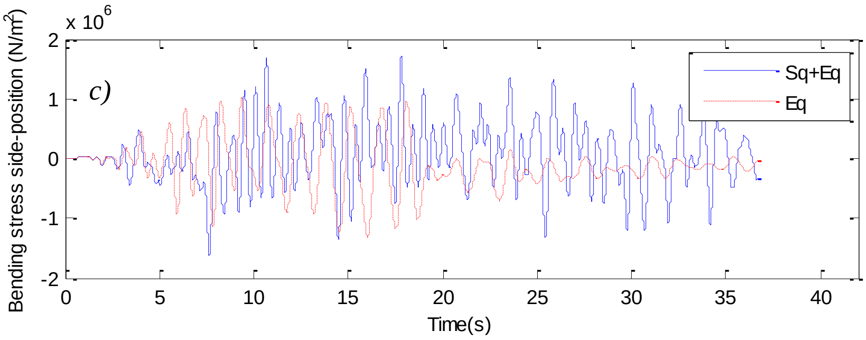



*Figure 9. Tunnel section at mid-span. (a) vertical displacement,(b) bending stress at top position,(c) bending stress at side position.*

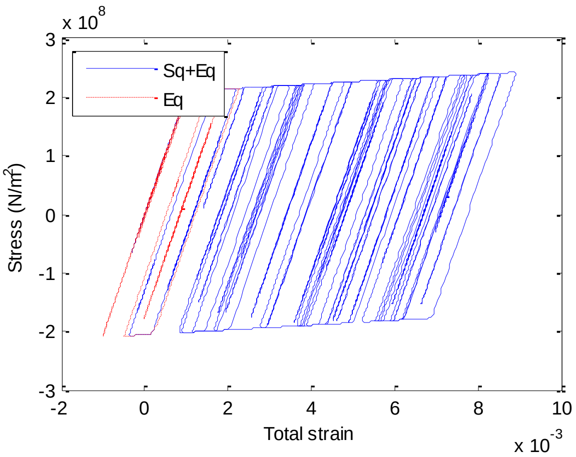



*Figure 10. Stress-strain behavior on top of the tunnel section at mid-span.*

Figures 9 and 10 are devoted to the illustration of the effect of the seaquake phenomenon; the results shown are somehow striking, with a dramatic increase in the vertical response and of the tunnel stresses, and deserve careful comment. In fact, it must be observed that the model here adopted is based on two conservative assumptions, namely on 1D wave propagation with no energy dissipation, which can lead to quasi-resonance phenomena in the "water column" below the tunnel. This can result in surprisingly high amplifications of the vertical response, with some effect also on the transverse motion (see stress on the side position on tunnel section figure 9c).

## CONCLUSIONS

In this paper a 3D model of a SFT has been developed to the aim of performing static, modal and transient seismic dynamic analyses. The model has been refined with respect to previous research work on the subject.
The results here shown points out two critical aspects of the SFT design and seismic response. First of all the tunnel sections close to the shore can undergo large amplification of stresses due to the constraint given by the end connection in vertical and transverse directions; the adoption of dissipative devices releasing this degree of constraint and dissipating some energy must be carefully considered. Secondly the seaquake effect, here introduced by means of a conservative but physically sound model, can significantly increase the vertical response of the tunnel in all sections: a model refinement seems to be necessary.

## ACKNOWLEDGMENT

This research work is dedicated to the memory of Elio Matacena, President of the "Ponte di Archimede International" company, who sadly passed away in August 2012. Without his enthusiastic and passionate support many of the research activities performed around the world on the Ponte di Archimede concept and applications would have been impossible.
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# WIND-INDUCED VIBRATIONS OF DRY INCLINED STAY CABLES IN THE CRITICAL REYNOLDS NUMBER RANGE

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## ABSTRACT

*Rapid development of long-span cable-stayed bridges in the past few decades has been followed by numerous incidents of stay cable vibrations. Large amplitude cable vibrations, of the order of several cable diameters, are facilitated by an inherently low structural damping and are attributed to various types of wind and parametric excitation. Adoption of effective means to suppress the vibrations rests on increasing the understanding of the excitation mechanisms. This paper addresses the aerodynamics of dry inclined/yawed cables in the critical Reynolds number range, in which the majority of stay cables operate. The cable response and the associated wind load characteristics are studied experimentally, with a 1:1 scale cable section, 0.162 m in diameter and 6.7 m long. The displacement (and acceleration) response was monitored simultaneously with 162 surface pressure taps, for inclinations of 60º, 77º and 90º. The data corroborated susceptibility of dry cables to respond to load variations throughout the drag crisis region, primarily in the across-flow direction. The most vigorous vibrations, with sustained amplitudes of 0.6D, were observed at 60º, with damping ratios below 0.2%. In the follow up wind tunnel study, it was found that the stay cable model was prone to dry inclined cable galloping even when fitted with a helical fillet representative for modern stay cables. When the nominally circular cable model was rotated about its axis, the absence of the perfect roundness (1% local deviation from the nominal diameter) was observed to influence greatly the response. Increased structural damping corresponding to a Scruton number of 5 was found sufficient to suppress the cable vibrations observed during the tests.*

## INTRODUCTION

Cable-stayed bridges are widely established as cost-effective structural systems for medium and long-span bridges (up to about 1000 m). Their rapid development over the past few decades has been followed by numerous incidents of large amplitude cable vibrations introducing a serviceability problem. Stay cables are characterized by a limited bending stiffness and damping levels as low as 0.1% of critical. They thus tend to 'absorb' the energy of the dynamic environmental forcing or that associated with the movements of the anchoring points, including the so-called parametric excitation. Elaborate surveys on stay vibrations have been presented in [1] and [2].

The adverse cable response typically arises at high reduced wind speeds, $U_r=U/(f_eD)$, (values of 20 to 100 or higher) in the presence of rain, snow or ice, which alter the cross-sectional shape and the cable surface roughness. Also dry stay cables are prone to vibrations in the critical Reynolds number range, where strong changes in the along- and the across-wind loading occur as turbulence propagates from shear layers in the near wake into the cable boundary layers ([3]). Some similarities between the dry cable vibrations and the rain-wind vibrations have been pointed out in [4] and [5]. Increased understanding of dry stay cable aerodynamics in the critical Reynolds number range is required in order to refine the criteria for the damping levels needed to suppress the adverse cable response or develop suitable aerodynamic alteration of the cable surface. During the past decade, considerable research efforts internationally have been concentrated on stay-cable aerodynamics in the drag crises. At the National Research Council Canada (NRC), four major investigations have been performed, two of which are presented in the following.

## EXPERIMENTAL INVESTIGATIONS AT NRC

### Test series

In 2001, wind tunnel tests dedicated to wind-induced dynamic response of a 6.7 m long cable section model in 1:1 scale were performed at the Icing and Propulsion Wind Tunnel of NRC. Response sensitivity to wind-cable angle, orientation of the principle response directions, damping levels and surface roughness were studied (Phase I, [6]). Violent cable response was observed at the wind-cable angle of 60° primarily in the across-flow direction and called for a thorough investigation of the load generation mechanism. The study of surface pressures on a 3.0 m long static cable model was performed in the NRC 2 m x 3 m Wind Tunnel in 2002 (Phase II, [7]). In 2008, a measurement campaign with the 6.7 m long dynamic cable model fitted with surface pressure sensors was performed, in order to bridge the gap between the separate response and load data available from the two previous studies. The tests were carried out as research collaboration between the National Research Canada, the University of Bristol and the University of Stavanger (Phase III, [8]). In 2011, the same experimental setup was adapted to examine the effect of helical fillets on the excessive cable response in the drag crisis region. The tests were conducted on the initiative of the US Federal Highway Administration (FHWA), with expert advice from the University of Bristol, Rowan Williams Davies & Irwin and the University of Stavanger (Phase IV, [9]). In the following, the main findings from the two most recent measurement campaigns are presented.

### Experimental setup, Phase III

The drag crisis aeroelastic behaviour of the cable section model, 0.162 m in diameter and 6.7 m in length, was studied in the NRC Propulsion and Icing Wind Tunnel that has a 3.05 m wide x 6.1 m high working section. This is an open-circuit wind tunnel fully opened to the atmosphere. The critical Reynolds number range was spanned by wind speeds from 10 m/s to 40 m/s. During calm days, the turbulence intensity in the test section was about 0.4 % in the along wind direction at wind speeds above 15 m/s, and approximately twice this value at 10 m/s. During very windy days, the turbulence intensity increased by about 50%. The nominally rigid cable section was made of a steel core with a high density poly-ethylene (HDPE) cover typical of the protection used for prototype stay cables. The average roughness depth to diameter ratio of the sanded HDPE surface was $6.5 \times 10^{-6}$. The cylinder was elastically suspended on a set of four orthogonal springs at each of its extremities. The springs provided the flexibility in the principal response directions of a prototype cable, in the cable plane and normal to it, hereafter termed heave and sway response directions. The upper end of the model protruded though the wind tunnel ceiling and was further suspended by an axial wire attached to the building roof, see Figure 1. The wire carried the majorority of the cable weight and contributed to the modal stiffness. The lower end of the model was completely immersed in the flow. The modal mass was 403.5 kg.

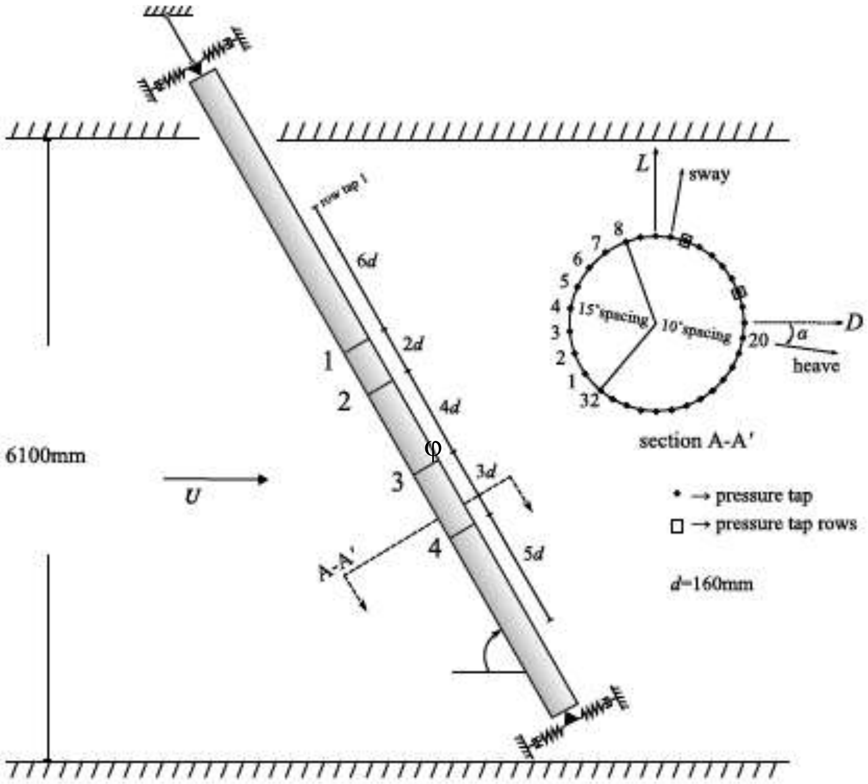


*Figure 1. Sketch of the section model setup, distribution of pressure taps and principal response directions.*

The cable model was fitted with 162 pressure taps grouped in 4 rings and two leeward lines, see Figure 1. The surface pressure signals were corrected in the frequency domain, for the distorting effect of the tubing system connecting the pressure taps on the surface of the model and the sensors embedded inside the model. The pressures were recorded simultaneously with displacement and acceleration data at a sampling rate of 500 Hz, typically for 3 minutes at each wind speed. Free vibrations tests in still air were also performed in order to establish the model eigen-frequencies and the damping ratio. More details about the test set-up and the flow conditions can be found in [8].

Phase III focused on the simultaneous surface pressures and response data on a nominally smooth cable surface in flow with low turbulence. Table 1 summarizes the test configurations examined. Angle $\phi$ is the wind-cable angle and $\alpha$ the angle by which the springs and the model were rotated about the model axis in order to represent a different orientation of principal response directions of a prototype cable. For the relation between angles $\phi$ and $\alpha$ and the prototype cable inclination $\theta$ and the wind yaw angle $\beta$ see e.g. [6]. The damping ratios in Table 1 are given for an amplitude of 3% of the cable diameter, i.e. about 5 mm. Increasing and decreasing wind-speed increments are indicated by arrow up/down under case number.

*Table 1 Test parameters, Phase III*

| Case number ↑ | Case number ↓ | Geometry $\phi$ [º] | Geometry $\alpha$ [º] | Sway $f_0$ [Hz] | Sway $\zeta_3$ [%] | Heave $f_0$ [Hz] | Heave $\zeta_3$ [%] |
|---|---|---|---|---|---|---|---|
| 1 | 2 | 60 | 0 | 1.37 | 0.06 | 1.35 | 0.12 |
| 3 | | .. | .. | 1.37 | 0.06 | 1.35 | 0.12 |
| 4 | 5 | .. | .. | 1.43 | 0.12 | 1.40 | 0.23 |
| 6 | | .. | .. | 1.41 | 0.13 | 1.42 | 0.17 |
| 7 | | .. | .. | 1.41 | 0.13 | 1.42 | 0.17 |
| | 8 | .. | .. | 1.41 | 0.11 | 1.42 | 0.13 |
| 9 | 10 | 60 | 55 | 1.41 | 0.15 | 1.42 | 0.12 |
| 11 | | .. | .. | 1.41 | 0.16 | 1.43 | 0.15 |

| Case number ↑ | Case number ↓ | Geometry $\phi$ [º] | Geometry $\alpha$ [º] | Sway $f_0$ [Hz] | Sway $\zeta_3$ [%] | Heave $f_0$ [Hz] | Heave $\zeta_3$ [%] |
|---|---|---|---|---|---|---|---|
| 12 | | 90 | 0 | 1.42 | 0.33 | 1.44 | 0.14 |
| 13 | | .. | .. | 1.42 | 0.33 | 1.44 | 0.14 |
| 14 | | .. | .. | 1.40 | 0.07 | 1.43 | 0.08 |
| 15 | 16 | .. | .. | 1.39 | 0.19 | 1.42 | 0.12 |
| 17 | 18 | 90 | 30 | 1.39 | 0.16 | 1.42 | 0.09 |
| 19 | 20 | 77 | 30 | 1.42 | 0.09 | 1.42 | 0.08 |
| 21 | | .. | .. | 1.42 | 0.09 | 1.42 | 0.08 |
| 22 | 23 | 77 | 0 | 1.41 | 0.09 | 1.42 | 0.12 |

## Wind-induced response of smooth cable model

Large amplitude cable vibrations were triggered in a number of cases, in particular at the inclination of 60°, see Figure 2. At this inclination, the sway displacement standard deviation exceeded 20% of the cable diameter in seven out of eleven cases. The largest sustained amplitudes were about 95 mm and were limited by an increase in structural damping, up to about 0.2% at the 70 mm amplitude. Structural damping was not intentionally modified but did vary throughout the tests. Figure 2 reveals that the large cable responses occurred for damping ratios below 0.2% at small amplitudes. The figure also shows that the tuned and nearly tuned sway and heave frequencies were associated with the largest responses.

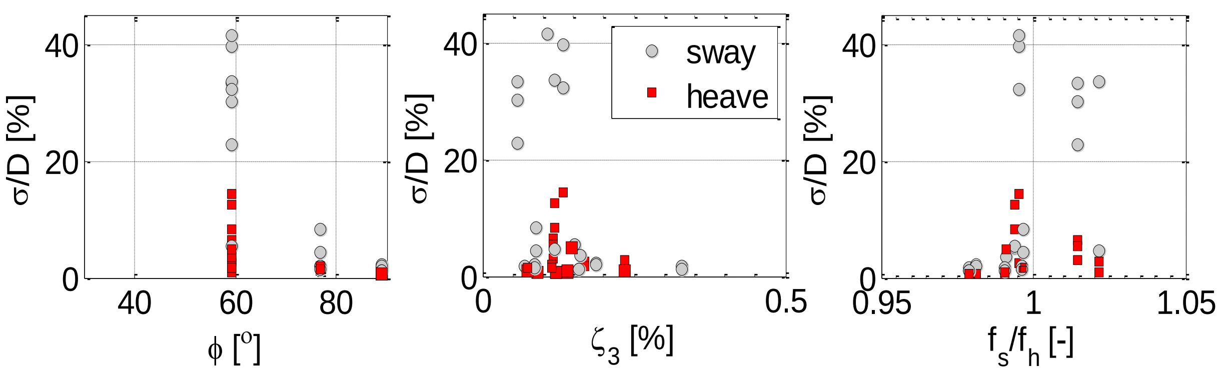


*Figure 2. Largest displacement response of the cable model (standard deviation $\sigma$ relative to cable diameter D) as function of inclination angle, damping ratio and frequency ratio.*

Figure 3 presents dependency of the displacement response on Reynolds number, as well as the displacement trace at Re=3.1 x $10^5$. The displacement orbit graphically presents the cable model susceptibility to the across-flow excitation, as in the case of galloping. In the present test campaign however, all the large responses were restricted to a narrow range of wind speeds, and in such a sense do not fully fit into the notion of classical galloping. For the cases of spring rotations $\alpha \neq 0°$, the along-wind response increased in magnitude, implying increased coupling between the two directions, but the across-flow response was still predominant.

In Figure 4, the entire set of dynamic response data is interpreted in terms of the mean force coefficients, inferred from the mean cable translations in the heave and sway direction and the associated modal stiffness. The largest dynamic responses in the individual runs are shown by circles and bullets, see e.g. Figure 3. The force coefficients define the loading in the plane

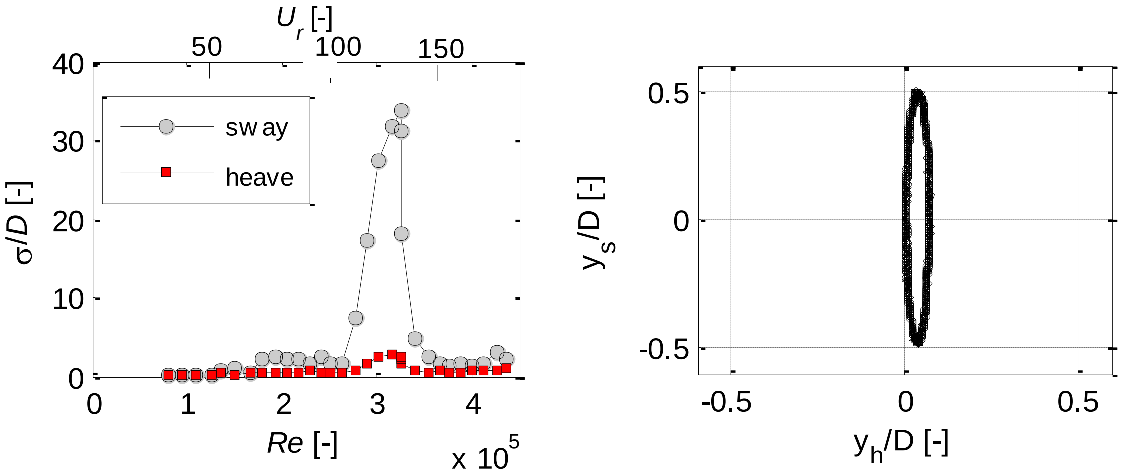


*Figure 3. Wind-induced displacement response in Case 5 (left) and displacment orbit ($y_h$: heave and $y_s$: sway) at R=3.1 x10$^5$ (right).*

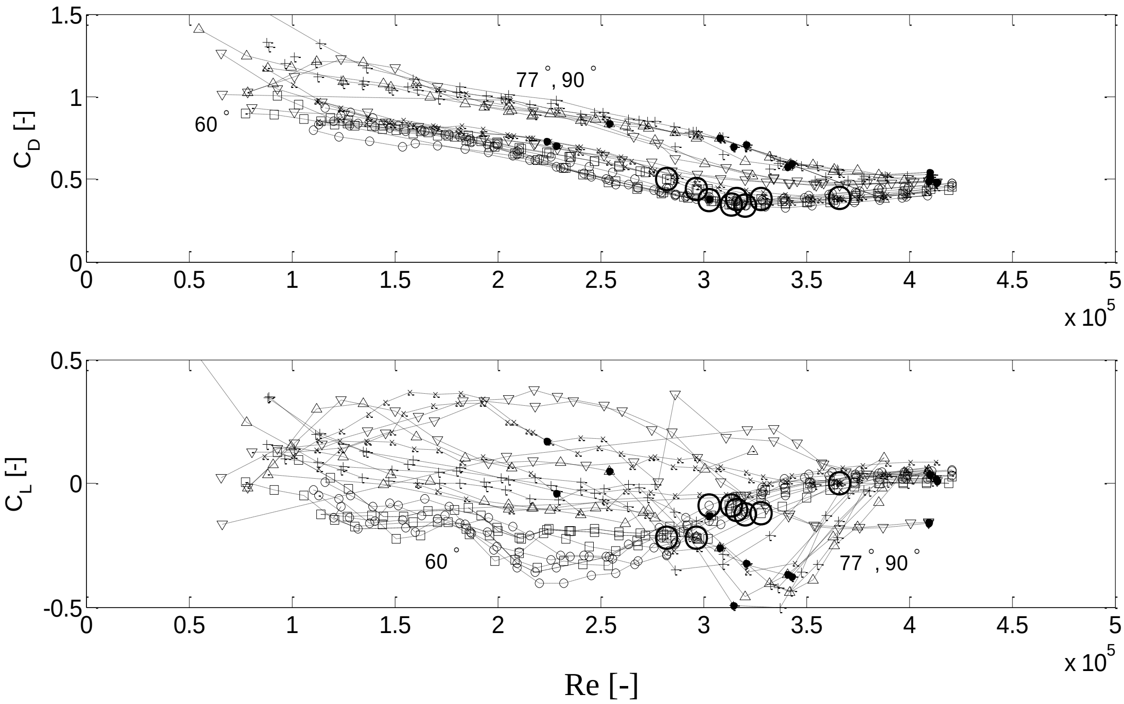


*Figure 4. Mean force coefficients based on mean displacements. Largest responses in each test case are highlighted by symbols: large responses ($\sigma$ > 0.1D) marked by circles and smaller responses ($\sigma$ < 0.1D) marked by bullets (•).*

normal to the cable axis but refer to the full wind speed (not its component normal to the cable axis). The largest vibrations are seen to cluster around the minimum normal (drag) force coefficient. Another, smaller group of maximum responses is found at 'the centre of the drag crisis', around Re=$2.3 \times 10^5$.

Among other factors influencing the aerodynamic cable response, occasional accumulation of insects with body dimensions of the order of 1 mm (excluding wings) on the windward side of the model was observed to inhibit large vibrations. Other findings on the cable response and the load generation mechanism can be found in [8].

### Test setup for model with helical fillets, Phase IV

In Phase IV, the tests focused on the effect of helical fillets on the aerodynamic behaviour of the cable model. A rectangular 2.3 mm thick and 2.4 mm wide fillet was fitted to the cable surface as a double parallel right-handed helix. The fillet pitch was 520 mm, i.e. 3.2 times the cable diameter and the helix angle was 44.5°, see Figure 5.


Figure 5. Double parallel helical fillets, 3.2 D pitch and 44.5° helix angle (left); Ladder grid 5.1 m upstream of the model producing approximately 5% along-wind turbulence intensity.

Table 2 Test parameters Phase IV

| Inclination $\phi$ [° ] | Spring rot. $\alpha$[° ] | Cable rotation $\gamma$ [° ] | Comments* |
|---|---|---|---|
| 60 | 0 | 0, -13.8, -30, -40, -50, -60, -75, -88, -90, -92, 5, 65, 50 | |
| 60 | 0 | -90 | medium damping, $\zeta$ = 0.12% at 20 mm |
| 60 | 0 | -90 | high damping, $\zeta$ = 0.25% at 20 mm |
| 60 | -54.7 | -90 | |
| 45 | 0 | 0, 27, -25, 90 | |
| 45 | -54.7 | -25, 0 | |
| 45 | -45 | -25 | |
| 45 | -12.5 | -25 | |
| 60 | -12.5 | -90 | |
| 60 | 0 | -88, -90, -92, 0, 50 | turbulent flow |
| 60 | 0 | -88,-90, -92, -54.7 | turbulent flow, no helical fillets |
| 60 | 0 | -93, -90, -54.7, 0, -2, 2 | smooth flow, no helical fillets |

*Baseline test case: with helical fillets, $\zeta$ = 0.07% at 20 mm, $f_s$=1.39 Hz, $f_h$=1.39 Hz, no turbulence generation devices. Angle $\gamma$ corresponds to -$\alpha$ in Table 1.

A special turntable was designed to allow cylinder rotation by an arbitrary angle $\gamma$ about its main axis independently of the spring system, and thus isolate the significance of the cable eccentricity on the vibrations in the critical Reynolds number range. Measurements prior to the wind-tunnel tests established that the nominally circular cable section had systematic deviations from the ideal shape up to 1% of the cable diameter, which may potentially influence the cable aerodynamics in the drag crisis.

The effect of turbulence levels representative of full scale conditions (approximately 5%) on the model response was also investigated as well as the effect of increased structural damping. Increased turbulence intensity was simulated using a ladder grid, see Figure 5. The flow conditions upstream as well as in the wake of the model were monitored by 4-hole Cobra probes. Both in the test Phase III and Phase IV a limited number of tests with the fixed, static model were also carried out. More details about the experimental conditions are provided in [9] and [10]. A summary of the test cases is presented in Table 2.

**Dynamic response of the cable section with helical fillets**

The tests with a model fitted with helical fillets revealed that the fillet attenuated the cable response for several but not all model orientations to the flow. As seen in Figure 6, sway oscillations with the amplitude of 0.5D were excited when the model was rotated by -90° about its axis. It was not possible to increase the wind speed beyond 36 m/s and to examine whether the response remains unstable at higher wind speeds or whether it was velocity restricted. At other cable rotations tested, the response was limited and driven only by buffeting forces.

To suppress the large cable response, the structural damping ratio was increased to 0.25% at 20 mm displacement amplitude, corresponding to a Scruton number of 5[4]. As illustrated in Figure 6 (right) such a damping level was found sufficient to mitigate the across-flow unstable response.

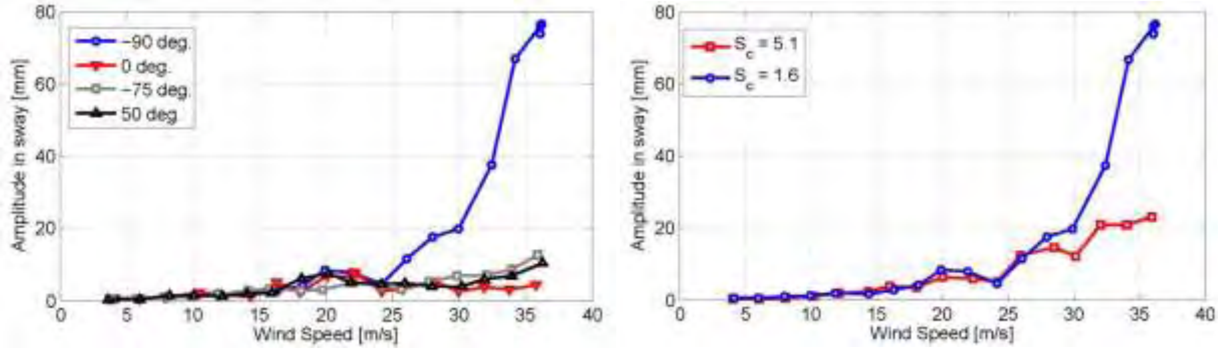


*Figure 6. Sway displacement response of cable with helical fillet inclined at 60° in nominally smooth flow. Response amplitudes as function of wind speed and cable rotations about its longitudinal axis (left) and response as function of wind speed and Scruton numbers at a cable rotation of -90° (right).*

A limited number of tests was performed without helical fillets and the cable response observed fitted into the trends established in the test Phase III. The mean force coefficients deduced from the surface pressure data, for the model with and without helical fillets, are depicted in Figure 7. The effect of the helical fillet is to reduce the Reynolds number for the drag crisis roughly by $\Delta Re = 1 \times 10^{5}$ compared to the smooth cable case, due to increased surface roughness. Significant magnitudes of the lift coefficient develop in the drag crisis region also in the presence of helical fillets. The drag coefficient in the supercritical regime increases to about 0.65. At the highest Reynolds numbers investigated, the lift coefficient has a mean value of about 0.2 in the presence of the helical fillets and a 0 value in case of the smooth model. The reason for the offset mean lift in the presence of helical fillets is that the model inclination causes the fillets on the two sides of the cylinder to be aligned differently to the flow, one towards the across-flow direction and the other one more or less parallel to the flow. The two angles between the flow and the fillets at cylinder shoulders in case of the model inclined at 60° are 105° and 15° degrees respectively. The 'across-flow fillet' then decelerates the flow locally, i.e. causes a local pressure increase just upstream of the fillet, while the 'along-wind fillet' interferes with the flow to a lesser degree. Both fillets are seen to create local kinks in the pressure distribution, similar to those associated with a single bubble formation ([3]). This is illustrated in Figure 8 where pressure distributions on average 'point away' from the 'across-flow fillet' in the upper part of the figure, resulting in a positive lift (downwards in the figure).

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[4] $S_c = \frac{\zeta_{str} m_e}{\rho D^2}$, where $\zeta_{str}$ is structural damping ratio, $m_e$ equivalent mass per unit length and $\rho$ air density.

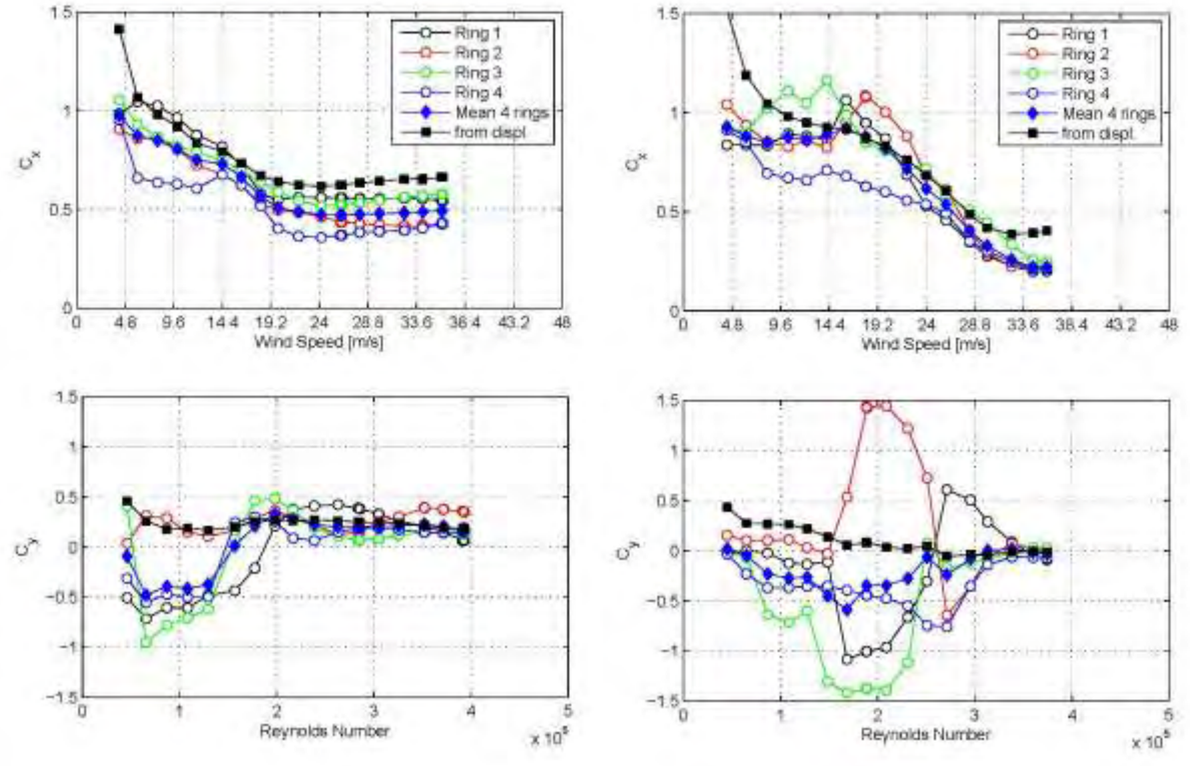


*Figure 7. Mean force coefficients for cable model at $\phi$=60° and $\alpha$=0°. Cable with helical fillets and $\gamma$=-90° (left), Cable without helical fillets and $\gamma$=-54.7° (right).*

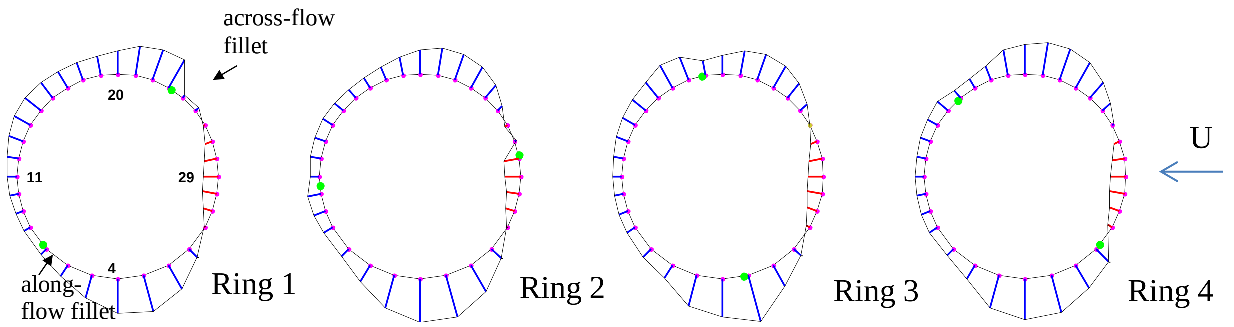


*Figure 8. Mean pressure distributions, model at $\phi$=60°, $\alpha$=0° and $\gamma$= - 90°, U=35.9 m/s.*

The response sensitivity to the model rotation about its longitudinal axis (see Figure 6, left) may be associated with the absence of perfect rotational symmetry of the model cross-section. The relative eccentricity of the model was quantified in terms of outside diameter measured at every 10° azimuth angle, as presented in Figure 9. The cable rotation by -90°, at which the largest cable vibrations with the helical fillets were observed, corresponds to moving the 'peak point' at -20° and the inflection point at about 4° to 70° and about 94° respectively, i.e. into a region defining the flow separation. For the smooth cable model, the largest response was observed for γ = -54.7°, which corresponds to rotation of the local peak from 35° to approximately 90° and the two inflection points (at 4° and 65°) to 60° and -65°. Such a variation of the cross-sectional geometry in the separation regions, including the uneven radial height around the two inflection points, might be defining for the across flow forcing. For the rotation γ= - 54.7°, magnitudes of the lift coefficients on different rings were largest among the test cases, both with and without the helical fillets. In addition to the cable degree of roundness, possible variations in surface roughness may also be governing the response, as postulated in [11].

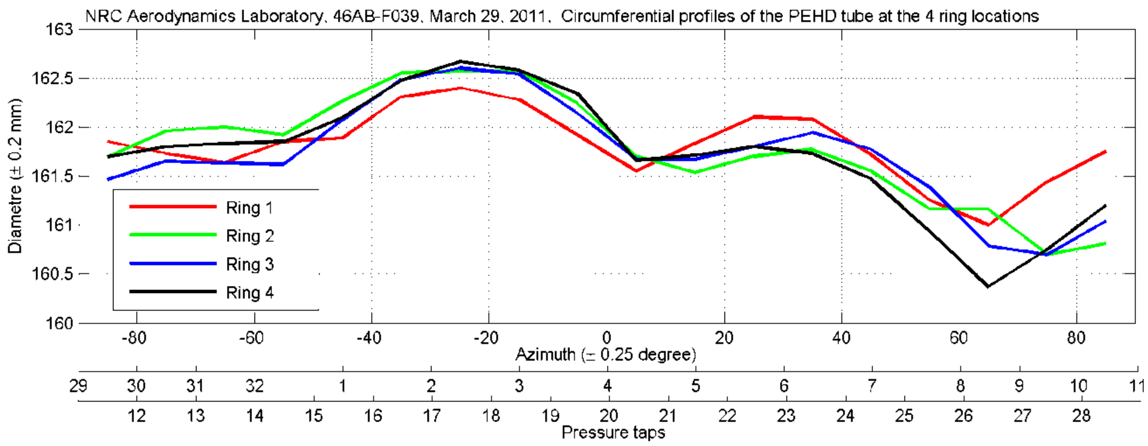


*Figure 9. Cable diameter measured at the locations of four pressure rings as a function of azimuth angle.*

Turbulence was found to reduce the Reynolds number for the minimum value of the drag coefficient by $1.0 \times 10^5$ for the model with the helical fillet and by $1.6 \times 10^5$ for the smooth model. Vibrations were attenuated to a certain degree but still present. In addition, end-to-end cable motion was excited. More details on the effect of various influencing parameters on the cable vibrations are given in [10].

## SUMMARY AND CONCLUSIONS

Wind tunnel tests on a 6.7 m long cable section model with a HDPE cover representative of prototype stay cables established a clear relationship between the large across-flow cable vibrations and the drag crisis region. The Reynolds number has thus been recognized as the major parameter governing the cable response. This is in line with recent full-scale observations on the influence of Reynolds number on the aerodynamic damping ([12]). The violent response of the smooth cable was found to be restricted to a narrow velocity range around the minimum value of the drag coefficient. A three-dimensional average surface pressure distribution was observed, indicating local variations of flow states along the cable model. Helical fillets triggered the drag crisis at lower wind speeds compared to the smooth cable surface but did not completely eliminate the vigorous across-flow cable motion. The large across-flow oscillations were triggered only at certain rotations of the cable model about its own axis. This suggests that the cable 'galloping' could possibly be governed by the absence of perfect rotational symmetry of the model, as well as possible local surface roughness variations. In the tests, damping ratio corresponding to a Scruton number of 5 was found to limit sufficiently the adverse vibrations.

## ACKNOWLEDGEMENTS

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# THE BJØRVIKA IMMERSED TUNNEL - ASSESSMENTS OF CALCULATED AND MEASURED SETTLEMENTS OVER A 3 YEAR PERIOD

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## ABSTRACT

*The Bjørvika Tunnel connects two existing rock tunnels on the primary road network in the heart of Oslo. The tunnel, approximately one kilometre long, consists of two cut-and-cover land connections and an immersed tunnel (675m). When opened in 2010, the tunnel realigned heavy traffic (AADT 90'000) from the quayside to beneath the sea level.*

*Along the tunnel alignment there is in general a deep deposit of soft normally consolidated clay but the base of the tunnel touches bedrock at some locations. The submerged tunnel was placed in a typically 10 m deep dredged trench that was later backfilled. The tunnel rests directly on the clay without any pile foundations. The paper discusses predicted settlements of the tunnel at the design stage, and compares this to settlements measured during the installation of the tunnel and during the first 3 years of operation. The settlement analysis provided both upper and lower bound values, depending on choice of soil parameters and time from trenching to installation of the tunnel. Special transition zones between two areas where the tunnel reached down to bedrock were designed. The paper also presents dredging requirements and constraints in addition to specifications for the construction.*

*The structural and geotechnical design of the immersed tunnel was undertaken by respectively Aas Jakobsen A/S and the Norwegian Geotechnical Institute. The client was the Norwegian Public Roads Administration. The contractor engaged to build the tunnel was AF Bjørvikatunnelen, a joint venture consisting of Skanska Norge AS, BAM Civiel BV and Volker Stevin Construction Europe.*

## IMMERSED TUNNEL AND CONSTRUCTION PRINCIPLES

### The immersed tunnel and its alignment

Figures 1 and 2 show a plan view and longitudinal section of the 675 m long 6-lane immersed road tunnel constructed across the Bjørvika and Bispevika bays, located within the inner harbour area of Oslo. As described by Markey and Narvestad (2009), the immersed tunnel tube has a gross height of H=9.5 to 10.0 m and width ranging from B=28.0 to 40.0 m. The tunnel elements were constructed in a dry dock as 6 elements each of length 112.5 m. Each element had a unique shape in terms of height, width and curvature, and each consisted of five 22.5 m long segments with an expansion joint and shear key in between. Figure 3 shows a typical cross-section of the tunnel box. The elements were numbered from one to six across the bay from Havnelageret to Sørenga.

The tunnel cuts through two existing piers constructed by land filling in the time period around 1920 – 1950, figure 1. The small river Akerselva has its outlet between these two piers.

The vertical tunnel alignment was chosen such that the roof of the tunnel would be slightly below the present sea bed, and at its deepest point lies at about elevation –11 m. Figures 1 and 2 also show the land approaches, which tie into tunnels going into bedrock at both the western and eastern sides of the tunnel. Note that on-off traffic ramps on the land sides of the tunnel are not shown in figure 1.


*Figure 1- Plan view of the immersed tunnel across the Bjørvika bay in Oslo*

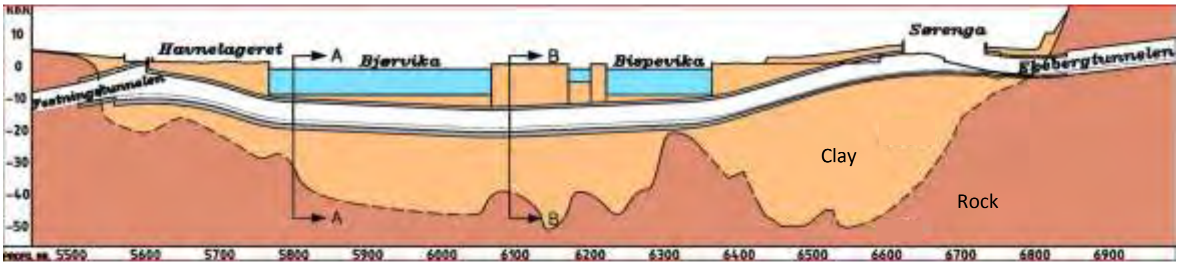


*Figure 2- Longitudinal section of the immersed tunnel showing clay(leire) and bedrock(fjell).*

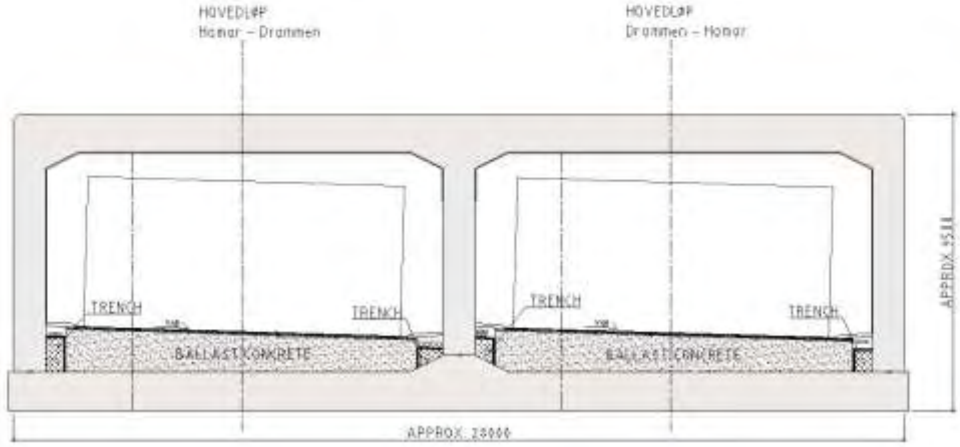


*Figure 3- Typical cross section of immersed tunnel box*

The tunnel elements were sunk down into pre-dredged trenches. To ensure stability of the trenches they were designed with side typical slopes of 1: 1.2. Figure 4 shows two cross sections A-A and B-B of the trenches made (see figure 1 for location), including later backfilling.

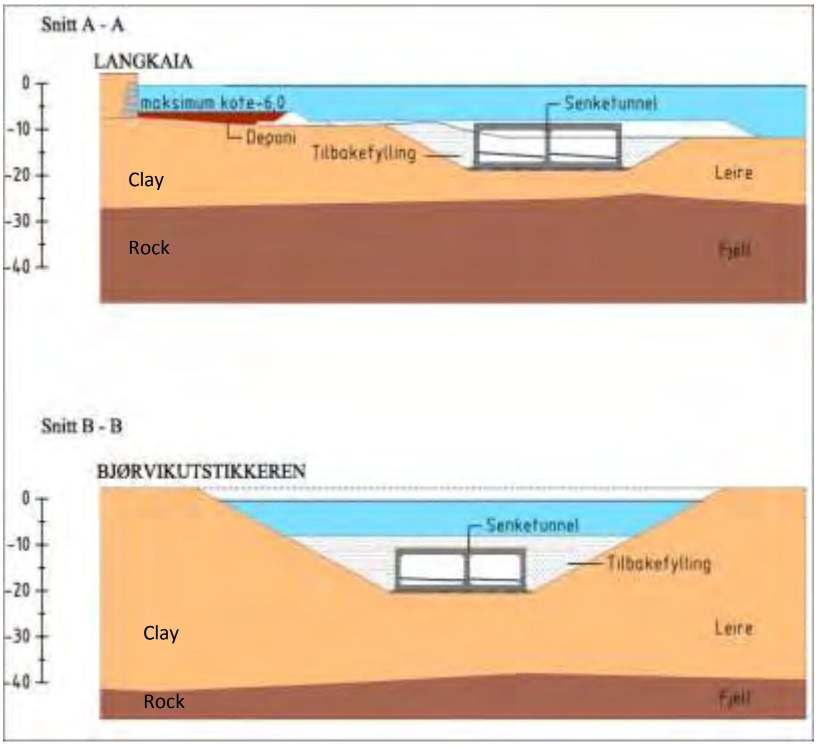


*Figure 4- Cross sections A-A and B-B of excavated trench and tunnel in place*

**Overall geotechnical conditions**

Part of the fill used for the piers which the tunnel cuts through, consisted of sawdust, saw mill cuttings, and was thus rather lightweight materials. The existing land areas at both ends of the immersed tunnel are also reclaimed land and had different types of quay structures at the water front. Most of these landfills were made of coarser materials, for a large part rockfill.

The original seabed level was along most of the alignment at around elevation -8 m (figure 1). Below the original seabed, there is mainly a soft normally consolidated marine clay deposit down to bedrock. As shown by the longitudinal section in figure 2, along the immersed part of the tunnel the bedrock level varies from Elevation – 18 to -48 m. The highest bedrock level corresponds closely with the base of the tunnel section.

The soft seabed clays were deposited in the post-glacial period about 8 to 10 thousand years ago. Along most of the alignment there is however, an upper 1 to 2 m thick layer consisting of loose highly polluted and organic harbour sediments from more modern times. These organic sediments have very high water content, generally in the range 80-120 %. Typical ranges of index properties of the marine clay deposit below this top layer are as follows:

Natural water content, w = 35 to 45 %

Total unit weight, $\gamma$ =18.0 to 18.8 kN/m$^3$

Plasticity index, $I_p$ = 17-25

There is a tendency that the water content, total unit weight and plasticity index decrease with depth below seabed. Geotechnical parameters used in settlement design analyses are discussed in subsequent sections.

**Construction principals**

A major design issue was if the tunnel should be founded on piles pre- driven to bedrock along its entire length, or it could be allowed to rest more or less directly at the bottom of the pre-dredged trench.

The tunnel represents a permanent unloading of the clay below its base. The tunnel would therefore be expected to only experience relatively small re-consolidation type settlements when the tunnel is placed at the bottom of the pre-excavated trench, ballasted down and backfilled along the sides and on top of the roof. This was confirmed by detailed settlement and interaction analyses, presented in later sections,, and formed the basis for the choice of a non-piled solution.

The methods of preparing for- and installation of- the immersed tunnel are briefly summarised as follows.

1. The top contaminated sediments were first dredged and transported to a safe disposal area.
2. The trench was then excavated using a box-type wire guided grab operating from a large barge. The position of the grab was GPS-controlled. Strict tolerances were imposed with respect to the shape of the trench at all times during excavation:
    - Local steep vertical escarpments were not allowed to be larger than 0.5 m.
    - The slope angle should not be larger than theoretical.
    - The final excavated level of the trench bottom should lie within ±0.3 m of the design dredging level, which was 0.7m under the tunnel base surface.

3. At the end of the main trenching operation, and maximum 4 weeks before placing a levelling gravel bed, the trench bottom should be documented to be free from cuttings or loose disturbed clay material that could have fallen out of the grab during excavation. This was documented by diver inspection and taking samples of the top half meter or so of the trench bottom by means of pushed-in plastic cylinders. The samples were to be taken at spacing not larger than 15 m. All samples were inspected by the client's representatives before the trench bottom was accepted as "clean" and ready for placement of the gravel layer.
4. It was chosen to place the gravel bed as "gravel strips" or berms transverse to the tunnel axis. The "gravel strips" were placed with a longitudinal spacing of approx. 2.6 m, and had a maximum thickness of about 100 cm at the "ridge top" and minimum thickness of 40 cm at the bottom of the troughs. For this purpose it was used a fallpipe mounted to a guide rail alongside a barge. As illustrated in figure 5, the fall pipe runs back and forth along the guide rail and across the trench as the gravel is pored through the pipe. The barge "Scradeway" was used by the contractor for this purpose. The intention with using this "gravel strip" concept was to ensure reasonably uniform contact stress across the base of the tunnel. The top of the gravel strips was 1.650 m from the size of the fallpipe. A tilt on the lower fallpipe "mouth" was also added to compensate for the vertical slope of the tunnel. It can be mentioned that this concept had been successfully used for several immersed tunnels in the past, e.g the Øresundtunnel . The tolerances set for the level of the gravel strings were ±30 mm. It was controlled by multi-beam echo soundings, see the example in figure 6. Also a full record of the fallpipe position was used for the control of each strip. It can be mentioned that the tolerances set forth were fully met for all tunnel elements.
5. The installation of the individual tunnel elements were to start within 1-2 days after the gravel string bed was prepared for the element. The elements were carefully ballasted down using water ballast tanks built inside the elements. Upon touch-down, the average contact pressure exerted on the bottom was only 3 kPa, based on a 3% safety required on the calculated weight/buoyancy ratio.
6. As each tunnel element was installed, backfilling along the two sides was carried out. It was required to do that in a balanced operation along the two sides of the tunnel, with a difference in filling level along the two sides never exceeding 1.0.m . This filling operation included placing about 0.5 m fill gravel to over the roof.
7. The ballast concrete at the bottom(see figure 3) was poured in the following weeks, increasing the contact pressure to a minimum of 6 kPa (6% safety weight/bouancy).
8. Final filling to the required levels as defined in figure 7 was finally undertaken. This filling level varied along the tunnel. Note in particular that the high level of backfilling from about Pr. 6155 to Pr. 6217, where the tunnel crosses under new piers.

In relation to the trenching requirements it should be mentioned that along the part of the tunnel were bedrock lies above the tunnel base (around Pr. 6300, figure 2), it was required to under-blast the bedrock by minimum 1.5 m and replace it with a base layer of crushed rock grade 20/120 up to 0.7 m below the tunnel base surface before placing the final gravel strings.


*Figure 5- Principal sketch of placement of "gravel string" bed with the specially equipped Scradeway barge*

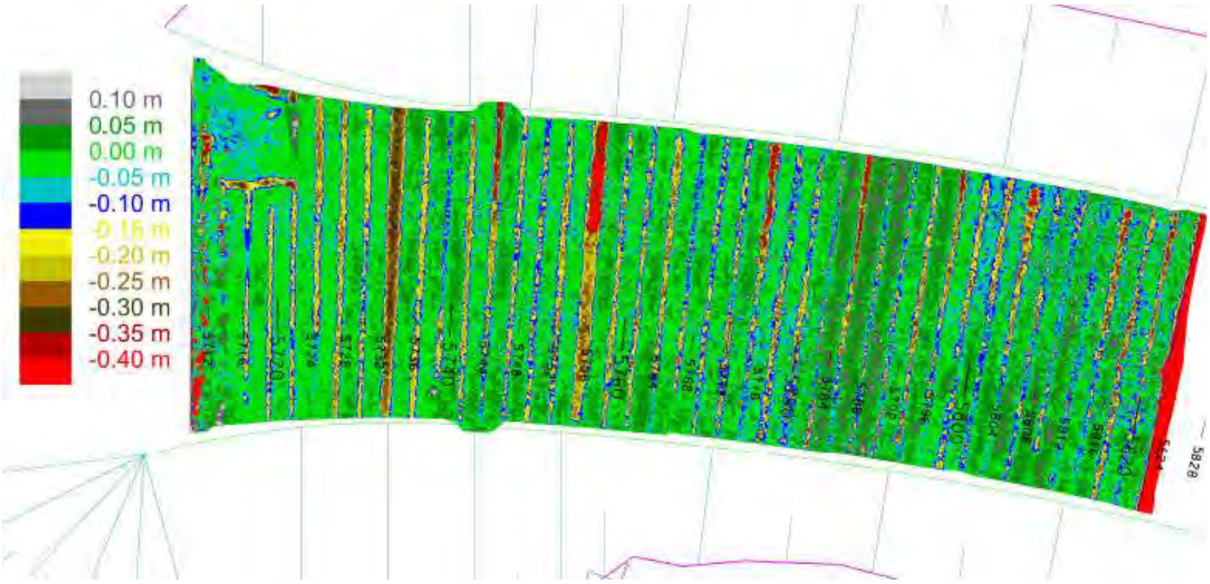


*Figure 6- Example of results of multi-beam echo sounding used for documenting final "gravel string" levels. Ridges are seen as light grey, the deepest troughs at red in colour*

Whereas the land side tunnel on the eastern side was also having a non-piled "floating foundation", the west landside tunnel was founded on piles to bedrock. The reason was complicated mixtures of fill and clay at the base level of the tunnel in this area and high levels of back fill combined with new quay area with large surface loads.. To ensure a smooth transition between the piled landside tunnel and the immersed tunnel element connecting into that, a special "relief pile" transfer foundation solution was adopted for this tunnel element. The principles of this "relief pile" type transfer foundation solution, covering the first western half of Element 1, are illustrated by figure 8. Briefly described it consisted of 610 mm diameter steel pipe piles driven from the trench bottom to bedrock at a centre spacing of 2.34 m, following the gravel strips. Pile hats with a diameter of 1.6 m were placed on top of the piles. The top of the pile hats lie 0.5 m below the base of the immersed tunnel. Coarse gravel was placed between the pile caps and up to 0.5m below base surface, last the gravel strips were placed on top of the piles.

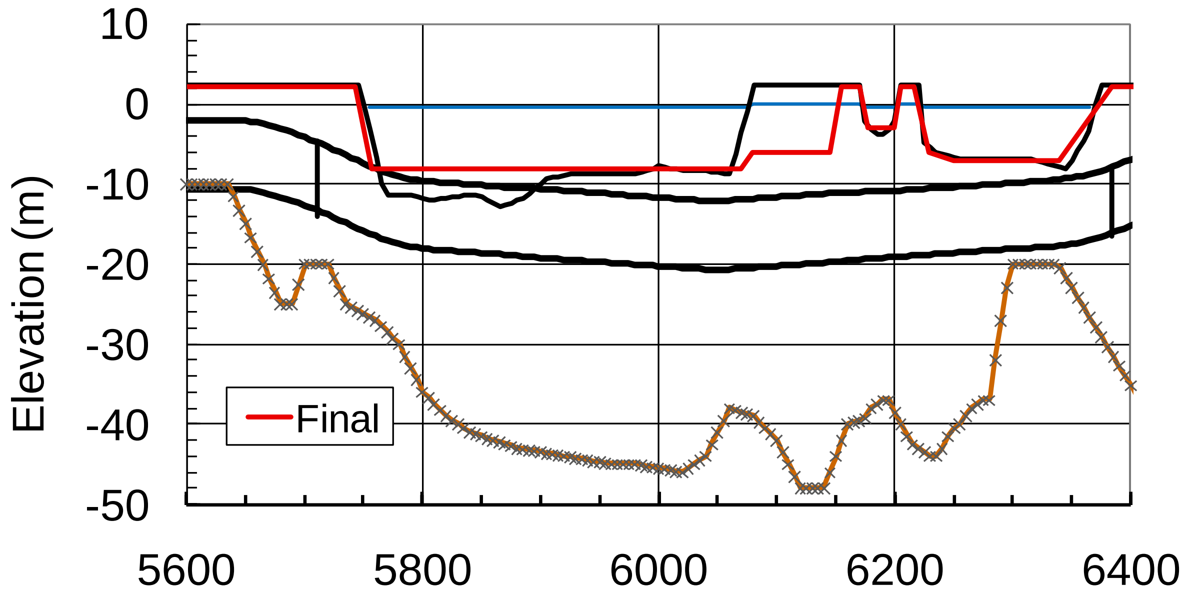


*Figure 7- Longitudinal profile showing tunnel, original seabed, backfilling level and bedrock level along the immersed tunnel*

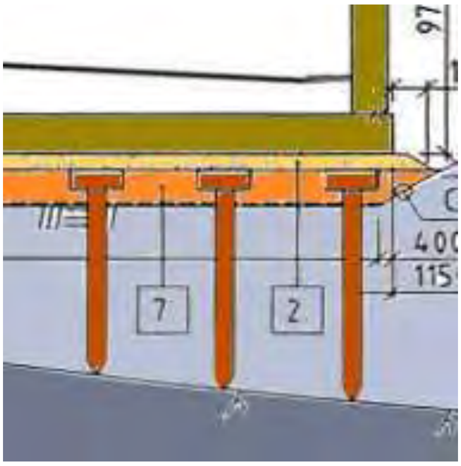

*Figure 8- Illustration of "relief piles" used under western half of Element 1*

## PREDICTED SETTLEMENTS AND ACCOUNTING FOR INTERACTION WITH THE TUNNEL

### Predicted settlements

As the trench for the immersed tunnel is excavated, the base of the excavation will undergo some uplift as a direct consequence of the unloading (undrained shear displacements). Thereafter, a gradual heave of the base of the excavation due to volumetric swelling of the clay below the base will take place.
After the immersed tunnel has been installed and starts to induce loads on the bottom, the following aspects will contribute to settlement of the tunnel elements:

1. Immediate settlements due to undrained shear displacements in the clay as a direct result of the imposed load changes, including both the submerged weight of the tunnel and back-filling against the sides and on top of the roof.
2. Settlements induced as results of volumetric compression of the layer of gravel strings, and any local shear distortions within the gravel strings or the clay below. Such shear-induced displacements will be induced when the tunnel base first touches the top of local "gravel string ridges". The load on top of the "ridges" will tend to push them down, gradually leading to larger contact areas. Shear displacements will stop when the contact area reach a certain value, which will be related to relative size of non-contact areas and imposed loads.
3. Volume changes within possible local thinner layers of clay "sludge" or lumps of severely remoulded clay still remaining at the trench bottom.
4. During grabbing, the clay just below the excavated depth may be somewhat disturbed by the actions of the grab. Such disturbed clay may upon loading undergo larger volume change than the intact undisturbed clay.
5. Settlement due to re-consolidation type volume changes in the clay induced by the imposed loads from the tunnel and backfilling.

The following describes the contribution from each of these 5 components as they were assessed at the design stage. Contribution 1-4 are short time deformation/settlements which is expected to come during the installation process and partly due to side fill and fill on the top of the tunnel.

1) The first component was calculated on basis of 2D finite element analyses using estimated undrained re-loading type shear modulus of the clay corresponding to G/su = 300. Even for the areas with the largest imposed loads this contributes to only about 8 to 12 mm settlement.

2) Volumetric changes within the "gravel strings", with a typical thickness of 70 cm, were estimated to be of the order 5-10 mm. Based on simplified analyses additional shear-induced displacements due to the tunnel elements initially riding on ridges was estimated to be limited to 1 to 5 mm.

3) This was a difficult component to estimate, as it depends strongly on the quality of the final "clean-up" of the trench bed. As the control of the trench bed showed that the clan-up was quite good, this component is probably less than 20 mm, maybe even zero.

4) It was assumed that disturbance caused by the grab would only have effects on the upper 50 cm or so below the excavated depth. Assuming that the main effect is that the clay partly looses its memory of stress history, the volume change was estimated to be 2-5 %, which would give 10 to 25 mm settlement in this layer.

5) The design analyses had a main focus on the settlements due to re-loading or re-consolidation of the un-loaded clay below the base of the trench.

A first and important issue was to try to establish the un-loading and re-loading volumetric compressibility parameters of the clay. The true un-loading and re-loading stiffness is in reality strongly stress- level dependant. This is illustrated by figure 9 presenting the tangent oedometer modulus during virgin loading, un-loading and re-loading from a given stress level. This tangent modulus depends far more on stress levels than just assuming a log-linear unloading and re-loading stiffness as given by the classical re-compression index or swelling $C_{rs}/(1+e_0)$ defined in most soil mechanics textbooks. (e.g. Lambe and Whitman, 1969).

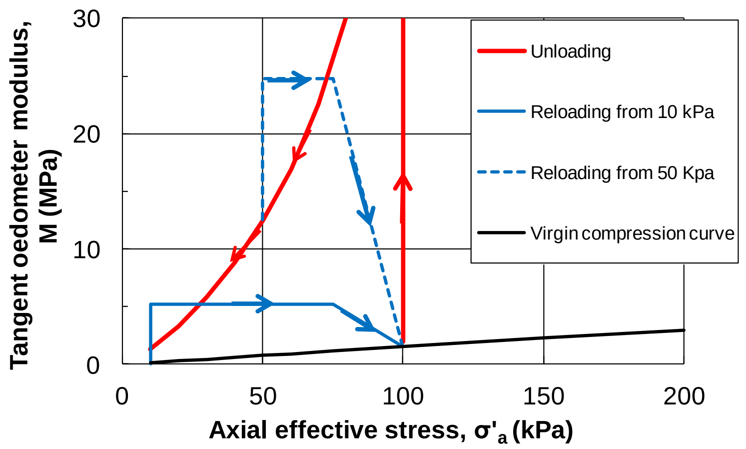


*Figure 9- Typical tangent modulus un-loading and re-loading behaviour of clay samples*

A large number of un-loading re-loading oedometer tests were carried out on piston samples taken at different depths and locations along the tunnel to try to define a general relationship for the un-loading and re-loading modulus. To avoid any impact of sample disturbance the samples were pre-consolidated to stresses well beyond the assumed in-situ pre-consolidation pressures and then un-loaded and re-loaded from that level. The following groups of stress levels were used:

a) The samples were first pre-consolidated to a stress of 100 kPa, gradually unloaded to stresses of 10 or 50 kPa, and then re-loaded again
b) The samples were first pre-consolidated to a stress of 800 kPa, gradually unloaded to stresses of 10 or 50 kPa, and then re-loaded again

Figure 10 shows examples of how the measured tangent unloading or swelling modulus decrease with the level of unloading. On this basis the following general relationship for the tangent swelling modulus was found to give a reasonably good fit to all the data:

$$M_s = a \cdot \sigma'_a \cdot (\sigma'_a / (\sigma'_{amax} - \sigma'_a))^{ns}$$

$\sigma'_a$ = current effective stress during unloading

$\sigma'_{amax}$ = maximum effective stress applied prior to unloading

a = constant = 250

ns = swelling exponent = 0.3

Note that the swelling modulus is extremely high for small levels of unloading compared to the maximum effective stress applied during virgin loading.

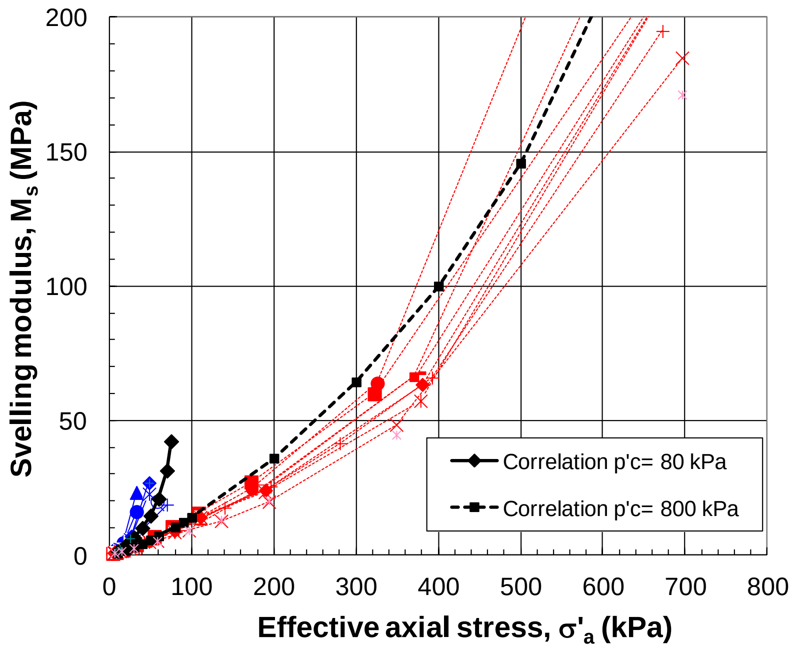


*Figure 10 – Tangent modulus measured during un-loading (swelling) in the oedometer tests*

Figure 11 presents values for the tangent re-loading modulus normalised with respect to the virgin compression modulus number, m and the maximum effective stress applied prior to un-loading, $\sigma'_{amax}$.

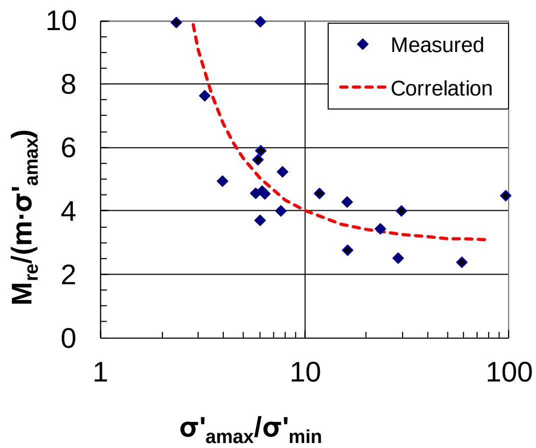


*Figure 11- Normalised tangent modulus measured during re-loading in the oedometer tests*

For reloading from the minimum effective stress applied the following correlation was found to give a reasonable fit to the data:

$M_{re} = m\cdot \sigma'_{amax} \cdot(b+(c/( \sigma'_{amax} /\sigma'_{amin}-1)^{nr})$

m= Janbus's modulus number for virgin loading (m= 16 to 19 was typical range)

b = constant = 2.5

c = constant = 14

nr = re-loading exponent = 1.2

In the settlement analyses typical average modulus values were calculated for the levels of un-loading and re-loading in question. This depends on the level of unloading due to the excavation and the level of re-loading due to the weight of the tunnel and also varies with depth due to geometry effects.

Figure 12 shows an example of how these average modulus values increase with depth for a typical case.

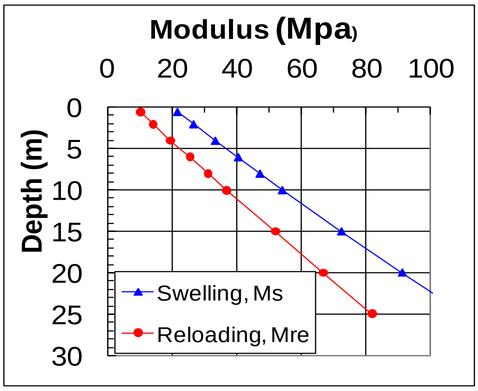


*Figure 12- Example of variation of average oedometer modulus with depth for a typical case*

The heave or swelling and subsequent settlements were calculated for a large number of cross sections by 1D finite difference based consolidation analyses, where the reduction in induced stress changes with depth (2D effects) were accounted for. Selected 2D FEM analyses were carried out for two selected cross sections, and showed excellent agreement with the 1D analyses. On basis of a large number of oedometer tests, the permeability of the clay was in the consolidation analyses taken as $0.8 \cdot 10^{-9}$ m/second.

Figure 13 shows an example of calculated deformations after excavation of the trench (heave), and subsequent settlements due to re-loading by the weight of tunnel. In this analysis the loads were applied instantaneously. Figure 13 shows results when re-loading was assumed to take place either 1 year or 2 years after excavation. It can be seen that the time allowed for swelling also has an impact on the magnitude of re-consolidation type settlements. For this case the reconsolidation settlement increase from about 40 mm when re-loading took place 1 year after un-loading, to 55 mm for 2 years.

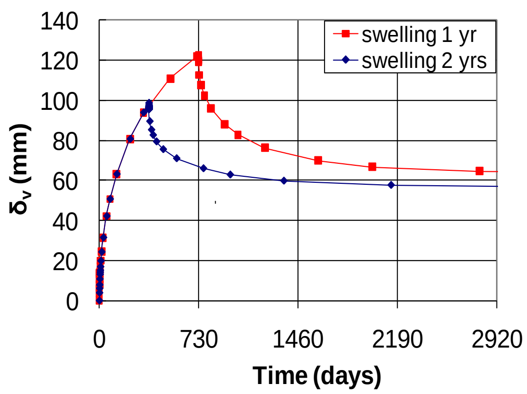


*Figure 13- Example of calculated heave following instantaneous excavation and subsequent settlement after placing tunnel and backfilling for Element 3*

In the design analyses it was conservatively assumed that re-loading would take place 2 years after trenching was completed. In reality the time from trenching to the full load was applied became varied from about 9 to 18 months.

## Analyses of interaction with the tunnel

The interaction between soil and tunnel was carried out by inputting the soil response as equivalent bed springs, defined by a spring stiffness, $k_s$:

$k_s = p/\delta$

p=average contact pressure at the base of the tunnel

$\delta$ = predicted settlement

The calculated settlements and thus, the equivalent spring stiffness, vary to some extent with local unloading and re-loading levels, but more importantly also with the thickness of clay layer below the base. It was found that the impact of clay thickness could well be accounted for by the following relationship:

$k_s = k_{s5m}/(H/5)^{0.4}$

$k_{s5m}$ = Spring stiffness for H= 5m

H = thickness of clay layer below the base of the tunnel

The calculated values of $k_{s5m}$ were mostly in the range 2000 to 3000 kPa/m, with the highest values in the areas under the old piers, where the clay relatively speaking is the strongest and stiffest.

Figure 14 presents the calculated distribution of settlements after complete re-consolidation and how it varies along the tunnel due to the variable loads. The black curve is based on analyses without accounting for the stiffness of the tunnel with its joint distribution, and the red curve when introducing the actual bending stiffness of the tunnel. It can be seen that the impact of tunnel stiffness is to smooth out the settlement profile, and is particularly important between about Pr. 6150 and 6230 where there is large variation in backfilling levels and thus, in permanent loads (figure 7). Note that these analyses do not account for settlements due to other components than re-consolidation of the clay.

Settlements due to other causes as listed under points 1) to 4) above come in addition. In total they may amount to a settlement of 25 to 70 mm. These settlements will partly take place before the tunnel elements are fully interconnected, and thus be of little consequence.

In the 3D design soil-structure interaction (SSI) analyses of the tunnel a series of parametric studies were carried out to assess the impact of variable vertical bed spring stiffness on stresses generated in the tunnel elements. These analyses accounted for, among others, local much softer vertical bed springs underneath the tunnel. These analyses gave an understanding of the response sensitivity to variable presumptions and were used for both the structural design and for the joint design.

SSI analyses were also carried out to address impact of temperature changes and earthquake loading. In these connections equivalent springs were also introduced to model the stiffness of the soil along the walls and under the base of the tunnel due to displacements in longitudinal and transverse directions to the tunnel axis. All these studies showed fully acceptable performance of the tunnel, but are not further documented in this paper.

## MEASURED SETTLEMENTS AND COMPARISON TO PREDICTED

Figure 14 presents the measured total settlements along the immersed tunnel compared to the predicted settlements due to re-consolidation of the clay below the base only. Measured values are given at a time of respectively 200 and 1200 days after the installation of the individual tunnel elements. The measured settlements are generally larger than these predicted settlements, but follow much of the same trend as the prediction. The difference can probably to a large extent be explained by the settlements that occur due to the reasons as presented under points 1) to 4) in Chapter 2.1 above.

The only main deviation from the predicted trend is for element 1, which relatively speaking seems to have experienced larger settlements than predicted. The reason for this is not clear.

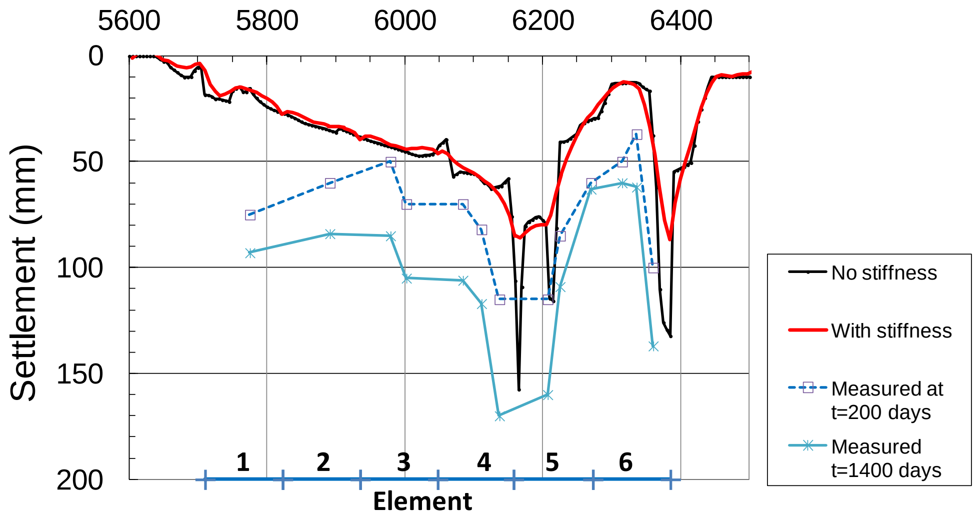


*Figure 14- Summary of calculated and measured settlements along the tunnel.*

Figure 15 presents a detailed plot of settlements with time for Element No. 4 during the first 250 days after installation. Herein is also shown the times when backfilling and ballasting took place, which is reflected in increases in settlements and settlement rates when these load changes occurred.

Figure 16 shows the complete measured settlement curves at 4 typical locations along the tunnel over the complete 1200 day long period that measurements have taken place so far. Note that due to installation of tunnel cladding and other installations in the tunnel there was a halt in the measurement program for a period ranging from about 120 to 240 days. The levelling bolts also had to be replaced during this period. The settlements during this period therefore had to be interpolated from the settlement trends before and after this halt in the measurements.

The data in figure 16 shows a very clear trend in decreasing settlement rates with time.  This agrees well with the decreasing settlement rates predicted due to re-consolidation of the clay alone, as typically predicted and shown in figure 13.

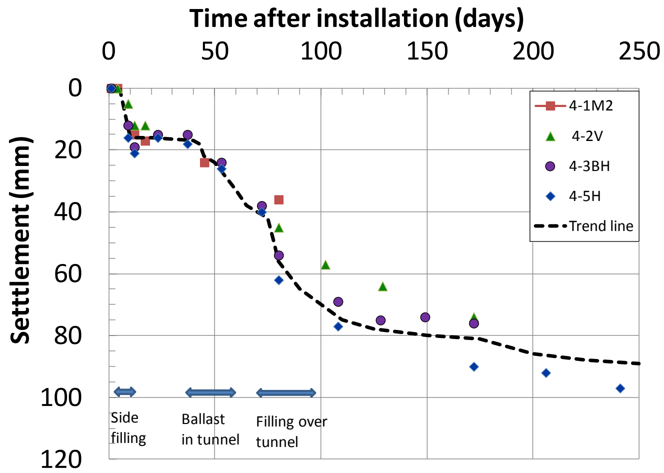


*Figure 15- Example of measured settlements with time for Element 4 during the first 150 days after the element was installed*

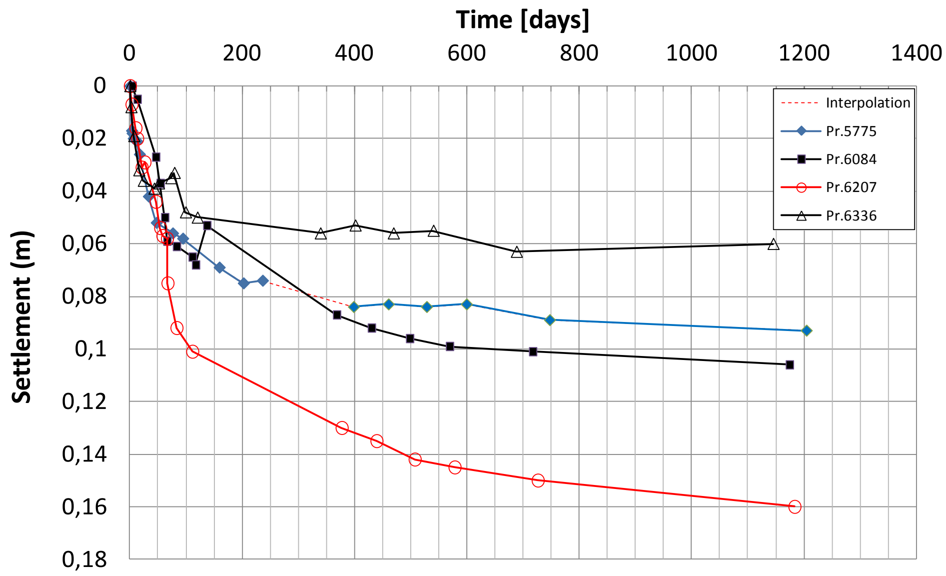


*Figure 16- Examples of measured settlements with time over the complete 1200 day observation period*

## CONCLUDING COMMENTS

The settlements measured along the immersed tunnel broadly follow what was predicted or expected at the design stage. It is in particular worth noticing that the settlements towards the end of the 1200 day long observation period so far tend to level off, much in agreement with the predicted rate of re-consolidation type settlements. The vertical displacements registered so far along the tunnel imply differential movements well below what is acceptable for the tunnel elements and the joints in the tunnel.

The performance monitoring program has also included measurements of displacement in the longitudinal and transverse direction of the tunnel axis, and displacements over the tunnel joints. All of these displacements have so far been very small and well below tolerance limits

As the tunnel is in an area with frequent city and harbour development it is necessary for the owner to monitor the tunnel movements. The tunnel will therefore also be monitored on a regular basis in the years to come.

The authors would finally like to thank the many colleagues that have taken part in the design of this immersed tunnel, and not the least the Norwegian Public Roads Administration for allowing this paper to be published.

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http://www.itaaites.org/index.php?id=75&tx_ttnews%5Btt_news%5D=33&cHash=19d6b88689d67b82fd9d1a04c3946f90

# RV. 13 RYFAST – WORLD'S LONGEST SUBSEA ROAD TUNNEL COMBINED WITH E39-EIGANES TUNNEL

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## ABSTRACT

***Rv. 13 – Ryfast***
*The Rv. 13 Ryfast project is the undersea tunnels which will connect the city of Stavanger to the region of Ryfylke, when completed the tunnels will replace the existing vehicle ferries which operate between Stavanger and Tau, and between Lauvvik and Oanes. Currently the combined daily traffic volume for both ferry routes is 4000 vehicles (AADT). Ryfast consists of two dual tube subsea tunnels, The Hundvåg tunnel, from the new E39 Eiganes tunnel to the Island of Hundvåg, and the Solbakk tunnel, which extends from the Hundvåg tunnel to Strand in Ryfylke. Construction time will be 5.5 – 6 years with expected opening in 2018/19.*

***E 39 – Eiganes Tunnel***
*Today's E39 bottlenecks in Stavanger, with traffic routed along heavily congested local streets. The E39 Eiganes tunnel will be the new "North / South" trunk route through Stavanger, extending the existing motorway beyond the Stavanger central business district, and significantly reducing local traffic above. The primary route for the project is 5 km long, comprised of dual tunnels 3.7 km long, and 1.3 km of ground level, two lane dual carriageway. In addition to the primary North and South Bound tunnels, the project will also include tunnelled on and off ramps, and the connection to the Ryfast Hundvåg tunnels. As a result of geometric constraints, and the existing topography a significant portion of the Eiganes tunnel will have 15-25 m of rock cover to the surface.*

*As the south western connection to the Ryfast project, theE39 Eiganes tunnel will be constructed and managed jointly with the Ryfast project. Together the Ryfast and Eiganes tunnel projects will produce 3.8 million m$^3$ of rock, and finding a suitable use for this material has been a*

*challenge, the solution has been to utilise this material for land reclamation at a number of sites adjacent to the project. Further project details and challenges will be discussed in the paper.*


*Figure 1* *Map showing location of the Ryfast project*

## GENERAL INFORMATION

*Project approved by the Norwegian parliament 12. June 2012.*

**Norwegian Public Roads Administration role:**
The Norwegian Public Roads Administration (Statens vegvesen or SVV) will manage the project through a traditional client role. As a result of the scope and size of the project, the management structure includes two sub-project managers for each of the two projects. They will be assisted by Site managers and inspection engineers, making sure that the contractors follow the required procedures and maintains a high focus on the health, safety and environment.

In addition to contract and project management personnel, the project organisation also includes a support team for communication, neighbour relations, environment, safety and a variety of additional tasks that’s needed to secure the overall quality of the project

**Construction method:**
In the early phases of the planning, alternative tunnelling options were assessed and evaluated. Among the assessed methods were different types of conventional drill and blast tunnelling (tunnels in Norway are almost exclusively constructed using drill and blast methods) and the use of tunnel boring machines.

Given the proximity of the tunnelling work to older urban areas, and a desire to secure an efficient and flexible construction program, conventional drill and blast tunnelling was seen as the best fit for the project.

**Contract terms/choosing the contractor:**
The contracts will be announced as fixed price contracts, with price, experience and methodology / program as the key selection criteria.

**Rough estimates:**
In total there's needed between 10 000 to 15 000 blasts to finish the projects. In urban areas these blast will be adjusted to give a minimal impact on peoples comfort. An equivalent of 15 000 truckloads of sprayed concrete will be used together with approximately 250 000 bolts to keep the tunnel secured during construction. There will also be extracted a total of 3.5 – 4 mill. m$^3$ of excavated rock from the tunnels. Finding the best solution with utilising the surplus rock has been a great challenge to the project, the geological properties / problems with the material make it unsuitable for use in road construction, but it can be used for land reclamation, and together with relevant local stakeholders the project has identified 3 locations where the material will be utilised. Upon completion of the project, we will have approximately 50 km of new roads, with the asphalt, concrete, pipes and cabling that goes with it, together with new areas for industry and business as well!

## RV. 13 RYFAST

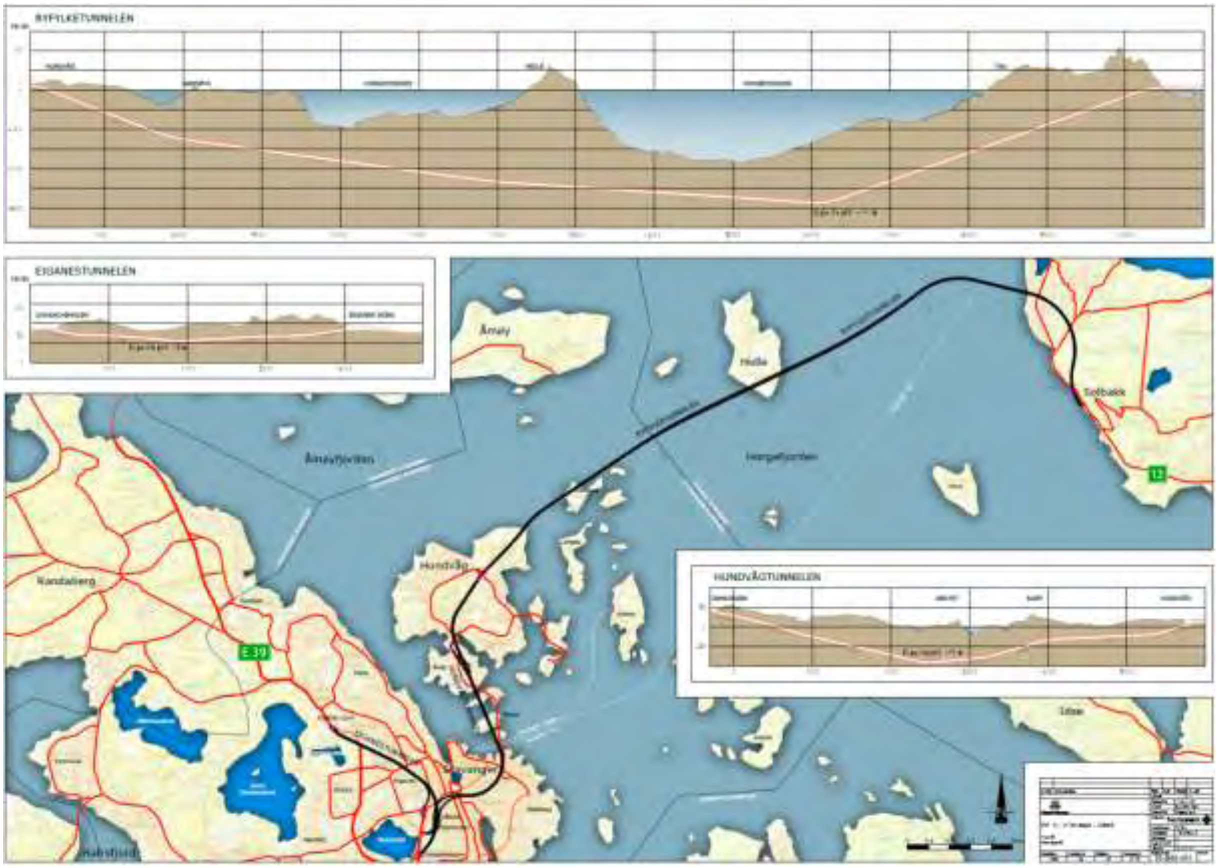

*Figure 2 Plan view and longitudinal profiles of the Ryfast and Eiganes tunnels*

Rv 13 Ryfast will connect Stavanger eastwards with the region of Ryfylke, replacing the ferry connections between Stavanger and Tau, and between Lauvvik and Oanes. Currently the combined daily traffic volume for both ferry routes is 4000 vehicles (AADT). Preliminary construction works commenced early in 2013, with tunnelling programmed to begin in August 2013. General details for the Rv 13 Ryfast project are as follows:

**The tunnels**
Ryfast consists of two "dual tube" tunnels, the Solbakk tunnel from Solbakk in Ryfylke, to the island of Hundvåg in the Stavanger municipal area, and the Hundvåg tunnel which extends from Hundvåg to an underground (tunnelled) motorway interchange with the E39 Eiganes tunnel. The Solbakk tunnel consists of two parallel tubes, stretching 14.3 km in length and reaching a depth of 290 meters below sea level. Traffic forecasts predict that upon opening in 2019, the Solbakk tunnel will carry 5000 vehicles (AADT) daily. Each of the tubes will be 8.5 meters wide, with emergency access tunnels connecting the main tunnels every 250 meters.

The Hundvåg tunnel is 5.5 km long and 95 meters deep. In addition to providing a link from the Solbakk tunnel to the E39 and Stavanger, the Hundvåg tunnel will provide an alternative route for commuters to the existing "Bybrua" bridge. Traffic forecasts predict that upon opening in 2019, Hundvåg tunnel will carry 10000 vehicles (AADT) daily. The standard width for the tunnel tubes in the Hundvåg tunnel is 9.5 meters.

**Funding**
The Ryfast tunnels are estimated to cost NOK 5.5 bn (Euro 0.65 bn), of this amount 90% of the financing will be provided by the Norwegian government, and repaid via tolls, with the remaining 10% provided by the local/municipal and county (Fylke) governments.
Today the toll for the Solbakk tunnel is estimated to be NOK 230 (30 Euros) for light vehicles, and NOK 30 (4 Euro) for light vehicles in the Hundvåg tunnel, the toll levied against trucks / heavy vehicles will be marginally higher.
The estimated toll fee makes the project the most expensive in Norway for road users.

Find more about the project at: http://www.vegvesen.no/Vegprosjekter/ryfast.
**E 39 EIGANES TUNNEL**


*Figure 3 Above map shows Eiganes North, the Eiganes Tunnel and the connection to Rv 13 Ryfast through The Hundvåg Tunnel*

Today's E39 bottlenecks in Stavanger, with traffic routed along heavily congested local streets. The E39 Eiganes tunnel will be the new "North / South" trunk route through Stavanger, extending the existing motorway beyond the Stavanger central business district (CBD), and significantly reducing local traffic above.

The project is 5 km long, and is comprised of dual tunnels 3.7 km long, and 1.3 km of ground level, two lane dual carriageway. Similar to the Hundvåg tunnel the standard width of the tunnel tubes is 9.5 meters. Traffic forecasts predict that upon opening in 2019, the Eiganes tunnel will carry approx. 20 000[5] vehicles (AADT) daily

Estimated cost for the project, including tunnels and surface road construction is NOK 2.2 bn. (250 million Euros). The Norwegian government will provide the funding for the Eiganes tunnel, with 70% of the cost to be repaid via tolling from the existing "North Jæren Toll Road Ring".

The E39 Eiganes Tunnel will be constructed concurrently with Ryfast, under the same project management. Construction time for the entire project, Ryfast and Eiganes tunnel is planned for 5.5 – 6 years, with the expected opening in 2019.

[5] It must be noted that this forecast is currently under review, and should only be treated as a rough estimate

Find more about the project at**:** [http://www.vegvesen.no/Vegprosjekter/e39eiganestunnelen](http://www.vegvesen.no/Vegprosjekter/e39eiganestunnelen)

## GEOLOGY

Geologically, this part of Rogaland is comprised mainly of Precambrian autochthon / autochthonous rock units of Caledonian covers. The Ryfast tunnel will be constructed through the overlying Allochthonous / long-Caledonian rock layers, consisting mainly of phyllite and gneiss which was pushed beneath the Caledonian orogeny for approx. 350 million years ago. From previous experience tunnelling in this region the project expects to encounter small leaks while tunnelling through the phyllite, both below the ocean floor and on land.
By way of a local comparison, there were small leaks during the construction of the Mastrafjord tunnel (Rennfast project, completed in 1993). According to geology reports, these areas are geologically similar to the Solbakk tunnel. The areas where the sediment is thinest are considered to pose the greatest risks of leakage along the Solbakk tunnel. These areas are located in Horgefjorden, close to Hidle. (Multiconsult, 2009). The project has in place proven work methodology to rectify any leaks encountered during construction.

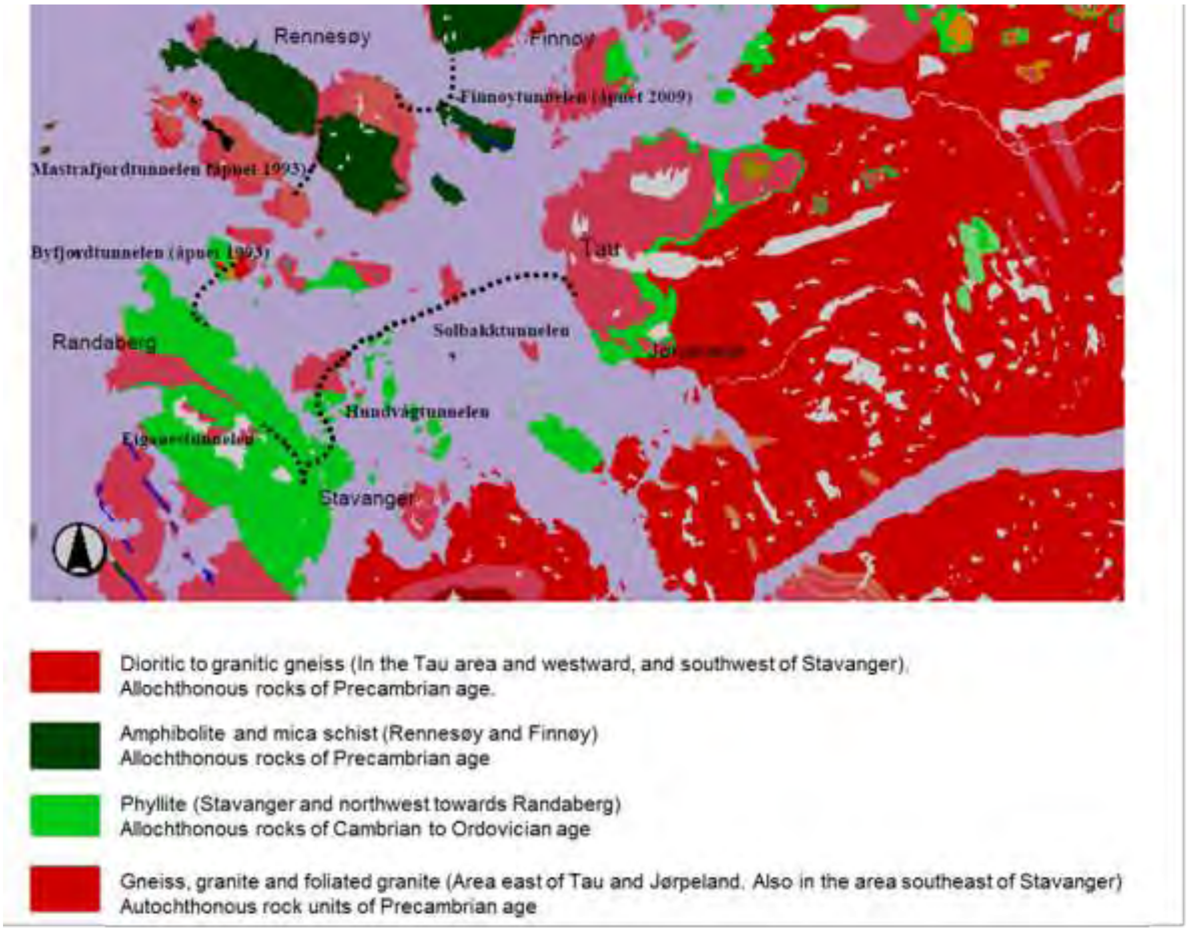


*Figure 4 Geological map of the tunnel area*

## CONTRACTS

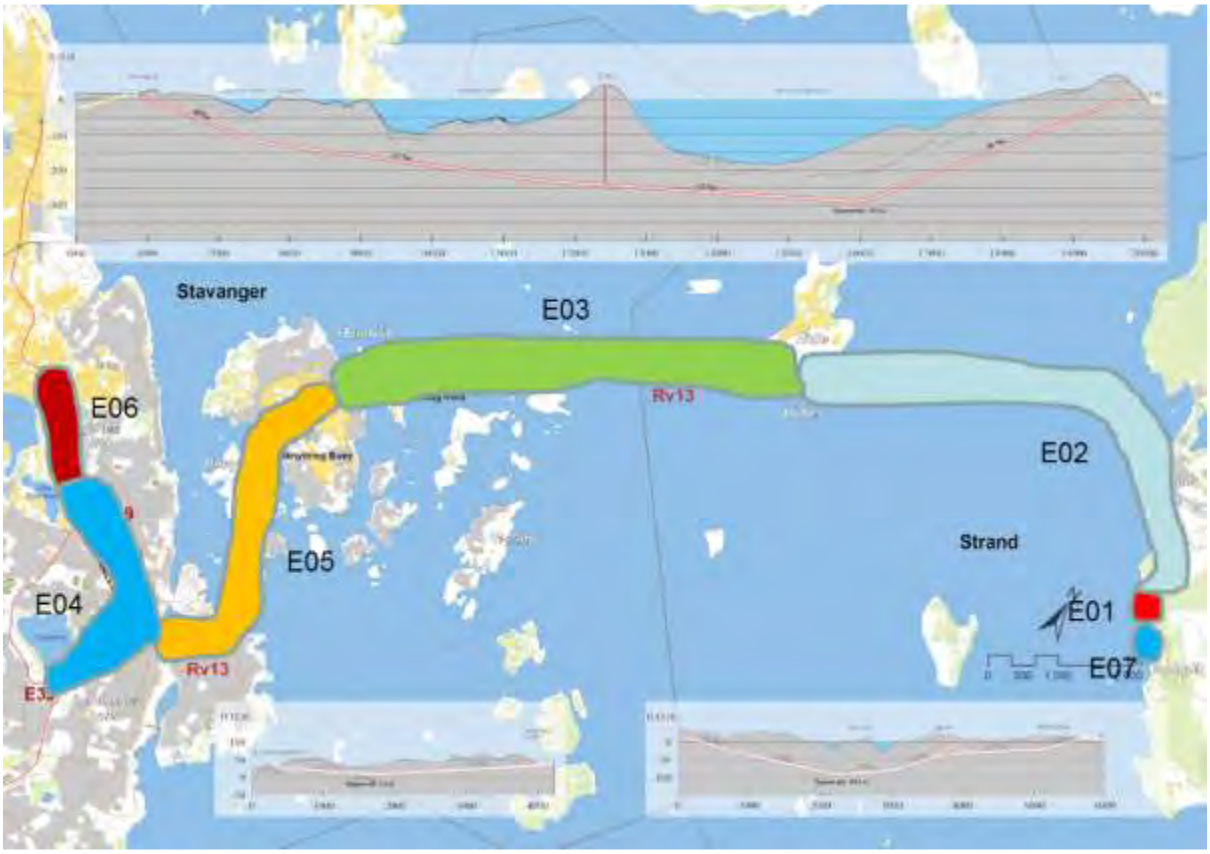


*Figure 5 Allocation of various contract sections*

**E01**: Preliminary work at Solbakk

The first construction contract for the Ryfast tunnel was signed on 9 November 2012, when a local contractor was awarded the tender for preliminary earthworks and stabilisation for the tunnel portal at Solbakk in the Strand municipality. These works were completed on 22 April 2013.

*Figure 6 3D-illustration of the entry point of the Solbakk tunnel at Solbakk*

**E02**: 7.6 km of Solbakktunnelen from Solbakk

On the 30 April 2013, the first of the major tunnelling contracts was awarded to Marti Contractors Ltd, IAV and Marti Norway AS. This contract covers the civil & tunnelling works for the first 7.6 km of the Solbakk tunnel, from Solbakk.

**E03**: 6.7 km of the Solbakktunnelen from Hundvåg

As of May 2013, the second major tunnelling contract, for the remaining 6.7 km of civil/tunnelling works in the Solbakk tunnel, from Hundvåg together with the construction of the Hundvåg interchange, and portal for the Hundvåg tunnel is out for tender.

**E04**: The Eiganes tunnel with parts of the Hundvåg tunnel

Stage 4 of the project, The Eiganes tunnel including the Hundvåg tunnel interchange, is expected to be the largest contract in the project. This contract includes all of the tunnelling for the Eiganes tunnel, the tunnelled interchange, and connection to the Hundvåg tunnel, additional on and off ramps for both tunnels, and the reconfiguration of the existing motorway to provide the southern approach to the tunnel, and as a connecting route to both tunnels, and the E39 southbound, from the Stavanger CBD and western parts of the city.

This stage of the project will pose a variety of challenges successful contractor, and is currently programmed to be tendered out in Summer/Autumn 2013.

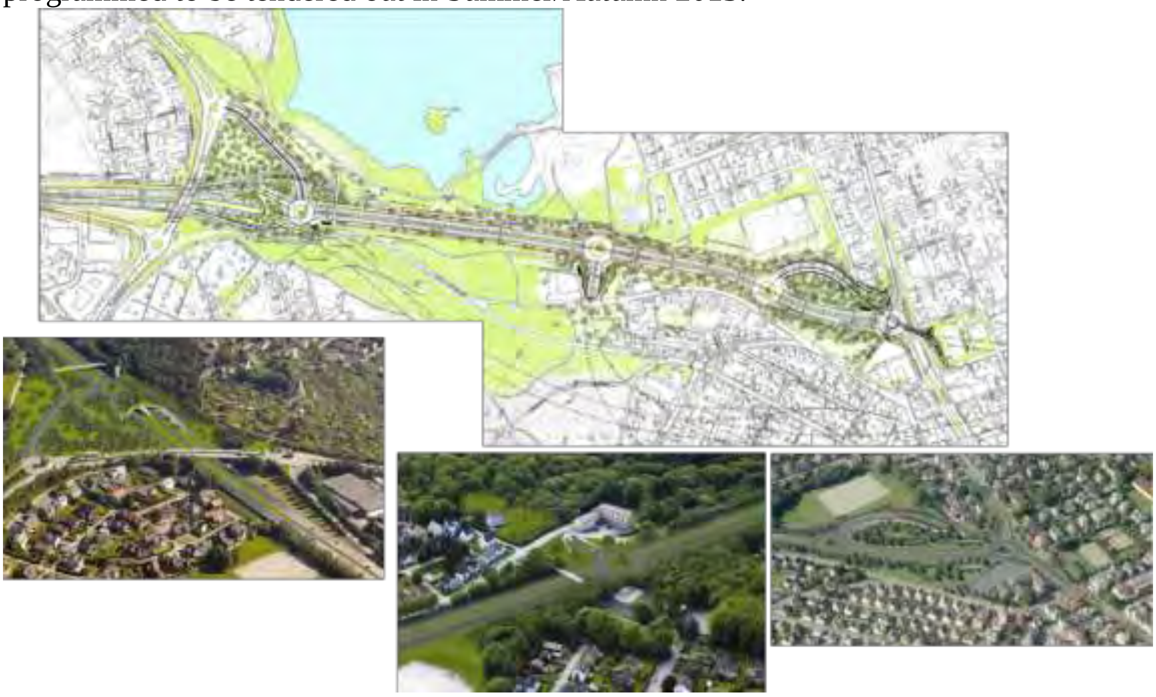

*Figure 7 Early project sketches from the development phase*

**E05**: Hundvågtunnelen

The last of the tunnelling contracts is stage 5 of the project; The Hundvåg tunnel. This contract involves tunnelling of on and off ramps for the Hundvåg tunnel from the Island – Buøy, and the tunnelling of the Hundvåg tunnel to Stavanger, and to Hundvåg from Buøy. Currently this stage of the project is planned to be tendered out in winter 2013/2014.

**E06**: Eiganes north

Stage 6 of the project, the northern approaches and motorway interchange for the E39 Eiganes Tunnel is currently planned to be tendered out in winter 2014/2015. This works package includes upgrading of the existing highway to Motorway standard, as well as the construction of a number of bridges. During construction of this stage of the project the contractors will need to maintain existing traffic, construct 4 lanes of motorway (2 Northbound, 2 southbound) and provide for construction traffic from Eiganes tunnel. As a consequence of the accelerated program for the construction of the Rogfast project, the final scope of this package of works is currently being revised to optimise the construction of both projects.

**E07**: Mosvannet Culvert / Land Bridge (which has now been included in E04 – Eiganes tunnel)

*Figure 8 Southern portals to the Eiganes tunnel, and the Mosvannet Culvert (Land Bridge) north of the portal.*

In addition to the contracts presented, there will be separate contracts for electricity, pumps, ventilation and control systems.

**CONCLUSION**

In summary, the project is a much needed measure for the people of Stavanger and inner Ryfylke. This infrastructure will improve traffic flows through the city, reduce congestion, and provide for the growth of the city of Stavanger. While at the same time providing a safe, reliable, all-weather route between Stavanger / North Jæren and Ryfylke, which will only enhance the economic development of the region as a whole!

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# METHODOLOGY FOR PREDICTING AND HANDLING CHALLENGING ROCK MASS CONDITIONS IN HARD ROCK SUBSEA TUNNELS

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## ABSTRACT

*The most challenging rock mass conditions in hard rock subsea tunnels are represented by major faults/weakness zones. Poor stability weakness zones with large water inflow can be particularly problematic. At the pre-construction investigation stage, geological and engineering geological mapping, refraction seismic investigation and core drilling are the most important methods for identifying potentially adverse rock mass conditions. During excavation, continuous engineering geological mapping and probe drilling ahead of the face are carried out, and for the most recent Norwegian subsea tunnel projects, MWD (Measurement While Drilling) has also been used. During excavation, grouting ahead of the tunnel face is carried out whenever required according to the results from probe drilling. Sealing of water inflow by pre-grouting is particularly important before tunnelling into a section of poor rock mass quality. When excavating through weakness zones, a special methodology is normally applied, including spiling bolts, short blast round lengths and installation of reinforced sprayed concrete arches close to the face. The basic aspects of investigation, support and tunnelling for major weakness zones are discussed in this paper and illustrated by cases representing recently completed, very challenging projects (Atlantic Ocean tunnel and T-connection) and projects planned to be built in the near future (Ryfast and Rogfast).*

## INTRODUCTION

During the last three decades more than 40 subsea rock tunnels have been built along the coast of Norway. Most of these are road tunnels, with the 7.9 km long Bømlafjord tunnel as the longest, and the Eiksund tunnel as the deepest, with its lowest section 287 m below sea level. Some subsea tunnels have also been built for the oil industry as shore approaches and pipeline tunnels, and some for water supply and sewerage.

Extensive site investigations, with offshore acoustical profiling, refraction seismics and in most cases also core drilling in addition to conventional desk studies and onshore mapping, are always carried out for the subsea tunnels. In addition, extensive investigations during excavation are carried out. In many cases, excavation of the Norwegian subsea tunnels has been completed without major problems related to the ground conditions. In difficult ground conditions, tunnelling challenges have in most cases been tackled efficiently by extensive investigation from the tunnel face and well planned procedures for excavation and rock support. The most difficult rock mass conditions in the Norwegian hard rock subsea tunnels have been represented by major faults/weakness zones with large water inflow.

This paper will discuss the challenges related to identifying zones of adverse rock mass conditions at the investigation stage, and methodology for tunnelling through such ground conditions, based on experience from the Norwegian subsea tunnel projects. For illustration, two relevant, recent cases (Atlantic Ocean tunnel and the T-connection) will be discussed in some detail, and two future very long and very deep subsea tunnel projects (Ryfast and Rogfast) will be briefly described. The paper is to a great extent based on the authors' experience from quality control and as members of expert panels for many subsea projects.

## PRE-CONSTRUCTION INVESTIGATIONS

The main pre-construction investigations for a subsea tunnel are:

1) Desk study
2) Onshore engineering geological mapping
3) Reflection seismics
4) Refraction seismics
5) Core drilling

The desk study, including review of geological maps, reports, aerial photos and experience from any nearby projects, represents the important first step of the investigations. The desk study is also important for the planning of further investigation of the project area. The onshore mapping includes conventional geological mapping to determine rock types, major geological structures, such as faults, dikes, lithological contacts, and any other features that may represent major weakness zones in the planned tunnel area, but has main focus on the following important engineering geological factors:

- Rock types; character, distribution and strength.
- Weakness zones/ faults; location, orientation and character. Each zone is evaluated and described individually.
- Jointing; including orientations of main joint sets, spacings, continuity, roughness and coating/filling (gouge material).

From the collected engineering geological information a site engineering geological model is developed. Samples are taken for laboratory testing of physical and mechanical properties. To avoid the effect of weathering in samples taken in outcrops, some blasting is often necessary.

Reflection seismic investigation (often referred to as acoustic profiling) is used for finding the depths to different geological layers (reflectors), including the depth to the bedrock surface where it is covered by loose deposits. The bedrock may be located below as much as 200m of sediments. The main target for this type of survey is to get an overall view of the soil distribution in the area to produce a map of the rock surface. These maps are of great importance for identifying favourable corridors for subsea tunnel crossing. Refraction seismic results are used for "calibration" of estimated sonic velocities.

Refraction seismic investigation is performed by positioning a cable with hydrophones on the sea bottom and detonating small charges of dynamite. Based on monitoring the arrival time of the refracted waves, the thickness of soil cover and sections of different sonic velocities are identified as illustrated in Figure 1. Interpretation of seismic velocities and thickness of the various layers is a complex process, and a great deal of operational experience is required for the results presented in a profile to be regarded as reliable.

In addition to the variations of the rocks, the in situ seismic velocities in rock masses depend on:

- The rock stresses; causing a general increase of seismic velocity with depth. Thus, direct comparison of velocities at the surface and at the tunnel level is not realistic.
- The degree of jointing; representing an important factor in interpretation of refraction seismic measurements to assess the block size.
- The presence of open joints or joints with filling.
- The presence of faults and weakness zones

Thus, seismic methods do not automatically give high quality results for all geological environments. Seismic velocities higher than 5,000 m/s generally indicate good quality rock masses below the water table, while the poor quality rock mass of weakness zones have velocities lower than 4,000 m/s. In some cases seismic velocities lower than 2,500 m/s, corresponding to the velocity of moraine, have been monitored for weakness zones.

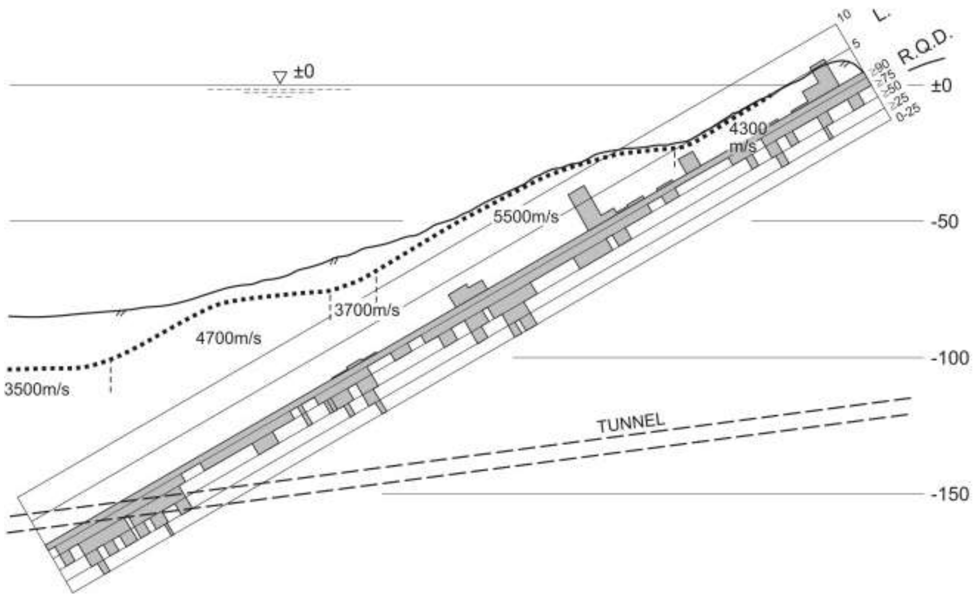


*Figure 1. Example illustrating the use of seismic investigation and core drilling for planning of subsea tunnel. The dotted line represents interpreted rock surface based on reflection and refraction seismics. The velocities of the various sections (3,500-5,500 m/s in rock and about 1,700 m/s in soil) are based on refraction seismics. RQD and Lugeon-values (L) are shown along core drill hole.*

Core drilling is used to obtain geo-information from volumes of rock masses that cannot be observed, and is often used in combination with geophysical measurement as shown in Figure 1. In most cases for subsea tunnels, core drilling is carried out from the shore as illustrated in the figure, but in some cases it is also carried out as directional drilling. In a few cases, when this has been considered necessary to prove the feasibility of the project, core drilling is also carried out from drill ships.

The purpose of a core drilling investigation is to:

- Confirm the geological interpretation.
- Obtain information on the rock types and their boundaries in the rock mass.
- Obtain more information of the rock mass structure.
- Study ground water conditions.
- Provide samples for laboratory testing and petrographic analyses.

In hard rocks dominated by discontinuities, core drilling is often carried out to study certain larger faults or weakness zones which are assumed to determine the stability and ground water conditions of the tunnel. The drillholes will, however, also give additional information where they penetrate the adjacent rock masses.

Considering the high cost of good quality core drilling, it is important to spend sufficient time and money for high quality core examination and reporting, including high quality photographs of the cores.

## INVESTIGATIONS DURING EXCAVATION

Even the most extensive pre-construction investigations cannot reveal all details regarding rock conditions. Some degree of uncertainty will still remain when tunnelling starts. To avoid any "unexpected conditions", and at all times have good control, systematic probe drilling during tunnelling is very important. Probing is normally done as percussive drilling by the tunnel jumbo. A common number of holes for probe drilling under water are 3-5, and the holes are drilled according to procedures as shown in Figure 2.

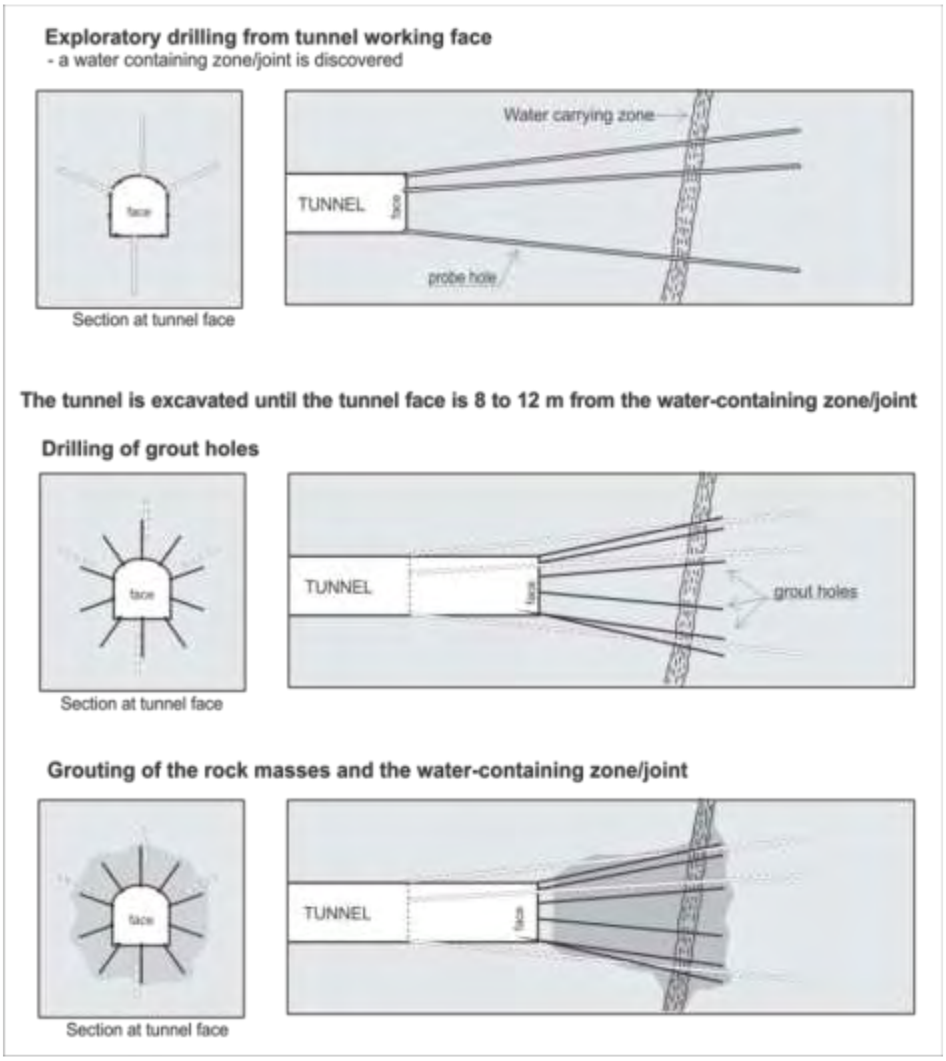


*Figure 2. Principles of probe drilling and pre-grouting. Typical length of probe drilling holes is 25-30 m, and the overlap is typically about 5 m.*

The most difficult rock mass conditions often occur in the fault zones at the deepest part of the tunnel. Any uncontrolled major water inflow here may have severe consequences. In such sections of the tunnel, core drilling is sometimes used for probe drilling.

Probe drilling also has the very important purpose of providing the basis for decision whether to grout or not as described in the next section of this paper.

In addition to probe drilling, continuous follow-up at the tunnel face by well qualified engineering geologists and rock engineers is of great importance. In Norwegian tunnelling this has become more and more realized, and time for such follow up is today included in the contract.

For the more recent projects, MWD (Measurement While Drilling) and DPI (Drill Parameter Interpretation) have been applied for predicting rock mass conditions ahead of the tunnel face. Three main factors describing the rock mass conditions are normally defined by this approach; rock hardness (strength), degree of fracturing and water conditions. An example illustrating the potential of MWD/DPI for estimating rock strength ahead of the face is shown in Figure 3.

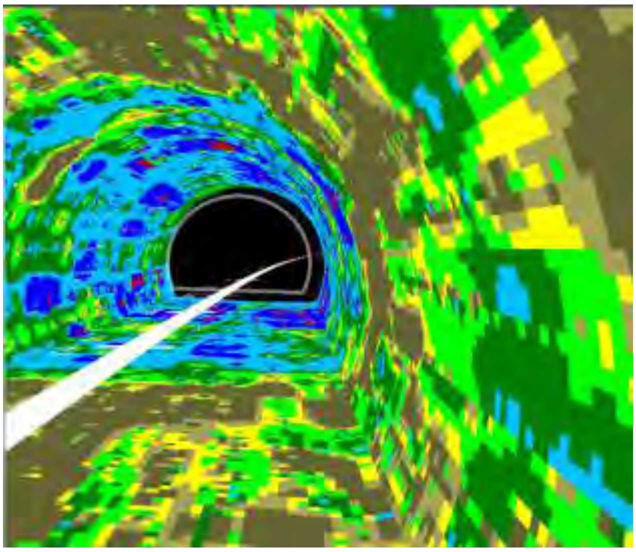

*Fig. 3. MWD/DPI-interpretation of rock hardness for section of the T-connection subsea road tunnel. Red and blue represent hard rock, yellow and brown weak rock (from Moen, 2011).*

Use of MWD/DPI has a great potential for predicting rock mass conditions ahead of the tunnel face. The method is however still at the development stage, and interpretation of data is often uncertain. As basis for the decision on whether to pre-grout or not, measurement of water inflow in probe drill holes as described above is therefore still the preferred method.

## METHODOLOGY FOR EXCAVATION IN DIFFICULT ROCK MASS CONDITIONS

All Norwegian subsea tunnels so far have been excavated by drilling and blasting, which provides great flexibility for varying rock mass conditions and is cost effective. The 6.8 km North Cape tunnel (completed in 1999) was considered for TBM, but also in this case drilling and blasting (D&B) was chosen as the final method. A main reason for not choosing TBM was that the risks connected to potential water inflow were considered too high. During tunnelling, water inflow was not a main problem. The main problem turned out to be thinly bedded rock causing stability problems in the D&B drives, which due to the uniform circular profile and less disturbance of the contour by TBM-excavation probably would have been less in a TBM drive.

Water sealing by pre-grouting is carried out when required according to criteria based on probe drilling. For a Norwegian subsea road tunnel today a maximum inflow of 3 l/min for one probe drill hole and a total of 10 l/min for 4 holes are typical action values for pre-grouting. By applying such criteria, the remaining inflow can be controlled and adapted to preset quantities for economical pumping (normally a maximum of 300 litres/min·km).

Grouting, when required according to probe drilling, is always carried out as pre-grouting in drillholes typically about 25 m ahead of the face, and with 2 blast rounds overlap. This procedure has been successful even in the deepest of the Norwegian subsea tunnels where grouting against water pressures of 2-3 MPa has been efficiently done with modern packers, pumps and grouting materials. Grouting pressures up to10 MPa are today quite common with modern grouting rigs as shown in Figure 4.

*Figure 4. High pressure pre-grouting with modern grouting rig.*

For rock support, a combination of fibre reinforced shotcrete and rock bolting is most commonly used. In good quality rock, spot bolting is sometimes considered sufficient, while in poorer quality systematic bolting is most common.

In difficult ground conditions spiling bolts are used, and sometimes also reinforced shotcrete ribs as shown in Figure 5. When the conditions are particularly challenging, reduced round length (down to 1-2 m instead of the conventional 5 used in good rock) and stepwise excavation of the face are applied. The trend today is that shotcrete ribs (sometimes supplemented with concrete invert) are used in poor rock conditions instead of concrete lining.

All rock support structures are drained, whether they are made of cast-in-place concrete lining, shotcrete ribs or shotcrete/rock bolting. Shotcrete in subsea tunnels today is most commonly applied as minimum 8cm thick, wet mix, polypropylene (PP) fibre reinforced.

Rock bolts have extensive corrosion protection. The preferred bolt type is the CT-bolt, which provides multiple corrosion protection by hot-dip galvanizing, epoxy coating and cement grouting applied on both sides of a plastic sleeve, and thus provides excellent corrosion protection for the subsea sections.

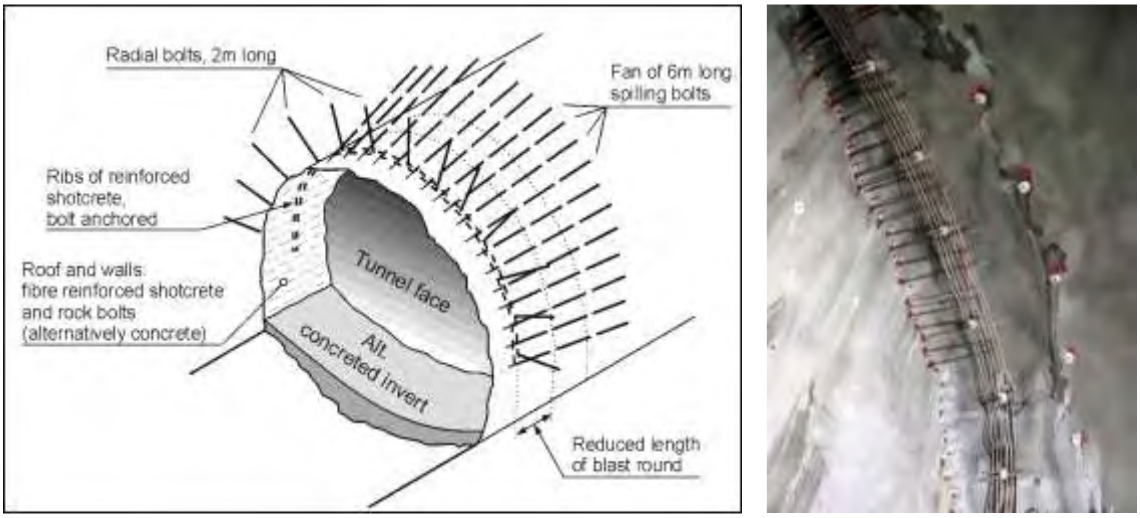


*Figure 5. Principle for excavation through poor stability weakness zones based on short round lengths, spiling and reinforced ribs of sprayed concrete (left, based on NFF, 2008), and photo illustrating spiling and shotcrete ribs in tunnel with heavy support also of the face (right).*

## CASE EXAMPLES

### Recently completed projects

To illustrate the very challenging rock mass conditions that may in some cases be encountered in subsea tunneling, and the way the problems may be solved, two relevant, recent cases will be briefly discussed; the T-connection and the Atlantic Ocean tunnel.

*The T-connection*

The “T-connection” represents a part of the future road connection between Haugesund and Stavanger on the SW coast of Norway. The tunnel system consists of 2 main tunnels: the 3.39 km long Karmsund tunnel and the 3.76 km long Fördesfjord tunnel, and in addition a 1.16 km long tunnel branch to Helvik. The main tunnels have a span of 9.5 m with 70m$^2$ cross sectional area (profile T 9.5). A large roundabout in rock is excavated at the junction between the three tunnels. The deepest points in the two main tunnels are 139 m and 136m and the slope is 5.5% to 7.5%. The tunnels were excavared in 2009-2011, and the project opened for traffic in 2013.

Early in the 1980s, tunnels for a gas pipeline (Statpipe) were excavated parallel with the T-connection tunnels only about approx. 1km further to the south, see Figure 6. The experience from excavation and results from the investigations performed for these gas pipeline tunnels provided very valuable information for planning of the T-connection, especially for the deepest sections with expected very poor and problematic ground conditions as indicated in Figure 6.

Because of the very difficult ground conditions encountered in the Statpipe tunnels, and since no core drilling was carried out at the pre-construction stage for the T-connection, exploratory drilling ahead of the tunnel face was performed for almost all the tunnel length. No significant water inflows were encountered, and the extent of pre-grouting therefore was moderate and focused on sealing minor inflows.

As shown in Figure 6, the T-connection tunnels were excavated in greenstone/greenshist, sandstone, phyllite and gneiss. The degree of jointing was mainly moderate.  There were, however, many small weakness zones (fault and shears) and a few large. Still, the T-connection tunnels did not encounter quite as problematic rocks as the existing gas pipeline tunnel.

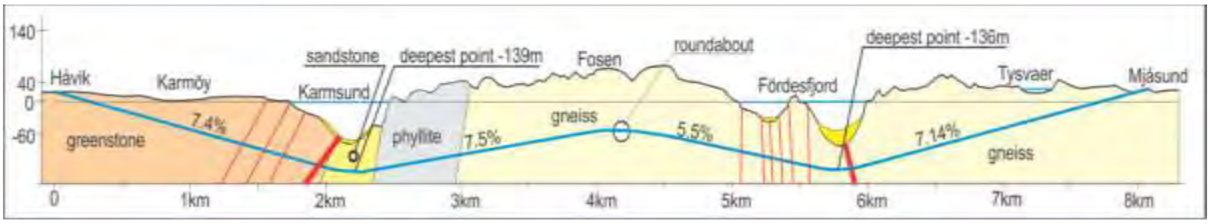


*Figure 6. Longitudinal profile with geology of  the T-connection subsea tunnels. Red lines indicate the main weakness zones encountered during tunnelling.*

Two large weakness zones (thick red lines in the profile in Figure 6) represented the most problematic tunnelling conditions.  Here, the blast round length was reduced from 5 to 3.5m, and 6-8 m long spiling bolts with 3m overlap were installed in roof and walls before blasting. Thick fibre reinforced shotcrete with rebar reinforced arches and rock bolts were used for work and permanent support.

*Atlantic Ocean tunnel*

The Atlantic Ocean tunnel, located on the central west coast of Norway, is 5.7  km long and has an excavated cross section of approx. 85 m$^2$. The tunnel was opened for traffic in 2009. A longitudinal profile along the tunnel is shown in Figure 7.

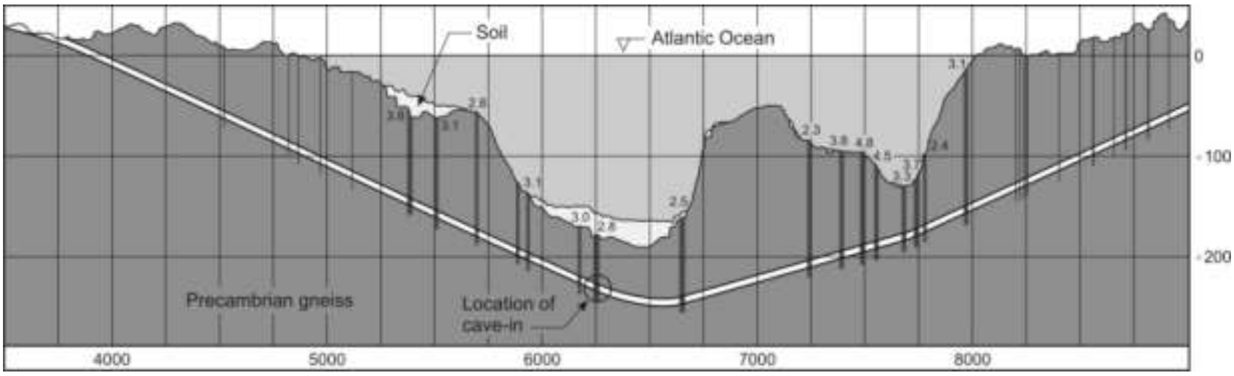


*Figure 7. Longitudinal profile of the Atlantic Ocean subsea road tunnel.  Assumed weakness zones/ seismic low velocity zones (with velocity in km/s) are indicated by vertical lines. Vertical scale is meter below sea level and horizontal scale is Station number in meters (modified after Karlsson, 2008).*

The bedrock is Precambrian granitic gneiss of mainly good quality. The conventional pre-construction investigations for this type of project were carried out, including reflection and refraction seismic investigations. Based on the latter, several low velocity zones, representing faults/ weakness zones under water were detected. Near the bottom of the planned tunnel zones with seismic velocities as low as 2,500 and 2,800 m/s were identified as shown in Figure 7. Based on overall evaluation of the rock mass conditions, a minimum rock cover of 45m was chosen, but it was realized that several of the low velocity zones under sea might be quite challenging, and this was taken into account in the planning of excavation and rock support.

Before entering a major zone at Station 6242, several nearby fault zones with seismic velocity down to 2.8-3.1 km/s, and even down to 2.4 km/s from the other side, had been crossed without major problems. These zones contained crushed rock and clay gouge, but very little water. Probe drilling indicated poor quality rock in the 2.8 km/s zone at St. 6242, but little water inflow. Thus, similar rock mass conditions as in the previous faults/weakness zones were expected. As extra precaution, the great water depth and limited rock cover taken into consideration, grouting was carried out in order to seal the joints and possibly also stabilize the zone material, and after that excavation was started with reduced round length (3m), shotcreting, systematic radial bolting and installation of 6m long spiling bolts.

The weakness zone proved to be of very poor quality, and after blasting the reduced round length there was a tendency of small rock fragments falling down between the spiling bolts. Attempts to stop this by applying shotcrete were unsuccessful, and after a few hours a 5-6 m high cave-in of the roof had developed, covering the full tunnel width and the 3m round length. Based on holes drilled later it was found likely that the cave in progressed about 10m above the tunnel roof.

In order to stabilize the tunnel, excavated material had to be filled up against the tunnel face and a more than 10 m long concrete plug was established to seal the tunnel. Probe drilling indicated considerable water leakage, and extensive grouting of the backfill material and the surrounding rock past the slide scar was required. Based on careful excavation with reduced round lengths, shotcreting/radial bolting and spiling with drillable rock bolts the tunnel face was re-established after 5.5 weeks at the same position as it was before the cave-in. Core drilling through the weakness zone showed that it was more than 25 m wide and had considerable water leakage.

Further tunnelling was based on a procedure including continuous pre-grouting, spiling, excavation with reduced round lengths/piece by piece, shotcreting/radial bolting and installation of reinforced shotcrete arches. The process was very time consuming due to extensive water leakages (up to 500 l/min in one single drill hole) at very high pressure (up to 23 bar). Tunnelling was continued approx. 20 m from the west side, and this position was reached about 10 months after the date of the cave in. The rest of the fault zone was excavated from the east side based on a similar procedure as described above.

More than 1000 tons of grout (mainly micro cement, but also standard cement and polyurethane) was needed to seal the leakages of the approximately 25 m wide fault/weakness zone. After completion of the tunnel in December 2009, the total leakage was only 500 l/min (or 88 l/min per km tunnel), which can be characterized as quite low for this type of tunnels.

## Planned projects

Several new, very long and deep subsea tunnel projects are scheduled to be built in the near future, including the Ryfast and Rogfast projects. These are located only about 30 and 20 km, respectively, south of the T-connection project, see Figure 8.

Ryfast includes two tunnels: the Solbakk tunnel and the Hundvåg tunnel, both with two tubes 12 m apart. Each tube will have a span of 9 m (70 m2 cross section) and cross passages for every 250 m. The Solbakk tunnel will be 14km long, and descend down to -290 m below sea level. It will pass through various gneisses, and several large weakness zones are expected. The Hundvåg tunnel will be 5.5 km long, with phyllites at the southern part, and gneiss in the rest. Construction will start in 2013 and planned opening of the link is in 2019.

The tunnels will be excavated by drill and blast. Difficult rock mass are to be expected for sections of the tunnel. It is estimated that 250,000 rock bolts and 100,000 m3 of shotcrete will be used for rock support, plus cast in place concrete lining in very poor ground conditions. The cost for the project is estimated at 5,500 mill. NOK.


*Figure 8. Locations of the planned Ryfast and Rogfast links.*

Rogfast, which is still at the pre-construction investigation stage, is also planned with two separate tubes, each with two lanes. Each tube will be about 25.5 km long and go down to a deepest level of about 385 m below sea level. The project is planned with connection approximately midway to the island Kvitsøy. The structural geology of the project area is very complex, with several major faults and thrust zones, and with phyllite as predominant rock type in south, gabbro and greenstone in the middle and gneiss in north. Ground investigation is particularly challenging because of the long sections under open, deep sea.

The conditions are expected to be very challenging for Rogfast, with several poor quality weakness zones as illustrated in Figure 9. Extensive investigations have been done already, but to further investigate the conditions under open sea, core drilling from drill ships at sea depths of up to 290 m is also planned to be carried out. The cost of the Rogfast project is estimated at 10,200 mill. NOK, and earliest start of construction is estimated to 2015.

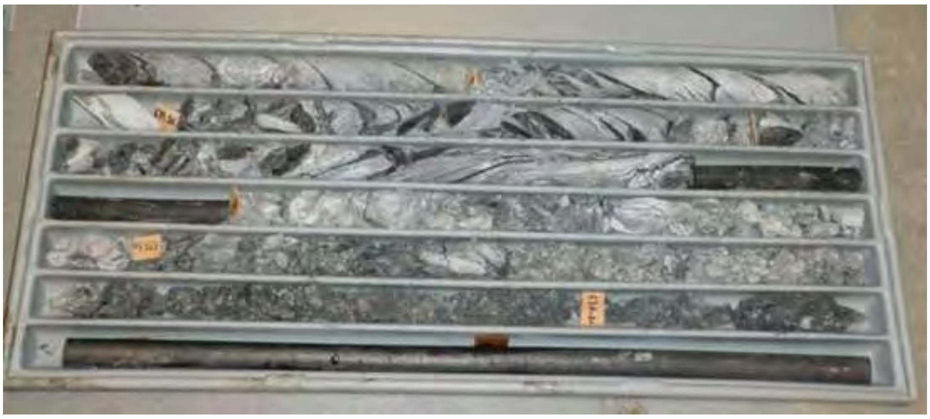

*Figure 9. Example of poor quality rock mass from core drilling at Rogfast (black, thinner sections are tubes representing core loss).*

## CONCLUDING REMARKS

This review of Norwegian projects illustrates that for subsea tunnels, even in hard rock, very challenging conditions are often encountered. The most difficult conditions are represented by major faults and weakness zones, particularly when very poor rock mass quality is combined with high water inflow. Even in such cases, the Norwegian projects have however demonstrated that with the technologies regarding pre-grouting, tunnelling and rock support which are available today, such challenges may be successfully coped with.

For any subsea tunnel project extensive, well planned and professionally performed pre-construction investigations, continuous investigations during tunnelling, appropriate procedures for excavation/rock support and high state of readiness are crucial. This applies even more for very challenging subsea tunnel projects like Ryfast and Rogfast which are planned to be built in the near future on the southwest coast of Norway. The long experience from the many completed subsea tunnels in Norway, and particularly the lessons learned from projects such as the T-connection and others, undoubtedly will have a great value for the planning and safe completion of these projects.

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# IMPACT ANALYSIS OF SUBMERGED FLOATING TUNNEL FOR CONCEPTUAL DESIGN

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## ABSTRACT

*Submerged floating tunnel is an innovative tunnel infrastructure passing through the deep sea independent of wave and wind so that high speed vehicle or train can run. It doesn't depend on water depth and is cost effective due to modular construction on land. The construction period can be reduced drastically. In this paper, a concept design of submerged floating tunnel is introduced and a method to analyze structural behavior of the body in case of collision with ships or submarines is proposed for securing safety. In this study, the local damage and global behavior of submerged tunnel in collision with submerged moving body are simulated via commercial hydrocode ANSYS LS-DYNA. In simulations, a conceptual tunnel section prepared in Korea is considered and various penetration and deformation responses with respect to impact velocity, applied materials and collision scenarios are obtained. Finally, for a conceptual design of submerged floating tunnel, maximum deformation, bending moment and impact forces are analyzed based on force and energy equilibrium.*

*Keywords* : Collision, Submerged floating tunnel, Penetration, Impact analysis, Hydrocode

## 1. INTRODUCTION

Recently, many submerged floating tunnel projects connecting inter-continents have seen the increase, especially those between Korea & China and Korea & Japan. The submerged floating tunnel technology can be used when developed to accommodate drive ways for vehicle and rails for train for 3 nations.

Submerged floating tunnel can maximize the use of undersea space and if a part of tunnel can be built transparent, it can be environment-friendly world class tourist attractions. Bridge is very difficult to build over the deep waters and is not easy to secure the safety over the sea lanes where there are heavy vessel traffics, floating debris or ices. However, submerged floating tunnel will be free from all those obstacles. There have been significant progresses made and technologies accumulated on the development of submerged floating tunnel technology from 1960s through 1970s, however, no such commercially operating tunnels are yet built. The technical gaps between domestic and foreign technology can be quickly reduced considering the level of construction technology sophistication of domestic firms. It is noted that time is rife for the domestic development of the technology over the technical licensing from overseas. Furthermore, it is desirable to develop those technologies employed in submerged floating tunnel within country which are also applicable to the building of offshore construction as gigantic sea platform construction.

This research to analyze the impact load and its characteristics was undertaken given consideration to those domestic industrial considerations. The existing structures for impact loads, except few cases, are not designed for with such details, and thus this study is done with consideration given to excess pressure loads. Ideally, tests are warranted for the performance and for movement of structures for impact loads, but due to its restrictions in both testing sites and measurements and for the lack of accuracy, movement analysis and its design are mostly conducted utilizing analytical tools.

This analysis was conducted using LS-DYNA, a commercial structure analysis program to analyze the impact loads for submerged floating tunnel.

Impact load is different from existing static analysis and dynamic analysis in that it takes place in a very short instance with big pressure loads, its response time is within the range of shock.

It thereby needs a new approach for other than existing analytical approaches. The study performed to design and for movement of structures when submarine collides with submerged floating tunnel and when the submarine sunk due to collision. This analysis was conducted under those realistic impact scenarios of submerged floating tunnel applying non-linear concrete material models.

## 2. IMPACT ANALYSIS OF SUBMARINE COLLISION IN OPERATION

### 2.1 Overview

This analysis has performed mesh modeling with specification of submerged floating tunnel described in Fig. 1. Partition wall has little effect in this analysis when it comes to resisting against actual impact loads. It is expected that when partition wall are built, it will have many configurations and therefore did not consider partition wall in this analysis as in Fig.2 and instead gave considerations to outer walls. In order to take into consideration of elliptical impact objects as submarines, the elliptical objects as in Fig.2 was modeled. And to take into consideration of elliptical objects as vessels and submarines for collision, the elliptical objects as 1,860 tons SON WON-IL class submarine (in Korea) with 50m length and 10m width has been selected shown in Fig.2 and was modeled with rigid solid elements. The impact velocity in operation was assumed to 5 m/s. The submarine was considered as a rigid body which did not permit any deformation. Considering the hydrodynamic behavior of submerged body, the added mass of both submarine and tunnel with approximate spherical shape was increased by 50%, respectively. Outer walls of tunnel are reinforced by rebar with 100mm cover depth on top and bottom and a reinforcement ratio of 0.2% was applied. 8 Node solid elements was used in modeling for impact structure and for tunnel and 2 Node Truss element was used for rebar modeling. Analysis used LS-DYNA v.971 and 72(*MAT_CONCRETE_DAMAGE) was used for concretes. The diameter of tunnel is 20m and thickness of wall is 1m and 100m for the length of tunnel. This analysis has performed mesh modeling with specification of submerged floating tunnel described in Fig. 1. Since partition wall is factored in the conceptual design phases for submerged floating tunnel, this modeling is attempted to model its typical cross section of tunnel's outer wall and partition wall in order to simulate as close as actual movements.

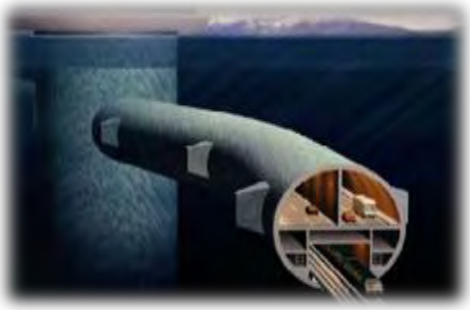


*Fig. 1 Trader submerged floating tunnel specification and planned tunnel modeling*

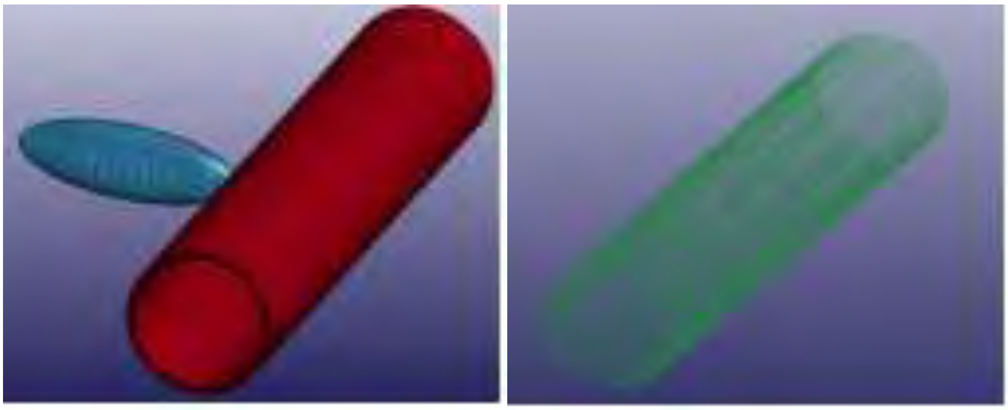
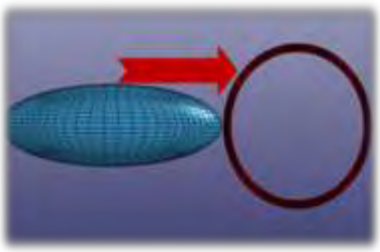


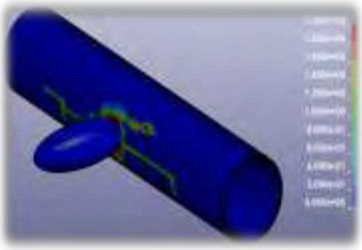
*Fig. 2 Submerged floating tunnel impact structure, tunnel (concrete) and rebar modeling*

### 2.2 Analysis results

Collision scenarios are categorized into 4 cases labeled to scenario A, B, C and D. Each scenario A, B, C and D is assigned to head-on impact, impact on middle of hemisphere, impact on top of hemisphere and sinking impact, respectively.

Fig. 4 and 5 show distribution of cracks and strains on tunnel when collided head-on with impact structures. It is indicated that the penetration failure of the submerged tunnel will take place in the event the tunnel is impacted with head-on collisions.





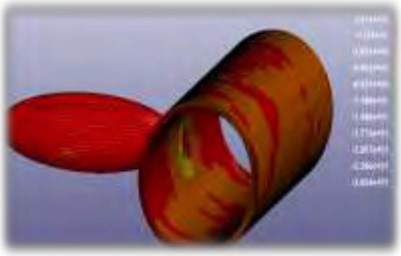


Fig. 3 Submerged floating tunnel impact scenario A (head-on collision)
Fig.4 Crack strain distribution after 1 sec impact
Fig 5. Principal stress distribution after 1 sec impact

Fig.6 shows the reduction of movement energy from its impact structure as it is progressing to the energy history of each impact scenario A~D.

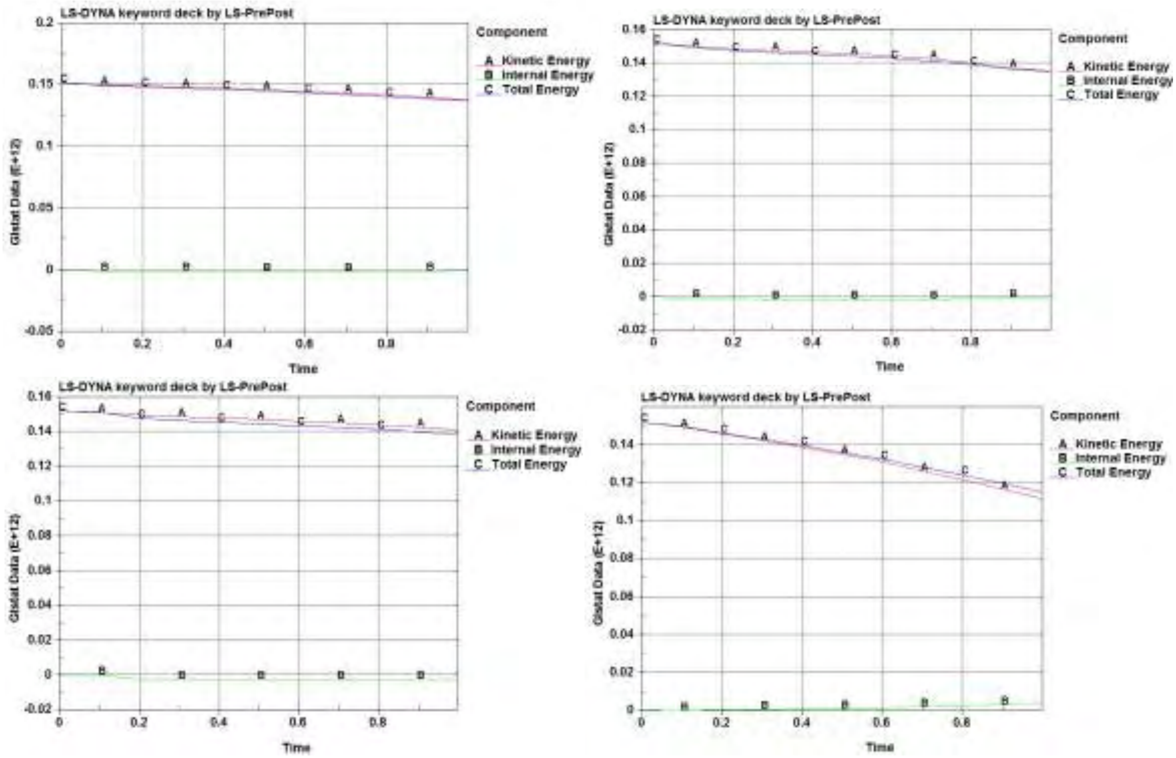
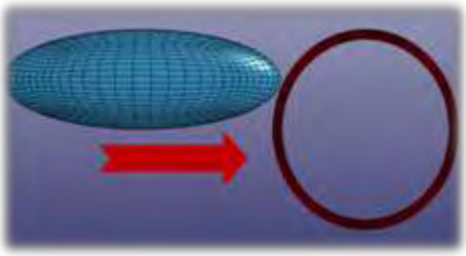
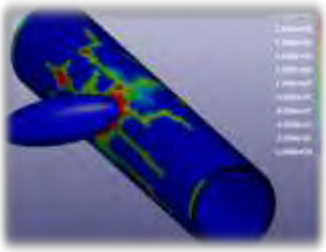



*Fig.6 Energy history of each impact scenario (A~D)*

Fig 8 and 9 indicate the distribution of cracks and strains on tunnel when collided with a structure on its sides. It is indicated that the penetration failure of the submerged tunnel will take place in the event the tunnel is impacted on middle of hemisphere. Fig.9 shows distribution of principal stress when impacted on its sides. On the whole, the stress when impacted shows that it does not exceeds the concrete pressure strength of 45MPa. Fig. B shows the reduction of movement energy from its impact structure as it is progressing to the energy history of impact scenario B.





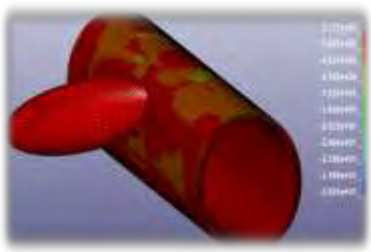


*Fig. 7 Submerged floating tunnel impact scenario B (impact on middle of hemisphere)*
*Fig.8 Crack strain distribution after 1 sec impact*
*Fig.9 Principal stress distribution after 1 sec impact*

Fig 11 and 12 indicate the distribution of cracks and strains on tunnel when impacted with structure on its sides. It is indicated that the penetration failure of the submerged tunnel will take place in the event the tunnel is impacted on top of hemisphere. Fig.13 shows distribution of principal stress when impacted on its sides. On the whole, the stress when collided shows that it does not exceeds concrete pressure strength of 45MPa.

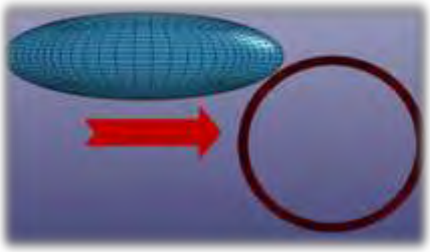
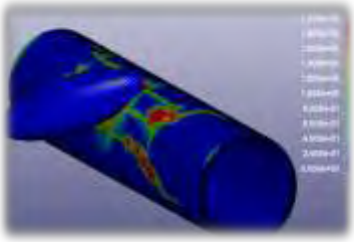
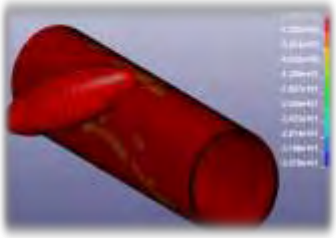




*Fig. 10 Submerged floating tunnel scenario C (impact on top of hemisphere)*
*Fig.11 Crack strain distribution after 1 sec impact*
*Fig.12 Principal stress distribution after 1 sec impact*

Fig 14 and 15 indicate the distribution of cracks and strains on tunnel when collided with sinking vessel structure. It is indicated that the penetration failure of the submerged tunnel will take place in the event tunnel is collided with such sinking vessel. Fig.17 shows distribution of principal stress when collided on its sides. On the whole, the stress when collided shows that it does not exceeds concrete pressure strength of 45MPa. Fig.6 shows the reduction of movement energy from its impact structure as it is progressing to the energy history of impact scenario D.

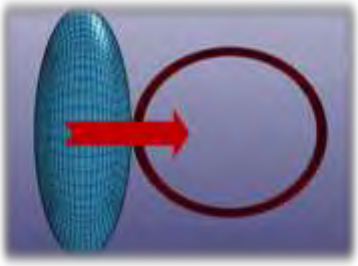
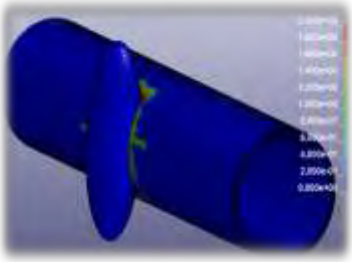




*Fig. 13 Submerged floating tunnel scenario D (sinking impact)*
*Fig.14 Crack strain distribution after 1 sec impact*
*Fig.15 Principal stress distribution after 1 sec impact*

## 3. IMPACT ANALYSIS OF SUBMARINE COLLISION WHILE SINKING

### 3.1 Overview

This analysis has performed mesh modeling with specification of submerged floating tunnel described in Fig. 1. Since Partition wall is factored in the conceptual design phases for submerged floating tunnel, this modeling is attempted to model its typical cross section of tunnel's outer wall and partition wall in order to simulate as close as actual movements. And to take into consideration of elliptical objects as vessels and submarines for collision, the elliptical objects as 18,750 tons Ohio class submarine with 100m length and 20m width has been selected shown in Fig.16 and was modeled with rigid solid elements. Outer walls of tunnel are reinforced by rebar with 100mm cover depth on top and bottom with collision speed of 0.463m/s and a rebar ratio of 0.2% was given consideration. 8 Node Solid elements was used in modeling for impact structure and tunnel and 2 Node truss element was used for rebar modeling. Analysis used LS-DYNA v.971(LSTC, 2007) and 72(*MAT_CONCRETE_DAMAGE, LSTC, 2007) was used for concretes. The external diameter of tunnel is 23m and thickness of wall is 1m and the length of 100m, 300m, 500m, 1000m tunnels were considered.

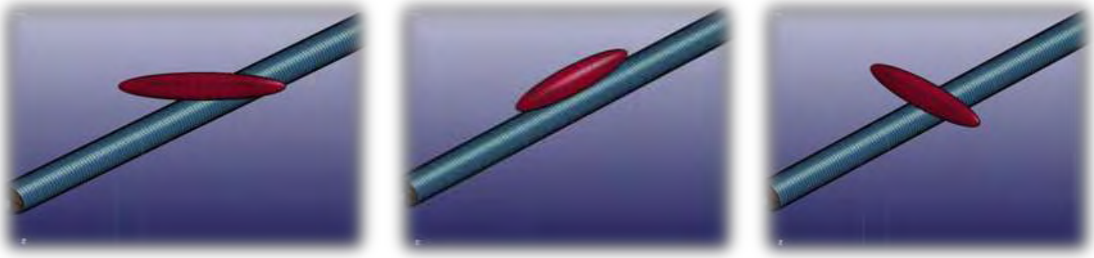


*Fig. 16 Impactor and tunnel mesh modelling*

**3.2 Analysis results**

Fig.17 shows the deformations and movements of sinking structure from its collision. It shows direction and the movements from its perpendicular, skew and parallel collision, assuming a 300m tunnel length. This data can still be used as reference to determine the scope of analytical modeling even though this data is expected to be quite different when the actual tunnel length of 1km is used.

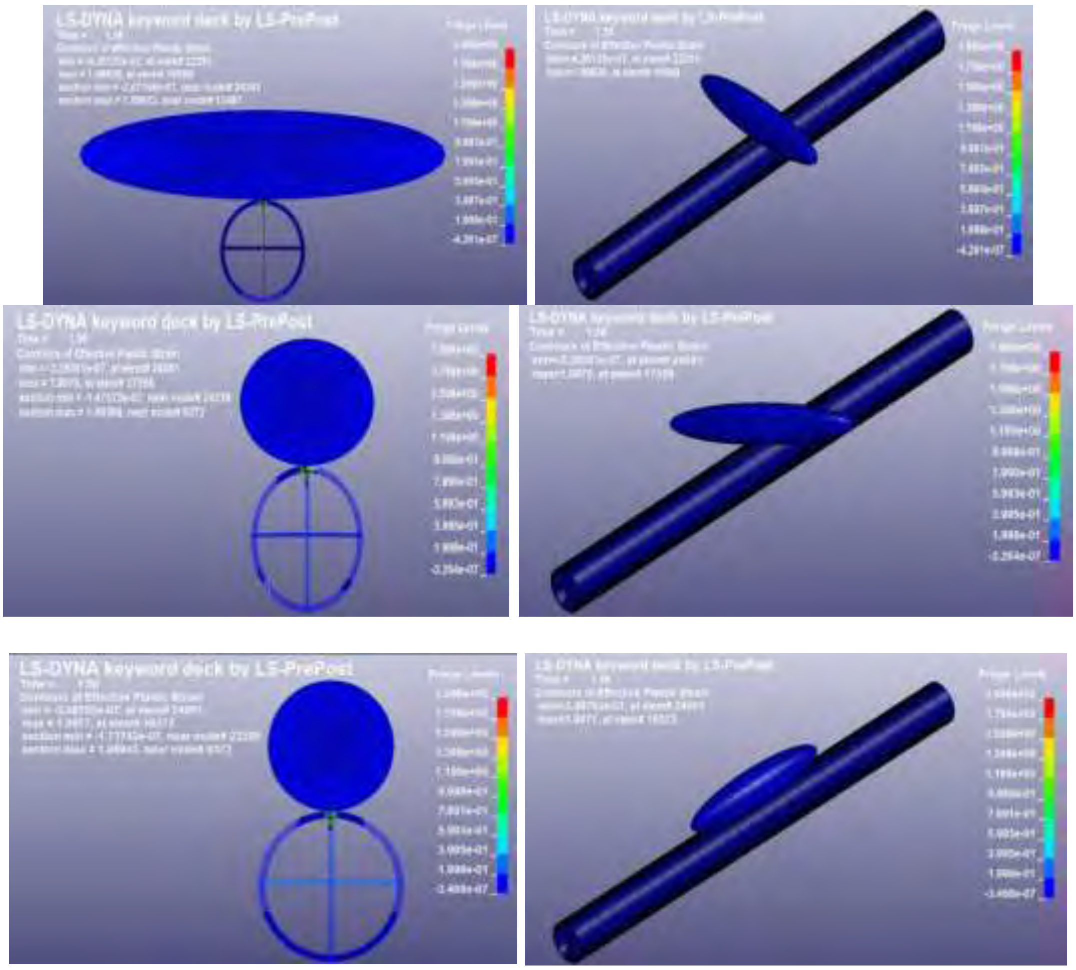



*Fig. 17 Deformations at impact (Tunnel length : 300m)*

Fig. 18 show deformations at impact in 1000m tunnels and indicates direction and the movement from its perpendicular, skew and parallel collision when a colliding object sinks. As were the cases in the aforementioned lengths, for 1000m tunnels, pressure deformation rate occurs all over central cross section Partition wall and over the surface of top colliding plane toward tunnel due to its bending moment. No damage movements penetrating tunnel from collision from perpendicular, skew and parallel collision resulted and showed colliding structure bouncing back in reverse direction from its colliding direction. Out of 3 impact scenarios, as it were for 100m and 300m cases, the collision impacted areas varied in accordance with the shape of colliding objects, however, since the length of tunnels used for testing were longer as the colliding objects get bigger, the relative scope and difference in area of collision were not significant.

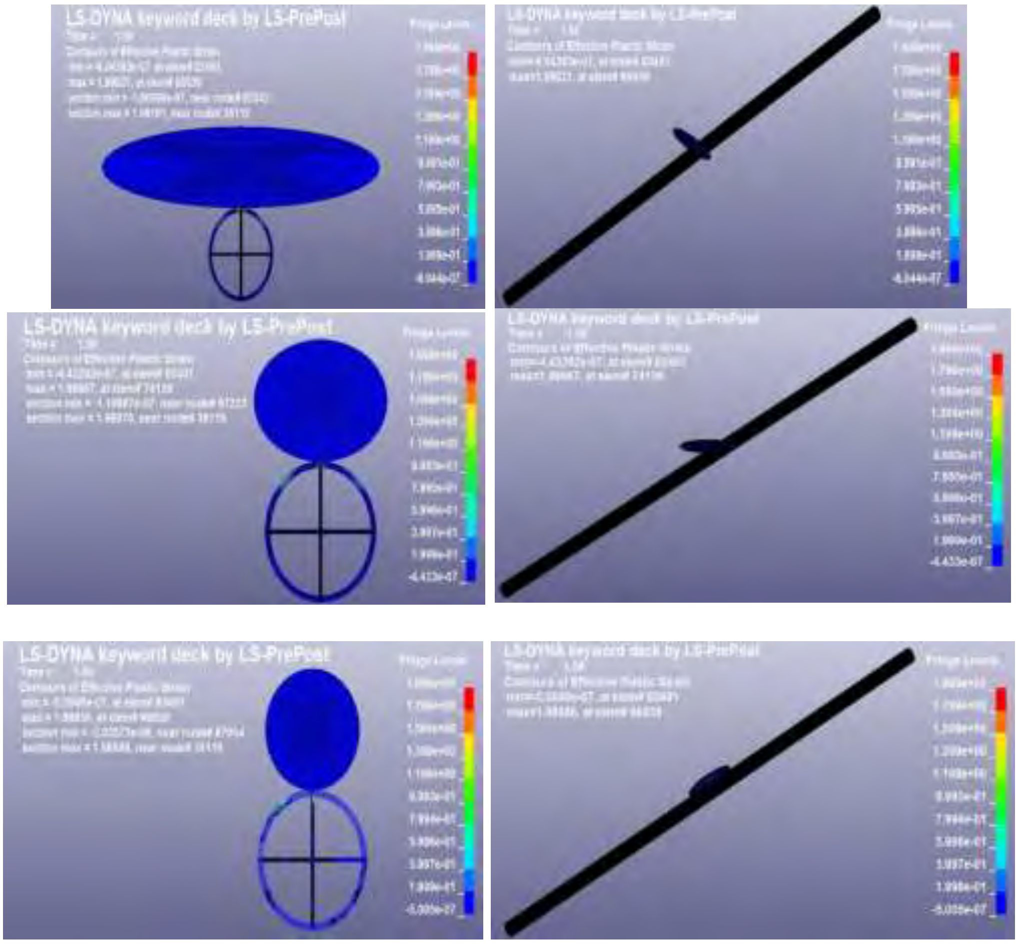



*Fig. 18 Deformations at impact (Tunnel length : 1000m)*

Table 1, 2, and 3 show maximum impact forces on the contact surface, when the impact forces are longitudinal, transverse, or sinking direction, respectively. The longitudinal and transverse impact forces demonstrated very little destruction, within 8% of the downward impact load, which could have resulted from the change of contact surface at the time of collision of the crashing object and the concrete part of tunnel. As seen in Table 1, the colliding force was the biggest when the crashing object was perpendicular to the longitude of the tunnel and the least when the crashing object was parallel to the longitude of the tunnel. It seemed the collision force is transferred less as the longitudinal contact surface is relatively less when collision is parallel than when it is perpendicular to the longitude of the tunnel.

*Table 1. Comparisons of longitudinal forces (X-force)*

| | X-force (UNIT: MN) | | | |
|---|---|---|---|---|
| | 100(m) | 300(m) | 500(m) | 1000(m) |
| X-force (perpendicular) | 1.9 | 2.4 | 2.1 | 2.2 |
| X-force (skew) | 0.9 | 0.8 | 0.75 | 1.15 |
| X-force (parallel) | 0.31 | 0.39 | 0.22 | 0.23 |

Table 2. Comparisons of transverse forces (Y-force)

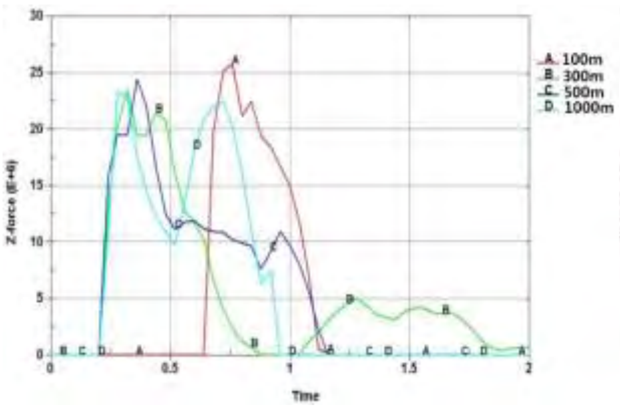
| | Y-force (UNIT: MN) | | | |
|---|---|---|---|---|
| | 100(m) | 300(m) | 500(m) | 1000(m) |
| Y-force(perpendicular) | 0.35 | 0.3 | 0.35 | 0.36 |
| Y-force (skew) | 1.3 | 1.2 | 1.18 | 1.1 |
| Y-force (parallel) | 1.4 | 1.3 | 1.2 | 1.42 |

Table 3. Comparisons of sinking direction forces (Z-force)

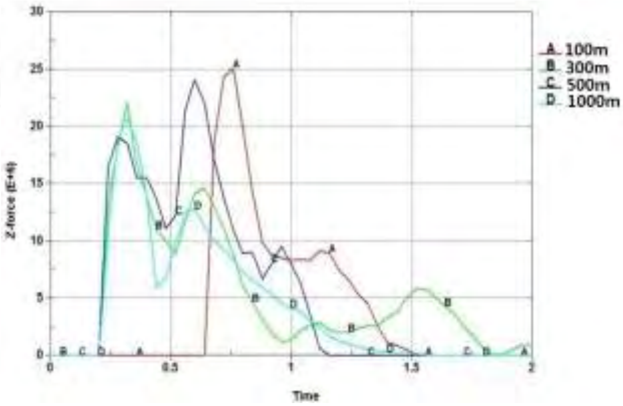
| | Z-force (UNIT: MN) | | | |
|---|---|---|---|---|
| | 100(m) | 300(m) | 500(m) | 1000(m) |
| Z-force (perpendicular) | 24 | 25 | 23.8 | 25.8 |
| Z-force (skew) | 20.5 | 24 | 22 | 25.7 |
| Z-force (parallel) | 46 | 19.5 | 20 | 26 |

As shown in Table 2, the collision force to the tunnel's vertical axis is the biggest when the impact of the crashing object is parallel and the least when the impact of the crashing object is vertical to the tunnel axis. It seemed the collision force is transferred most as the longitudinal contact surface is formed relatively larger when impact is parallel to the tunnel axis.

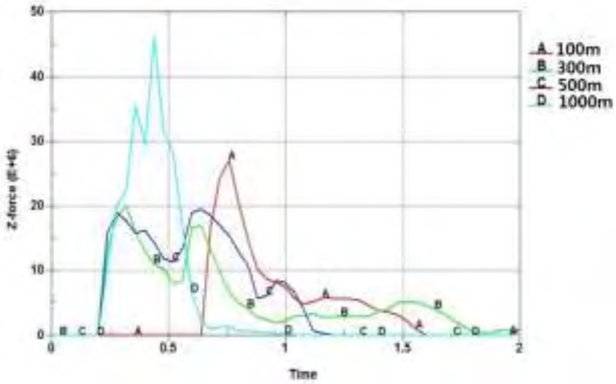
As shown in Table 3, except when the crashing object is longitudinally collided on 100 meters long tunnel, the maximum impact force appears to be approximately 20 to 25 MN. It is estimated that impact surface is formed momentarily wider throughout the short length of the tunnel in case of the horizontal impact of the 100 meters long tunnel, and the result would be similar of the submerged floating tunnel longer than 1 km in length. Fig. 19 shows the change of resultant force against time in three impact analysis of different tunnel lengths. It shows the longer the tunnel, the longer maximum impact duration occurred, and the resultant force disappears as the crashing object is bounced off in the reverse direction after the impact.




a) perpendicular impact




b) skew impact




c) parallel impact

*Fig. 19 Comparisons of resultant forces*

## 4. CONCLUSION

### 4.1 Impact analysis of submarine collision in operation

The impact surface decreases little even as the collision progresses because impact surface is comparatively smaller than the crashing object when vessel or submarine collide head-on or on their the sides, however, it decreases rapidly as the collision progresses because impact surface is very large in the event when colliding with sinking vessel or submarine. The x- directional force is not that great when massive vessel of submarine collides head-on or on their sides while moving forward, but x-directional force is larger by far when the impact happened while sinking as in case D. Though the estimated sinking speed would be slower than a colliding speed while in operation which is 5 m/s, since the whole mass would be burdened with very large impact surface if the two speeds are the same and the most impact force and mass would be transferred in case of D in progress, these data should be considered seriously while calculating the realistic impact force of submerged floating tunnel in the future. The y-directional force on top part of cross sectional of the tunnel is almost nonexistent when near the center of submerged floating tunnel's outer wall was impacted, however, y-directional force occurs when the impact is made off center as in B and C. These data should be considered seriously while calculating the realistic impact force of submerged floating tunnel in the future.

### 4.2 Impact analysis of collision when submarine sinks

This research performed analyses of collisions to design submerged floating tunnel against possible impact loads from anticipated colliding submerged objects and concludes as follows:

The loads vertical to the tunnel-axis were found to be less than 8% of loads along the direction of collision. This is due to the deformation occurring on the colliding surface against concrete surface of tunnel. And the maximum loads along the direction of tunnel occurred when impact object crashed vertical to the tunnel. This is due the minimum impact surface results perpendicular to the tunnel-axis direction when collision occurs along the direction of tunnel-axis. This impact in turn minimizes the loads from collision in comparison.

The collision force to the tunnel's vertical axis is the biggest when the impact of the crashing object is parallel to the tunnel. It seemed the collision force is transferred most as the longitudinal contact surface is formed relatively larger when impact is parallel to the tunnel axis. When designing the structure for submerged floating tunnel against possible collision, the impact loads from crashing objects along its colliding direction is the most critical aspect in design. The study shows that maximum load lies in the range of 20 ~ 25 MN when the length of tunnel is 500m and 1000m. This maximum loads should be carefully examined when designing the submerged floating tunnel in future, since the submarines operating in Korean waters are 10 times lighter than the one being considered for this analysis, U. S Ohio class submarine with 18,750 ton.

## ACKNOWLEDGEMENTS

This work was supported by Korea Institute of Ocean Science and Technology (KIOST) and greatly appreciated.

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# THE ROGFAST PROJECT

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## ABSTRACT

*Rogfast is the name of a 25.5 km long subsea tunnel being planned to pass under Boknafjord in Rogaland. The tunnel will contribute to shortening the travel time on the Coastal Highway Route E39 between Stavanger and Bergen by about 35 minutes. In the Norwegian National Transport Plan, it is assumed that the start-up will be in 2015.*

*The projected tunnel is planned as a twin tunnel with a cross-section of 2 × T 10.5. There will be an arm up to Kvitsøy and a two-level underground interchange. The maximum depth will be 392 m and maximum gradient 7%.*

*The tunnel will be ventilated along its length and three double shafts are planned. The design fire will have a 200 MW effect at the outset. A complete risk and vulnerability analysis has been carried out to determine what standard must be incorporated to achieve a satisfactory safety level with respect to fire and other undesirable events.*

*Roughly speaking, Rogfast will generate surplus rubble and spoil amounting to approx. 6 million* $m^3$*. Municipal zoning plans will be approved in all three municipalities concerned (Randaberg, Kvitsøy and Bokn) in May this year.*

*The financing of the project is mostly based on toll money. It is expected that the Parliament will approve the plans for the project and give license to collect toll money late in 2014. The bidding process will start as soon as the Parliament has approved the project.*

*A study has been carried out with respect to the use of TBM for the project. Later on it will be decided whether there will be developed alternative bidding documents which will make it possible to have TBM as an alternative to the traditional drill & blast method.*

## BACKGROUND AND HISTORY

Development of Coastal Highway Route E39 is a link in the strategy to develop efficient north – south traffic routes between key areas in Western Norway, and to develop this part of the country’s contact with the continent. Coastal Highway Route E39 is therefore a prioritised road system for all the counties in Western Norway. Interacting with important harbours along the coast, the Coastal Highway Route E39 is seen as one of the pillars of future economic development, both for this part of the country and for the country as a whole.


*Figure 1 The tunnel alignment crosses Boknafjord from Harestad in Randaberg municipality in the south to Arsvågen in Bokn municipality in the north.*

A high standard, ferry-free road along the coast of Western Norway has long been an overriding goal for Rogaland and a common goal for the counties of Western Norway. Crossing Boknafjord is currently done by means of a ferry connection between Mortavika and Arsvågen. The connection has experienced very strong growth in traffic since its opening in 1992 and has had great problems with respect to capacity during periods with heavy traffic. New, gas-driven ferries in use from 2007 have increased capacity, but do not remove the basic obstacles to an efficient transport solution which any ferry connection represents.

Travel time between Bergen and Stavanger today is about 4 hours and 15 minutes by car. With Rogfast, this travel time will be reduced to 3 hours and 40 minutes.

The overall goals for E39 Rogfast can thus be summarised as follows:

- To contribute to developing the E39 into an important connecting road for Western Norway as part of a national transport corridor along the coast of Western Norway.

- To contribute to development of a continuous transport system for goods servicing key harbours and other junctions along the coast of Western Norway.

- To develop a common residential and labour market for North Jæren and Western Norway.

Boknafjord is a sill fjord that is deep far from the mouth, but has a sill in the shape of a bow from Tungenes in the south, via Kvitsøy to Arsvågen in the north. To avoid long and steep gradients, a subsea tunnel must follow this sill. Conditions are favourable for a link to Kvitsøy, which will provide a ferry-free connection to the mainland for this island community.

The project consists of 25.5 km long twin tunnels between Harestad in Randaberg and Arsvågen in Bokn. The arm to Kvitsøy is 4 km long and will be built as one tunnel.
The Zoning plan for the Project is newly approved in Randaberg, Kvitsøy and Bokn municipalities. The project is estimated to cost NOK 10.2 billion and is planned to be financed mainly by road toll money. In the Norwegian National Transport Plan (NTP), the State’s share of the cost is assumed to be NOK 1000 million. The project start-up is assumed to be in 2015.

Work is now in progress on a comprehensive programme of sub-surface surveys (seismic, core drilling from both land and ship). This will continue throughout 2013 and 2014.

## GEOLOGY

SINTEF and Cowi has compiled a geological report based on all the sub-surface survey, reports, and a number of technical reports and Master theses in several different disciplines such as geology, refraction seismology, acoustics, magnetrometry and engineering geology. In addition, experience from completed tunnel sites in the region has been incorporated. In a report summary, it is stated that the Rogfast tunnel goes through an area of complicated geology in the form of several thrust nappes and faults, where many types of rock are represented. Four thrust nappes, consisting mainly of granitic and dioritic gneisses, have been defined over the bedrock.

The tunnel is expected to go through phyllite and mica schist for the first approx. 6.3 km from Randaberg, until it meets an expected fault zone in this area. If the phyllite is thin, the tunnel may reach bedrock before it reaches the fault zone. Depending on the vertical throw of the fault, the tunnel is then expected to enter the Karmøy Ophiolite which has gabbroic gneisses, hypabyssal rock types, volcanic rock types and sediments, as well as ultramafic rock types (dark and quartz-free). If the vertical throw is greater, the tunnel may enter the Storheia Nappe, which consists of granitic and dioritic gneisses and lies under the Karmøy Ophiolite, or go right down to the phyllite.

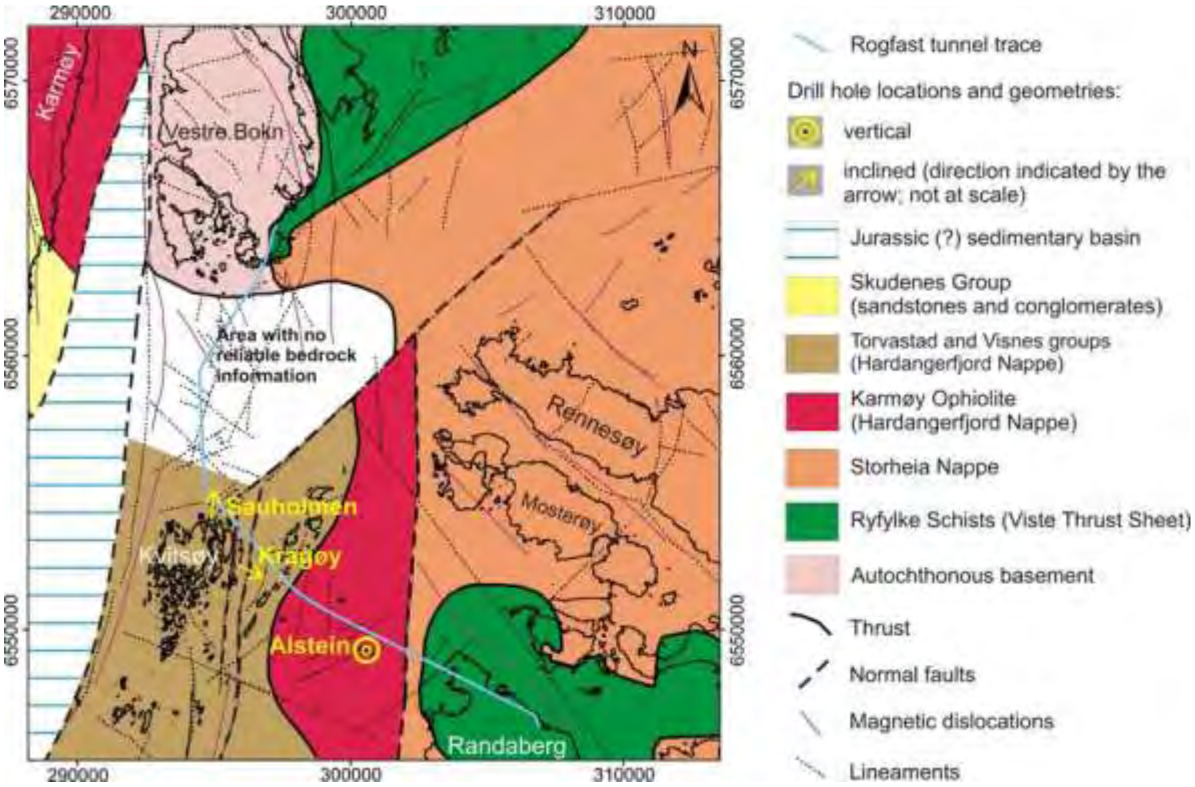


*Figure 2 Geological map of the area. The blue line indicates the tunnel alignment.*

It is assumed that there are several fault zones between Alstein and Kvitsøy that could result in variations of the rock types mentioned in the tunnel. On Kvitsøy and the small islands east and south of Kvitsøy, green schist and chlorite schist are also found in addition to the rock types mentioned in the Karmøy Ophiolite. The distribution of rock types between Kvitsøy and Bokn is very uncertain but it is expected that the tunnel will go through a large fault zone just a kilometre north of Kvitsøy, and thereafter into a thin layer of the above-mentioned nappes to eventually enter bedrock gneisses. Folding or undulation of the nappes can lead to the tunnel entering and exiting the nappes several times. Near Arsvågen at Bokn, the tunnel will go through phyllite up to tunnel entrance.

The joint measurements performed show that there are both a dominant joint set and foliation in a north-northeast-south-southwest direction at both Randaberg and on Kvitsøy. The foliation has a low dip angle towards the west at Randaberg and varies considerably due to folding on Kvitsøy. At Arsvågen, the main joints and the foliation are approximately north-south with a varying dip towards the east. In many places there is one set of cross joints. The tunnel will pass under many deep channels that have low seismic velocities. Based on the measurements in the region, moderate to high horizontal stress is expected in the bedrock along the Rogfast tunnel.

Refraction seismic measurements in almost all the deep channels along the tunnel alignment show velocities from 2200 m/s to 6300 m/s with 7% of the length less than 4000 m/s and almost 90 % over 4500 m/s. The figures are converted to rock mass quality values according to the Q-system and calibrated against experience from tunnels in the vicinity to form a basis for reinforcement estimates.

A study was carried out to address the potential for water leakage in the tunnel, based on experience from tunnels in the same area. There is little experience of leakage in phyllite and mica gneiss, while there has been moderate leakage in the other rock types, depending on the stresses and joint directions. The same study indicates there is little need for injection throughout 66% of the tunnel length and little to moderate need for injection over a further 27% of the tunnel length. It is estimated that only over 7% of the tunnel length will there be great to very great need for injection.

## THE TUNNEL ALIGNMENT

The tunnel is planned as twin tunnels with cross-sections of T10.5. The arm up to Kvitsøy is planned as one tunnel. The line location is designed according to design class S9, i.e. with a design speed of 100 km/hr. Due to the length, an attempt to break the monotony will be made by placing large horizontal curves between 3,800 m and 20,000 m.

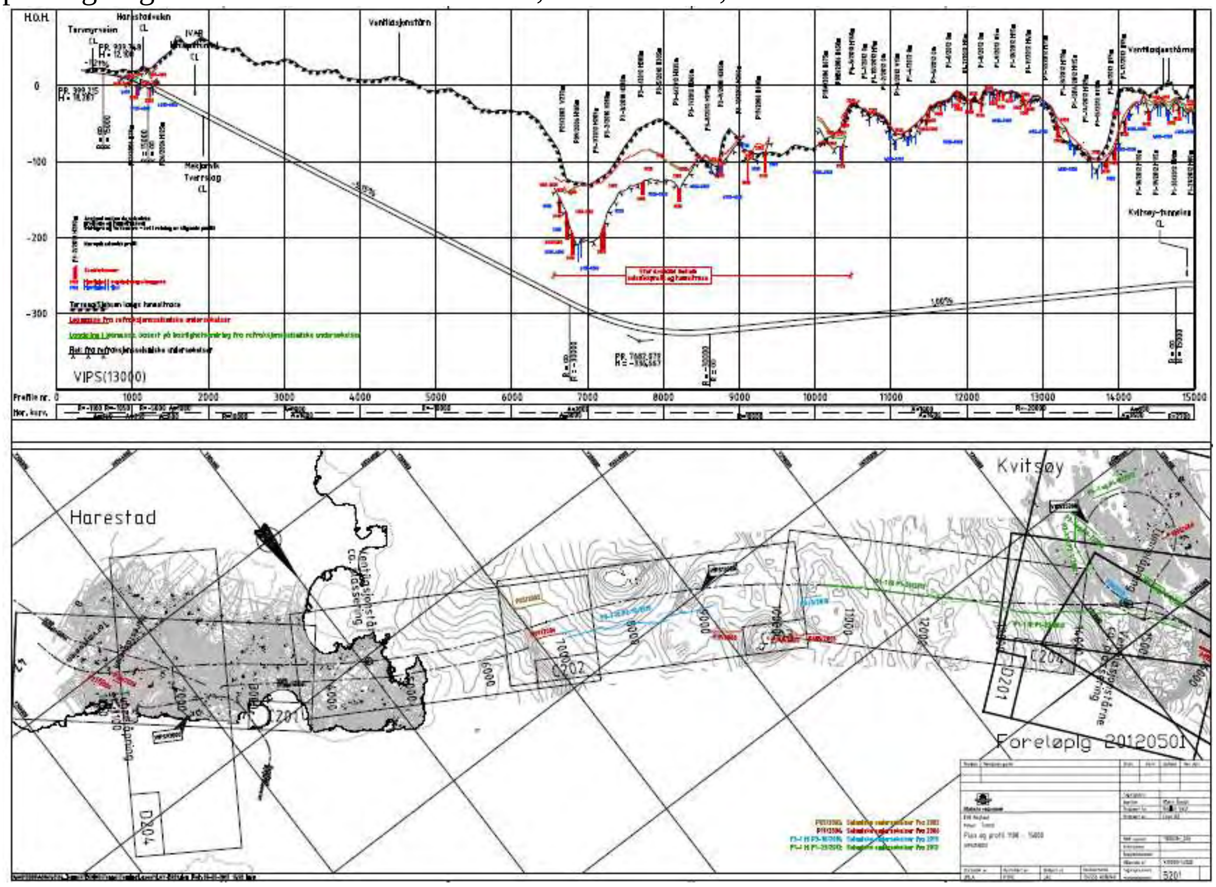


*Figure 3 The tunnel alignment from Randaberg to Kvitsøy.*

The maximum gradient of the tunnel is 7%. This occurs in the ascent up towards Arsvågen and has a length of approx. 3 km. Otherwise gradients are at 5% or less. An additional climbing lane will be considered for the 7% gradient up towards Arsvågen. At the moment there is a discussion whether this gradient should be decreased to 5%. The tunnel will then be about 1 – 1.5 km longer, but tunnel safety will be increased.

The maximum depth is 392 mbsl. This occurs about halfway between Kvitsøy and Arsvågen.

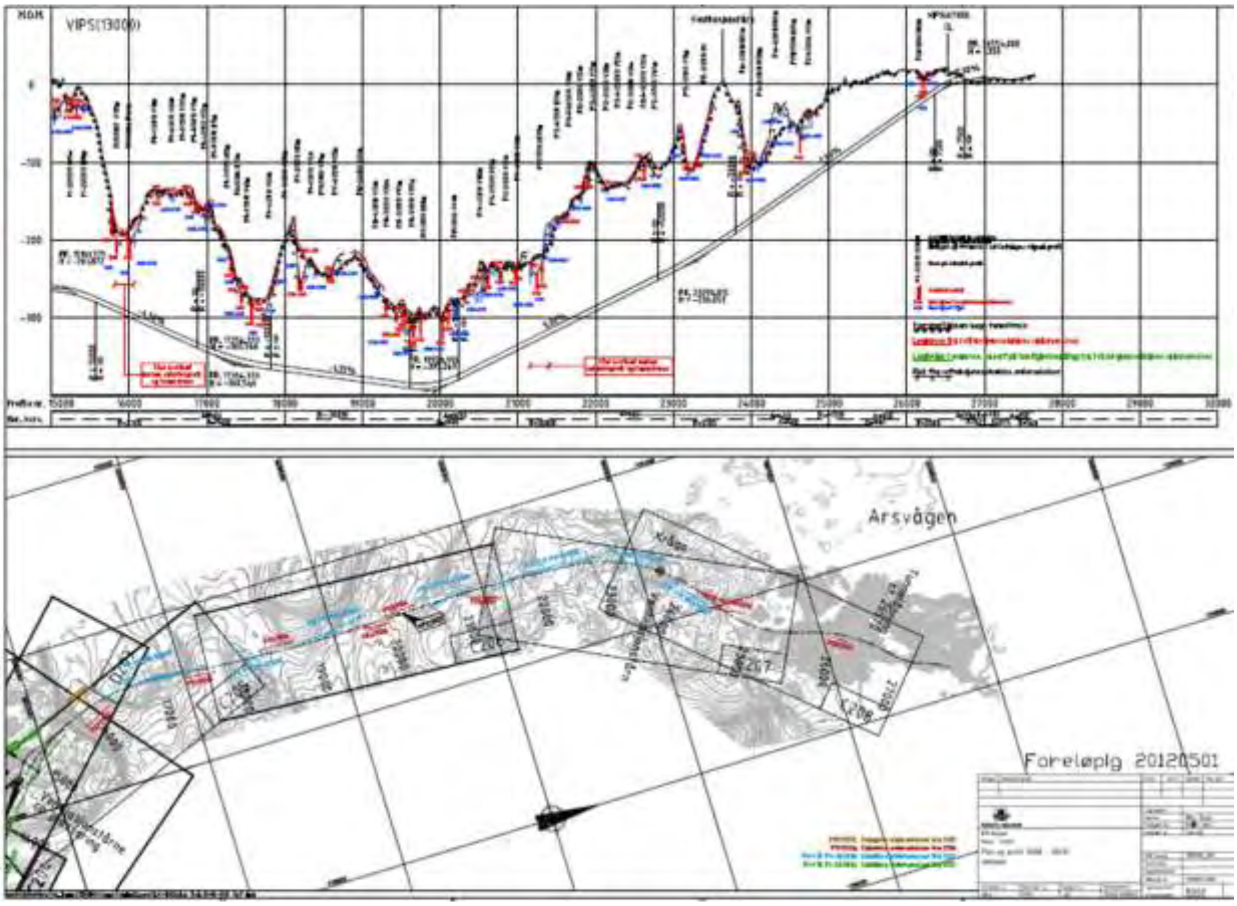


*Figure 4 The tunnel alignment from Kvitsøy to Randaberg*

The interchange of Rogfast and the tunnel arm to Kvitsøy is proposed to be made as a two-level interchange with four ramps. The tunnel interchange for the Trans European Road Network (TERN) is not permitted if the EU tunnel safety directive is observed. However, the Directorate of Public Roads has granted an exception from this requirement in connection with the municipal sub-plan for Rogfast. However, the interchange does not comply with the requirement that the two courses in the main tunnel must be separate fire cells. A possible solution to this could be the introduction of fire doors in the tunnels to Kvitsøy.

## VENTILATION

As a basis, a ventilating principal of lengthwise ventilation and three double shafts has been chosen. This provides a simple, flexible ventilation facility that can be developed as the traffic increases. The three shafts will be located on the golf course at Tungenes on the mainland in the south, on Kvitsøy and on Kråka Island near Arsvågen in the north.

Ventilation calculations that were carried out show that the piston effect from traffic will for long periods be sufficient to push air along the tunnel as long as there is one-way traffic. In two-way traffic (deviant situation) and during highly trafficked periods in one direction, extra thrust from impulse fans hanging in the tunnel will be required.

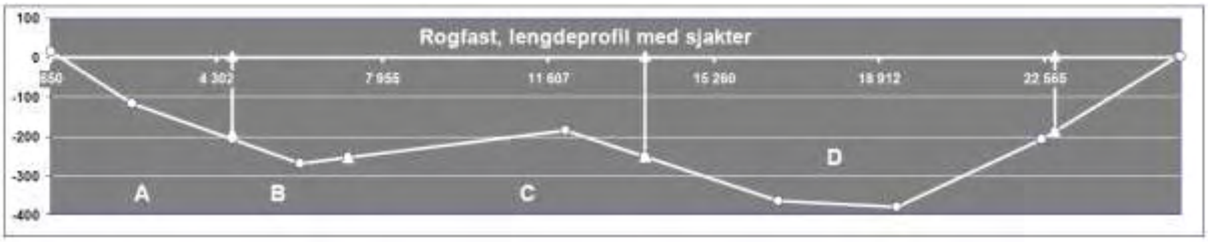


*Figure 5 Longitudinal section including three ventilation shafts*

At the outset, the tunnel will be designed for a 200 MW fire. The risk and vulnerability analysis has clarified scenarios with greater fire effects. In the main tunnel, there will be airtight locks between the courses. In case of fire in the main tunnel, smoke will be led in the direction of the traffic to the nearest shaft or portal. The interchange at the arm to Kvitsøy is especially challenging, but the main principle will be that the difference in pressure between north- and southbound tunnels will be regulated so that there will be enough excess pressure to avoid smoke spreading to both tunnels in the interchange area. In case of a large fire in the northbound tunnel near the portal at Randaberg or in the southbound tunnel near the portal at Arsvågen, hot smoke will result in a strong buoyancy force (stack effect).

At the outset, no pollution abatement facility is planned for dust and soot. When traffic approaches 10,000 vehicles per day, an assessment must be made as to whether to establish such a pollution abatement facility to increase the capacity of two-way traffic and reduce energy needs when there is normal traffic.

## RISK AND VULNERABILITY

The Rogfast Tunnel will be between 25 and 26 km long, and during its opening year will accommodate traffic of about 6,000 vehicles a day. When road tolls lapse after about 20 years, traffic will increase to about 13,000 vehicles a day. Standard requirements for Norwegian road tunnels are described in Manual 021 – Road tunnels. These standard requirements apply to tunnels of up to 10 km in length. For longer tunnels, special evaluations must be made. Rogfast is in this category and a risk and vulnerability analysis is a very important tool for defining a standard for the tunnel that will provide a good tunnel with high traffic safety. Such an analysis was carried out in 2012 in connection with the zoning plan for the project.

The Rogfast tunnel has at the outset been placed in safety class (E), two tunnels with two lanes in each tunnel, with appurtenant safety equipment. In principle this means that this is a tunnel with a lot of built-in safety. However, overriding safety assessments indicate that there are factors that one must be especially aware of.

Rogfast is a very long, subsea tunnel with in parts steep gradients. The risk of a serious accident and two concurrent events must be specially evaluated. Experienced risk is another aspect that must undergo a thorough evaluation. The classification in Manual 021 does not take lengths greater than 10 km into consideration. The design fire effect in Manual 021 is set at 100 MW. Investigations as to whether any safety benefits might be gained by increasing the design fire effect to 200 MW has been carried out and the greater effect is recommended.

Intersections in tunnels are a special case and in principle are not allowed in the Trans European Road Network (TERN). Even though an exemption has been approved on this point, the intersection has been given special attention in the risk and vulnerability analyses.

Examples of measures that has been evaluated:

- Pedestrian cross-connections between the tunnels (emergency exits). Minimum distance according to Manual 021 is 250 m. The effect of reducing that distance to 125 m has been evaluated, and the conclusion is that the distance between the emergency exits should not exceed 125 m when the gradient is 5% or more.
- Drive-through opportunities for emergency vehicles in the cross tunnels has been evaluated. The chosen solution is to establish drivable cross tunnels every 1250-1500 m along the tunnel.
- The tunnel arm to Kvitsøy was originally planned as a single tunnel. The possibility of making it a twin tunnel has been considered, but the conclusion is to keep it a single tunnel.
- Climbing lane for the ascending sections that exceed a 5% gradient. This will apply to an approx. 3 km rise up to Arsvågen, where it may be advisable to construct a third climbing lane to provide the heaviest vehicles with their own climbing lane.
- Manual 021 specifies a distance of 500 m between emergency lay-bys in the tunnel. Evaluation of a reduction of this distance or replacement of emergency lay-bys with a separate, continuous emergency lane will be made later in the Project
- Drivable cross connections for use in problem situations such as traffic accidents and maintenance/road work has been considered. This will entail two-way traffic in one tunnel for parts of the tunnel. An example of this would be to establish drivable cross-connections for all traffic every 4 km.
- Driving in long tunnels can lead to tiredness, inattention and anxiety. Measures that will be evaluated are varying the cross section through the tunnel to help break the spatial monotony to which road users are exposed: use of light, marking of special points etc.
- The effectiveness of an active extinguishing system will be evaluated (deluge, sprinkler, water mist).
- The effect of a separate parking space for hazardous goods and emergency vehicles will be evaluated with respect to fire/emergency services. In addition, the possibility of a separate road traffic centre for tunnels in the area and an operations building for fire/-emergency rescue services at the tunnel entrance must be evaluated.

## THE DISPOSAL OF RUBBLE AND SPOIL

Rogfast will generate a large surplus of rubble and spoil. Roughly estimated, this surplus will amount to approximately 5.4 million $m^3$. An optimal performance of project construction involves excavation of 8 tunnel faces simultaneously to minimise construction time. This implies three initial points: Randaberg, Kvitsøy and Bokn. From Kvitsøy, the arm will be excavated first and thereafter excavation will continue in both tunnel courses towards both Randaberg and Bokn. A strategy for tunnel excavation such as this will mean that approx. half of the rubble will be removed to Kvitsøy, while a quarter will be removed at Bokn and at Randaberg, respectively.

The county governors are now requiring that the excess rubble be utilised for publically beneficial purposes. This means that they will not accept excess rubble deposits without purpose. Early planning of the use of excess rubble has therefore been an important part of the project planning.  Bokn and Randaberg are favourable places for the use of the rubble as fill either at sea or on land, where the areas created may be used for industrial purposes. On Kvitsøy, however, it is somewhat more difficult because all the fills will have to be located in a vulnerable archipelago. If this results in it not being possible to excavate a major part of the tunnel from Kvitsøy, it will affect construction time, and the project could become significantly more expensive. In addition, allowance must be made for significantly larger rubble deposits at Randaberg and Bokn .

## FURTHER PLANNING

The municipal sub-plan for Rogfast was finally approved in 2010. The plan also includes plans for road links in the day zones in the Randaberg, Kvitsøy and Bokn areas. Based on the municipal sub-plan for Rogfast a more detailed zoning plan has been developed for the whole project. The zone plan includes the preparation of area and technical plans to provide the basis for processing in the municipalities concerned. In addition, the technical planning has proceeded to a point where it is possible to prepare a cost estimate with ± 10% uncertainty. The zoning plans are now completed and will be processed in the municipalities concerned in May this year.

The financing of the project is mostly based on toll money. The Government and the Parliament has to approve the financing of the project. This also includes the technical solutions which the project is based upon. This process will probably start late in 2013 and we expect the project to be approved by the Parliament late in 2014.

In the meantime The Public Road Authority will develop the project further, and in the end make a full description and bidding documents for the project. The goal is to start the bidding process as soon as the Parliament has approved the project.

A study has been carried out with respect to the use of TBM for the project. Limited opportunities to deposit rubble on Kvitsøy can make the use of TBM appropriate to reduce the construction time. On the other hand rubble from TBM can make it more difficult to deposit it in the sea. The zoning plan includes the necessary area for the use of TBM and in the detailed planning phase of the project it will be considered whether TBM is feasible. If so alternative bidding documents will be developed. This will make it possible to have TBM as an alternative to the traditional drill & blast method. Price and quality will then be the factors that decide which method will be preferred.

# CROSSING THE OSLOFJORD - AN EARLY STRATEGIC ANALYSIS
## - Technical challenges and consideration of Feasible solutions

Ove Solheim MSc - Norwegian Public Roads Administration

**ABSTRACT**

*Feasibility studies are in progress for a fixed link across the southern part of the Oslofjord covering an area from Tønsberg in the south to Drøbak in the north. The present Oslofjord tunnel at Drøbak has shown itself not to be a preferred alternative to the ferry between Moss and Horten, which is the ferry link in Norway carrying most vehicles. New potential crossings will be considered including a railway link in combination with the road link. Regional impacts as well as other relevant conditions related to the transport sector will be evaluated. The study is scheduled to be completed by summer 2014. Various combinations of bridge and tunnel solutions will be considered, including the effect of improved ferry connections related to current and predicted traffic volumes. The sailing lane requirements for the Oslofjord will probably be 1500 m wide (alternative: 2 x 750 m) and 75 m high due to the frequent passage of large size ships. At the most relevant sites for future crossing the fjord is 1,8 - 8 km wide and the bedrock has depths down to -400 m.*
*This paper presents a choice of technical alternatives at this very early stage related to the topographic and ground conditions at the various prospective sites.*

## GEOGRAPHY – BACKGROUND

As may be seen in Figure 1, all the roads and railway lines around the Oslofjord are channelled towards the capital Oslo. It follows therefore that the focus of those living west of the fjord naturally is on the capital Oslo, and likewise for those living on the east side of the fjord. The fjord is thus effectively preventing those living close to the fjord from having a mutual labour- and housing market. At present there are also regrettably few social and educational contacts across the fjord. And the industry is not cooperating either. This ought to change.


*Figure 1 South of Norway and the Oslofjord*

This feasibility study considers different concepts to substantially improve transport and communications across the Oslofjord south of Drøbak as indicated in Figure 1. The object of the technology part of the present study is to ensure with sufficient certainty that what is proposed as fixed link solutions, can be built, and at a cost the country can afford. Adding to the complexity of this task, a combined crossing for road and rail is also be investigated and developed.

At this point all possible crossing sites and all methods of crossing is to be considered. No concept is at present excluded from the investigations. It will therefore be totally misleading to present any concept, as "a chosen one"!

## HISTORY

The model in Figure 2 is from 1988 and shows a planned road bridge crossing the fjord just north of Drøbak. If having been built, it would have destroyed many summer cottages and reduced the value of scenery and nature in one of the most visited parts of the Oslofjord. The navigational channel is quite narrow at this point.


*Figure 2 Model from 1988 showing possible bridge crossings north of Drøbak*

The inhabitants of the area and local politicians reacted strongly against this concept. The minister of Transport and Communication then decided to build a subsea tunnel instead.

The two acoustic profiles shown in Figure 3 was the sole information of bedrock available for the southern part of the Oslofjord when this feasibility study was started. One of them covers the distance from the Bastøy Island to the Jeløy Island, while the other covers the distance from Bastøy to Rygge on the eastern mainland. These profiles were obtained in 1988.
As may be seen, the profiles show sediments having thickness up till 120 m in the deeper parts of the fjord. This strongly indicate that the depths to the bedrock is more than the depths to the sea bed as shown in public seamaps.

A great need for additional data of the bedrock arose, and a seismic exploration program was drawn up trying to take all eventualities into consideration. Interests were focused on rock formations through which drill and blast tunnels could be constructed or upon which bridge foundations could be placed. This program was executed August 2012, and provided some unexpected results.

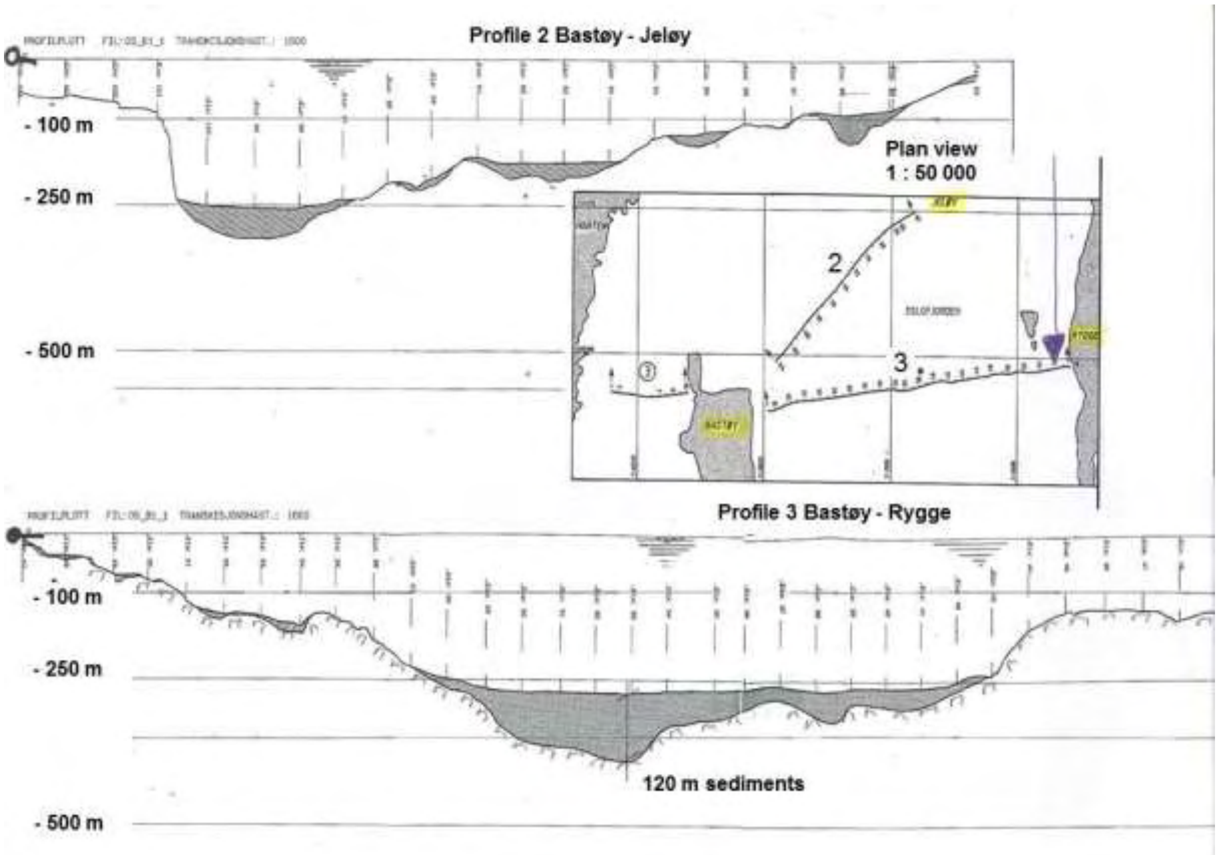


*Figure 3 Showing location and result of two acoustic profiles in the southern part of the fjord*

## CONDITIONS IN THE FJORD - BEDROCK

Figure 4 gives an adequate view of the location of seismic profiles in order to obtain depths to bedrock and the thickness of sediments. Dark red is very deep, red is deep, yellow is still deep and green is less deep, whereas blue represents shallow water or naked rock.
In all 6 narrow corridors and 3 more extensive areas were explored.

The result shows that the Drøbak Channel represents a typical U-shaped valley. Here is mostly naked rock having only small layers of sediments at the bottom. Depths of 200 - 300 m to bedrock prevent solutions like tunnels in this part of the fjord. The fjord in the channel is barely more than 2000 m wide and is therefore favourable to large conventional bridge constructions.

South of the Hurum Peninsula the rock formations in the fjord become more varied and unpredictable. Here great variations occur in the depth to bedrock. In some places gigantic potholes in the rock were found filled with sediments up till two hundred meter in thickness.

The fjord has a general strike north - south.

The resulting bedrock map however has large uncovered areas where additional knowledge is required before rock tunnel alignments or the location of bridge foundations may be recommended. Supplementary seismic exploration is to be carried out this summer ending with a final report in September. Until September this year it is therefore impossible to conclude on or recommend any of the crossing possibilities between Moss and Horten. And it will be a challenge to decide upon a recommendation for the alignment of a subsea tunnel.

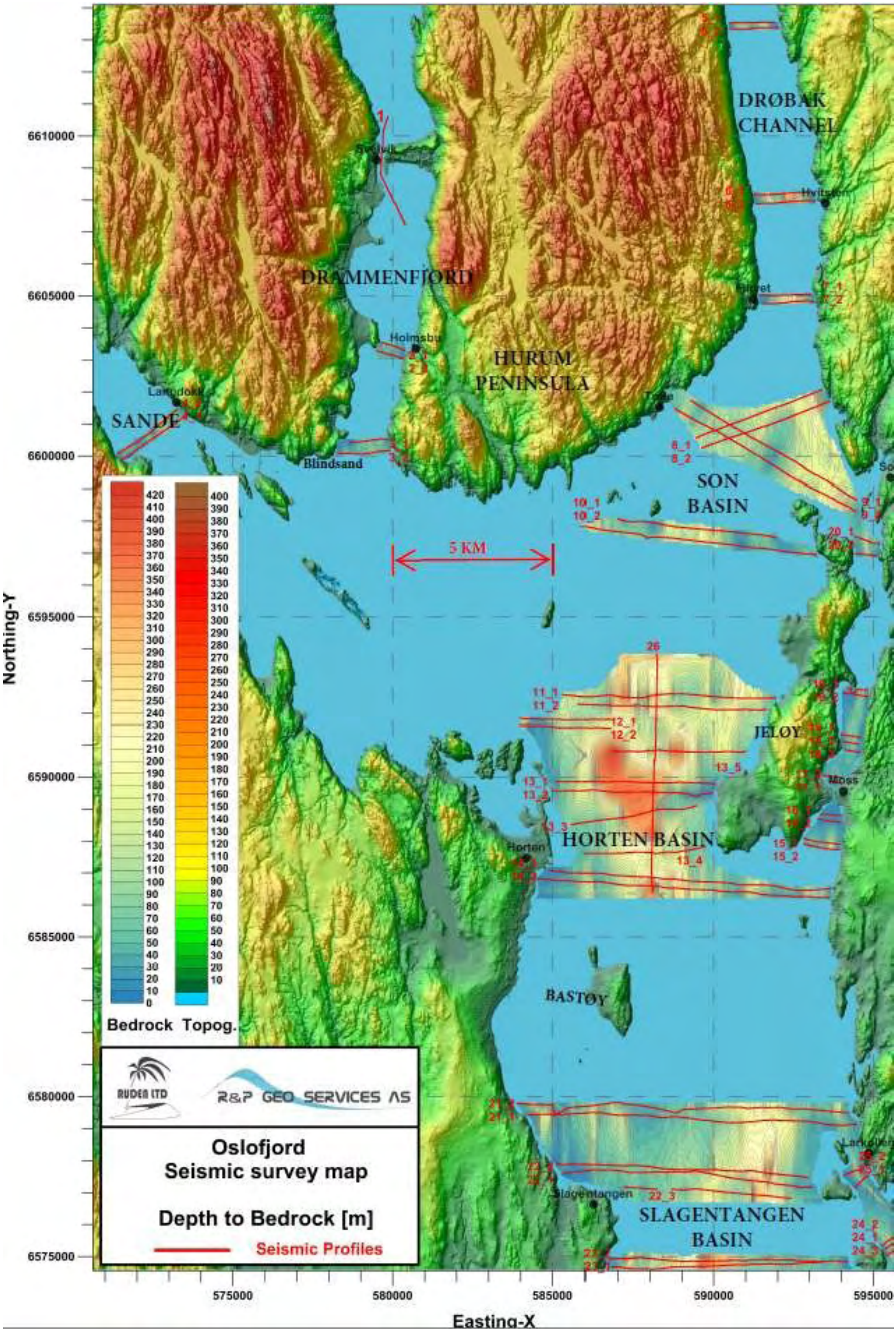


*Figure 4 Showing the location of seismic profiles and bedrock topography in the middle part of the Oslofjord*


*Figure 5 Areal view from the south-east looking north-west towards Moss City and the Jeløy Island*

Figure 5 gives an aerial view to the northwest where the City of Moss and the Jeløy Island can be seen. Further away we can see the Son Basin, the Hurum Peninsula and Drøbak Channel. Observe that the fjord becomes much wider south of Hurum. A ferry can be seen in Moss Harbour.

**A SELECTION OF CROSSINGS OVER THE DØBAK CHANNEL USING HURUM PENINSULA AS A "STEPPING STONE"**

The object of the ongoing feasibility study is to find and define favourable places where to cross the fjord. In Figure 6 only some of the considered crossings via the Hurum Peninsula are presented. To obtain the full potential of a crossing of the fjord at Drøbak, it will be necessary to cross the Drammensfjord as well. Drammensfjord is a sidefjord to the Oslofjord.

A is the position of the bridge which nearly was built in 1988 (see Figure 2).

XX is the subsea tunnel which was built with one single tube and opened in the year 2000. This tunnel lies in a rock formation crossing the fjord. It reaches its deepest point at -134 m and has a gradient of 7%. The EU-recommended gradient is maximum 5%. Some serious fire incidents due to overheating of brakes have occurred in this tunnel. At present a tube no. 2 is being planned having the same gradient of 7%. An additional task was added to this feasibility study early this year: to develop and propose a bridge near to the present tunnel as an alternative to building tube no. 2!

The project has also been asked to identify and recommend any feasible combined railway/road crossing. B is a crossing where a foundation can be placed in comparatively shallow water at

-35 m depth in the middle of the fjord and thereby cutting the main span in half. B seems therefore to be the "best buy" for a combined rail/road crossing.

C represents a suspension bridge having a main span of 1500-1700 m, see figure 7. This type of bridge can technically be built anywhere across the Drøbak Channel and is an example of a suspension bridge composed of 4 vehicle lanes and a pedestrian/bicycle lane in the middle. This type of bridge can be placed nearly at any crossing in the Drøbak Channel. As the foundations are placed in shallow water, the risk from colliding ships is small.


*Figure 6 Alternative potential crossings of the Oslofjord and Drammensfjord. Blue lines represent bridges and dotted lines road/rail on dry land.*

D is located where a shallow foundation site is situated at 1/3rd of the distance which reduces the main span to 1200 m and can perhaps thereby offer another combined rail/road crossing.
E is the most southern crossing for a bridge in this part of the Oslofjord and is a suspension bridge as described in figure 7.
F is a concept unlike the others in the Drøbak Channel. The rocktunnel is very long, and the connecting suspension bridge J over this part of the fjord will probably have a record span of 2000 m. At present it is thought of as being a road concept. It may also be combined with the K bridge.
G and I are places where to cross the Drammensfjord by bridge.

H represents an immersed tunnel for a combined solution with 2 lanes rail/4 lanes road.
K is a bridge spanning the Sande Bay. Here thought of as cable-stayed. It can be combined with the I bridge.

Examples of road connections between E-18 and E-06 are shown. Similar for rail.

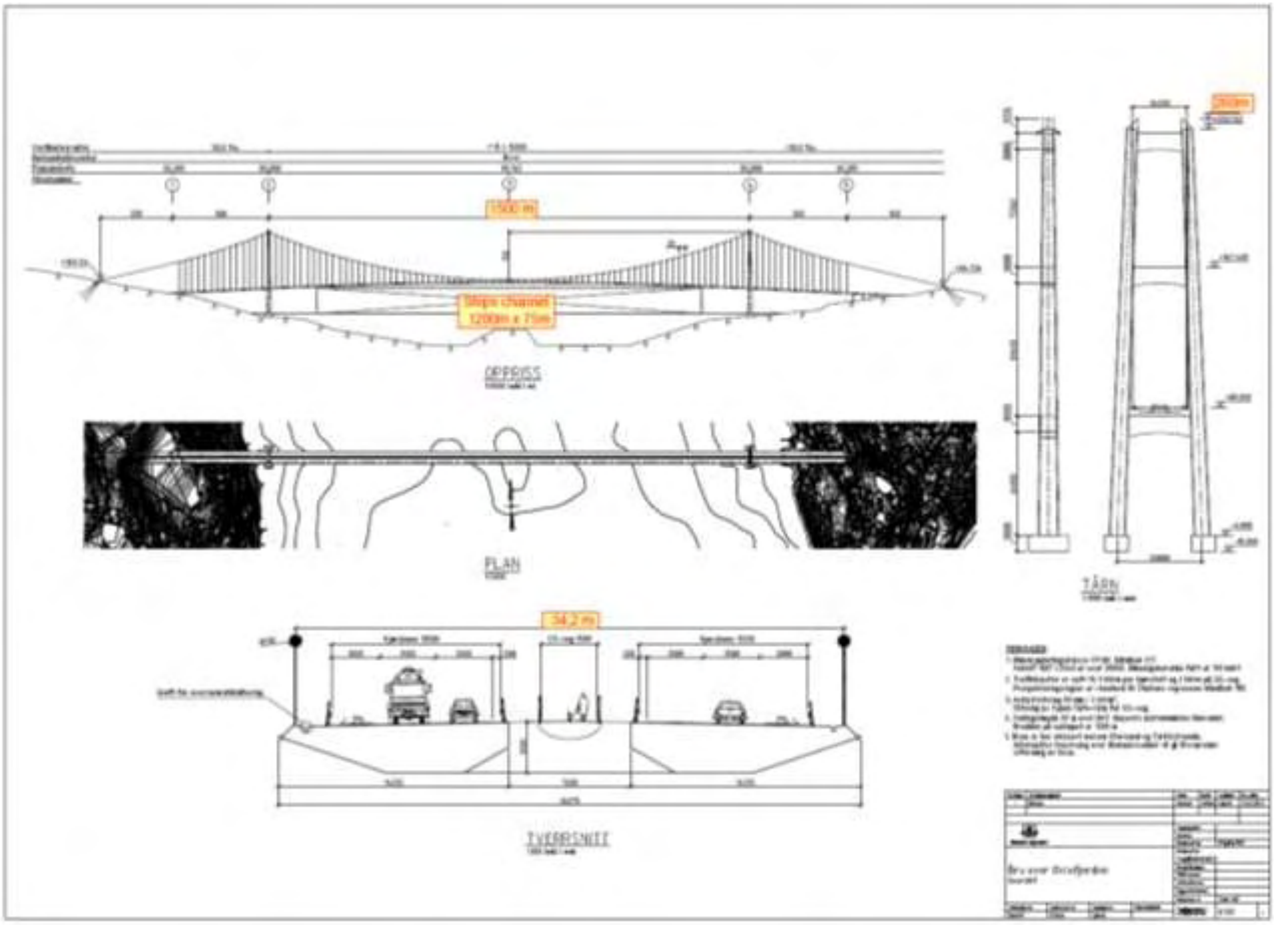


*Figure 7 Example of suspension bridge with 4 vehicle lanes and a pedestrian/cycle lane in the middle.*

The drawing in Figure 8 shows a combined rail/road bridge at crossing B with 2 rail lanes and 4 road lanes. This is a cable stayed bridge 2470 m long having 3 main towers. A cable stayed bridge is sturdier with regard to a fast rail track than that of a suspension bridge.

The free height of the ships channel in the Oslofjord has been generally defined to be 75 m and the ships shall have 2 x 750 m in each direction or a single width of 1500m in which to manoeuvre if geographically possible. In the Drøbak Channel the width is reduced. The superstructure consists of a concrete road deck and a sturdy steel girder beneath. On the south side of the bridge there is a 3.5 m wide pedestrian/bicycle lane.

This bridge has a main foundation placed on a -35 m shallow in the middle of the fjord. Due to the risk of colliding vessels, this foundation has to withstand up to approximately 30-40.000 tons of impact force. The road and rail continues into a tunnel westward, and to the east it crosses a 500 m viaduct.

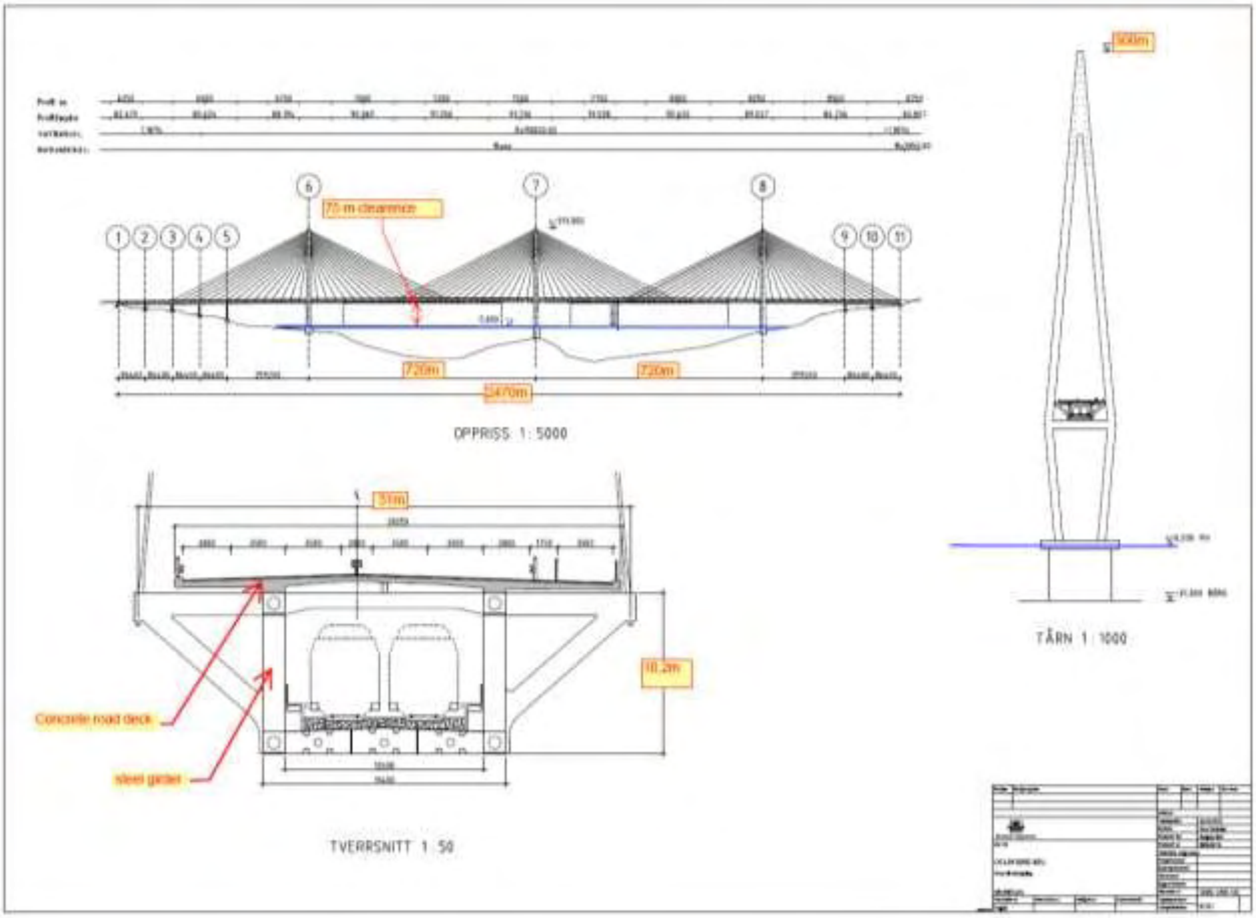


*Figure 8 Example of a combined road and rail bridge at crossing B.*

In Figure 9 the cable stayed bridge is shown in 3D. Multiple cable stayed bridges are usually pleasant to the eye when they are symmetrically built and have the superstructure high up in the air.

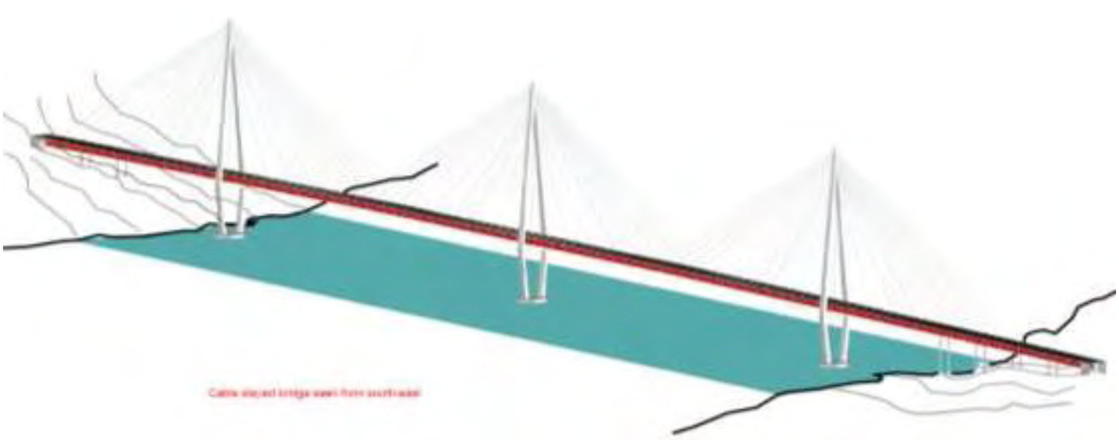

*Figure 9 Three dimensional view of cable stayed bridge.*

As will be apparent from these examples, existing technology is expected to be used when crossing the Drøbak Channel.

**TUNNELS**

EU has stated new requirements for the gradient of road tunnels as shown in Figure 10. This gives a subsea road tunnel a length of 16-20 km under the Oslofjord. A rail tunnel can here end up being nearly 50 km long! This makes it near to impossible to combine rail and road when building deep tunnels.

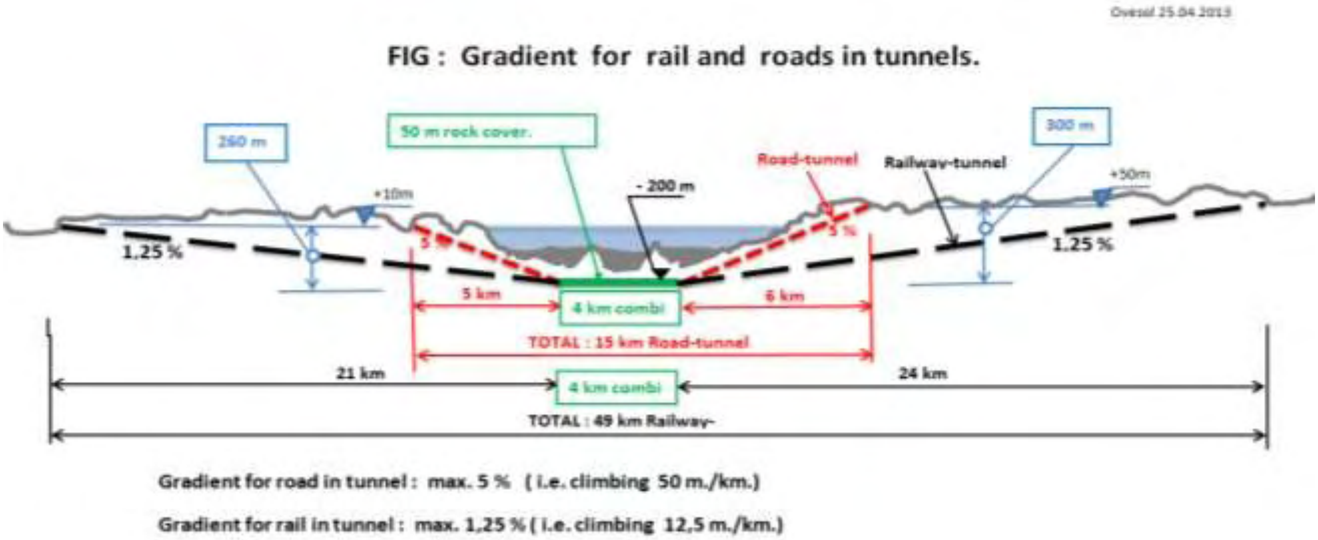


*Figure 10 Example of tunnel lengths with gradients adjusted to EU requirements.*


*Figure 11 Areal view of basin between the Cities of Moss and Horten.*

**CROSSING THE FJORD SOUTH OF THE HURUM PENINSULA**

The photo in Figure 11 gives an aerial view over the Horten Basin between the Cities of Moss and Horten. This is the shortest distance over the fjord south of Hurum Peninsula. The forest and agricultural areas which are protected by law, can be clearly seen. If a crossing is proposed which includes the Jeløy Island, a tunnel has to be placed in the rock under the island.

The ferry is the only link at present between the two sides of the fjord and runs between the Cities of Moss and Horten.  On workdays as many as 5 ferries will be busy carrying cars and people across the fjord. A ferry can be seen at the quay in Moss.

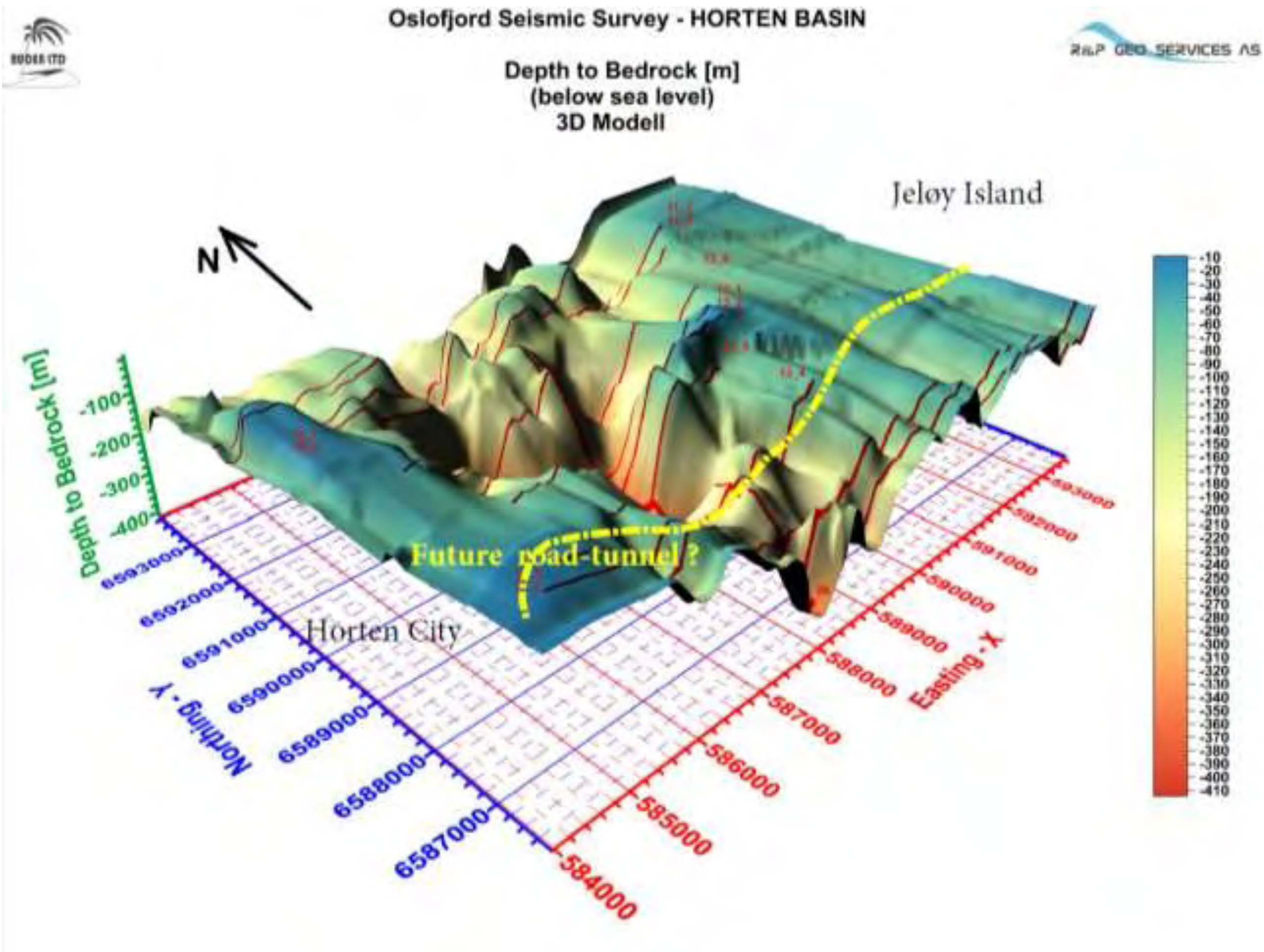


*Figure 12  3D impression of the varying undersea bedrock "landscape" of the Horten Basin.*

The 3D map in Figure 12 gives an impression of the varying bedrock "landscape" in the Horten Basin. It gives an impression of the difficulties that will be met when trying to locate suitable places for the foundations of bridges.

A subsea tunnel must waver its way inside the ridges which crosses the fjord. Here the supplementary seismic exploration being carried out this summer is strongly missed.

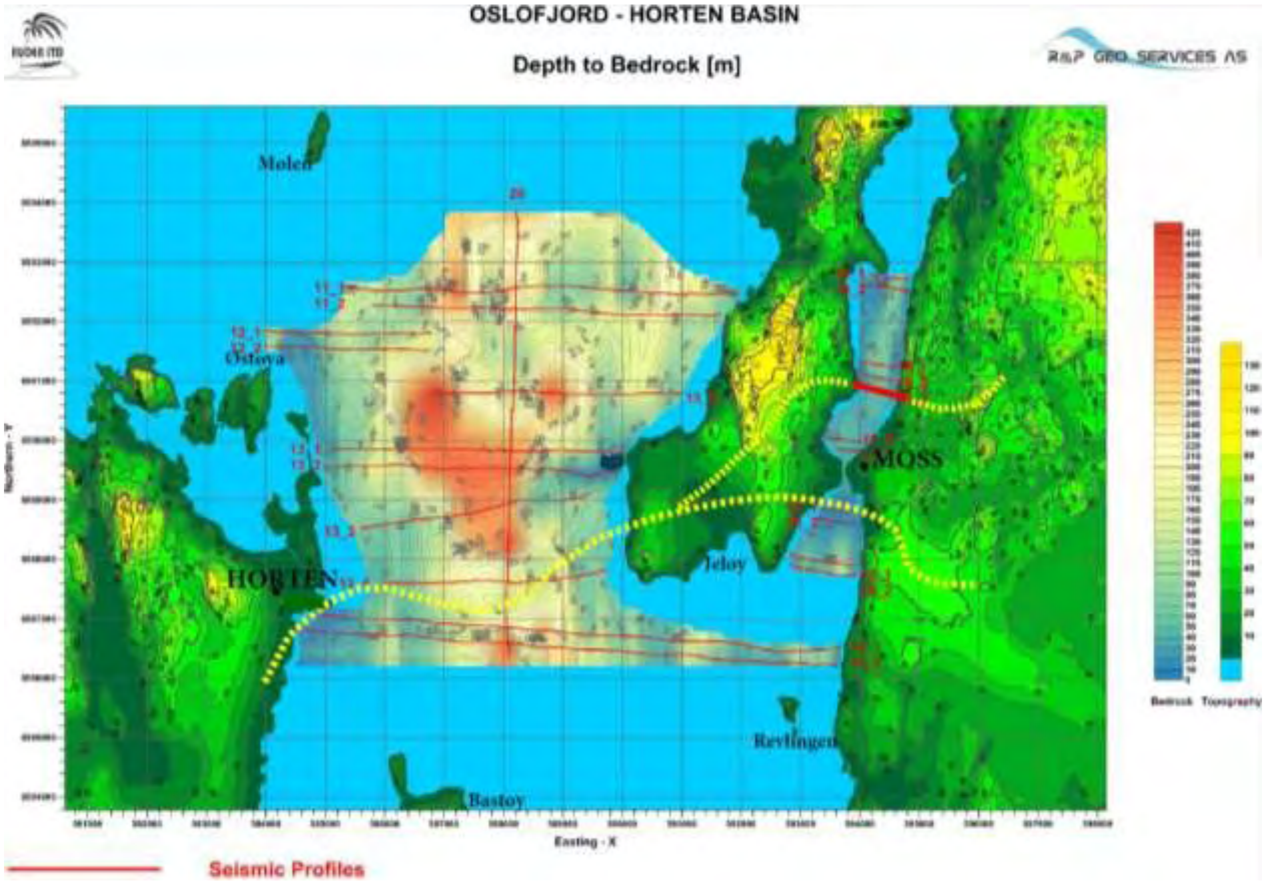


*Figure 13 Map showing bedrock surface in the Horten Basin*

In Figure 13 the bedrock in the Horten Basin is shown on an ordinary underwater map. There is a huge pothole between Horten and the Jeløy Island containing 3 km$^3$ of sediments. And the amount of trapped freshwater in this pothole is estimated to be near 1 km$^3$ - enough to satisfy the total needs for all Norwegians for 1½ year.
Between the Jeløy Island and the mainland the great fault zone in the Oslofjord is situated. This zone can make it necessary to divert a possible rock tunnel under the Jeløy Island and around Moss Bay. The map shows an example of such a road tunnel.
A bridge spanning the Oslofjord at this location will require very deep foundations - or a very long main span or floating foundations, if that is possible. The height over the water and the danger of ship collision can make such a solution near to impossible.
To illustrate the difficulties presented by the subsea terrain, a combination of rock tunnelling, an immersed tunnel and an underwater bridge is shown between Horten and Jeløy.

In Figure 14 alternative crossings between Larkollen and Slagentangen are also shown. These alternatives present similar difficulties as in the Horten Basin, perhaps more so, since the width of the fjord is greater at this location.

In order to consider what sort of crossing is most suitable for this part of the fjord, the results from the further seismic surveys to be performed this summer, is needed.

When considering the crossing possibilities south of the Hurum Peninsula, new technology will have to be implemented to varying degrees. Much of this new technology will come from concepts developed for the Coastal Highway Route E39 Project along the western coast of Norway.


*Figure 14 Alternative crossings between Larkollen and Slagentangen (blue line bridge – dotted line tunnel)*

## CLOSING REMARKS

At the present stage of this preliminary feasibility project for crossing the Oslofjord there is not enough knowledge of the fjord geology and topography to present proper solutions for a crossing south of the Hurum Peninsula. All possible crossing-scenarios are still being considered.

The present Strait Crossings and the request for a technical paper came one year to early. During next spring the feasibility project will hopefully be able to present one or two viable crossing concepts.
And thereafter it will be most interesting to see what the Norwegian government will decide with respect to the submitted concept(s) for a new link across the Oslofjord.

# THE NORDHORDLAND BRIDGE – TWENTY YEARS IN SERVICE

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## ABSTRACT

*The Nordhordland Bridge across the Salhus Fjord in Western Norway is one of Norways two floating bridges and to the author´s knowledge the world´s longest floating bridge of this kind. It replaced the most heavily used ferry route in Norway between Steinestø and Knarvik in September 1994. The total length of the bridge is 1615 m, of which the floating bridge is 1246 m and a cable stayed bridge 369 m long, with main span 163 m across a ship channel, 50 m with sailing height 32 m. The floating bridge consists of a steel box girder, which is supported on 10 concrete pontoons and connected to the abutments with transition elements in forged steel. The scope of this paper is to give a short version of the project background and construction, with focus on details relevant for operation and maintenance, an update on the owner´s experiences with respect to maintenance operations and costs, and finally some aspects concerning traffic increase and further consequences of this.*

## INTRODUCTION

The design and construction of the bridge has been reported on several occasions and this paper will therefore only give brief technical information (see reference list for more details). The Nordhordland Bridge, together with its smaller sister The Bergsøysund Bridge (845 m, finished 1992), are two bridges that were a major challenge to technology, and still after nearly 20 years are of great interest to visitors. Since this conference takes place close to The Nordhordland Bridge, we feel it right to use the opportunity to present this unique construction for this session of Strait Crossings.

## BACKGROUND

When The Nordhordland Bridge was opened to traffic on 22 September 1994 it replaced the most heavily used ferry route in Norway between Steinestø and Knarvik. The floating bridge together with the cable stayed bridge form the total bridge length of 1614,75 meters crossing the Salhus Fjord, that gave the project its original name: The Salhus Bridge.

## DESIGN CHALLENGES – ALTERNATIVES

Floating bridges with lateral moorings had already been built at depths of up to 140 metres in the U.S.A. and Canada. This lateral mooring technique could not be used in the Salhus Fjord, which reaches depths of 500 m and omitting this complicated element is proven to be a very robust and maintenance free solution. The local topography of the site meant that a 30 meter-deep underwater foundation could be built on the Klauvaskallen ridge, 170 m from shore, making a navigation channel under the high bridge section (see photo below).

*Figure 1  The Nordhordland Bridge*

**STEEL BOX GIRDER**

The steel box girder is the main load carrying element of the bridge together with the pontoons. The box section has an octagonal shape, 15.90 m wide and 5.50 m high and has an overall length of 1246 m. The free height below the girder down to the waterline is only 5.5 m but allows for passage of small boats. A combination of a ramp and viaduct lifts the carriageway up to the level of the high-level bridge for a length of approximately 65 + 350 m. The height of the carriageway ranges from 11.0 to 34.4 m above sea level.

The bridge follows the tidal variations by elastic deformations of the girder. A special type of hinge is provided at the abutments to relieve the bending moments about the horizontal axis transverse to the bridge axis (see details below).

The load from the steel box girder is transferred to the pontoons where the box girder has local strengthening at the connection points. This design of the pontoon connections were developed in the initial period of the contractors work and was favourable in terms of segmental production (40 similar pieces), partly welded with semi-automatic equipment. These segments were thereafter installed in the ten 42 m sections with high degree of accuracy. See Figure 2.

As the box girder is such a vital part of the bridge, it is satisfying that this part of the structure has proved to be a good choice with low costs in terms of maintenance and repair. However there are some concerns related to part of the painting system, especially the bottom plate of the box girder. The interior is well protected with two sets of dehumidifying units, keeping relative humidity at 40% or below.

*Figure 2 Connection segments, 2 units*

## CONCRETE PONTOONS

The 10 pontoons are constructed in light weight concrete. This was decided to achieve optimum hydrodynamic behaviour of the bridge but also seems to be favourable in protecting the reinforcement. The pontoons are all 42.0 m long, 20.5 m wide (half circle in each end).

The pontoons were constructed in a 300 m x 40 m dry dock, allowing 5 pontoons to be made at the same time. The walls of the pontoons were slip formed, which was very efficient due to the number of pontoons under production.

## FLEXIBLE PLATE CONNECTIONS

The steel box girder is connected to the abutments with 4 flexible plates in each end of the bridge. The steel plates are arranged in line, horizontally, in the centre of gravity of the box girder. They are installed as pairs, 3.5 m long, 1.6 m wide and 126 mm thick.

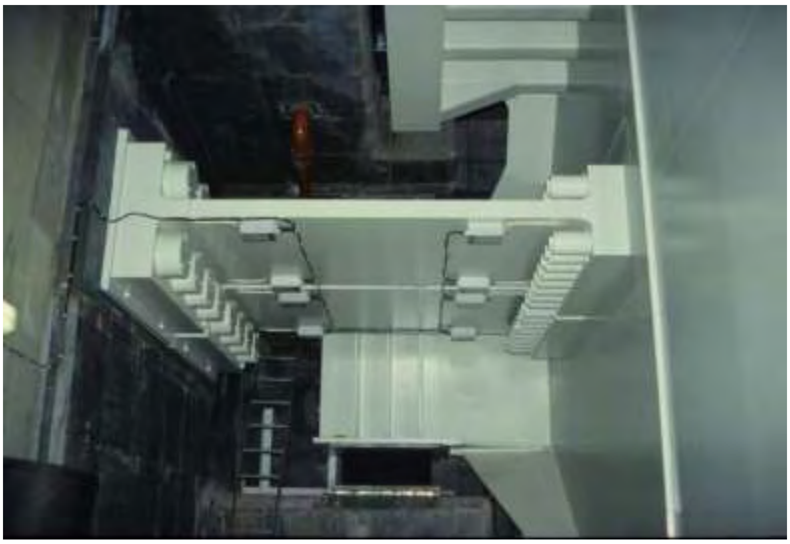

*Figure 3 Flexible plate connections (2 pairs in each end)*

Vertical shear and torsion is handled by neoprene bridge bearings installed on a console at each side of the box, as an extension of the vertical side plates (can be seen on Figure 3 in the background). These bearings have an estimated lifetime of 25 years and are easily replaceable.

This design of the connection between the box girder and the abutments with flexible plates simplified the installation, reduced the cost and is expected to require less maintenance compared to traditional bearings. During the in-service time up to now, no repair has been needed. An important service operation is to observe the stress level in the stud bolts connecting the box girder to the flexible plates. The stress in the stud bolts should be kept in accordance to specifications to avoid fatigue.

## ABUTMENTS

*The Klauvaskallen anchorage (south)*
The shore anchorage sits on the end of an underwater ridge at a depth of 30 m.
The Klauvaskallen anchorage was built in three steps: a 21 m x 20 m concrete box was produced as a separate caisson and floated to the site, on top of this the abutment block for the floating bridge and at last the pillars for the junction of the main span of the high bridge with viaduct of the floating bridge.

*The Flatøy anchorage (north)*
This is on dry land and consists primarily of a massive concrete block, 22 m long, 20 m wide and 14.5 m high. The abutment block has been cast directly on bedrock in the blasted-out foundation pit.

## TRANSPORT AND ASSEMBLY

The 11 preassembled steel box girder modules, with lengths between 92 and 156 meters, were transported from Fredrikstad on North Sea barges to the assembly site in Lonevåg, a sheltered fjord about 7 nautical miles from the bridge site, where the bridge assembly took place.

On three of the modules, the viaduct sections (connection between floating and cable stayed parts of the bridge) were installed prior to the barge shipping. This was a challenging operation, where all factors had not been taken into consideration. When the last, end-section module, with the highest part of the viaduct, was under tow, parts of the viaduct were lost at sea. This piece of bad luck hampered the installation and eventually delayed the opening of the bridge with almost 6 months.

## INSTALLATION

From the assembly site in Lonevåg, the whole floating bridge was towed to the bridge site by two powerful tugs together with three smaller tugs positioned along three of the pontoons for better manoeuvring.

The installation was a complex operation with small margins. Techniques developed for installing offshore platforms were used to hook the bridge to the abutments and making the connection permanent.

As a consequence of the floating bridge, the clear headway for the main ship route through the Salhus Fjord was reduced to 32.0 m.

## OPERATION AND MAINTENANCE

For a floating bridge lack of buoyancy in any of the pontoons is critical. Also a direct impact on the steel box girder may cause reduced capacity or even full closure. The obvious incident is a ship colliding with the bridge. To avoid this, a radar beacon system is installed in the ship channel. But any system is depending on qualified operation and that action is taken if a ship is out of course. During the 19 years of operation only one incident (June 2009) has been detected, when a small cargo vessel collided with the anchorage on Klauvaskallen. The concrete construction could stand the impact better than the ship (see facsimile from local newspaper BA)!

*Headline says: "Is probably due to illness"*


*Figure 4 Ship collision with Klauvaskallen anchorage (June 2009)*

OECD recommends a maintenance budget of 1 % per year of the investment cost. Compared to an estimated new value, this would be equivalent to NOK 10 mill per year. Since cost has just passed a total of (nominal) NOK 20 mill (€ 2.65 mill) during the first nearly 19 years in-service, The Nordhordland Bridge has been "a low cost object" to maintain up to now. In addition to this the cost of electrical power is charged on a different account (internal systems and road light).

The expansion joints and the bearings on the viaduct carriageway have turned out to be a weak point and extensive repair work has been carried out during the period from the opening of the bridge.

Another problem (related to the joint construction) is the generation of noise from the joint fittings, which is a nuisance for the traffic, but most for the neighbours ashore. Part of the problem is that the joints are not positioned over the column axis, where displacements and rotations caused by the traffic load would be at a minimum.

Even so that the traffic increase has been considerable during the years in service, the renewing of the asphalt has been accomplished only once. The operation was done in two stages; the high bridge carriageway in 2002 and the floating bridge with viaduct in 2003. Due to load restrictions and the overall end result it was needed to mill off the top surface. Asphalt specification is Ab composed with a rock material which is wear resistant, also wear from studded tires used in the winter period.

*Figure 5 Road surface with upper viaduct joint (April 2013)*

According to maintenance plans, the steel box girder of bridge was scheduled for a refinish (top coating) several years ago (12 years after opening) and normally this should be done without extensive repairs. However, the environmental conditions have proved to be much harsher than expected (remember that the bridge girder is situated between 5.5 and 11.5 m above sea surface), and this gives a situation where the chlorides are a continuous threat for the protection system. An inspection report carried out recently, revealed serious problems on the protection system, concentrated to the bottom plate of the box girder [12]. This part of the box girder is exposed to accumulation of the chlorides from the salt water below. Surprisingly, the upper parts and side panels, which are flushed by occasional rain, were better than expected. The report states comprehensive problems both with white rust and also full exposure of the steel surface due to rust "break-through" of the protection system. Not unexpected, the viaduct with columns and superstructure are far better off, being elevated with more distance from the sea surface. Detailed plans for a full recoating of the bottom plate are in process (2013), and there is certainly a risk of higher cost for this operation than expected. The viaduct has already (2011) been repainted (top coat on the underside of the superstructure).

*Figure 6 General situation on the bottom plate surface*

The problems with the protection system on the steel structures, are at this stage not serious, but will develop rapidly if not corrected. In the abutment areas and inside the steel box girder, the maintenance situation is satisfactory.

The pontoons and the deep water abutment on south shore seem to be well protected by the cathode protection system, and testing shows that the anodes protect the reinforcement far above the water line. There are however some concern for the top deck on the pontoons. The sea gulls produce (tons of?) guano which is an aggressive substance on the top deck. Probably this will need a treatment with a membrane combined with at top layer for protection.

The electrical installation also includes detectors on doors and hatches (on the pontoons). These have been renewed. The dehumidification installation (2) in the box girder gives ideal conditions in this area, not only for the electronics but also for the box girder itself. It has also saved painting of 90.000 sqm. inside the box girder.

On the high bridge the only significant problem has been related to the cable stays. To avoid vortex induced vibrations on the stays under certain wind and weather conditions, it was already before opening of the bridge decided to install interlocking ropes (cables) between the stays. These are highly stressed and exposed to shifting dynamic loads causing fracture in the ropes.

## TRAFFIC AND MAINTENANCE

Today´s traffic volume is hard to understand, compared to the capacity of three big ferries running in this connection until the bridge opening in September 1994. Average day traffic the last year was near to 4400 units (AADT). By the second quarter of 2013, the traffic is passing 16.000AADT.

The traffic on the bridge is a great challenge when the maintenance and inspection works are carried out. Without the pedestrian/bicycle lane (width 3.00 m) outside the traffic area, this work would have been extremely difficult to execute (see photo below). But any major repair work on the bridge will undoubtedly cause problems for the traffic flow and in worst case higher risk of accidents.

Higher demands for general safety in road work, leads to a question whether bridges like this should be planned with more space for maintenance and repair work. In practice a dual carriageway solution?

*Figure 7 Morning traffic April 2013*

**TRAFFIC INCREASE AND FUTURE**

A more fundamental question related to traffic volume is what scope we have for a transport system like this. There is an ambiguity in the fact that we are only planning for a capacity that is barely sufficient according to the estimates for the traffic needed to pay back the toll loans. And in the same time there seems to be an underlying wish that the traffic do not increase too much! And when the traffic increases far above the estimates made, we don't have proper explanations.

For the case of the Nordhordland Bridge, no complete before-after survey was accomplished. There is therefore a lack of knowledge relating which processes are steering the increase of traffic when shifting from a ferry based transport system to a fixed link. From a similar fixed link, The Askøy Bridge, near the centre of Bergen, several travel surveys were carried out and from these some comparisons and conclusions can be relevant. According to one of the reports the number of trips increased between Askøy and the centre of Bergen when the bridge opened. Result from the transport model (general tool for estimating traffic), show the opposite development. This is a consequence of the way the transport model is defined. The transport model is not able to calculate change in mode choice correctly. A better description of the relationship between car-availability-groups will probably improve the transport model. This will influence results of trips distribution and mode choice [9].

From another report analysing benefits of different road projects, it is observed that three cases (The Nordhordland Bridge included) replacing ferries proved to have higher traffic than the highest expected traffic volume from the models. There are no definite conclusions why this is the case, but it seems that replacing a ferry with a fixed link gives a higher benefit/gain than earlier estimated [10].

The replacement of a ferry with a fixed connection, gives at least two obvious improvements: An opportunity to travel at any time, with all the flexibility this gives (no timetables) and a significant capacity increase compared to the ferry. These "obvious" observations seem to be partly confirmed in the references.

The practical problem is that more of our new bridge connections are not planned or built for these increase in traffic. Nevertheless, the floating bridge has been designed for using the footpath and cycle track as a future lane No.3 for traffic. The added traffic load was therefore planned for by extra ballast in the pontoons, and this will be removed when the third lane is taken into use. The pedestrian-cycle lane has to be moved onto a cantilevered structure (possibly both sides of the bridge), but no preparations have been made during the initial construction phase. This option seemed many years into the future when the bridge was finished, but now, only 19 year after, it will soon be in the planning stage.

## FINAL COMMENTS AND CONCLUSIONS

The floating bridge project has proved to be a robust bridge with no serious problems related to the main elements of the structure (included the high bridge). In spite of the problems mentioned, to my opinion, the construction and operation of The Nordhordland Floating Bridge has been a great success. In addition to the protection system on the steel box girder the main problems are related to all movable parts and also the electrical system, The costs for maintaining and, eventually, re-establishing the protective paint system will still sum up to a cost estimate much lower than the 1% figure foreseen by the recommendations from OECD.

## KEY FIGURES – SUMMARY

The bridge investment was 910 mill. NOK (€ 120 mill) in nominal value (accounts for 1987-1995) and was financed by 95 % toll. The construction costs included "everything" except interest on the loans. The repayment period was estimated to 13-14 years, but already on 31. December 2005 it was close-down for the toll collection, 11 years, 3 months and 9 days after opening. The total repayment on the loans plus interest and toll collecting costs, summed up to 1 457 mill NOK (about € 192 mill.).

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**Photographs**

Figure 1, 2, 3, 5, 6, 7: Jan Olav Skogland, NPRA.

Figure 4: Facsimile from Bergensavisen, June 2009. Copyright Photo by Eirik Hagesæter. Permission 04.04.2013

# THIRTY YEARS OF EXPERIENCE WITH SUBSEA ROAD TUNNELS IN NORWAY

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## ABSTRACT

*The first Norwegian subsea tunnel was opened in 1983. Subsequently，31 other subsea tunnels have been built. As a result in the 2012 a total of 32 subsea tunnels with a total length more than 126 km are open to traffic. Some new are under construction and several other subsea tunnels are also in the process of being planned，including tunnels of up to 24 km length. Most of the tunnels have one tube with two lanes，but some tunnels have an extra lane (overtaking lane) when the gradient is more than 6 %. The subsea road tunnel projects are located on the trunk roads along the coast replacing often congested ferry connections，and establishing ferry-free connection from the main land to the communities. Experiences from these tunnels are the theme for this presentation, but also design criteria, safety aspects and constuction methodes. Generally speaking，building costs for subsea tunnels have been reduced over the years. However，costs vary a great deal from project to project. Operation and maintenance costs also vary considerably. Costs for reinvestment and equipment are particularly high. Water ingress has diminished over time，so that the need for pumping leakage water has been reduced.*

## 1 INTRODUCTION

There are more than 1000 road tunnels in Norway
Until the end of the seventies in the 20th century numerous large bridges were built over Norwegian fjords or between some of the many islands along the coast. As the bridge span and lengths increased so did the costs and alternative methods of crossing such as pontoon bridges immersed- tube tunnels and subsea rock tunnels were considered.

The first Norwegian subsea road tunnel was built at Vardø, Norway's most easterly town (Figure.1) between 1979 and 1983. After the Vardø tunnel was opened in 1983, 31 others have been built and opened to traffic. Today we have about 126 km of subsea road tunnels in Norway. Most of the tunnels have only one tube with two lanes, but some tunnels have an extra lane (overtaking lane) where there are steep gradients. Some of the tunnels have also two tubes.

Figure.2 shows the length and depth of some subsea tunnels in Norway.


*Figure.1  Maps showing subsea tunnels*

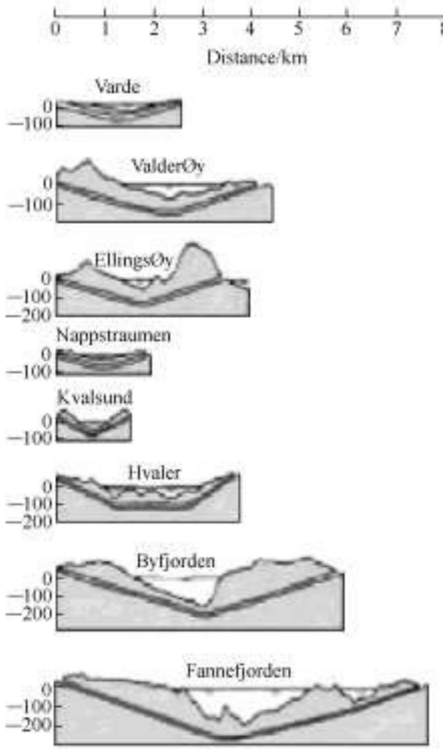


*Figure.2  Lengths and depths of some subsea tunnels*

## 2 EXPERIENCE

The following tunnels (Table 1) are now in use in Norway.
The annual average daily traffic (AADT) varies considerably from tunnel to tunnel, from AADT approximately 100 veh/day to 100000 veh/day.
The gradient in Norwegian subsea tunnels varies from 6% to 10%.
The following table shows also the length of the tunnels that are now open to traffic, *AADT and the year of opening.*

*Table 1 Subsea tunnels that are in use in Norway*

| Tunnel name | County | Road No. | Length (m) | AADT | Years of opening | For short |
|---|---|---|---|---|---|---|
| Hvaler | Østfold | Rv 108 | 3751 | 1300 | 1989 | HVA |
| Oslofjard | Akershus/Buskerud | Rv 23 | 7252 | 6000 | 2000 | OSL |
| Flekkerøy | Vest-Agder | Rv457 | 2327 | 1500 | 1989 | FLE |
| Byfjorden | Rogaland | Ev 39 | 5875 | 3000 | 1992 | BYF |
| Mastrafjord | Rogaiand | Ev 39 | 4424 | 3000 | 1996 | MAS |
| Talgjefjordtunnelen | Rogaland | Rv 519 | 5685 | 1000 | 2009 | |
| Karmøytunnelen | Rogaland | RV 47 | 8900 | 4000 | 2013 | |
| Bjorøy | Hordaland | Fv 207 | 2000 | 350 | 1996 | BJO |
| Bømlafjord | Hordaland | Ev 10 | 7900 | 2500 | 2000 | BØM |
| Halsenøy | Hordaland | RV 544 | 4150 | 750 | 2008 | |
| Knappetunnelen | Hordaland | RV 557 | 5400 | 14000 | 2013 | |
| Skatestrsum | Sogn og Fjordane | RV 616 | 1902 | 250 | 2002 | |
| Fannefjorden | More og Romsdal | Rv 64 | 2713 | 1500 | 1991 | FAN |
| Freifjord | More og Romsdal | Rv 70 | 5086 | 1850 | 1992 | FRE |
| Ellingsøy | More og Romsdal | Rv 658 | 3520 | 3000 | 1987 | ELL |
| Valderøy | More og Romsdal | Rv 658 | 4222 | 3000 | 1987 | VAL |
| Godøy | More og Romsdal | Rv 658 | 3844 | 750 | 1989 | GOD |
| Eiksund | Møre og Romsdal | Rv653 | 7765 | 1500 | 2008 | |
| Atlanterhavstunnelen | Møre og Romsdal | Rv 64 | 6766 | 1500 | 2009 | |
| Hitra | Sor-TrØndelag | Rv 714 | 5645 | 1100 | 1994 | HIT |
| Froya | Sor-TrØndelag | Rv 714 | 5305 | 550 | 2000 | FRO |
| Skansentunnelen | Sør Trøndelag | Rv 706 | 715 | 8000 | 2010 | |
| Nappstraumen | Nordland | Ev 10 | 1780 | 600 | 1990 | NAP |
| Sloverfjord | Nordland | Ev10 | 3200 | 250 | 1997 | SLO |
| Tromsoysund* | Troms | Ev 8 | 3376 | 7000 | 1994 | TRO |
| Kvalsund | Troms | Rv 863 | 1650 | 500 | 1988 | KVA |
| Maursund | Troms | Rv 866 | 2122 | 600 | 1991 | MAU |
| Ibestad | Troms | Rv 848 | 3398 | 400 | 2000 | IBE |
| Ryatunnelen | Troms | RV 858 | 2660 | 2500 | 2011 | |
| Varde | Finnmark | Ev 75 | 2892 | 700 | 1983 | VAR |
| Nordkapp | Finnmark | Ev 69 | 6826 | 300 | 1999 | NOR |
| Bjørvika | Oslo | E 18 | 1100 | 100000 | 2010 | |

### 2.1 Construction Costs

The total construction costs to day, based on current technical requirements, typically vary from NOK 120 000 per meter per tube to NOK 180 000 per meter per tube, depending on size of the cross section and geological conditions. All costs are based on year 2012.

### 2.2 Operation and maintenance General

The most extraordinary problem in subsea tunnels is algae. This phenomenon exists in a number of tunnels. It appears to be no connection between types of rock and the presence of algae.
Experience particularly from Vardø tunnel, would tend to suggest that the algae population expands to a certain leve1 before collapsing and starting all over again.

Seawater leakages on the asphalt surface make the asphalt quite slippery, possibly because of the algae.

Shotcrete is broken down by seepage，particularly in salt water. Poor quality shotcrete is much more susceptible than high quality shotcrete. Consequently，new and more stringent rules have been made for the use of shotcrete in tunnels.

So far corrosion has not resulted in any great problems for subsea tunnels. However，electrical equipment，pumps and piping have had to be replaced in several tunnels because of corrosion. In future tunnels，more attention must be paid to the choice of corrosion resistant materials. Materials for installations in subsea tunnels must be supplied in stainless steel.

Damage to aluminum linings by salt water has been registered in the Freifjord and Fannefjord tunnels as well as in some others. The corrosion damage to the aluminum linings due to seawater is of such a scale that the linings must be replaced. Replacement work has already started at Freifjord and Fannefjord tunnels.

### 2.3 Operation and maintenance Cost

When the first subsea tunnel at Vardø was opened in 1983，there was little relevant experiences from operation and maintenance of subsea tunnels. One of the main problems with the Vardø tunnel was too little capacity in the buffer reservoir for water leakages when the pumps broke down. Problems with emergency power，pumps and a special alga resulted also in high costs. Annua1 operation costs were more than NOK 1 600 per meter per year (costs in 2012). These costs have now been considerably reduced, but vary from tunnel to tunnel according to traffic volume, technical installations and type of lining.

Typical operation and maintenance cost is from NOK 400 to NOK 1500 per meter per tube per year, depending of tunnel length and traffic volume.

There are a number of installations in the tunnels that have to be periodically replaced. These include pumps，drainage pipes，electrical installations and water and frost linings. The annua1 costs for these items are not included in operation and maintenance cost above. The costs of improving and replacing installations and water and frost linings in some of the tunnels can be quite expensive in the year it is affected

The dominant operation and maintenance costs are attached to electrical power supply for lighting and ventilation.

Figure.3 shows how costs are distributed between lighting，ventilation，pumping and other uses in the Ålesund tunnels. Ventilation costs take the highest share.

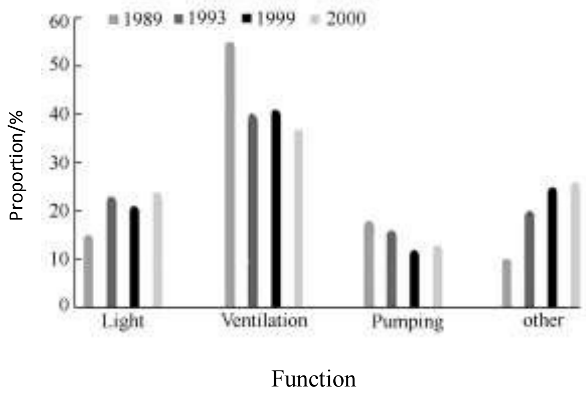


*Figure 3 Typical electrical power supply cost in subsea tunnel (Ålesund tunnels)*

Pumping costs are relatively low. In virtually, all subsea tunnels water leakages have been reduced after the tunnels have become operational. In some tunnels, the reduction has been more than 50% in relation to initial water leakages. It would appear that the tunnels have a certain self-sealing capacity.

The most probable reason for this is that particles in the rock cracks move and reduce cavities, and that minerals in the rock swell and close the cracks.

## 3.0 GEOMETRIC DESIGN

### 3.1 General

Tunnels are different from open roads in respect of conditions such as:
• little or no lateral movement
• other winter conditions
• regular lighting throughout the day and year, except from the entry zone
• difficulties in estimating gradients
• difficulties in estimating distance to vehicle in front
• other safety measures, breakdown services, etc.
These require that a number of design elements will differ to those of the open road.
Maintenance and operations shall ensurea constant level of safety in the tunnel.
Important elements in this connection are:
• selection of the appropriate construction method and equipment in the planning and construction phases
• uniform standard for tunnels along the same road with corresponding traffic type and volume.
The demands placed on standards increase correspondingly with traffic volume and tunnel length.
Tunnels are therefore placed in categories which determine the required geometric specifications and features.

### 3.2 Selection of tunnel category

The traffic volume is normally given in AADT (Annual Average Daily Traffic volume).
AADT is the total annual traffic a year divided by 365 and is given as the total traffic volume in both directions.
The tunnel category is determined according to the estimated traffic volume twenty years after opening, AADT(20).
Where the traffic volume varies throughout the day or over the year, or where there is considerable uncertainty in calculating AADT(20), the tunnel category may be based on selected criteria.
The tunnel categories are based upon traffic volume (AADT) and tunnel length (Km). (Figure 4).
The tunnel categories are the basis for a specific cross-section, number of tubes, need for emergency lay-bys and turning points together with safety equipment.
Example: 2 x T9.5, means two tunnel tubes, each with a width of 9.5 m.

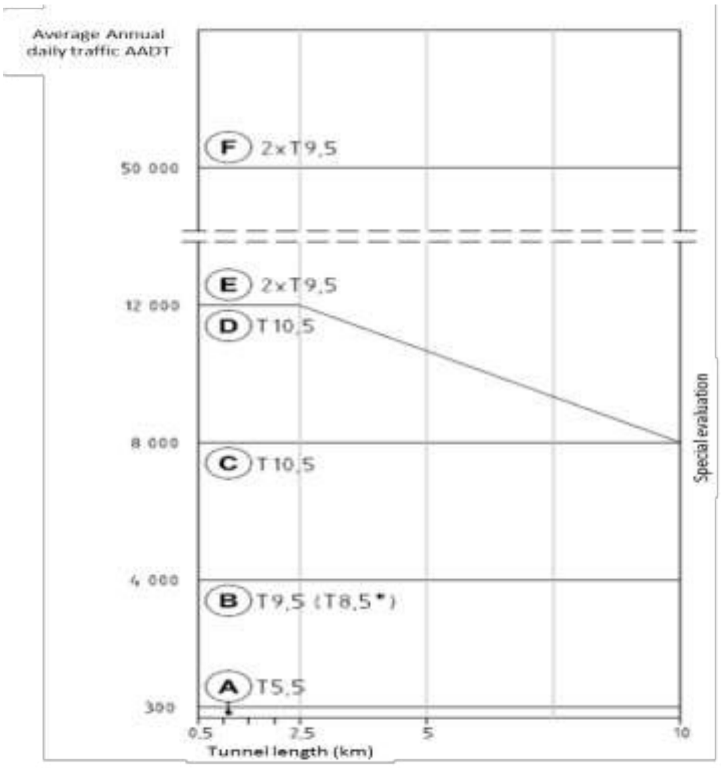


*Figure 4 Tunnel category*

### 3.3 Tunnel cross-sections

The tunnel cross-sections are designated according to the total width of the road surface (Figure 5).
The vertical clearance requirements in tunnels are 4.6 m. The vertical clearance specifications apply to the vertical distance measured on the carriageway boundary.
Normal cross-sections will be in excess of this to allow for:

• Extra clearance for subsequent road resurfacing, normal tolerance for tunnel linings, water and frost protection / concrete linings (total deviation = 0.1 m)
• Requirements for vertical clearance including kerbstone.

Normally the tunnel cross-section will also include space for traffic signs and technical installations. The need for extra width locally must be considered in each individual case. The minimum height for
technical equipment must be 4.8 m above the carriageway. For laterally-mounted equipment such as traffic signs etc., the clearance must be individually determined.

With consideration to emergency exits laterally mounted signs should be placed such that the minimum height below the sign is at least 2.0 m.

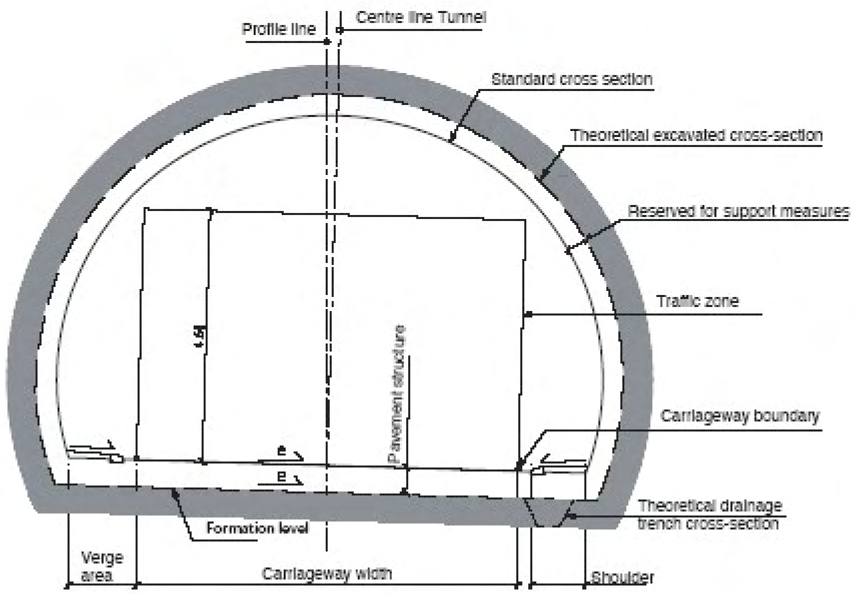


*Figure 5. Cross-section*

**3.4 Design and location of emergency lay-bys and turning points**

Emergency lay-bys enable parking outside of the carriageway in the case of emergency.
Emergency lay-bys are designed as in Figure 6.
Turning points are built into two-way tunnels. Emergency lay-bys can also function as turning points for light vehicles.
Turning points for heavy vehicles are designed as in Figure 7.
Technical equipment is located in separate niches with an enclosing wall along side the traffic lane. These niches should be located together with the emergency lay-bys.
The distance between the lay-bys is determined by the tunnel category. The distances given are approximate. The location will depend upon the local circumstances including rock mechanics and geometric considerations. Further, consideration must be made to designing niches for several purposes (for example, technical room, pump station etc.). Deviations in location should be within ±50 m for emergency lay-bys and ±100 m for turning points.

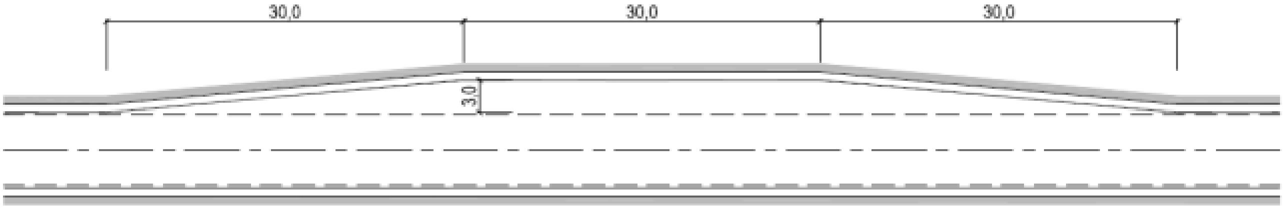


*Figure 6 Emergency lay-by*

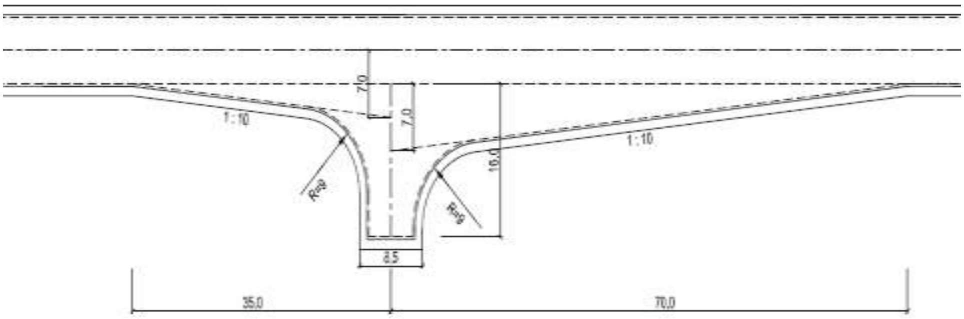


*Figure 7 Tourning point*

### 3.5 Vertical curves for subsea tunnels

The maximum gradient for subsea tunnels is shown in Figure 8
Where an overtaking lane is constructed, the values in Figure 8 may be increased by 1%.
Tunnels with local characteristics and low traffic volume, together with urban tunnels outside the main road network, may be constructed with a gradient of up to 10% for subsea tunnels of local character and low traffic volume.
The AADT values for one-way traffic in Figure 8 apply to both tunnel tubes in aggregate.

| | Two-way traffic | | One-way traffic | |
|---|---|---|---|---|
| AADT (20) | 0 - 1 500 | > 1 500 | < 15 000 | > 15 000 |
| Max.gradient | 8 % | 7 % | 7 % | 6 % |

*Figure 8 Permitted gradients for subsea tunnel*

### 3.6 Overtaking lanes

The need for an overtaking lane is based upon estimated capacity. In tunnels with two-way traffic and a gradient of > 6% over a stretch exceeding1 km, a separate overtaking lane shall be constructed when the AADT(20) is > 2500.vehicle/day

## 4.0 INVESTIGATIONS

### 4.1 General

Before the construction of a subsea tunnel a geological site investigation has to be carried out. The investigation will determine the length and location of the tunnel. During the building period the nature of the preliminary investigations is carefully examined and compared with the last sample regularly taken along the proposed route of the tunnel.

A relatively simple geological investigation has been carried out for most tunnels， supplemented by acoustic measurements and seismic profiles where necessary. Rock core sampling from hard rocks has only been used to a limited degree on a few projects. These relatively simple investigations worked satisfactorily until the construction of the Bjorøy tunnel. The same simple procedures were used here for the preliminary investigation. However，during construction， problems arose when a fault with younger rock had to be crossed. The fault was very difficult to work in， and made progress difficult for the contractor. At a later date， problems arose in parts of the Nordkapp tunnel， and the same occurred in the Oslofjord tunnel， where a fault filled with sand and grave1 had to be frozen in order for the tunnel to be driven. These setbacks have resulted in more thorough preliminary investigations for the new projects. Comprehensive rock sampling from boreholes was used in the preliminary site investigations for the Frøya and Eiksund tunnels. This is a complicated and lengthy process that can easily take several years to complete.

### 4.2 Rock cover for subsea tunnels

The minimum thickness of rock cover is a decisive factor for deciding tunnel length. The less cover is permissible, the shorter the tunnel is. However，the chances for problems during construction increase with a reduction in cover， and therefore a minimum rock cover should not be less than 50 m， unless reliable investigations of the rock surface are available and for rock of good stability and good quality.

## 5.0 CONSTRUCTION METHODS

Norwegian specifications are based on the drill and blast method, and we use mostly the bid – build model and unit price contracts for our tunnel projects. The Norwegian tunnelling method regards the rock that we are tunnelling in as a construction material, and support methods are determined by assessment of rock quality at the tunnel face. This implies that actual quantities may differ from the contract's bill of quantities, and we require a flexible contract for adjusting quantities for both support ahead of the tunnel face and support measures, and a clause to adjust construction time accordingly.

All Norwegian subsea road tunnels have been built by conventional drill and blast methods. Construction time is dependent on support measures that must be taken, and particularly those that have to be done at the tunnel face. In recent years methods for shotcrete and concrete shuttering have been improved so that these operations are both faster and better results are achieved.
Bolting is the mostly used method of support and is commonly used in conjunction with shotcrete. Figure.9 and 10 shows the relationship between planned and actual amounts of bolting and shotcrete.

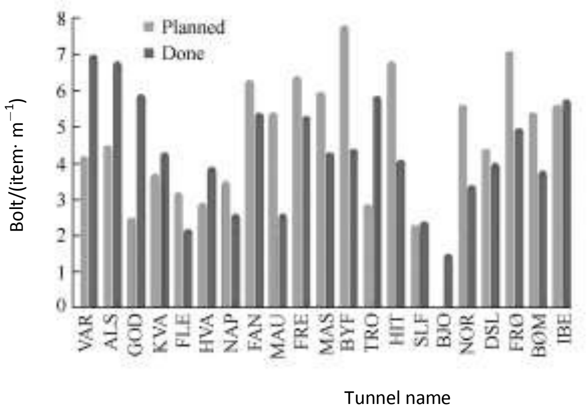


*Fig.9 Bolting per meter tunnel: planned and executed*

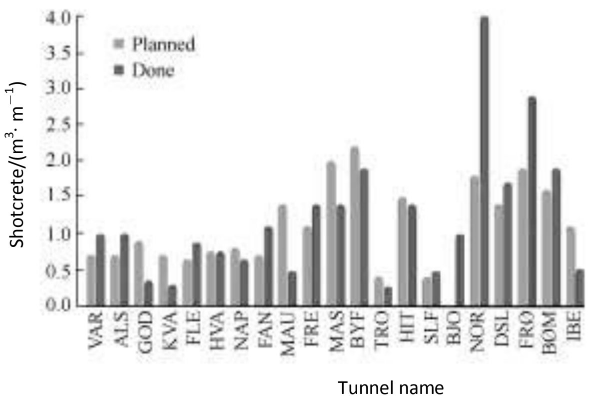


*Fig.10 Shotcreting: planning and executed*

The use of shotcrete has increased from 0.7–1.0 $m^3$/m tunnel to 1.5–2.0 $m^3$/m tunnel in some of the last completed tunnels. In the first tunne,l Vardø, concrete of C25 quality was used for temporary support. Experience has shown that C25 is of a too poor quality to use in tunnels, and it has now been replaced by C45 for shotcreting below sea level.

Initially, much use was made of concrete lining, but with time and experience, there has been a noticeable reduction in this expensive and time-consuming method, figure 11. However the Nordkapp tunnel is an exception to the rule, as the extremely poor rock conditions have resulted in almost 50% of the tunnel being lined with concrete. This also explains the high costs for the tunnel.
Water ingress, and the need for its prevention is very difficult to ascertain prior to construction. Factors governing leakage are rock types, crack patterns and the amount of clay in the cracks.
Figure.12 shows the amount of water leakage from each tunnel at the time of opening. This can be

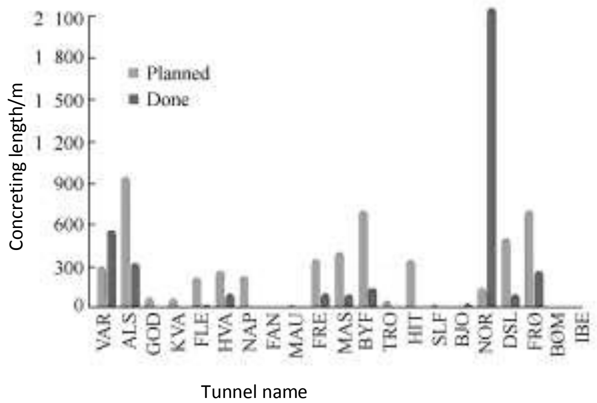


*Fig.11 Concreting, planning and executed*

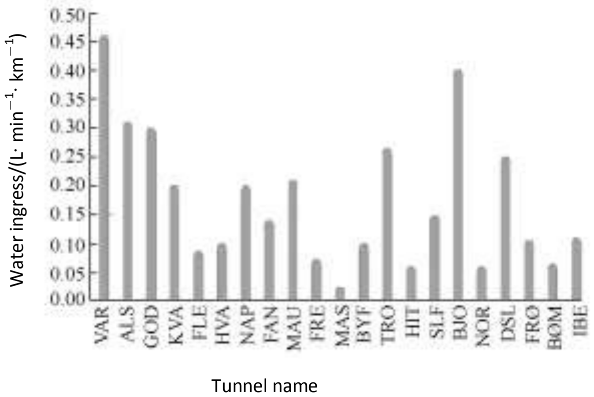


*Fig.12 Water ingress at time of openings*

compared with the amount of injection that is shown in Figure.13. Up to the present time, the Vardø tunnel has the highest water leakage rate, but it must be said that it is also one of the tunnels with the least amount of injection.

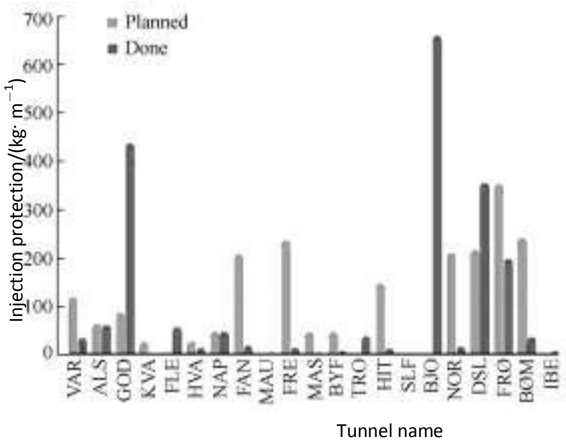


*Figure 13. Injection：planned and executed*

Figure.14 shows the amount of water/frost protection installed in Norwegian subsea tunnels. Apart from the large quantity used in Vardø and Ålesund， there is very little correlation between the quantity of water ingress and the amount of water/frost protection

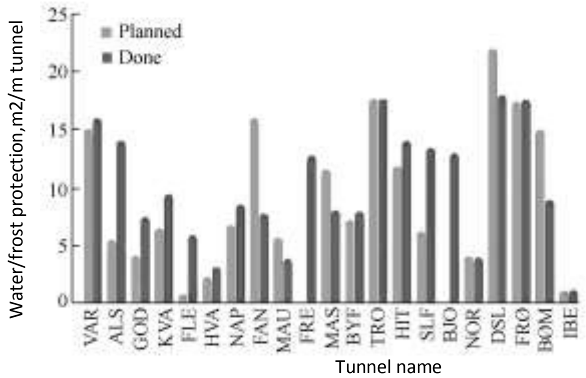


*Fig.14 Lining - Protection against frost and water, planned and executed*

## 6.0 SAFETY

### 6.1 General

The level of safety in a tunnel shall resemble that of the open road. Control that safety objectives have been achieved, are made with the aid of a risk analysis.
The tunnel category determines the specifications of safety equipment in the tunnels. The tunnel categories are illustrated in Figure 4,
Tunnels with a length of between 250 m and 500 m are placed in a tunnel category which is lower than that suggeste by the designed traffic volume.
For tunnels shorter than 250 m, demands are made only upon lighting.

The principles for evacuation are based upon road-users making their way out on foot or using their own vehicle. In tunnels with two-way traffic, facilities shall be available allowing road users to turn and drive out again. Turning points and emergency lay-bys are constructed as given in Section 3.3

Tunnels with two parallel tubes, emergency escape is made through footway interconnections.

Figure 15 shows the safety equipment in various tunnel categories.

| EQUIPMENT | TUNNEL CATEGORY | | | | | | NOTES |
|---|---|---|---|---|---|---|---|
| ● Obligatory<br>◷ Evaluated | A | B | C | D | E | F | |
| Emergency lay-bys | | ● | ● | ● | ● | ● | See Ch. 4 "Geometric design" |
| Turning points | | ● | ● | ● | | | |
| Escape possibility by foot | | | | | ● | ● | Interconnections every 250 m |
| Power supply, lighting and ventilation | See Chapter 10 "Technical equipment | | | | | | |
| Emergency power supply | | ● | ● | ● | ● | ● | Lighting in the event of power failure<br>See Sections 602.201 and 1003.6 |
| Emergency exit lighting | | | ◷ | ● | ● | ● | Approx. every 62.5 m.See Sec. 602.202 |
| Emergency Exit sign | | | | | ● | ● | Also obligatory in other categories if the tunnel is constructed with alternative emergency exits,e.g.interconnections. See Sections 602.203 |
| Emergency telephone | | ● | ● | ● | ● | ● | Category B: Approx. every 500 m 1)<br>C: Approx. every 375 m 1)<br>D: Approx. every 250 m (both sides) 1)<br>E: Approx. every 500 m 1)<br>F: Approx. every 250 m 1) |
| Fire extinguishers | ◷ | ● | ● | ● | ● | ● | Category B: Approx. every 250 m 1) 2)<br>C, D: Approx. every 125 m 1) 2)<br>E: Approx. every 125 m 1)<br>F: Approx. every 62,5 m 1) |
| Water for fire extinguishing | | ● | ● | ● | ● | ● | Alternative solutions in Section 602.206 |
| Flashing red stop signal | | ● | ● | ● | ● | ● | See Section 602.207 |
| Remote controlled barriers | | ◷ | ◷ | ◷ | ◷ | ● | Evaluated on basis of expected frequency of use. See Section 602.208 |
| Changeable signs | | ◷ | ◷ | ◷ | ◷ | ◷ | See Section 602.209 and 603 |
| Lane signals | | | | | ◷ | ◷ | See Section 602.209 |
| CCTV surveillance | | | | | ◷ | ◷ | See Section 602.210 and 603 |
| Communication and broadcasting equipment | | ● | ● | ● | ● | ● | See Section 602.3 |
| Mobile telephone | | ◷ | ◷ | ◷ | ◷ | ◷ | Determined in conjunction with mobile teleph. company |
| Height control barrier | ● | ● | ● | ● | ● | ● | See Section 602.211 |

1) Emergency telephone and fire extinguisher additionally installed outside each tunnel entrance (See Section 414.2)

2) Fire extinguishers mounted on one side at given intervals. In addition, fire extinguishers are located together with all emergency tele phones on the opposite side

*Figure 15 Safety equipment in the various categories*

A solid circle indicates that the stated equipment is obligatory.
An open circle indicates that the need to install this equipment has to be considered, and the equipment shall only be installed if it can be documented that there are special circumstance which deem this necessary.

In Figures 16 and 17, the location of emergency lay-bys, emergency telephones and fire extinguishers are indicated, as an example for tunnel categories B and E.
In addition, an emergency telephone and fire extinguishers shall be placed outside each tunnel opening.

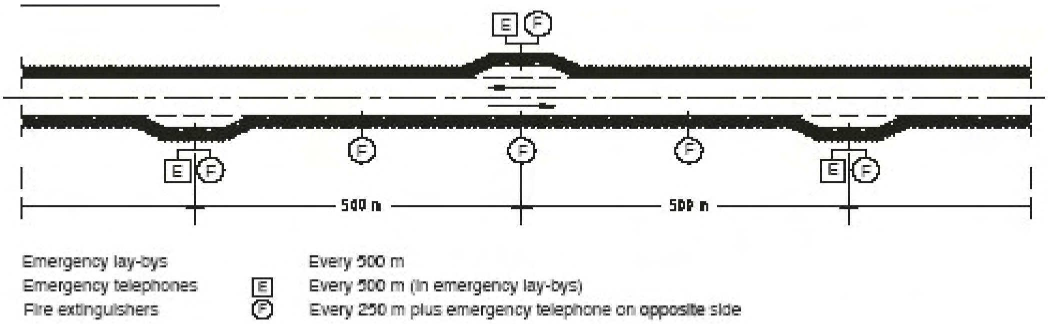


*Figure 16  Lay-bys and safety equipment, tunnel category B*

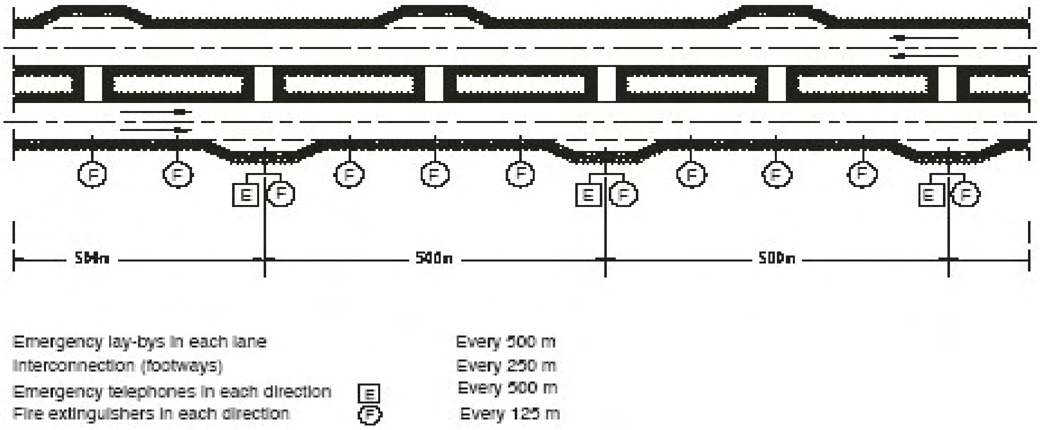


*Figure 17   Lay-bys and safety equipment, tunnel category E*

# 7.0 TECHNICAL EQUIPMENT

## 7.1 Specifications for technical equipment

All equipment shall be CE approved.
The lifespan must be evaluated for each individual component based on estimates of approximate lifespan costs.
The atmosphere in the tunnel is corrosive. This is due to condensation arising from warm, damp air The water may be weakly acid due to nitrous acid and nitric acid from the nitrous gases in the exhaust.
Equipment must therefore be protected from corrosion or be constructed from corrosion- resistant materials such that the minimum recommended lifespan is achieved. In tunnel categories C, D, E and F, together with tunnels characterised by a particularly corrosive environment, (for example sub-sea tunnels), cable duct bridges and light fittings must be supplied in stainless steel.

## 7.2 Ventilation

In Norway we always use longitudinal ventilation system for road tunnels.

Mechanical longitudinal ventilation is based on the use of impulse ventilator fans. In long tunnels and those with a high traffic volume, or where specific pollution regulations apply in the areas around the
tunnel entrances, ventilation with the aid of a ventilation shaft will be appropriate.

The ventilation equipment shall be designed to cope with expected pollution levels 10 years after opening of the tunnel (AADT(10)).
With a normal mixture of exhaust gases it is only necessary to determine the level of permissible concentration of carbon monoxide (CO-gas) and nitrogen dioxide (NO2 gas). The concentration of the other toxic gases does not present a health hazard if a sufficient dilution of CO and NO2 gases is achieved. In order to attain sufficient control of gas concentration in the tunnel, measuring instruments should be installed in the middle of the tunnel and at both ends. In one-way tunnels, measuring equipment is not required in the entrance zone.
The measurement range for CO shall be a minimum of 0–300 ppm and 0–25 ppm
for NO2

A system of longitudinal ventilation may be constructed with or without ventilation
shaft/side-adit. The air flow may be calculated as pipe flow and a simple equation for air movement in the tunnel may be formulated.
The forces which generate ventilation in a tunnel are of three types:
* mechanical ventilation force
* meteorological ventilation force
* piston effect from vehicles.

### 7.3 Fire ventilation

Where ventilation equipment is installed in a tunnel, this must be designed with regard to fire ventilation requirements. In the event of fire and smoke, the air speed should be able to be reduced to the lowest possible speed, yet sufficient to direct the smoke in the desired direction.
The ventilation direction will be determined in consultation with the fire authorities and incorporated into an emergency plan.
The ventilation equipment must be designed so as to be able to control a fire from 20 MW to 100 MW, dependent upon the tunnel category.
In order to achieve the necessary control of air flow in the tunnel, the ventilation shall be designed such that:
• it has the capacity to counteract the build-up of pressure in the tunnel.
On account of the general upward buoyancy, external wind and the fire itself, together with natural draughts on account of temperature differences inside and outside the tunnel.
• sufficient capacity is ensured in the ventilation equipment for the necessary time for the tunnel to be evacuated.

## REFERENCES

**(1)** This paper is based on a publication about subsea road tunnels in Norway from year 2002. The title of the publication is "Publication no. 98，Subsea road tunnels in Norway" of Directorate of Public Roads，Road Technology Department.

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**(4)** Moderne vegtunneler – Drift og vedlikehold av vegtunneler. Hovedkostnader – 2011-12-02.

# DESIGN BASIS FOR STRAIT CROSSINGS

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## ABSTRACT

*In case a bridge or a submerged tunnel is selected for a strait crossing, the design basis must accurately be determined to ensure safe construction and operation. The design basis relates to the physical environment and to traffic conditions as well as to relevant accidental conditions and selection of an appropriate safety level.*

*In this paper we will discuss the selection of safety level for the construction and operation phase as well as the combination of loads to be considered. It will be suggested that learning from oil and gas projects be taken into account.*

*Furthermore, emphasis will be on the most important loads from the physical environment. Wind and potential for vortex shedding and flutter are normally given high attention in bridge design and it is obvious that the wave effects must be accounted for should parts of the bridge be supported on bottom founded structures or on buoyancy elements.*

*The importance of currents will in this paper be highlighted with emphasis on extreme tidal effects in combination with storm surge. In a large storm, the water level in a fjord system can be raised to a high level due to the storm surge effects and a strong outgoing current can be expected after the storm event. To obtain reliable statistics is impossible and modeling of potential extreme values for design is important.*

*Finally we will discuss relevant accidental loads and call for proper risk analysis to identify realistic worst case scenarios.*

## INTRODUCTION

There is to day an ongoing discussion in Norway on how to promote the construction of bridges and tunnels to ensure easy communication across the country. It is the general opinion that a communication link without ferries represents an advantage from a commercial point of view. Huge time and cost savings are expected and such a communication link will represent larger flexibility to choose time for transport and travelling. The total savings for transport of goods from Kristiansand to Trondheim could be a reduction in travel time from 20 hours to 13 hours in case no ferries are needed for the distance along the West coast of Norway, [1].

The crossings of the wide Norwegian fjords could in this scenario be considered as Strait Crossings and would in some cases represent very challenging design and construction projects, [2].

So far the discussion has been concentrated on the huge investments needed for such a transport link and the potential savings for the society and the commercial actors. Furthermore, the Norwegian Road Administration in their report [2] puts emphasis on the potential fjord crossing concepts and their feasibility in terms of technical robustness and costs.

If, or when, a decision to proceed with the planning is made, concept selection for several of the fjord crossings will be taken. The concept selection will have to be based on a number of factors, including technical feasibility, maintainability and life cycle cost issues. Furthermore, aspects of safety will play a major role in the concept selection.

In order to ensure the technical and economical robustness for a project, a design basis must be collected. Some of the fjord crossings considered will represent world records with respect to tunnel lengths or bridge spans and the design basis must take into account all parameters that could influence on the concept selection and on the concept design. Learning from successful projects in the oil and gas industry should in this respect be useful and should be taken into account. In these projects all work starts from a defined design basis which in some cases might have to be updated causing needs for design updates.

The required format for a detailed design data base for the Norwegian Fjord crossings should be of general interest for all those considering Strait Crossings and some of the key technical requirements are discussed in this paper.

We should diversify the design basis to cover:

- the temporary construction phase
- the in place built phase (the Design phase)
- the strait crossing operations where procedural measures could be taken to reduce risk to the public, for example from falling ice. This is normally done by closing the crossing for the public.

## DATA FOR PHYSICAL ENVIRONMENT FACTORS AND ASSOCIATED LOADS

Physical Environmental data are most important for the design. The crossings shall be operational for most weather conditions with a minimum of closure time (operational requirement) and shall be sufficiently strong to survive the actual design loads which occur with a specified annual probability of exceedance (design requirement). In the oil and gas industries the design requirement is represented by the load effect having an annual probability of exeedance of 1% for elastic design using the appropriate load factors and material factors. Furthermore, the structures shall be able to sustain without collapse the load effect having an annual probability of exceedance of 10 Similar criteria must be agreed for the strait crossings.

For the temporary construction phase it is of importance to ensure that damages due to harsh weather are limited to acceptable levels. In fall and winter the weather conditions can be very strong necessitating measures to avoid structural damages or hazards to the construction workers.

The question then arises as to which environmental factors are important and how to obtain extreme values for these factors.

### Wind

The wind load is most important for the design of a bridge, both with respect to the overall load and the variability of the load. It is suggested that the wind load be established through:

- Measurements of the average wind profile at the site and the associated wind gusts.
- Modeling (in a wind tunnel) using the topography of the site in a wind tunnel to ensure

that all possible wind conditions are taken into account. Of particular concern is large downward flowing air from the mountains (squalls), a situation that is known to cause damages to several buildings at Møre during the Storm of January 1 1992, [3], [4]. This storm was reported to be a storm situation with an expected return period of 200 years.

- The wind tunnel modeling of the local site conditions must be followed up with proper wind tunnel modeling of the bridge under the estimated extreme and possible resonance wind conditions.
- Numerical modeling of the bridge to sufficient details as to reveal all degrees of freedom where resonances could occur due to various types of wind excitation [5]. After all, the strongest design wind may not represent the largest vibrations of the bridge which will occur in case of resonances. For bridges longer than existing bridges, special care should be taken to avoid potential resonance situations.
- We should also note that strong side winds could cause operational limitations and possible temporary closure of the bridge

**Waves**

Wave effects on bridge piers and in particular on floating bridge elements, if selected, must be given sufficient attention:

- It is expected that wave statistics be collected and that the extreme situations be calculated using documented statistical models. One should notice that the highest waves will occur where the wind has the largest fetch length. Special analysis using wind and wave data from unfavorable directions should be undertaken. Possible limited amount of wave data for these directions will influence the statistics. Special wind conditions, for example squalls, should be considered.
- With respect to wind generated waves, the locals should be consulted to ensure that no "freak" phenomenon occurs due to local topography.
- Normally, tsunami generated waves are not considered. There are, however, locations, also in Norwegian fjords, [6], where tsunamis caused by rock fall or landslides may form. In case of such risk, the strait crossing concept should be adjusted so as to avoid a potential catastrophe. It should be noted that effects of potential global warming may give rise to more unstable slopes and increase the possibility for large landslides, also along Norwegian fjords.

**Currents**

For any strait crossing where the current in the strait is important, the estimation of the design current shall be given particular attention:

- The design current consists of a number of components where the tidal component represents the daily variation of the water flow through the strait.
- The effect of storm surges in a wind blowing towards land shall also be accounted for. During this wind situation, the general water level in an enclosed area (for example a fjord) increases. Following the wind, the water level will return to normal, associated with a strong return surface current. It is very difficult to obtain reliable statistics for this phenomenon and physical modeling might be necessary in a model environment.
    - During the design of the Troll to Mongstad oil pipeline, the design current in Fensfjord was estimated, however, there were large uncertainties with respect to the quality of the storm surge extreme estimates.
    - With respect to Fensfjord also colder and therefore heavier Atlantic waters from time to time flow along the bottom and over the outer sill of the fjord. This creates a strong current along the seafloor, [7], that does not influence the surface

current. The pipeline was, however, laid along the fjord and the current's influence was limited.

For a strait crossing, the storm surge current component can represent the design event and the current estimates should be carefully prepared.

- With respect to storm surge currents, the locals should be consulted regarding possible "freak" phenomenon. It would be expected that "elders" would have sufficient historical knowledge to report on extreme situations even though these are expected to be very infrequent.

Furthermore, strong tidal currents in some straits or narrows cause erosion under bridge piers, requiring large repair works to be undertaken.

- o An example is the Tromsøsundet Bridge connecting the Tromsø Island to the mainland where repairs have been undertaken regularly. In this respect the design should consider:

- Scour protection (rock dumping) around the bridge piers to avoid erosion of the bridge foundation.
- Regular inspection program to keep track of possible scour.

**Atmospheric icing**

Icing on bridges could represent operational limitations:

- It is known the bridges across the Danish straits close from time to time due to danger of falling ice caused by atmospheric icing on cables. In 2006 the Great Belt Bridge was closed for hours while the Øresund Bridge connecting Denmark and Sweden was closed for up to one day. According to [8], Denmark's Great Belt Bridge was, over the 3-year period 2004-2007, closed for 12 hours per year because of falling ice.
- Atmospheric icing can also give rise to wind-ice vibrations. As e.g. reported on long hangers of the Great Belt Bridge, [16].

**Foundation aspects**

Foundation failure may cause severe consequences and should by all means be avoided:

- Some strait crossing concepts depend heavily on anchoring to the sea bottom. It is therefore of utmost importance that the bottom soil conditions are mapped carefully prior to final design and prior to signing any fabrication contract.
- Piles in tension might not represent sufficient safety should buoyancy elements be selected for the elements of the crossings. This could be buoyant pipes or tension cables fixed to the ground. It is, therefore, recommended that gravity elements be used to ensure proper on the bottom stability.
- Piles drilled into rock and fixed by grouting would, however, give sufficient safety providing the required safety factors are properly implemented.

**Sea ice**

Sea ice is most relevant in certain areas of the world. The 13 km long Confederation Bridge connecting the Prince Edward Island with New Brunswick mainland Canada has been designed to resist loads from drifting ice. As ice loading is not so relevant for European Strait Crossings, we will rather direct the reader to proper references than include a discussion of the design of bridge piers against loads from drifting ice, see [9] and [10].

## TRAFFIC CONDITIONS

International and national standards related to traffic loads on bridges are implemented in Norway, [11], [12].

- As Strait Crossings are exceptionally costly, it could be considered to increase the design "point load" locally to allow very heavy objects to pass over the bridges in certain situations.
- One could also consider the design total traffic loads, although it should be realized that the crossing may see large traffic accumulating in case of a stop in the traffic flow, however, operational measures could be put in place to limit the number of vehicles in case the traffic comes to a standstill.

Another design requirement is sailing height for vessel in case of bridges where the sailing height should reflect the height of vessels normally passing the strait. This could in many instances be set to the sailing height of cruise ships where heigh tide conditions are taken into account. It should be noted that limited sailing height may restrict the passage of drilling rigs for the oil and gas industry.

Also the sailing depth will be an important parameter to include in case a submerged floating tunnel is selected as concept for the strait crossing. The criteria must be agreed with relevant authorities.

## SAFETY CONSIDERATIONS AND ACCIDENTAL SCENARIOS

Large efforts must be undertaken to identify design accidental situations. These conditions will include but not be limited to the following:

### Ship impact

An impact could occur between a bridge pier and a drifting ship or with a ship out of control, [13]. Strong currents could cause such impacts. The piers of the bridge should be designed to withstand a "design impact" which has been identified after a proper risk analysis where the traffic in the strait is being analyzed. Mitigating measures would be procedures (e.g. reduced vessels speeds, requirement for captain to be on the bridge) or design of protection barriers around the bridge pier.

### Tunnel fire

A tunnel fire could cause extensive damage to the built structure besides representing large risk to the public. A strait crossing tunnel made of concrete or with concrete lining would need a layer of fire resistant material.

### Water filling

Water filling would represent the ultimate hazard for the public and might cause the loss of the tunnel be it a buoyant structure. Only through the risk analysis will causes and design mitigating measure be identified to ensure that such a situation has extremely low probability. Water filling could, for example, be caused by dragging anchors, earthquakes damaging the connection to the land or design events not properly analyzed in the design phase.

## NEEDS FOR HAZIDS AND RISK ANALYSIS

In order to ensure that hazardous design situations are identified and accidental design loads are being defined and implemented, there is a need for proper hazard identification and risk analysis.

### Acceptance criteria

Acceptance criteria for project have to be prepared, [14]. This includes criteria for allowable structural damage and financial loss, allowed levels of pollution (which in the case of a strait crossing should be very low) and required safety level for the construction workers and the public.

We will suggest that the construction workers should be protected similarly as the workers in the oil and gas industry. A recommended goal would a FAR level of 5 although the project should strive towards no fatalities. A FAR level of 5 means that there are 5 fatalities for every 10  hours worked on the project.

The public should be protected to the same level as for new roads where the roadways are divided by barriers. A FAR level of 1 to 2 should be obtained for public using the strait crossing.

### Hazard identification

A very thorough hazard identification exercise (Hazid) would be needed for a strait crossing project as such project would involve new challenges and new risks through non-identified causes for hazards. The Hazid team should involve experts from several disciplines and with experience from design, project execution and operational aspects. Some project managers are not open to identification of hazards and see some Hazid experts as potential project busters. The Hazid team should, however, be seen as contributing to giving the project the necessary background for project sanction. A proper hazid is therefore a most important tool for those making decisions and sanctioning the project funding.

### Risk analysis

In a risk analysis the probability for an event as well as mitigating measures to reduce the consequences of the event is being identified. Mitigating measures could be constructive including protection devices or they could be procedural. First a qualitative risk analysis is carried out to identify quantitatively the risk and this is possibly followed by a quantitative risk analysis to quantify that the mitigating effects to ensure that he risk is being reduced to the acceptable level for the crossing project.

## QUALITY REQUIREMENTS TO ENSURE LOW LIFE CYCLE COSTS

Some of the Norwegian road bridges are not up to the standard expected by the public. Poor workmanship and overly optimistic design assumptions, in particular with respect to the required amount of concrete coverage of the rebar have caused severe chloride penetration and corrosion of certain bridges [15]. The design and in particular the construction must take account of this phenomenon and the following should be specified in the Design Basis:

- Sufficient concrete cover over the rebar is required to avoid chloride penetration into the rebar. In the oil and gas industry 40 mm is required. To save some concrete seems to be a very poor idea for life cycling cost control
- The concrete must be dense to avoid chloride attack on the concrete. Dense concrete fabrication will be ensured through vibrating the concrete during the concreting work.
- The concrete should not get into contact with sea water during the concreting work to avoid initial chlorides in the concrete.
- The aggregate should be strong and chloride free. Granite of good quality is recommended while limestone may not be recommended.
- Impressed current is often used to serve as anodes limiting the development of bridge corrosion. The use of such system should be planned initially.

## CONCLUSIONS

A Strait Crossing is considered to represent an investment of large importance for a nation, a Strait Crossing project must therefore be very well planned and it is expected that the cost and schedule estimates are kept.

The design basis for a Strait Crossing project is a very important document. It should be ensured that design data are available to avoid excessive design modifications in later project stages, normally associated with huge cost overruns or project delays.

The design basis should be extended to include a risk analysis of accidental scenarios to ensure that the Strait Crossing is designed to resist the likely accidental situations that may occur and that the safety for the public is to the standard set in the accept criteria document of the project.

One could argue that implementing the learning from the oil and gas industry would lead to very costly projects. On the other hand, it could be argued that some of the offshore oil and gas projects have been handled with great professionalism, in particular those where a proper design basis laid the ground for a well-defined concept and a smooth project execution.

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# FACTORS AFFECTING OPERATION, MAINTENANCE AND COSTS IN SUBSEA TUNNELS

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## ABSTRACT

*In Norway we now have 32 Subsea road tunnels, and more are to be built in the coming years. Up to now operation and maintenance cost has been underestimated. Partly because there has been a relatively low standard in the road tunnels in Norway, but also because of the new regulations given among others from the EU. Another parameter is that operation and maintenance cost has not been sufficiently taken into account when planning and building new tunnels. The objective of the operation and maintenance of subsea road tunnels: • Maintenance of all functional requirements for the tunnel and installed infrastructure • Optimization of the maintenance level and frequency. General definition of operation and maintenance -Operations are all the tasks and procedures that are required for a construction or an installation to work as planned - Maintenance is all of the measures that are necessary to maintain a plant or a construction on a fixed-quality (maintain a certain minimum quality requirement). The choice of solutions based on the optimal life span for equipment will become more and more important for operation and maintenance. Subsea road tunnels have a characteristics of steep gradients when you drive in and out of the tunnels. And of course the water coming in to the tunnels has to be pumped out. A special program called PLANIA is used to take care of the planning of maintenance and also the documentation of the work we have done in the different tunnels.*

## 1. INTRODUCTION

Subsea road tunnels in Norway are built to standards mostly governed by traffic intensity; Low volumes of traffic – low standards. High traffic intensity – high standards.
In both cases, safety and security inside the tunnel shall match the same on the outside roads.


*Figur.1. Map showing the geology in Norway.*

*Figur2. Map showing the road tunnels in Norway.*

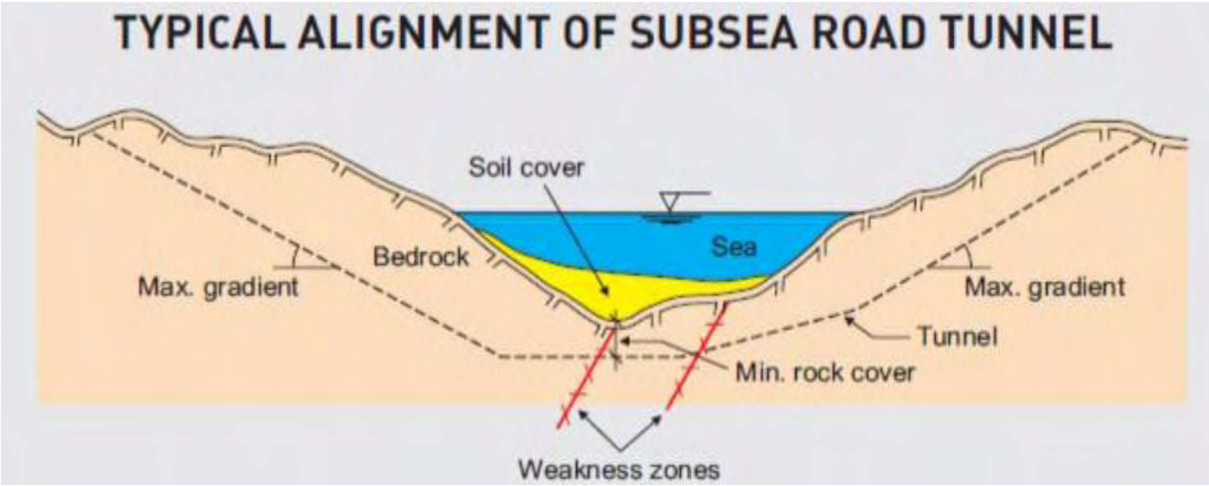
The accidents records over many terms of years also show this to be true. In fact, there are fewer accidents per length of road inside the tunnels than on the outside.

During operation of the tunnel projects there are other essential key issues which are related to such topics as; ventilation, illumination (normal operation and emergency mode), safety aspects, rescue operations, long term stability and durability of rock support, drainage and water handling, contingencies, rescue and evacuation plans. These are important for an effective operation of any tunnel project, but local regulations and standards may govern the details. Norwegian tunnels have been associated with a cost and time efficient tunneling concept – even in the long run including maintenance and repair. An operational and maintenance function must contribute positively to the function of the tunnel in relation to the expenses that have been used. There is only one way the tunnel owner can contribute in this context, and that is availability = quality concerning the flow of traffic. An optimal effort is therefore required in order to accomplish this. Conditions affecting tunnel maintenance are already determined from the moment the planning of the tunnel begins. Already in the early stages of planning, as one starts to describe the design of the tunnel, knowledge is needed about which conditions that can affect operation and maintenance. The standards and solutions that are chosen will always influence future operational procedures and maintenance requirements.

Compared with "conventional tunnels" under land, subsea tunnels are in many ways special. Regarding engineering geology and rock engineering, the main characteristics are:

• The main part of the project area is covered by water. Special methods for investigation are therefore required and the interpretation of investigation results is more uncertain than for tunnels under land.

• The locations of fjords and straits are defined often by regional faults and weakness zones. The deepest part of the fjord, and hence the most critical section of the tunnel, often coincides with particularly distinct zones.
The potential of water inflow is unlimited, and due to the gradients, all leakage water has to be pumped out of the tunnel.
The corrosive character of leakage water represents considerable problems for tunnel excavation and rock support. In Norway, we have nearly 1100 road tunnels. 32 of these are subsea road tunnels.
Corrosion is a general problem in subsea tunnels and there are instances of safety equipment that is heavily corroded.




*Figur 3. Typical alignment of a subsea tunnel*.

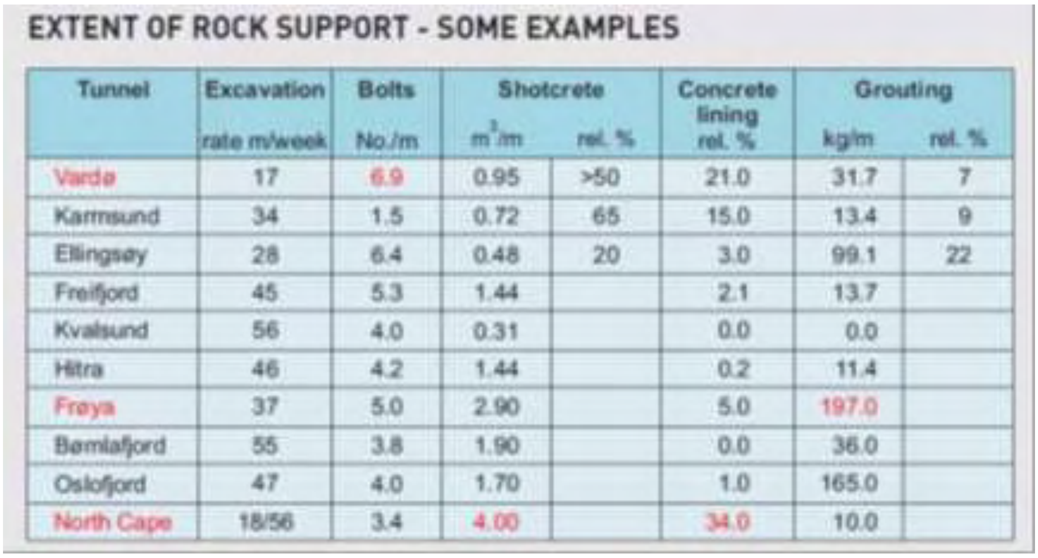
## 2. DEFINITION OF OPERATION AND MAINTENANCE

The objectives of the operation and maintenance of road tunnels are:
• Functional requirements for the tunnel and its equipment should be maintained
• Functional security should be attended to
• Safety equipment must comply with the given requirements
• To aim for equal standards for comparable systems
• New constructions should ensure future needs
• To aim for an optimization of the level of maintenance.
• Operations are all the tasks and procedures that are required for a building or an installation to work as planned
• Maintenance is all of the measures that are necessary to maintain a plant or a building on a fixed-quality
The aims must be more closely linked to:
• maintenance costs
• operational availability
• life span
• accessibility

• injuries
• security

An optimal maintenance means the lowest possible maintenance costs, short shut-down time, good operational availability and operational security, the longest possible life span of constructions and equipment and the safeguarding of the level of security.

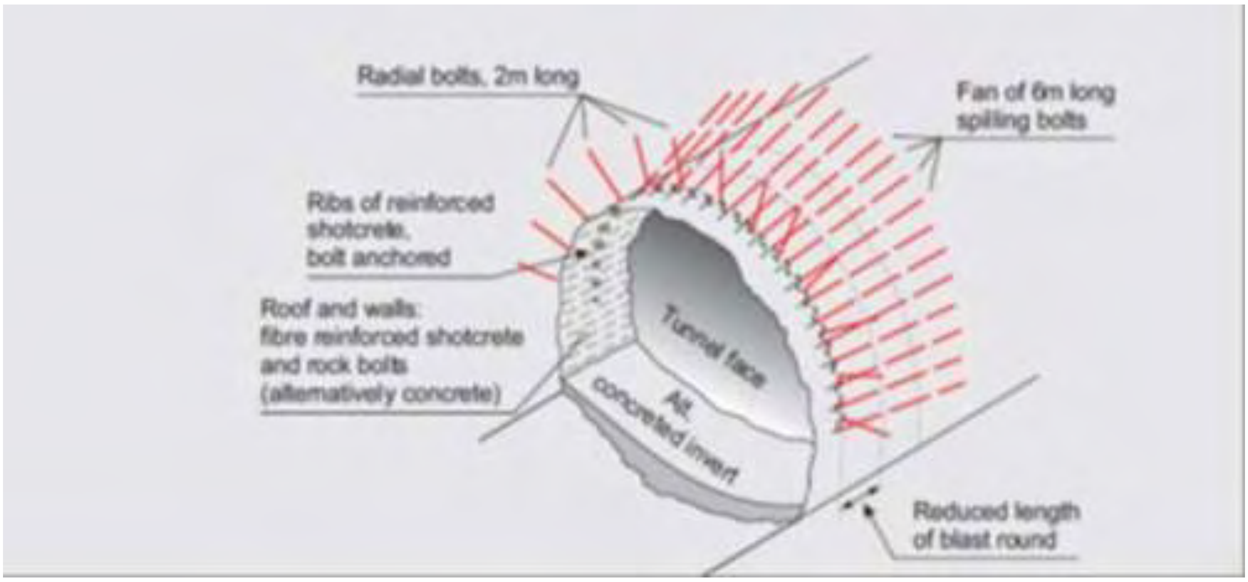
In order to realize the optimization of operation and maintenance in a tunnel, it is important that both the availability and maintenance-friendly solutions are sufficiently emphasized from the moment the planning begins. Inspection and control procedures must be carried out by competent personnel.



EXTENT OF ROCK SUPPORT - SOME EXAMPLES

| Tunnel | Excavation rate m/week | Bolts No./m | Shotcrete m³/m | Shotcrete rel. % | Concrete lining rel. % | Grouting kg/m | Grouting rel. % |
|---|---|---|---|---|---|---|---|
| Vardø | 17 | 6.9 | 0.95 | >50 | 21.0 | 31.7 | 7 |
| Karmsund | 34 | 1.5 | 0.72 | 65 | 15.0 | 13.4 | 9 |
| Ellingsøy | 28 | 6.4 | 0.48 | 20 | 3.0 | 99.1 | 22 |
| Freifjord | 45 | 5.3 | 1.44 | | 2.1 | 13.7 | |
| Kvalsund | 56 | 4.0 | 0.31 | | 0.0 | 0.0 | |
| Hitra | 46 | 4.2 | 1.44 | | 0.2 | 11.4 | |
| Frøya | 37 | 5.0 | 2.90 | | 5.0 | 197.0 | |
| Bømlafjord | 55 | 3.8 | 1.90 | | 0.0 | 36.0 | |
| Oslofjord | 47 | 4.0 | 1.70 | | 1.0 | 165.0 | |
| North Cape | 18/56 | 3.4 | 4.00 | | 34.0 | 10.0 | |

*Figur 4. Normal Rock support*




*Figur 5. Rock support.*

### 3. MANAGEMENT, OPERATION AND MAINTENANCE - MOM

If operational and maintenance tasks are to be resolved in the most cost-effective manner, the utilisation of the resources must be optimal. This requires a certain degree of predictability, which currently is not quite satisfactory. This predictability can be improved by continuously preparing guidelines for "The administration, operation and maintenance of tunnels". Such guidelines will provide the basis for new strategies, which in turn ensure the predictability of further production tasks. This will also make choices of strategy for production considerably easier.
The following objectives of such guidelines are suggested:
• Management, operation and maintenance (MOM) should ensure that the accessibility in the tunnel should be as good as on the roads in general.
• MOM should ensure that:
- the level of security in the tunnel's entry zones (that is 100 m before and the first 100 meters inside the tunnel) should be at the same level as the adjacent roads
- the level of security inside the tunnel should remain at the same level as on the adjacent roads without junctions and pedestrian-/bicycle traffic
• MOM should ensure appropriate preparedness for unforeseen events in the tunnel
• MOM should ensure that current environmental standards are complied with
• MOM should ensure that the tasks are carried out in an economic way for the community (LCC)

**3.1 Management**

Management consists of:
a. An administrative section.
b. A contingency plan section.
c. An operation and maintenance section.
d. Action plans
e. Documentation

***Administrative section***

The administrative section should contain a description of the actual structure.
• Name of tunnel
• Type
• Year of construction
• Key data
Moreover, it should provide an overview of the organization and the different areas of responsibility. The plan of the organization should describe the boundaries of responsibility for personnel related to the tunnel. The administrative procedures for quality-assurance should be described the filing and preparation of documents and the maintenance of the administrative system.

***The contingency plan section***

In road tunnels there is a need for fixed procedures for the determination of who should respond to an emergency call, and how fast this should happen when errors are discovered or accidents occur. This kind of contingency plan should contain a list of who is to be notified and when notification should occur depending on which events take place.

***The operation and maintenance section***

All of the elements of the tunnel are gathered and registered in the MOM-programme *PLANIA* with the preparation of maintenance procedures and the documentation of these procedures.

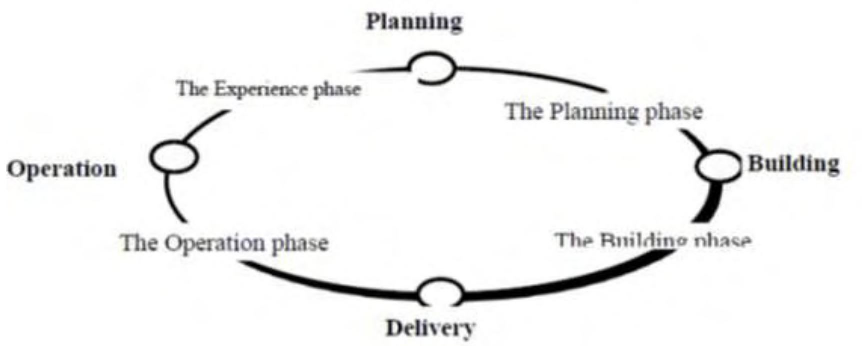
***Action plans***

For tunnels with so much technical equipment and costly solutions, plans should be drawn up based on a condition assessment including anticipated future repair- and replacement needs

***Documentation***

This can consist of:

• Technical documentation
• Legal documentation
• Financial documentation
• Quality assurance material




*Figur 6. Phases in tunnelling*

### 3.2 Operations and maintenance

Operation and maintenance should be organized so that the fundamental conditions that existed when the tunnel was planned, are continued throughout the entire production phase.
The standards and solutions that are selected here will affect future operation and maintenance needs.

• the preparation of a register for the individual tunnel
• a register for maintenance tasks, including a description of the individual elements
• a register for historical data, where experiences can be taken from.
• function agreements
• washing
• cleaning
• inspection
• electric power
• the flow of traffic
• clearing
• supplemental safety measures
• drainage plant
• maintenance of water safety measures
• maintenance of electrical safety measures

- pumps
- ventilation and waste water treatment plants

We must always be aware of the fact that it is the operational phase of 100 years, we should plan and build for. In other words, such guidelines should form a basis for our methods.

*Table 1. Subsea tunnels in Norway.*

| Name | Length (m) | Muo. | Opened | Road | County | Emancipality |
|---|---|---|---|---|---|---|
| Bømlafjordtunnelen | 7888 | 263[1] | 2000 | E 39 | Hordaland | Stord, Sveio |
| Eiksundtunnelen[2] | 7765 | 287 | 2008 | 653 | Møre og Romsdal | Ulstein, Volda, Ørsta |
| Oslofjordtunnelen | 7306 | 134 | 2000 | 23 | Akershus, Buskerud | Frogn, Hurum |
| Nordkapptunnelen | 6875 | 212 | 1999 | E 69 | Finnmark | Nordkapp |
| Byfjordtunnelen | 5875 | 223 | 1992 | E 39 | Rogaland | Randaberg, Rennesøy |
| Atlanterhavstunnelen | 5779 | 250 | 2009 | 64 | Møre og Romsdal | Averøy, Kristiansund |
| Talgjefjordtunnelen[3] | 5685 | 200 | 2009 | 519 | Rogaland | Finnøy, Rennesøy |
| Hitratunnelen | 5645 | 264 | 1994 | 714 | Sør-Trøndelag | Hitra, Snillfjord |
| Frøyatunnelen | 5305 | 164 | 2000 | 714 | Sør-Trøndelag | Frøya, Hitra |
| Freifjordtunnelen | 5086 | 130 | 1992 | 70 | Møre og Romsdal | Gjemnes, Kristiansund |
| Mastrafjordtunnelen | 4424 | 133 | 1992 | E 39 | Rogaland | Rennesøy |
| Valderøytunnelen | 4222 | 137 | 1987 | 658 | Møre og Romsdal | Giske, Ålesund |
| Halsnøytunnelen | 4120 | 136 | 2008 | 544 | Hordaland | Kvinnherad |
| Godøytunnelen | 3844 | 153 | 1989 | 658 | Møre og Romsdal | Giske |
| Hvalertunnelen | 3751 | 120 | 1989 | 108 | Østfold | Hvaler |
| Ellingsøytunnelen | 3520 | 144 | 1987 | 658 | Møre og Romsdal | Ålesund |
| Tromsøysundtunnelen[4] | 3500 | 102 | 1994 | E 8 | Troms | Tromsø |
| Ibestadtunnelen[5] | 3396 | 112 | 2000 | 848 | Troms | Ibestad |
| Sløverfjordtunnelen | 3337 | 112 | 1997 | E 10 | Nordland | Hadsel |
| Vardøtunnelen | 2892 | 88 | 1983 | E 75 | Finnmark | Vardø |
| Fannefjordtunnelen | 2743 | 101 | 1991 | 64 | Møre og Romsdal | Molde |
| Ryatunnelen | 2660 | 87 | 2011 | 858 | Troms | Tromsø |
| Flekkerøytunnelen | 2327 | 101 | 1989 | 457 | Vest-Agder | Kristiansand |
| Melkøysundtunnelen | 2316 | 62 | 2003 | Kun for petroleumsvirksomhet | Finnmark | Hammerfest |
| Maursundtunnelen | 2122 | 93[6] | 1991 | 866 | Troms | Nordreisa, Skjervøy |
| Bjorøytunnelen | 2012 | 88 | 1996 | Fv207 | Hordaland | Bergen, Fjell |
| Skatestraumtunnelen | 1902 | 91 | 2002 | 616 | Sogn og Fjordane | Bremanger |
| Festningstunnelen | 1800 | 45 | 1990 | E 18 | Oslo | Oslo |
| Nappstraumtunnelen | 1776 | 63 | 1990 | E 10 | Nordland | Flakstad, Vestvågøy |
| Kvalsundtunnelen | 1650 | 56 | 1988 | 863 | Troms | Tromsø |

| Name | Length (m) | Muo. | Opened | Road | County | Emancipality |
|---|---|---|---|---|---|---|
| Bjørvikatunnelen | 1100 | 20 | 2010 | E18 | Oslo | Oslo |
| Skansentunnelen | 715 | 14 | 2010 | 706 | Sør-Trøndelag | Trondheim |
| T-forbindelsen | 8900 | 139 | 2013 | 47 | Rogaland | Haugesund |
| Ringveg vest | 5400 | 29 | 2013 | 557 | Hordaland | Bergen |

## 4. DESIGN

The profile is defined for drill and blast methods:

• Theoretical blasting profile

• Normal profile

A theoretical blasting profile is the profile the tunnel should have after the tunnel is initially blasted. In addition to the roof and walls, the base of the tunnel is also included in a theoretical blasting profile.

A normal profile is the profile tunnel should have after safety measures relating to the lining protecting against water and frost insulations have been installed, the surface of the road has been added and the side areas have been built.

These solutions will require a minimum profile of T-9, 5; 3.5 m driving lanes and 1.0 m shoulder and blasted straight walls. In the last few years, approvals have been given for vehicles with a width up to 3.4 m with a mirror, without exemption. However, in order to also meet the needs of special transports, we need to build the tunnel so that the middle section - 4.0 m wide - has a height of 5.0 m.

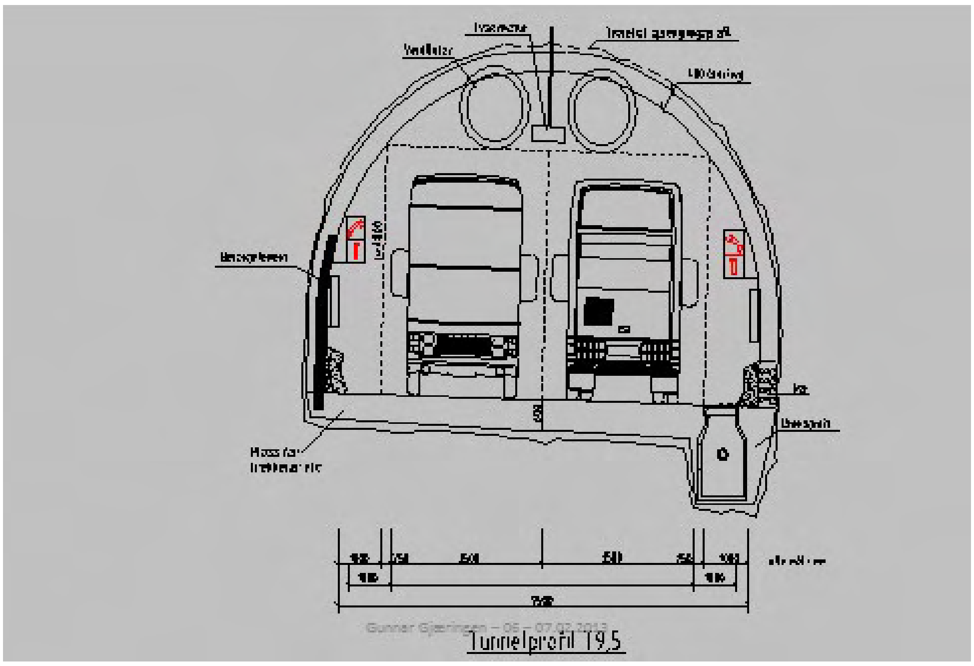


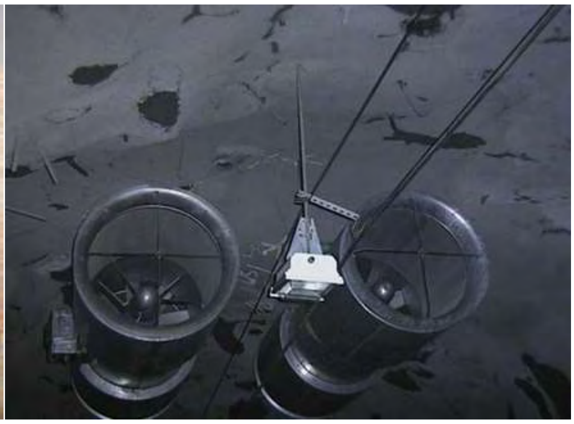

*Fig 7 . Normal tunnel profile.*

## 5. LIFE CYCLE COSTS

It has been gradually acknowledged that operation and maintenance costs to a large extent are determined by the decisions that are made as early as in the planning phase. This means that the need to provide a tool that makes it possible to focus on an overall operational-optimization is becoming more and more of current importance.

Throughout the life span of a tunnel project renewals and upgrades of the technical equipment are carried out due to wear and tear and / or technical development. The main aim of any acquisition is an optimal life span, with the lowest possible costs.


*Figur 8. Old ventilator after corrosion.*



*Figur 9. New ventilators*

## 6. METHODS

The operation and maintenance work in a tunnel should take place as far as possible without unnecessary obstacles to road users.
The safety of both road users and maintenance personnel should be assured through the equipment that should be built into the tunnel's security system.
To achieve this, a systematic maintenance is necessary. The MOM-programme *PLANIA* should be used to set the correct maintenance routines. These maintenance routines will to a large extents be based on empirical data from a corresponding tunnel element and on the documentation of equipment that the builder gains access to at the acquisition of the completed tunnel. What kind of maintenance routines that should be used depends on, among other things, length, profile, AADT (Annual Average Daily Traffic) and what kinds of elements that are present in the tunnel. The NPRA defined as early as in 1988 that a system to manage the work with operation and maintenance in road tunnels should be developed.

*Figur 10. and 11. Examples of broken cable bridges due to corrosion in Norwegian tunnels.*

A system was developed together with a private company, where NPRA as owner of the tunnels, defined how often and how extensive the work should be performed in each tunnel.
Frequency of action is based on AADT, length, the amount of equipment, function, and then of course, type of equipment found in the various tunnels. It has both a web version and a normal version.

## 7. CONTRACTS

All works will be advertised for bids. In this way the contractors must compete to get these works. In these contracts it is listed a range of tasks in each tunnel. It is also specified how often, and when to be performed. The contracts last for up to 5 years and a contract may cover several tunnels.
There are several types of contracts. These are divided according to subject areas as electrical work, work with the road and of course work with the tunnel construction.
The contractor must document that the pre-defined tasks are performed. They do so by signing out of the so-called work orders. In this way we get a clear overview of all types of work being carried out, when they are performed and also about some tasks have not been done at the right time. If not, the contractor gets a fine. Statistics and graphs are produced to illustrate the performed work.

## 8. MAP

We have made a module map showing all road tunnels in the country with the specific location and other data. Clicking on the tunnel name, the layout with the tunnel data appears.

## 9. DOCUMENTATION

These data give a good documentation of what kind of operations done in each tunnel, that the tasks are performed as well as the status of the equipment - functionality, etc.
This gives us then a relatively simple overview of replacement needs. It also provides a simple overview of the costs.

Special conditions are reported from contractor to owner immediately
It is also an invaluable tool in relation to the requirements concerning fire and electrical installation and systems. We can thus easily provide the documentation of these requirements.

## 10. ORGANISATION

The operation manager must choose an optimal maintenance organization that also must take into account other traffic-related tasks in addition to the technical issues connected to current maintenance tasks. Guidelines and instructions for all kinds of routine tasks, the repair of safety equipment and for larger rehabilitation assignments are necessary. The cost to society in the form of traffic problems will generally increase when the tunnel is closed because of long-term maintenance. Through planning and construction, one can take into account ways to carry out the operation and maintenance reducing the need for closure.

## 11. REDUCTION OF COSTS

In order to reduce the costs of operation and maintenance an optimal maintenance will primarily depend on the type of tunnel and the investments (that have been) made in the planning phase. Moreover, the conditions for and accessibility to operation and maintenance will be central issues.

## 12. CONCLUSIONS

The registration of experience has formed the basis and conditions for the proposed measures that have come forward in order to contribute to more optimal tunnel maintenance with optimal costs.

Choices and decisions that are made in the planning phase have a decisive influence on the costs in the operation- and maintenance phase. These choices must be selected based on experience! The optimization of maintenance requires life span calculations based on data from previous experiences throughout the different processes. This requires a high degree of availability and the selection of maintenance-friendly solutions and materials in the early stages of the planning and construction phase.

NPRA want to prevent fires and queues in tunnels as the ones shown below. We also want to lighten up the tunnels by whitening the walls to prevent accidents.



*Fig11: Fire in Seljestad tunnel. Fig12:. Queue in a tunnel. Fig13: Tunnel with white walls*

The management system should in other words answer all questions regarding the administration, operation and maintenance of a tunnel. A systematic execution of operation and maintenance ensures good workmanship, a long life span and lower costs.

Operation and maintenance requires a necessary choice of profile that provides sufficient space for installations and the availability of rational methods of maintenance.
A significant amount of capital has already been invested and will in the future be invested in Norwegian road tunnels.

The administration of this capital should first and foremost consider a long-term ownership. If this is to happen, it is difficult to see how it can be done without adding a much stronger emphasis to life cycle costs in both investment and operation and maintenance.

In order to work towards optimal operation and maintenance procedures, one needs knowledge about which conditions to monitor, which factors affect their management and which factors need to be influenced in order to achieve the desired results. Therefore we use the MOM system PLANIA. In Norway a common system is used for planning and documenting maintenance both in subsea tunnels and in other road tunnels.

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# LONG SPAN CABLE STAYED BRIDGES FOR RAILWAY

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## ABSTRACT

*Most cable stayed bridges constructed to date carry road traffic and/or pedestrian traffic. However, a number of major cable stayed bridges for heavy railway load are in operation and today it is feasible to design and construct cable stayed bridges with a main span of 5-700 m or longer for railway loading. Depending on whether the bridge also carries roadway traffic the girder may be arranged as a one-level deck or a two-level structure.*

*The paper describes some of the early examples of cable stayed bridges for railway, but focuses on the recent development which has demonstrated that significant spans can be achieved without compromising safety and comfort of the passengers.*

*The following projects are described in depth: The Øresund Fixed Link (Denmark-Sweden), the Third Tagus River Crossing (Portugal), the Fehmarnbelt Fixed Link (Denmark-Germany) and the Puente Nigale across Lake Maracaibo (Venezuela).*

Keywords: Cable stayed bridges, railway, two-level, girder, runability.

## INTRODUCTION

The use of cables as main load carrying elements in bridge structures has proven very efficient as the high strength-to-weight ratio of the cable material will decrease the escalation of the dead load otherwise related to longer spans. Today cable supporting is applied for most spans above 250 m.

Most cable stayed bridges carry only road traffic and/or pedestrian traffic. However, cable stayed bridges have also been designed to carry light rail or heavy railway load, often in combination with road traffic. The requirements to maximum slope, deflections and rotations are stricter for a bridge carrying railway than for roadway traffic only. In particular the differential angular rotations under live load at expansion joints between adjacent girders need special attention. Furthermore, the loads in the longitudinal direction of the bridge due to braking and accelerating trains are significant, [1].

In comparison with suspension bridges, cable stayed bridges are stiffer structures which is an advantage under railway loading as limitation of deformations is a governing factor. Other factors that influence the choice between a suspension and a cable stayed bridge include geotechnical conditions, required main span length and construction time. Cable stayed bridges are usually designed as self-anchored structures and are consequently less reliant on good soil conditions that allow transfer of large horizontal forces as compared to an earth-anchored structure like a typical suspension bridge. On the other hand the self-anchored system introduces significant compressive forces in the stiffening girder which are not found in an earth-anchored system. As a consequence, longer spans can be achieved with suspension bridges than with cable stayed bridges.

In terms of construction time the self-anchored system allows installation of the superstructure to start before completion of the pylons – as soon as the pylon reaches a certain height above the lower stay cable anchorage points. For a suspension bridge pylons, anchor blocks (or anchor chambers) and main cable have to be completed before installation of the superstructure can start (unless a substantial temporary supporting system is provided), which leads to a more sequential and typically longer construction programme than for a cable stayed bridge.

The dynamic amplification of the loads shall be taken into account in the structural design and the accelerations shall be evaluated in terms of comfort and safety of the train passengers. A runability analysis is carried out to verify the dynamic train-track-structure interaction due to moving loads on the flexible structure. For an example and general principles refer to [2]. Furthermore, special accidental load cases are defined to cover various scenarios of fire or derailment resulting in loss of structural elements, possibly including stay cables.

One of the early examples of a cable stayed bridge carrying railway is the twin bridges across the Paraná River in Argentina (Zárate-Brazo Largo Bridge I and II) which opened for railway traffic in 1978. These two identical cable stayed bridges with main spans of 330 m carry four lanes of roadway traffic and a single railway track over the two main branches of the Paraná River. The railway track is located eccentrically next to the traffic lanes on the one-level girder. One of the stay cables suddenly failed in 1996 and subsequently all stay cables were replaced, [1].

The Skytrain Bridge in Vancouver, Canada, opened in 1990 and carries two tracks of light rail across the Fraser River. The superstructure is a one-level prestressed concrete girder. The main span is 340 m and the bridge carries no road or pedestrian traffic, [1].

The Kap Shui Mun Bridge in Hong Kong S.A.R. completed in 1997 has a main span of 430 m and carries road on the upper deck and MTR (light rail) on the lower deck which also provides two lanes for cars when the upper deck has to be closed for traffic due to high wind speed.

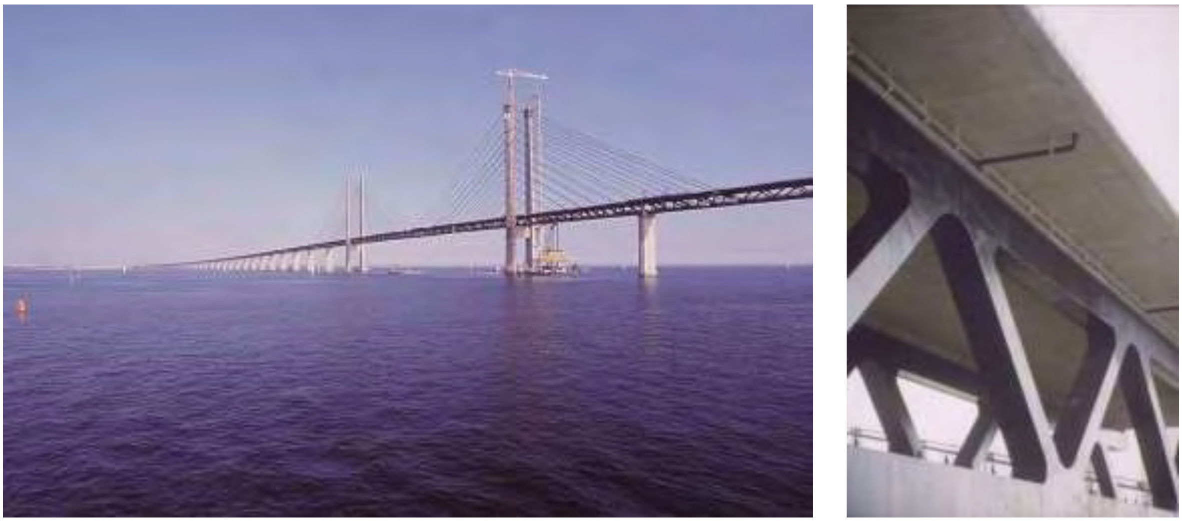

*Figure 1 The Øresund Fixed Link between Denmark and Sweden, completed in 2000, carrying two tracks of railway on the lower deck and four lanes of road traffic on the upper deck. Courtesy of COWI A/S.*

The cable stayed bridge of the Øresund Fixed Link between Denmark and Sweden opened for traffic in 2000 and has a main span of 490 m, [3]. The stiffening girder is a two-level truss structure with the double track railway located on the lower deck. The trusses and the lower deck are in steel whereas the upper deck is in concrete. The depth of the girder is 11 m which provides significant stiffness and distribution of local, concentrated loads. The pylons are H-shaped with the cross girder located underneath the girder. The stay cable spacing is 20 m at deck level and the stays are arranged in a harp system, *Figure 1*.

The current (2013) world record for a combined road and rail cable stayed bridge is the Tianxingzhou Bridge across the Yangtze River in Wuhan, China, with its main span of 504 m. The bridge carries six traffic lanes and four railway tracks, including high speed railway. The bridge opened to traffic in 2008, [4].

A number of long span cable stayed bridges carrying railway are currently in the planning stage. The Third Tagus River Crossing in Lisbon, Portugal, is planned to carry four tracks of railway, two tracks for conventional railway and two for high speed railway, in addition to six lanes of roadway traffic (according to the project requirements in the 2008/2009 project). A significant bending stiffness of the girder is needed for the planned 540 m main span requiring a deep two-level truss girder.

*Figure 2 Proposed bridge solution for the Fehmarnbelt Fixed Link: Multi-span cable stayed bridge with two main spans of 724 m carrying two tracks of railway on the lower deck and four lanes of road traffic on the upper deck. Visualisation by Dissing+Weitling, courtesy of Femern Bælt A/S.*

A number of bridge and tunnel solutions have been investigated for the planned Fehmarnbelt Fixed Link between Denmark and Germany. The preferred bridge solution is a two-span cable stayed bridge to accommodate two tracks of railway and four lanes of roadway traffic. The conceptual design investigations comprise a structure with two main spans of 724 m and a two-level truss girder with the railway located on the lower deck, *Figure 2*.

A different girder design is adopted on the Puente Nigale link across the Maracaibo Lake in Venezuela. The link will carry two railway tracks and four lanes of highway. Due to the phasing of the project it is necessary to design and construct the link with separate and independent superstructures supported on a common foundation. As a consequence the current design of the main bridge across the navigation channel is based on a 430 m main span cable stayed bridge with a one-level concrete girder carrying the railway traffic and a similar parallel bridge for the roadway.

The four projects Øresund, Tagus, Fehmarn and Puente Nigale are described in more detail in this paper with a view to identifying general trends in design and construction of cable stayed bridges for railway. In addition to being major fixed links of general interest, COWI A/S have direct involvement in design of all four projects which has allowed - for the purpose of this paper

- access to the relevant project information and details of the designs which allows a comparison across these four significant cable stayed bridges for railway.

## RAILWAY LOADING

Railway loading is characterized by a high vertical load intensity and significant longitudinal loading due to braking and traction. In addition the cyclic loading may lead to fatigue in the structural elements including stay cables.

According to Eurocode the nominal load intensity of normal rail traffic is typically taken as 80 kN/m over a length that is unlimited, Load Model 71, [5]. Alternatively, a load intensity of 133 kN/m shall be applied over a limited length of 2 x 15 m separated by a gap, Load Model SW/0, [5]. Heavy rail traffic is represented by Load Model SW/2 with a load intensity of 150 kN/m over a limited length of 2 x 25 m separated by a gap, [5]. Even heavier loads may be specified for the project depending on local conditions and special activities. Countries outside the Eurocode framework have different requirements. An advantage of the Eurocode system is that it provides load combinations including both rail and road traffic which is fundamental for design of bridges carrying combined traffic.

Rails, sleepers and ballast help distribute the concentrated wheel and axel loads. It is also possible to arrange a ballastless track. This provides some weight saving and in addition reduces the need for maintenance to restore the desired track geometry otherwise needed at regular intervals with a ballasted solution. There are a number of proprietary systems available in the market. However, the ballastless track poses strict requirements to construction tolerances of the supporting structure and also the initial cost is higher than with a traditional ballasted solution. The higher cost of the ballastless track may to some extent be balanced by a saving in structural quantities because of the weight saving.

## ØRESUND

The 7.7 km long Øresund Bridge is a major part of the Øresund Fixed link, see *Figure 1* and *Figure 3*. Other significant elements include an artificial island and a 4 km submerged tunnel.

The owner prepared two options for the bridges, described by Definition Drawings and Illustrative Designs. The contract was tendered on a design-build basis leaving the responsibility for basic and detailed design with the contractor who in this case opted for the two-level solution.

The landmark of the link is the 203.5 m high H-shaped pylons, see *Figure 1*. The outer shape of the visible parts of the structures was determined by the owner in the Definition Drawings.

The bridge girder for the high bridge is arranged as a steel truss girder with an upper transversely post-tensioned concrete roadway deck and a lower deck for the railway, designed as a closed steel box. The inclination of the truss members is arranged to match the inclination of the cable stays which are anchored to the girder on outriggers, see *Figure 4*. The tracks are ballasted over the entire length of the bridge, [3]. The approach spans are in general 140 m spans with a lower concrete deck instead of the closed steel box.

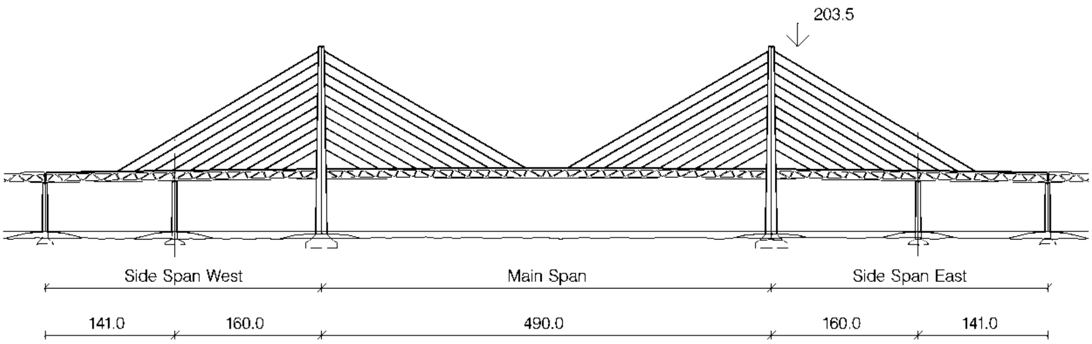


*Figure 3 The Øresund High Bridge, [3].*

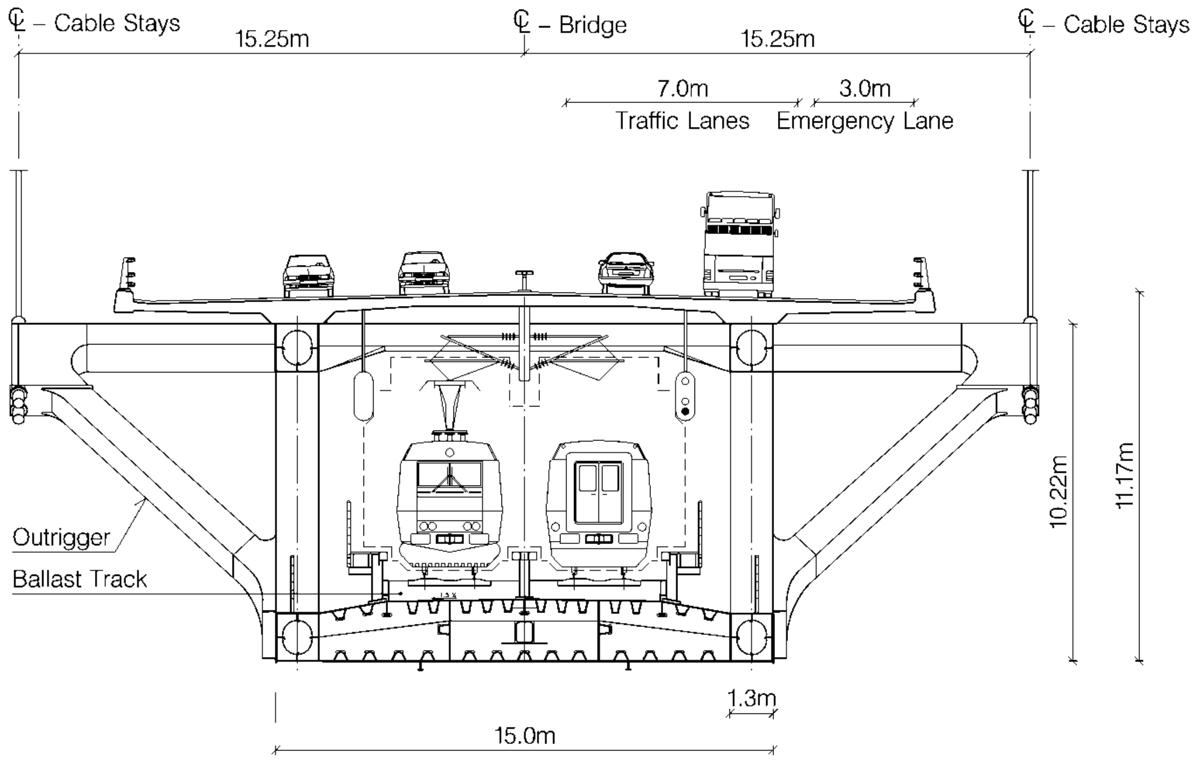


*Figure 4 The Øresund High Bridge, cross section, [3]. Dimensions in [m].*

**THIRD TAGUS RIVER CROSSING**

According to the owner's requirements presented in 2008 the total length of the Third Tagus River Crossing in Lisbon is around 7.2 km and consists of a cable stayed main bridge with a main span of 540 m, two secondary navigation bridges and a significant length of approach bridges with spans in the range 90-120 m. In the owner's design the girder was arranged as a two-level structure with the four tracks of railway on the lower level and highway traffic on the upper deck. The girder had two vertical truss planes and stay cables anchored on outriggers. The pylons were inverted Y-shaped.

In the end three groups submitted a tender for the design and construction of the Third Tagus River Crossing. However, the project was put on hold before a contract was signed due to the general economical situation.

The team that included COWI plus companies from Portugal, Spain and Brazil carried out an analysis to optimize the design considering the contractor's preferred construction methods. This resulted in a proposal where the girder in the cable stayed bridge was modified to include three truss planes supported by three cable planes in order to save weight and quantities in the transverse system. Furthermore, the outer trusses were inclined to optimize the cross section, see *Figure 5*. The pylon was maintained as an inverted Y, but with a modified shape in the anchorage zone in order to accommodate three cable planes.

Both the upper deck and the lower deck are concrete structures that act compositely with the steel truss girder. The design is based on precast concrete slabs to reduce construction time. The tracks are ballasted and located inside a prefabricated concrete trough.

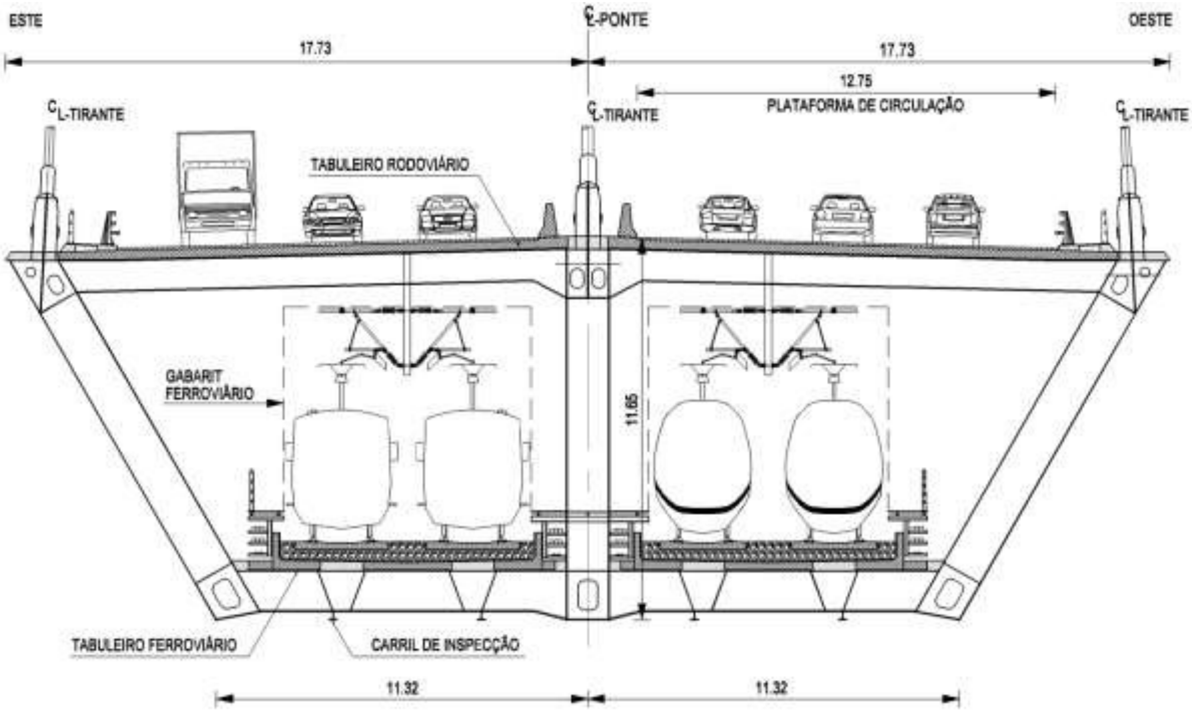


*Figure 5 Third Tagus River Crossing, cross section, tender design. Dimensions in [m].*

**FEHMARNBELT FIXED LINK**

Due to navigational requirements the Main Bridge proposed for the Fehmarnbelt Fixed Link consists of two main spans of 724 m each and three pylons as shown in *Figure 6*. The longitudinal support of the 2.4 km long superstructure is provided at the central pylon only. A significant stiffness is required to deal with asymmetrical vertical load and the central pylon is arranged as a stiff A-frame in the longitudinal direction.

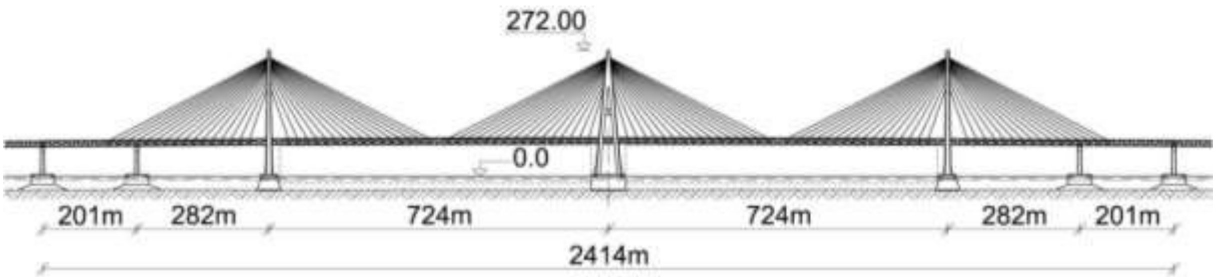


*Figure 6 Fehmarnbelt Fixed Link, elevation of the Main Bridge.*

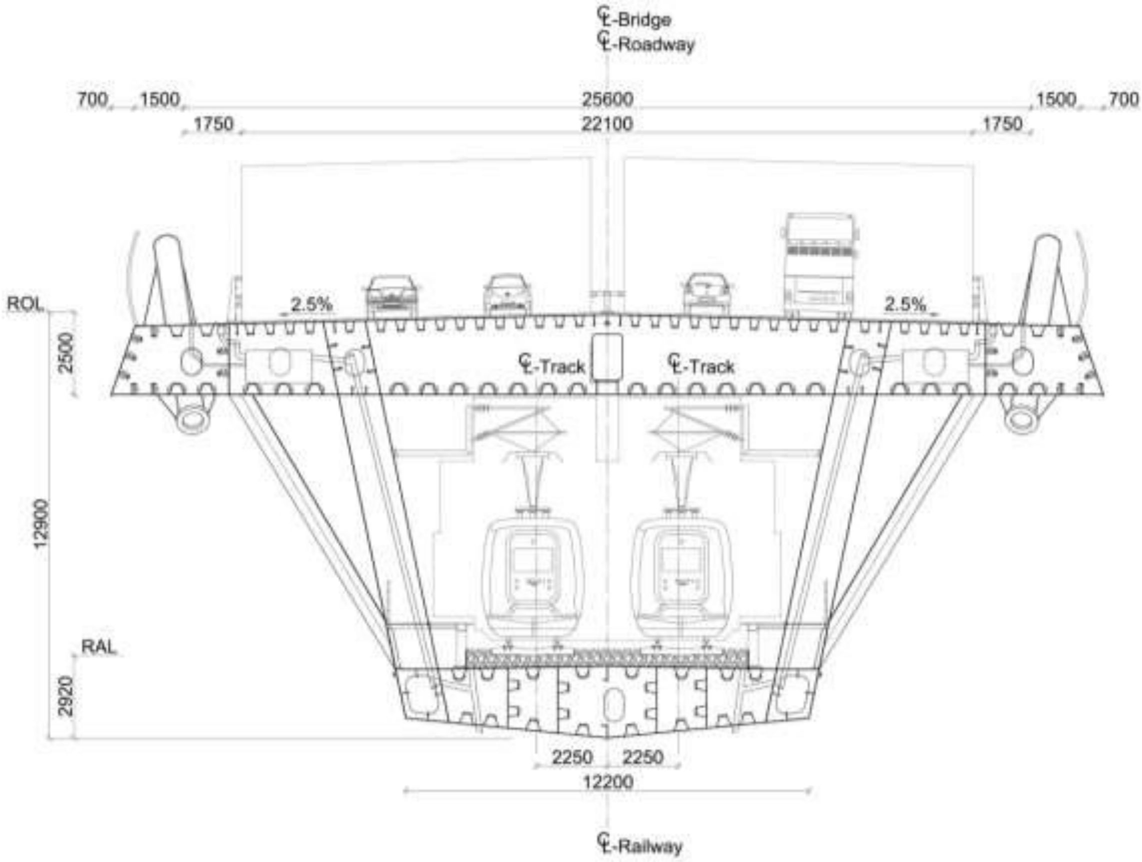


*Figure 7 Fehmarnbelt Fixed Link, typical cross section of the Main Bridge. Dimensions in [mm].*

The superstructure consists of a two-level truss girder. Both the upper roadway deck and the lower railway deck are arranged as closed steel boxes, see *Figure 7*. The main reason for choosing a steel solution in this case was to save weight and avoid double stay cables which would need more space in the pylon for anchorage. As it is the case for the Øresund Fixed Link and the project for the Third Tagus River Crossing, the track on the Fehmarnbelt Fixed Link was assumed to be ballasted over the entire length.

The trusses, cable planes and pylon legs are inclined outwards (i.e. V-shaped in the transverse direction), which gives the main bridge a distinctive appearance, see *Figure 2*.

The total bridge length is nearly 18 km and the approach bridges have a typical span length of 200 m with the above described cross section, only the upper deck is a transversely post-tensioned concrete slab instead of a closed steel box.

Following a phase of conceptual design for both a bridge and a tunnel solution it has been politically decided to progress the design based on an immersed tunnel solution. Consequently, the bridge solution will not be developed beyond the current level of detail.

**PUENTE NIGALE**

The design adopted on the Puente Nigale link across Lake Maracaibo in Venezuela is significantly different from the three projects described above. Due to the time schedule and phasing of the project as well as important aspects of construction, it is deemed advantageous to design and construct two parallel bridges with independent superstructures for road and rail, respectively, but with common foundation.

This naturally leads to superstructures of the one-level type and in this particular case concrete is the preferred material because of supply of materials and tradition of maintenance. Again the tracks are ballasted over the entire length of the fixed link, see *Figure 8*.

The cable stayed bridges are designed with central cable plane and freestanding mono-column pylons above deck, see Figure 9. The connection between pylons and girders is monolithic. It is anticipated that the two pylons will be connected by a cross beam below girder level in order to take advantage of the two parallel structures to increase the transverse stiffness.

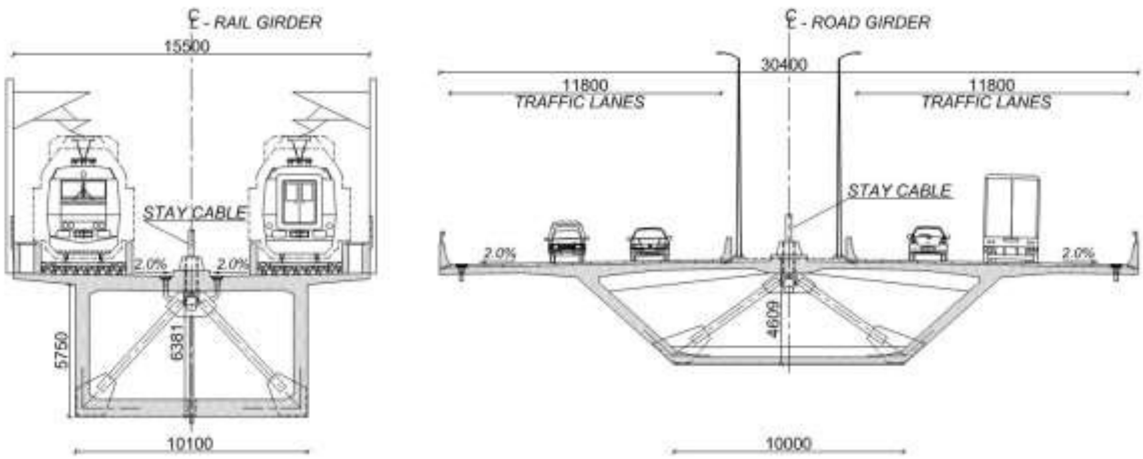


*Figure 8 Puente Nigale, typical cross section in the two parallel main bridges across the navigation channel. Dimensions in [mm].*

Figure 9 Puente Nigale, rendering showing the two main bridges across the navigation channel (rail bridge to the right). Courtesy of COWI A/S.

The central cable plane is quite unique for a bridge of this scale carrying heavy railway traffic, and there has been special focus in the design on twist of the girder and transfer of shear stresses due to torsion.

The total length of the crossing is nearly 12 km. Because of the lower girder depth of a one-level solution the approach bridges are not able to span as long as two-level girders, 75 m in this case.

## COMPARISON OF GENERAL TRENDS IN DESIGN AND CONSTRUCTION

As described in the previous sections cable stayed bridges carrying railway load have been designed - and constructed - with both one-level and two-level stiffening girders. In general two-level girders seem to be the preferred solution for the longer spans, where the stiffer superstructure allows bigger stay cable spacing and limits local deformations. Examples of two-level girders include the Øresund Bridge (490 m), Tianxingzhou Bridge (504 m) and the projects for the Third Tagus River Crossing (540 m) and Fehmarnbelt Fixed Link (724 m). However, the choice of a deep two-level girder is actually often linked to optimization of the arrangement in the approach bridges. As the length of the approach bridges is often quite significant for a major fixed link, the majority of the total cost of the link is related to the approach bridges - despite the higher unit cost of the main bridge. Arranging a two-level superstructure will allow longer spans in the approach bridges and result in savings in the substructure.

A drawback of the two-level superstructure is that it is necessary to arrange a transition where the traffic on either the upper level or the lower level is shifted horizontally to separate the alignments and allow traffic on both levels to reach ground level on land.

On the Øresund link two different strategies are adopted: On the Swedish side the transition takes place on land after the abutment, where the trains on the lower deck continue into a short tunnel while the road traffic on the upper deck is shifted horizontally. At the western transition, located on an artificial island in the strait, the rail traffic on the lower deck is shifted horizontally away from the alignment inside a gallery structure which allows the road traffic on the upper deck to be brought down to ground level on the artificial island. Subsequently, the traffic continues into separate tunnels to the Danish coast.

On the Fehmarnbelt fixed link gallery structures similar to the above described were designed at both ends.

For a span of 430 m on the Puente Nigale - identical to the main span of the two-level Kap Shui Mun Bridge described in the introduction - a solution based on two parallel structures with one-level girders is currently taken forward and will go into the detailed design phase in 2013. The central cable plane is unique for the Puente Nigale as compared to the other structures described in this paper.

If the bridge carries both road and rail traffic and is planned for simultaneous opening for operation, a two-level solution may seem the obvious solution. Other aspects to consider in the choice of superstructure include construction method, lifting weight, risk, cost, operation and maintenance, environment and noise. The complexity of the issue is illustrated by the fact that the Øresund link was tendered with both a one-level and a two-level girder solution for a main span of 490 m. The girder depth of the one-level solution was approximately 6 m and the stay cable spacing 6 m, to be compared with the girder depth of 11 m and stay cable spacing of 20 m for the two-level solution, see also *Figure 4*.

The two-level solutions are truss girders, with steel trusses and either steel or concrete (composite) decks. Typically two truss planes and two cable planes are arranged, even though Tianxingzhou has three truss planes and three cable planes. The proposal described above for the Third Tagus River Crossing also assumed three truss planes and three cable planes.

The stay cables may be arranged as single stays or double stays if the capacity of available stay systems is insufficient for a single stay cable to accommodate the demand.

## CONCLUSION

This paper illustrates that it is feasible to design and construct cable stayed bridges with a main span of 5-700 m or even longer for railway loading. This span length allows spanning over a significant range of waterways where girder bridges may not be cost effective because of the cost of foundations in deep water or because the required navigation clearance cannot be achieved with a girder bridge. In these cases cable stayed bridges are considered to offer a good solution for crossing of rivers, straits and lakes.

The trend for these long span cable stayed bridges carrying railway seems to be towards two-level girders, even though designs based on one-level girders are also found. The preference seems to be for ballasted tracks. A variety of different pylon shapes have been adopted and there seems to be no general preference.

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# FLOATING CONCRETE STRUCTURES

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## ABSTRACT

*Straits of considerable width need to be crossed by planes, helicopters or ships. These planes, helicopters and ships need terminals. The terminals may have a variety of functional requirements.*

*This paper will deal with the case were floating terminals are selected.*
*Floating structures have a number of benefits as compared to land based structures; examples may be one or more of:*

- *There is not enough space on land*
- *The land is expensive*
- *We do not like to have the activity on land*
    - *For environmental reasons*
    - *For security reasons*
    - *For cost reasons*
- *Buoyancy may allow novel solutions*
- *Floating structures may be relocated*

*Marine concrete structures are structures where the robustness of concrete is quite helpful.*

*Fortunately we have a lot of experience with marine concrete structures for the oil and gas industry. This paper describes some of the experiences with marine concrete structures, some of them sitting on the seabed in their operational life, but all floating at least during construction and installation. Future developments and examples are commented upon.*

## INTRODUCTION

Concrete has been used for floating structures for more than a century. Particularly during wars, when steel is used for other purposes, concrete ships and barges have been popular. But typically they are heavier than their steel counterparts, and therefore more costly to operate, and a frequent destiny has been grounding and use as a silo.

The American Concrete Institute Committee 357, reference [1], has written several publications on barges and floating structures. The properties of the barges are very good, although a bit heavy. The weight issue is partly a design issue; concrete ships should not be designed as a steel ship. As an example the bow and stern structure should not have concave elements as the steel ship has.

Fig. 1 shows Sylvestre, built in Norway and accidentally grounded in 1927, almost 90 years ago. It is not in a good shape, but the sections are thin and no maintenance has been performed. The fiord where she sits is periodically frozen. The picture is from the summer of 2012.

Fig.'s 2 and 3 show better bows, one modern and one more than 1000 year old, the Gokstad Viking ship. These Viking ships crossed the North Atlantic.

*Fig. 1 The Sylvestre*

*Fig. 2 The Oseberg Viking ship*  *Fig. 3 The X-Bow of Ulstein (steel)*

Not all floating structures are made for transport, some are meant for stationary activities, or maybe infrequent transport. The latter is important; the ability to relocate a floating structure. We may imagine terminals placed in catastrophe prone areas (earthquake, tsunamis), war zones, environmental sensitive zones (oil spills in the arctic), and temporary mankind activities. In such cases equipment and goods may be stored on board the terminals, and manpower hastily shipped or flown in when activated. The US for example has worked on developing such a terminal large enough to land aircrafts on. Oil and gas installations typically have landing facilities for helicopters. One may argue that these examples do not represent Strait Crossings, but they are similar in nature. And for a Strait Crossing the benefit of being able to relocate a terminal may become helpful when future needs dictate other routes to travel.

**EXPERIENCE FROM THE OIL AND GAS OFFSHORE CONCRETE STRUCTURES**

There are 50 major offshore concrete structures serving the oil and gas industry. Most of them sit on the sea bottom, connecting the reservoir far underneath the sea bed to a production facility on top of the platform. Some of them also store oil.

The typical construction scheme is to use a suitable dry dock for the lower part, and then commence construction afloat. The topside facilities may be built simultaneously, and the two, topside and platform, may be coupled inshore and towed to location. Installation may be as simple as adding water, and will typically take only a few days.

This method of construction favours deep fiords, particularly if the topside weight is large. Fig.4 shows the tow out of the Troll A platform, displacing 1 million tons, with its 22,000 tons topside 150m above sea level.

Fig.5 shows the typical construction process: the mono-tower platform Draugen is used for illustration. This platform is operated by Norske Shell, and it has performed very well since the installation in 1993.

In fact all the concrete platforms perform well, which may seem surprising, considering the harsh environment some of them sits in, as exemplified in Fig.6. Modern platforms have longevity of more than 200 years, exceeding by far the production span of the oil or gas reservoir. The first concrete platform was installed in 1973, 40 years ago, and many platforms have surpassed their design lives but still in good shape. Fig. 7 shows work of inspection.


*Fig.4. Troll A during tow out*

*Fig.5. Construction of a gravity based concrete platform*

*Fig.6. The harsh environment of the North Sea*

*Fig. 7 Inspection*

The state of platforms is described in reference [2].

The 50th concrete platform was installed last summer, east of Sakhalin Island in Russia. It is operated by ExxonMobil. It has been designed to resist sheet ice. The 51st, also to be operated by ExxonMobil, is now under construction in Canada, and shall resist icebergs. In general earthquake is considered and designed for, as well as waves and other environmental loads.

Several construction sites exists, the ones in sheltered and deep fiord are favourable. The ExxonMobil Adriatic LNG Terminal reference [5] was built in Spain before being towed to the Adriatic and installed.

*Fig. 8 Construction site, near Bergen*

*Fig. 9 The Adriatic LNG Terminal*

**DESIGN OF STRUCTURES**

Designing marine structures involves the water, hydrostatic and hydrodynamic disciplines. Practical testing is helpful, and enjoyable.

*Fig. 10   Practical design of marine structures*

Due to a very large amount of loadcases there are logistics and numerics to be accounted for, as may be envisaged in Fig. 11:


*Fig. 11   Numerical design of marine structures*

We now have the tool of running nonlinear global finite element analyses, including also the Modified Compression Field method of Prof. Michael P. Collins and his colleagues at the University of Toronto. We thus have a tool for both optimising the design of the structure, and also the possibility to document strength under severe conditions.

This is a positive development; the design of large reinforced concrete shell structures has until now been based on linear-elastic structural analyses and nonlinear sectional analyses, whereas the use of nonlinear structural analyses has been limited to local, critical regions. Although the superposition principle has clear practical advantages, the stiffness incompatibility between structural and sectional analyses may constitute a significant uncertainty in the design, especially for structures designed to satisfy strict performance criteria. The new FE–based approach allowing for global nonlinear response predictions has been developed which eliminates some of the drawbacks normally associated with nonlinear simulations. By the so-called consistent stiffness method, the nonlinear response is obtained by iterative, linear analyses in which the effective element stiffness is repeatedly updated from the cracked shell section analysis.

Motivated by the shortcomings of a linear-elastic design approach, practical ways of eliminating the incompatibility between the linear structural analysis and the nonlinear sectional analysis has long been searched. Despite the progress in development of complete constitutive models for reinforced concrete shells, current commercial nonlinear FEM programs still have limited applicability in practical design. A pragmatic approach for performing global nonlinear analysis and design of shell structures that overcomes most of the previously described limitations is presented.

The method is an iterative elastic procedure that calculates the effective stiffness from the sectional analysis based on the sectional geometry, material behavior, linear strain distribution over the section and equilibrium of forces. The calculated stiffness parameters are fed back into the linear – elastic structural analysis which is rerun resulting in a new distribution of forces and deformations. This is repeated until a specified stiffness convergence criterion is satisfied, see Fig. 12. The stiffness parameters are updated based on the secant stiffness for all degrees of freedom at each design section for each iteration. At convergence, the stiffness properties in all sections are consistent to the actual load level and material layout. This procedure is possible due to the use of a special linear elastic element type.

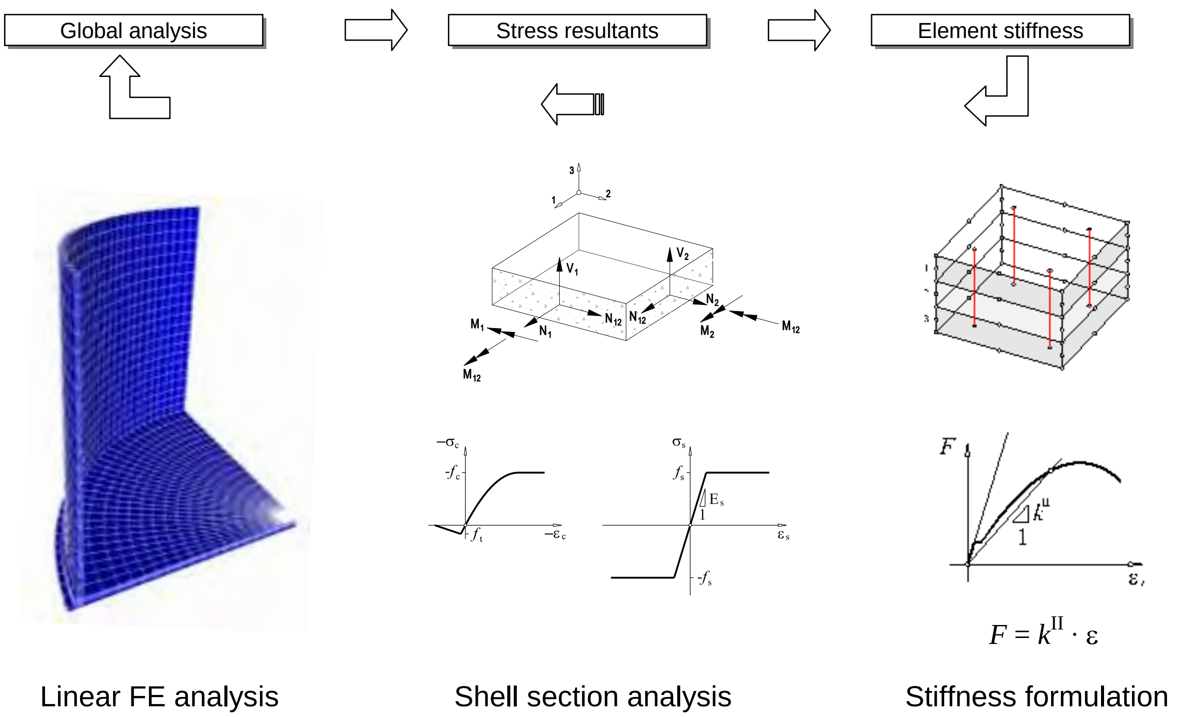


*Fig. 12 The iterative approach for predicting nonlinear behaviour of shell structures*

This procedure has shown very useful in real problems. Fig. 13 shows one of these, were we managed to document sufficient strength of the Troll A platform subjected to new loading.

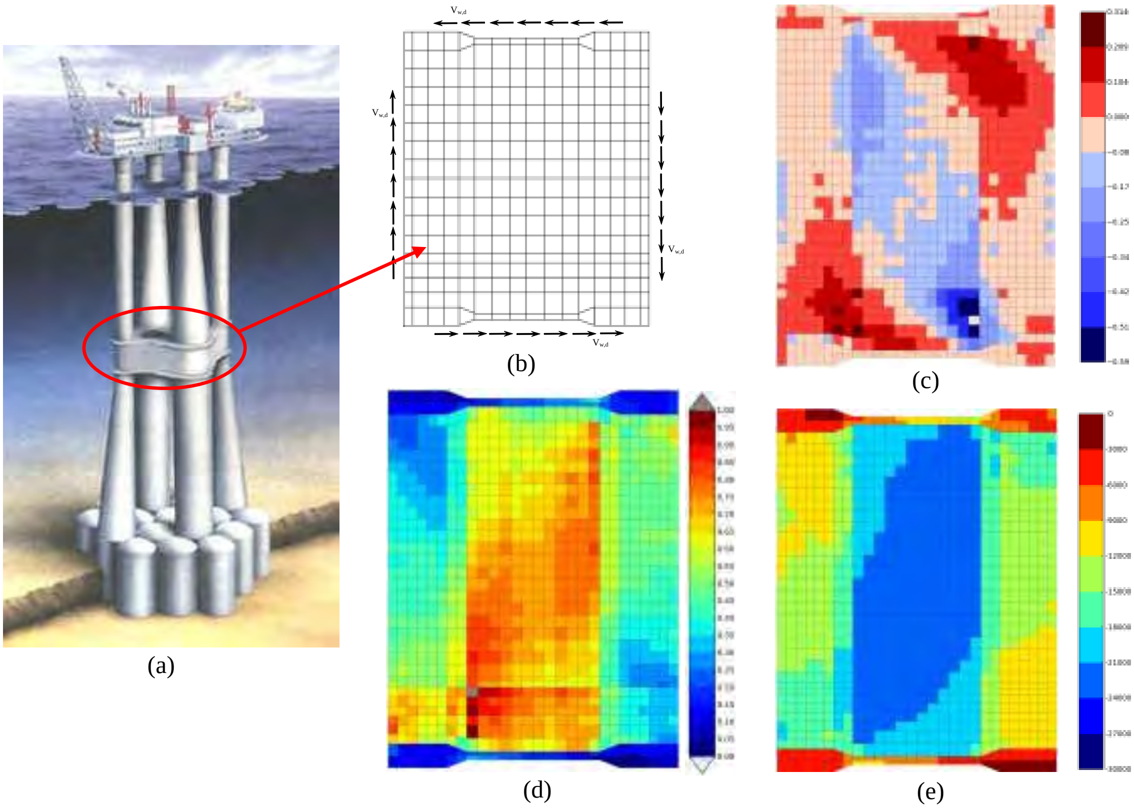


*Fig. 13 Results from the nonlinear analysis of the Troll A Riegel: (b) Simplified geometry and applied load. (c) Difference between the utilization of concrete in linear and nonlinear analysis. (d) Contour plot of the concrete utilization. (e) concrete principal compressive stress*

## CONCEPTS

There are many concepts presented for terminals and strait crossing. One of them, for the Messina Strait crossing, includes, in addition to the bridge, large floaters that are intended for use by people; living, shopping, etc., etc. (Fig. 14, reference [3]).


*Fig. 14 Concept for crossing the Messina Strait*

There are lots of creative concepts shown on the internet, reference [4], of which two are shown in Fig. 15:

*Fig. 15 Creative concepts*

Terminals may contain hotels, as shown in Fig. 16, or liquids, Fig. 17, or parking.

*Fig. 16 Terminal with hotel and parking*

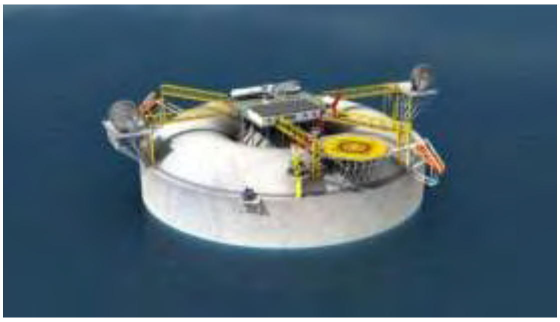

*Fig. 17 Terminal with liquid storage or parking*

Terminals for ship landing may take various shapes and additional functions.

Fig. 18 illustrates a floating monument accessed through a submerged floating tunnel with glass for closeness to the marine environment. The floater itself may very well be used for educational purposes, leisure activities; something more than just a ticket office for the ferry.

Fig. 19 illustrates a large landing place, for cruise ships in this case. Not exactly a strait crossing situation, but a facility bridging land and the sea. In this case the concept is shown outside Monaco, a place where land is scarce and cost of land is very high. There are many places with limited space; the usual way around it is to build high, as in New York City, several Asian cities, and Middle East cities. The alternative may be using the sea.

*Fig. 18 Ship landing with social activities*    *Fig. 19 Ship landing with urban development*

Also in densely populated areas the shoreline is often used for railroads and highways. Historically one can see the reason for this, but now more and more land is captured for pleasant activities like dwelling, shopping and leisure, and roads and rails are put in tunnels, either on land or in the sea. These are described in other presentations in this Strait Crossing Conference.

## CONCLUSIONS

The use of technology developed already, also by the oil and gas industry, may be used to develop novel applications for floating terminals for the world's transport requirements.

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5. ExxonMobil home page

# MONITORING OF EXISTING STRESSES IN PRESTRESSING STEEL STRAND FOR PC BRIDGES

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## ABSTRACT

*In the presented paper, a literature review on the existing effective stress test methods for pre-stressing tendons was carried out. A modified electromagnetic measurement technique was proposed to determine the effective stresses of prestressing tendons in PC bridges. Firstly, a large-scale beam was designed and fabricated, in which prestressing tendon stresses were tested by using electromagnetic flux sensors and strain gages. Secondly, existing stresses of tendon were calculated by a theoretical method. In practical projects, a medium-term existing stress of prestressing tendons of PC box-girder was monitored by using the above-mentioned technique, so as to verify the theoretical results. Eventually, by comparing the tested and calculated data, the existing theoretical calculation method was revised further.*

**Key words:** Monitoring, Prestressing steel tendon, effective stress, electromagnetic techniques, calculation method

## 1 INTRODUCTION

Calculation of prestress losses is a big concern in concrete bridges. Existing stresses of prestressed steel tendons in PC members are vital to ensure the load-bearing capacity of PC bridges. It is quite complicated to accurately calculate the prestress loses due to the tendon geometric alignment, duct friction, shrinkage and creep of concrete. Hence, monitoring or inspection is a way to recognize the real pre-stresses distribution state along the PC steel tendons.

A lot of testing technologies [1-2] were proposed to determine/monitor the evolution performance of pre-stresses with time. They include the technologies of fiber optic sensors, grating sensor, magnetic flux sensor and so on.

Bragg grating sensors can offer good accuracy of 2~3$\mu\varepsilon$, and can be arranged along the PC steel tendons. When sensors and steel tendons simultaneously deform under external forces, the stresses along the steel tendons can be determined via the obtained strain data from Bragg grating sensors. The disadvantage is that the sensors fixed on the steel tendons are easily broken due to tendons' squeezing mutually when the PC steel tendons are stretch-draw. For this reason, the accuracy and reliability of tested data are reduced greatly.

Distributed fiber optic sensors have high strength, flexibility and good durability [3]. They can be used in special and severe environmental conditions. The accuracy of fiber optic sensor reaches 70με which is lower than that of Bragg grating sensors. The sensors face the same problem as Bragg grating sensors while installing within the steel tendons.

Different from the above-mentioned sensor technology, magnetic flux sensor is one kind of non-contact test technology. Generally, the sensors are installed outside of grouting duct and they are able to catch the signals of magnetic flux across the PC steel tendons while deforming under external forces. The testing accuracy can reach 150με. The residual pre-stresses can be calculated via the relationship of magnetic flux vs. deformation. The sensors cannot be damaged by the steel tendons' stretch-draw.

The residual pre-stresses, in general, are theoretically obtained through calculating the difference between the designed pre-stresses and the calculated pre-stress losses. The calculation of prestress losses can be divided into three kinds: Time-dependent iteration method of pre-stressed losses, refined estimate calculation method of pre-stressed losses, and approximate estimate calculation method of total pre-stressed losses.

Time-dependent iteration method of pre-stressed losses adopts iteration calculation with FEM or FDM [4,5], in which the mutual influence of the shrinkage, creep of concrete and steel tendon relaxation are introduced. The calculation time is divided into small time steps. In each step the pre-stress losses are computed numerically with separately considering the effects of shrinkage, creep of concrete and steel relaxation. Eventually, all losses are superimposed to determine the total losses.

Refined estimate calculation method of pre-stressed losses is developed to separately calculate each loss in instantaneous losses and time-dependent losses which occurred by duct friction, anchorage deformation, reinforced retraction and joint compression, concrete elastic compression, steel tendon relaxation, concrete shrinkage and creep. Refined estimate calculation method is mostly adopted to calculate the pre-stress losses in current specifications, such as AASHTO LRFD, BS 5400, European DD ENV 1992, JICE and Chinese standards [6-10].

Approximate estimate calculation method of total pre-stressed losses, pre-stressed long-term loss calculation is more complicated, in order to simplify and estimate the prestressed losses rapidly and efficiently, the United States AASHTO-LRFD [7] introduced a simplified method which is able to process the calculation by simple choose some factors and values from experienced tables and data.

Traditionally the pre-stresses nearby the anchorage of PC steel tendons can be measured by load cell. Although they are often used to represent the average pre-stresses along the tensioned steel tendons, this cannot be applied to calculate the pre-stress distribution, which is vital to design and evaluate the load-bearing capacity for PC bridges.

The presented paper is aimed at monitoring the evolution of existing pre-stresses with time along the tensioned steel tendons by using specially-developed electromagnetic flux sensors. Based on the experimental results, the calculation method for pre-stress losses which is widely used in current technical standards is expected to be modified and improved.

## 2 RESEARCH METHODOLOGIES

### 2.1 Experimental equipment and methods

A technology of electromagnetic sensors was specially developed for this research study by the author's research team [11]. The Technical specification is listed in Table 1. The sleeve-type sensors were amounted at the specified positions (anchorage, quarter point, midpoint etc.), and let the PC steel strand go through them before casting the PC members. The installation schematic of electromagnetic sensors and steel strand is shown in Figure 1.

Table1 Technical specification of sensor

| Strain range | Accuracy | Relative error | Working temperature |
|---|---|---|---|
| 0~ yield strain | 100με | ≤2% | -40~80°C |

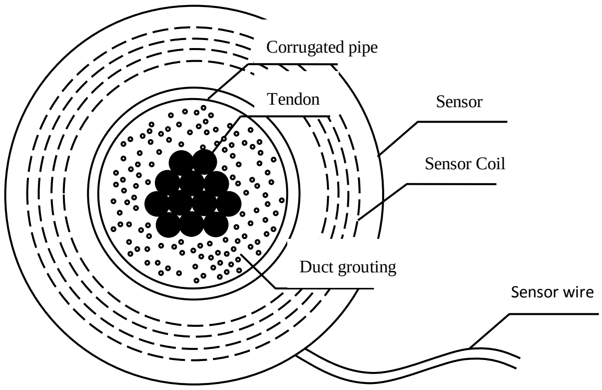


*Figure 1 Schematic for electromagnetic flux sensor and steel strand*

### 2.2 Experimental design

In the presented study, two series of experiments were carried out both in lab and in real bridge. A large-scale model beam with dimension 24×0.5×1.7m was designed and fabricated in lab (see Figure 2). The test was lasted for 360 days under self-weight load. The properties of concrete, steel bar and PC steel tendon are same as those applied in real bridge (see Table1). The model beam was used to verify the efficiency of the developed sensor technology. 4 sensors were installed along the longitudinal direction for each of two PC steel tendons. They are used to monitor the pre-stresses' evolution at the positions of stretch-drew points, quarter point and midpoint (positive bending moment zone). The sensor arrangement was shown in Figure 3.

*Figure 2 Large-scale experimental beam*

Table 2 Materials' Properties

| Concrete | | Steel bar | | Steel tendon | | Grouting duct |
|---|---|---|---|---|---|---|
| C55 | | HRB335 | | 12-$\Phi^{s}$15.2 | | PE |
| strength | 55MPa | tensile strength | 335MPa | tensile strength | 1860MPa | Φ90mm |
| E-value | $3.5\times10^{4}$MPa | E-value | $2.0\times10^{5}$MPa | unit weight | 13.212kg/m | - |
| | | Diameter | 25mm | E-value | $1.95\times10^{5}$MPa | |
| | | | | Diameter | 15mm | |

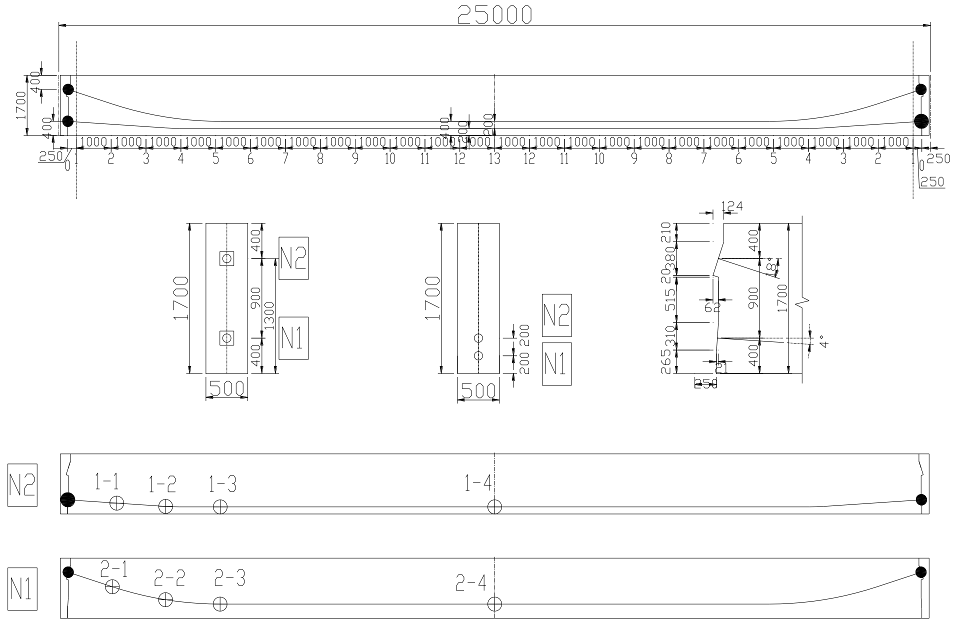


*Figure 3 Large-scale PC model beam and sensors' layout*

A real PC bridge constructed in Zhejiang Province was selected to conduct the in-situ test. The PC continuous box-girder bridge elevation is 3×60+11×86+60m. The test went on 220 days under self-weight of concrete box-girder. Top straight steel strand of 82m length (T10), web curved steel strand of 40m length (F5) and bottom close-up steel strand of 52m length (Z5) for a main span of box-girder were selected to install the sensors. The sensors' layout is shown in Figure 4. The response feature of structural behavior for end, quarter and mid beam is considered in the sensor's layout.

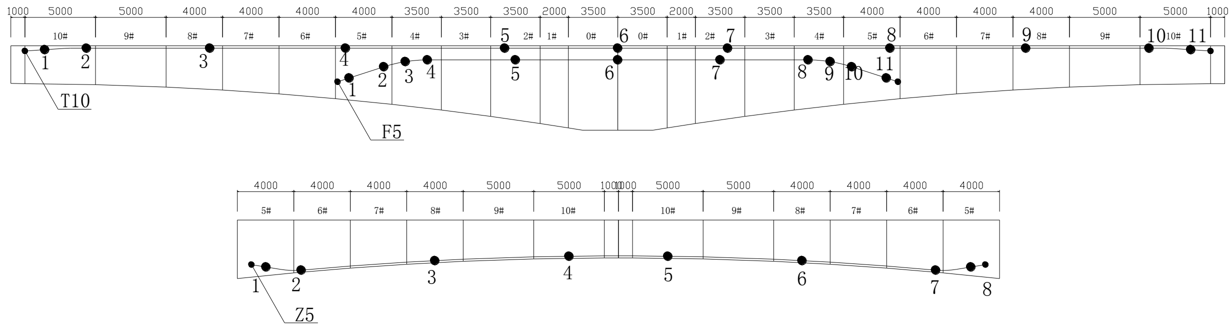

*Figure 4 PC box-girder and sensors' layout*

# 3 EXPERIMENTAL RESULTS

## 3.1 Stress distribution and evolution of prestressing strand with time for PC model beam

For the measured and calculated results, their stress difference at the end of tendon N1 separately reaches 0.15% and 1.86% at tensioning moment and 180 days. The stress difference at the anchorage is 7.13% and 4.12% at tensioning moment and 180 days respectively (see Figure 5). A same analysis is carried out for tendon N2. The difference in pre-stresses between measured and computed values reaches 6.09% and 2.75% at tensioning and 180 days respectively. The difference value at anchorage end is 6.97% and 5.04% at tensioning and 180 days, respectively (see Figure 6). It is obvious that the stress difference for straight tendon N2 is lower than that for curved tendon N1.

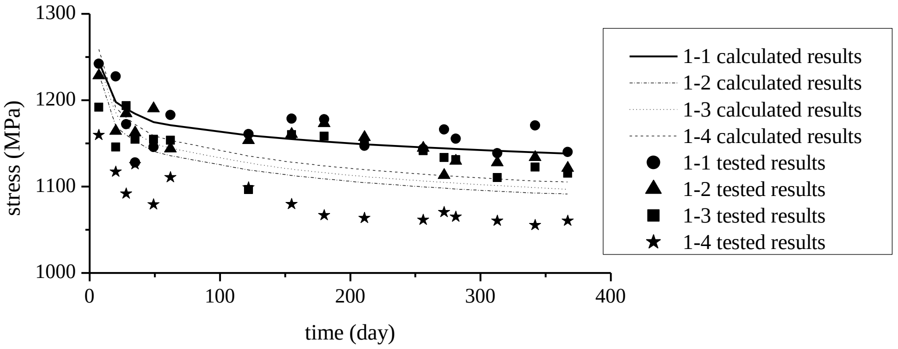


*Figure 5 Existing stresses in prestressing strand for PC model beam (Tendon N1)*

Generally, the stress difference between the measured and theoretical values is getting smaller with time. This indicates that the pre-stresses are transmitting with time and gradually approaching the calculated value in accordance with the technical specifications. The small stress difference also means that the short length and small curvature have less influence on the pre-stress distribution in steel strand, so the calculation method provided by the technical standards is suitable when the PC steel strand has short length and lower curvature. For the long steel tendon, further study should be conducted to test the pre-stresses' distribution and their evolution with time.

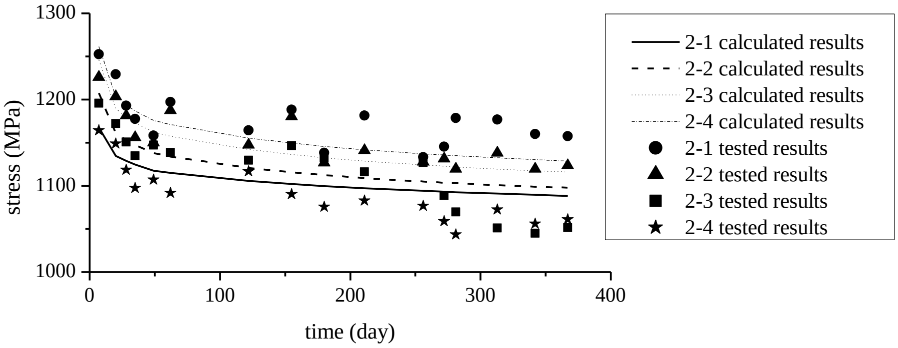


*Figure 6 Existing stresses in prestressing strand for PC model beam (Tendon N2)*

### 3.2 Stress distribution and evolution of prestressing strand with time for PC bridge

The tested results show that the pre-stresses are reducing with time and getting stable. The stress difference between the measured and calculated/designed values at the end nearby anchorage of steel tendon is almost remaining constant with time, while the difference at the midpoint of steel tendon is slightly increasing.

For steel tendon T10, the stress difference between the measured and calculated/designed values at the end nearby anchorage reaches 9.38% and 3.32% at tensioning moment and at 152 days respectively. The difference at the midpoint is 16.82% and 18.12% at tensioning moment and at 152 days respectively (see Figure 7).

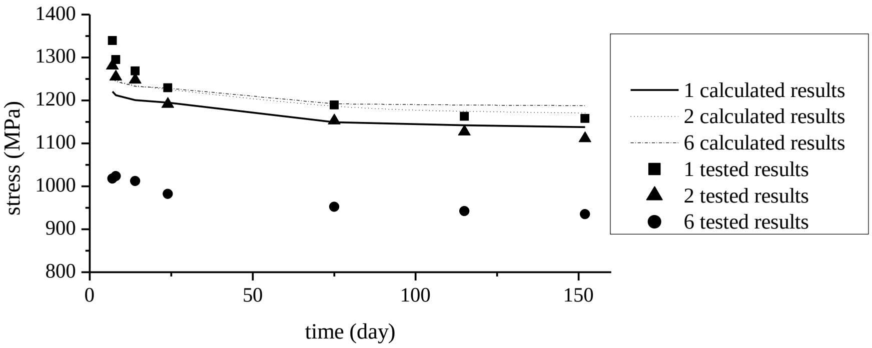


*Figure 7 Existing stresses' evolution in prestressing strand T10 with time*

For steel tendon F5, the stress difference between the measured and calculated/designed values at the end nearby anchorage reaches 13.63% and 3.79% at tensioning moment and at 220 days respectively. The difference at the midpoint is 18.90% and 22.85% at tensioning moment and at 220 days respectively (see Figure 8).

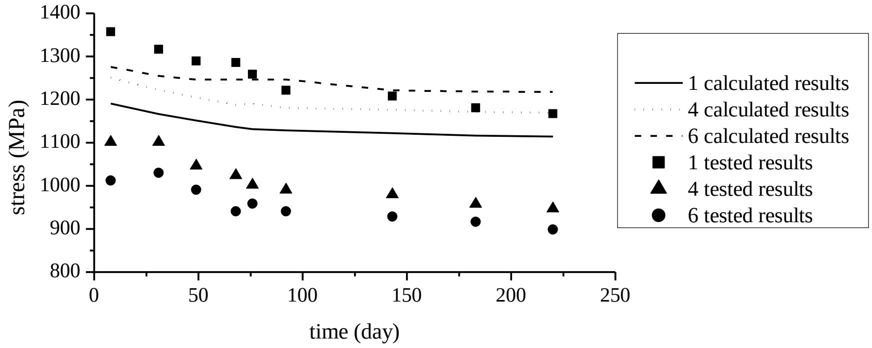


*Figure 8 Existing stresses' evolution in prestressing strand F5*

For steel tendon Z5, the stress difference between the measured and calculated/designed values at the end nearby anchorage reaches 7.46% and 5.48% at tensioning moment and at 85 days respectively. The difference at the midpoint is 7.13% and 1.72% at tensioning moment and at 85 days respectively (see Figure 9).

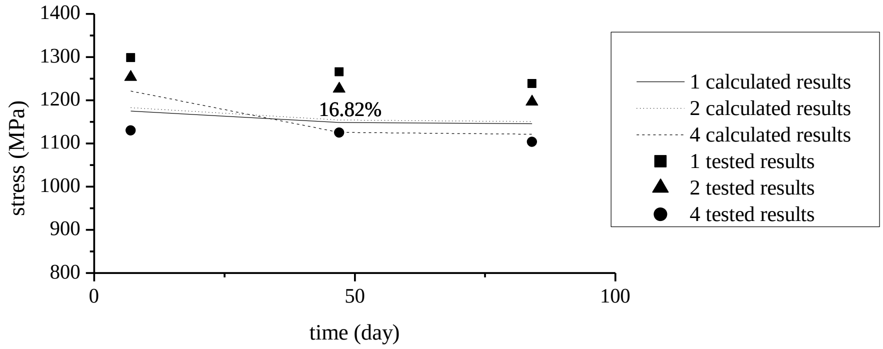


*Figure 9 Existing stresses' evolution in prestressing strand Z5*

It can be seen from the above results that the difference between the tested and calculated values both for long-straight and for long-curved tendons are larger from tensioning time as compared to the results of model beam test. The difference at the anchoring end is getting smaller with time, while the difference at the midpoint is becoming large. This indicates that the increase in stress difference is caused by increasing the tendon length and curvature, which leads to the enhancement in friction resistance.

### 3.3 Estimation of real friction resistance based on tested results

Figure 10 to Figure 12 show the pre-stress distributions along steel strands T10, F5 and Z5. As compared to the calculated results, generally, the real pre-stresses are lower than those calculated by the method of technical standards (see Figure 7 to Figure 9) except for those at the anchorage ends since tensioning steel strands.

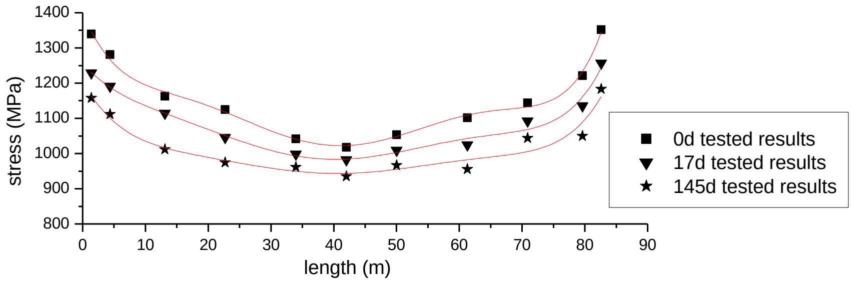


*Figure 10 Stress distributions along prestressing strand T10*

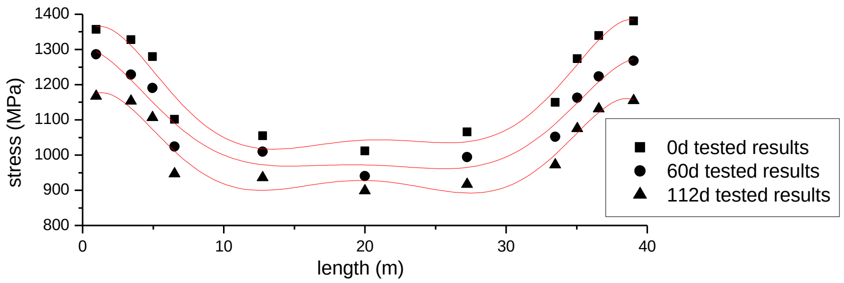


Figure 11 Stress distributions along prestressing strand F5

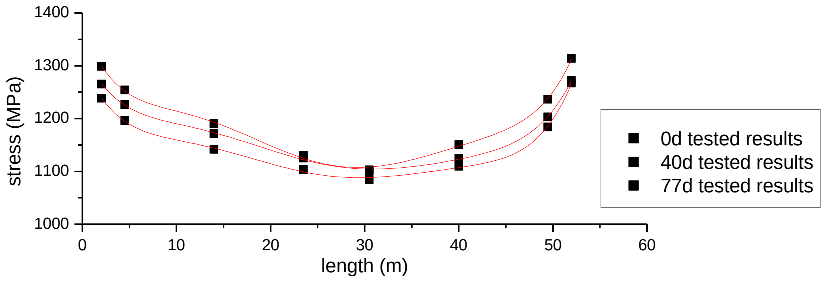


*Figure 12 Stress distributions along prestressing strand Z5*

The difference value is probably caused by the friction resistance which is large than the given values in technical standards. Hence, the test results were used to calculated the real friction coefficients with using the following formula [6].

$$\sigma_{s-m} = \sigma_{con} - \sigma_{l1} - \sigma_{l2} = \sigma_{con} \cdot e^{-(\mu\theta(x)-kx)} - \frac{\sum \Delta l}{l} E_p \tag{1}$$

Where $\sigma_{\text{s-m}}$ - effective stress of prestressing steel strand; $\sigma_{\text{con}}$ - controlled stress of prestressing steel strand; $\sigma_{l1}$, $\sigma_{l2}$ – losses in prestressing steel stress due to friction, anchorage deformation, elastic shortening and joint compression; $\mu$ - friction coeff. between the strand and the duct; $k$ - affecting coeff. of partial deviation of duct per meter; $\Delta l$ - values of anchorage deformation, elastic shortening and joint compression at tensioning end; $l$ - distance from the tensioning end to anchoring end; $E_{\text{p}}$ – elastic modulus.

The suggested values of friction coeff. from bridge design specification of China [6] are: $\mu$=0.14~0.15, $k$=0.0015. By introducing the real prestressing steel stress into the equation (1), the real friction coeff. $\mu$ and affecting coeff. $k$ for prestressing steel strands can be solved. The suggested values and real values are listed in Table 3. It is obvious that suggested values are much lower than the real ones.

*Table 3 Suggested and tested coeff. $\mu$ and k*

| Tendon No | Tendon type | Suggested values of China specification (JTG D62-2004) | | Inverse calculated values | |
|---|---|---|---|---|---|
| | | $\mu$ | $k$ | $\mu$ | $k$ |
| T10 | long and straight | 0.15 | 0.0015 | 0.37 | 0.0035 |
| F5 | long and curved | 0.15 | 0.0015 | 0.36 | 0.0071 |
| Z5 | long and slightly curved | 0.15 | 0.0015 | 0.08 | 0.0039 |

# 4 CONCLUSIVE REMARKS

For the short straight and slightly curved tendons, the difference in prestressing steel stress between the tested and calculated results is small and acceptable for bridge design. This means that it is suitable to calculate the prestressing steel stresses by using the method of bridge design specification.

For the long straight and long curved tendons, the difference in prestressing steel stress between the tested and calculated results is much larger both at the tensioning moment and at later time. It means that the suggested values of friction coeff. $\mu$ and affecting coeff. $k$ are underestimated during designing bridges. The reason is that the actual friction resistance and duct deviation are larger than the recommended ones in bridge design specifications [6]. Hence, the real friction coeff. $\mu$ and affecting coeff. $k$ should be tested before tensioning, and then introduced the calculation formula.

For the long and curved steel tendons, the difference in prestressing steel stress between the tested and calculated values both at tensioning end and at midpoint of steel strand are very large, when steel strands are just tensioned. At the tensioning end of steel strand, the stress transmission is taking place with time, so the difference between the tested and calculated values gradually decreases. At the steel strand midpoint, the tested value is less than the calculated value, and the tested value is gradually reduced with time. This is probably caused by the long-term prestressing steel stress losses due to concrete shrinkage and creep, and it leads the tendon relaxation and continuous reduction of the tested stresses. As a result, the later stress difference is bigger than the former one.

**ACKNOWLEDGEMENTS**

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# MOORING CONCEPT FOR DEEP WATER CROSSINGS

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## ABSTRACT

*A new mooring concept applicable for floating bridges is presented. An idea adopted from the tension leg platform concept is used. The basic idea is to obtain side anchoring of long floating bridges at crossing sites with deep water (> 500 m). The concept is constructed with two separated pipelines, installed across the fjord. An artificial seabed at water depth 30 m is obtained by tensioning the pipelines towards each other. Floating bridge pontoons could now be anchored to the artificial seabed following the side anchoring idea. For ship passage in mid fjord, a submerged floating tunnel is proposed. A preliminary design is performed on the anchoring concept, primary for feasibility checks and early phase cost estimates. Dimensions from Bergsøysundet floating bridge is utilized for the floating bridge part while a cylindrical cross section of 12 m diameter from Sulafjord is assumed for the submerged floating tunnel. Dimensions known from the Heidrun tension leg platform is checked against the loads obtained from the floating bridge and submerged floating tunnel. Capacity checks performed so far shows that the concept is feasible within the framework of known dimensions, material qualities and installation methodologies.*

## INTRODUCTION

New initiatives have been presented in Norway recently to challenge existing technology to cross fjord spans in the range of 1000m to 5000m with water depth up to 1250 m. To date, the world longest suspension bridge Akashi-Kaikyo spans 1991 m while the longest floating bridge is the side anchored Governor Albert D. Rosellini Bridge in Lake Washington crossing 2300 m at shallow water. Some twenty years ago, two end anchored floating bridges were constructed in Norway, Bergsoysundet bridge (980 m) (Fig. 1) and Nordhordlandsbrua (1240 m). Bergsøysundet bridge was the world first end anchored floating bridge while Nordhordlandsbrua is the world longest. End anchoring was chosen in Norway primary due to the water depth at the two sites. As shown by Langen (1980), the initial idea was to construct the bridge as a continuous floating box. This was reevaluated after the Hondo Channel disaster in 1979 and discrete pontoons were used with a separation of about 100 m.

A new concept is now proposed that allows the side anchoring philosophy to be applied at larger water depths. Side anchoring is of interest at deep water due to the technical challenges with direct connection of anchors at deep water together with sloped seabed bathymetry.

*Figure 1. World first end anchored floating bridge – Bergsøysundet bridge – Norway.*

## NEW FLOATING BRIDGE CONCEPT

A crossing method combining the well-known side anchored floating bridge concept with the submerged floating tunnel idea is presented. The new structure consists of three parts:

- Submerged anchoring system (SAS)
- Floating pontoon bridge (FPB)
- Submerged floating tunnel (SFT)

### Submerged anchoring system

Anchoring system consist of two steel pipelines crossing the water way as indicated in Fig. 2. The pipelines are tied together with trusses to get stress level about half of yield. Standard offshore carbon steel pipelines with diameter $D$ = 1066 mm (42 inch) and wall thickness $t$ = 35 mm is commonly used dimensions for oil and gas transport pipelines. The ratio $D/t$ = 30 means that the pipline is neutral in water. Pipes and trusses represent now an artificial seabed suitable for side anchoring a floating bridge. Governing horizontal loads are primarily environmental loads due to wind, waves and current on the SAS, FPB and SFT, respectively.

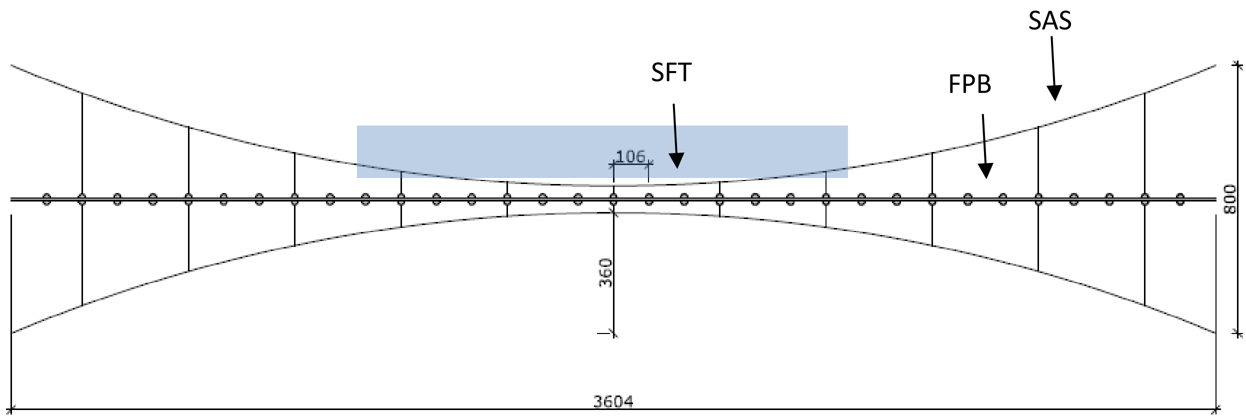


*Figure 2. Top view of the new concept of crossing.*

When robustness is concerned, water tightness of the mooring pipes in SAS is a key issue. A bundle configuration consisting of individual pipes strapped together is considered as illustrated in Fig. 3. Monitoring of water inclusions in the SAS may also be considered for redundancy.

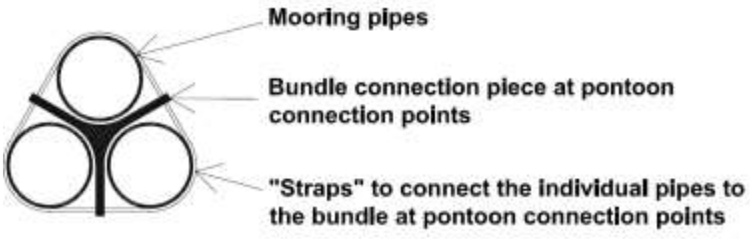


*Figure 3. Bundle of strapped pipes*

Termination of SAS to land is considered as shown in Fig. 4. A tunnel could be made where the anchoring pipeline should be terminated. The tunnel will be available for dry inspection during the structure lifetime.

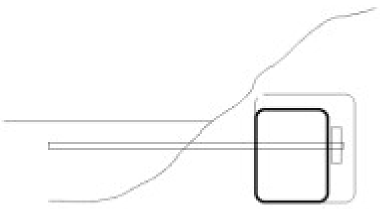

*Figure 4. Land anchor of SAS in a dry rock tunnel.*

**Floating pontoon bridge**

A floating structure inspired from the Bergsøysundet bridge is applied in the study. Every forth pontoon is connected to SAS with trusses. The pontoon separation is about 106 m as indicated in Fig. 2. Cross section details of FPB are shown in Fig. 5. The dry weight of pontoons is about 2000 tons each while the truss girder is 8.2 ton/m.

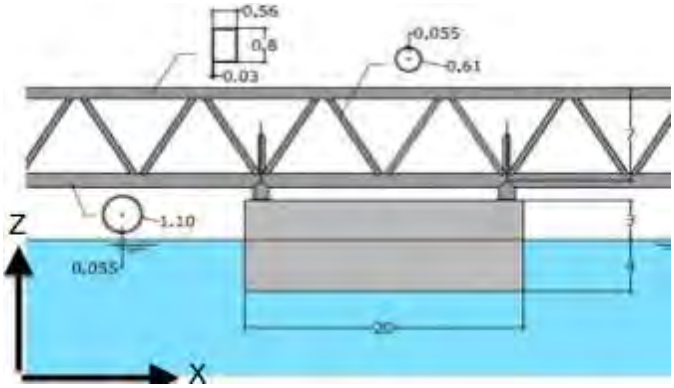


*Figure 5. Typical pontoons and truss girder inspired from the Bergsøysundet bridge.*

**Submerged floating tunnel**

The submerged floating tunnel (SFT) concept got its patent approval in the mid 1960'ties but has never been constructed in full scale. Different cross sections have been evaluated and documented feasible for construction. The ship passage is a governing structure for long crossings and a significant cost driver for floating bridges. After evaluation of different kinds of air spanning bridge concepts for ship passage, it has been found cost effective to have a SFT instead of a classical air spanning bridge. Preliminary calculations have been performed with the cross-section from the Sulafjord concept study sumarized by Jakobsen et al., (2009) as shown in Fig. 6. The weight of the tunnel is assumed to be about 200 ton/m while the hydrodynamic diameter is about 12 m.

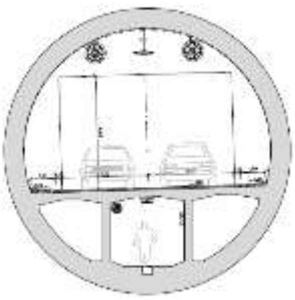

*Figure 6. Submerged floating tunnel cross section (Jakobsen et al., 2009).*

**Numerical model**

The proposed geometry is modeled with beam elements in FE software ANSYS (Fig. 7) to obtain preliminary magnitudes of responses from relevant environmental loads. Linear elastic beam elements (PIPE16) are applied for all pipe cross sections.

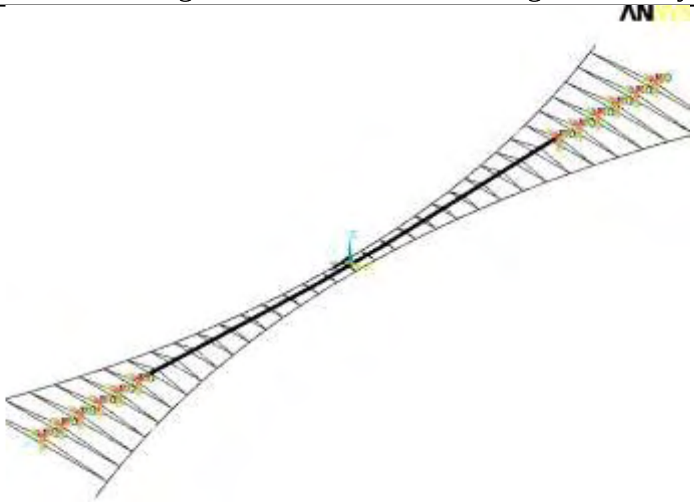

*Figure 7. FE model of anchoring system and floating bridge including the submerged floating tunnel.*

**Applied loads**

Governing loads for the concept is together with traffic and accidental loads, loads from waves, wind and currents. In the concept development phase, environmental loads are handled as quasi static loads of Morrison type (Faltinsen, 1990). Special attention is given to the surface piercing part of the SFT and the transition to the FPB. The concept is, by its slender nature, vulnerable to slowly varying loads from winds and waves. Available methods to handle the long periodic dynamic effects may differ in design codes for bridges as HB 185 (2012) compared to codes traditionally applied for offshore structures as DNV-RP-205 (2007). This is planned to be addressed in the further evaluation work of this concept.

## RESPONSE CHARACTERISTICS

Preliminary calculations have been performed with the above mentioned geometries and loads. Static and dynamic response characteristics have been outlined based on the early phase findings. So far horizontal responses have been studied only.

**Static response**

Static response in sway (horizontally) is worst case estimated to be in the range of 15-20 m. This has to be evaluated in the light of functional criteria for the bridge structure.

**Dynamic responses**

The longest eigenperiod of the system is presently about 130 seconds in sway (Fig. 8). For such a flexible system, a major response contribution from wave and wind low frequency effects is to be expected.

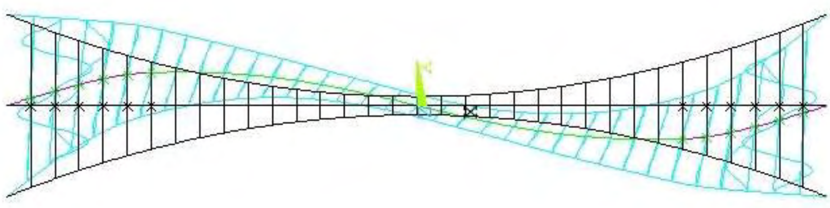

*Figure 8. First Eigenmode, T = 130 sec.*

Thus as part of the project, numerical tools are developed for handling second order wind and wave loads and response calculations. The phenomenon of low frequency response is well known from offshore floating structures where natural periods in the range of minutes are handled. The major difference here is that the present flexible structure is exited both in the low and high frequency range, whereas a rigid floating platform by its low mooring frequency possesses minor sensitivity to first order wave response.

Based on static and dynamic response calculations the following evaluations emerge:

- The SFT with its large mass governs the dynamic response in the system and thus the amount of slowly varying loads
- Second order wind on the FPB as driven by the low frequency of the system comes out as a major load contribution
- Second order wave drift response is of the same magnitude as slowly varying wind effect.

**DISCUSSIONS**

A new concept for side anchoring a floating bridge is presented herein. Fig. 9 shows in an artistic view how the side anchoring philosophy can be applied at deep water. By omitting direct seabed anchoring, it is believed that a side anchored concepts may be proven cost effective.

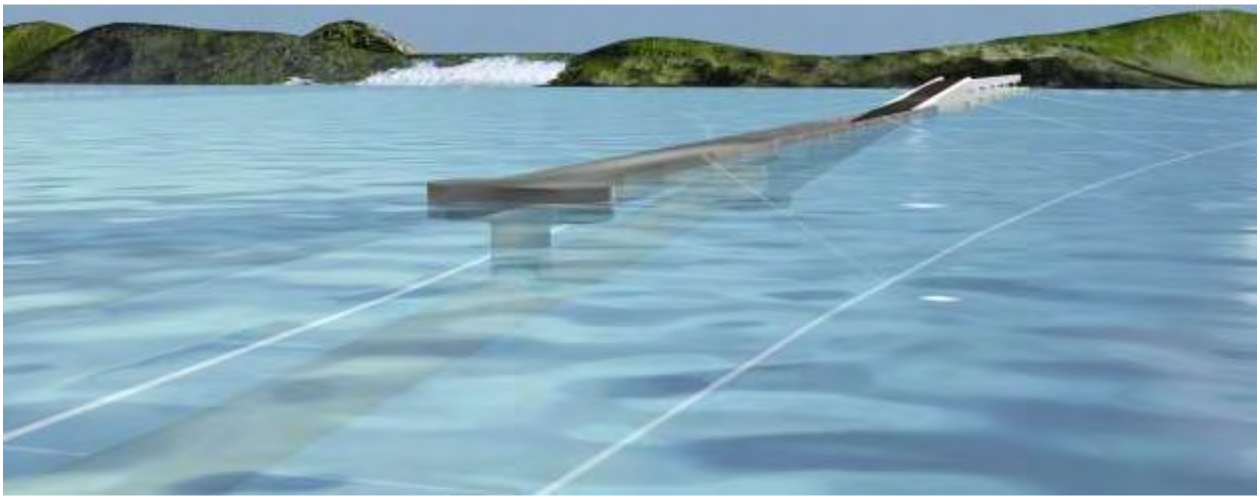

*Figure 9. Floating bridge concept including the proposed artificial seabed.*

The present concept enables a floating bridge combined with a submerged tunnel by a mooring system where stiffness is defined by built in geometry and axial stiffness. The mooring system can be pre-installed and thereby serve as a support during coupling of the complete bridge.

It is foreseen that the submerged part must be a self-floating structure with capacity to resist wave and wind forces during transport and up-coupling, whereas the floating bridge may arrive in sections and be connected at site. A major challenge is then to come up with the self-carrying mid part. A solution here is to have the slender tunnel section as short as possible like 400 m as required for ship passage, and then let the transition part be stronger in the water line. The bridge now acts like a sideway moored system with no axial force and can be straight and axially fixed at one end.

The horizontal geometry of the mooring system is defined by the transverse connectors and as such may be optimized for the actual bridge concept. With a heavy central part consisting of SFT and transition pieces more of the initial geometry curvature should be devoted to the mid part and close to straight cables along the lighter floating bridge at both ends. Such a tuning process on stiffness and mass is to be made at early stage of the design to smoothen the deformation modes.

As a minimum of subsea welds is desired in the main pipes the transverse connectors are strapped on to the main pipe, alternatively to the bundle connection piece in Fig. 3. The strapping has to be documented and to be easy for inspection by ROV. A typical mooring is then shown in Fig. 10.

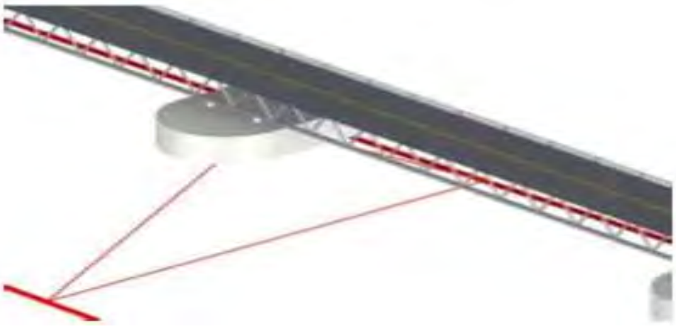

*Figure 10. Connection between anchor and pontoon/truss.*

The connection between mooring line and pontoon as illustrated in Fig. 10 is at the pontoon bottom, however, it is foreseen that the mooring lines enter at the pontoon deck where a winch allows adjustments during installation.

Axial stiffness of the main steel pipes together with initial geometry defines the lateral characteristics of the mooring system in sway motion. The pipe dimensions used are within proven technology for offshore platforms and experience is gained on pipe fabrication and welding. It is foreseen that offshore steel qualities are applied for which welding procedures are well defined.

The pipes arrives from fabrication yard in joints of 12 m length and a welding site at land is foreseen feeding pipe into the sea where the pipe is anchored. Each of the two main pipes are then installed in full length with geometry controlled by tugs before cross connectors are established. Depth control is made by buoyancy elements attached to the pipes before final tensioning.

## CONCLUSIONS

The paper presents the major characteristics of the artificial seabed mooring system, and presents a bridge concept in the form of floating bridge and a submerged floating tunnel as attached to the system. The major benefits by the proposed artificial seabed are as follows:

- The mooring system is independent of water depth at site.
- The sway stiffness for the completed system is taken by the artificial seabed. By combining pre stressing and geometry there is the option to optimize the stiffness in view of the mass of the bridge.
- The mooring system being installed allows sections of the floating bridge to be attached, thereby reducing the structure length to be handled during installation.
- The system allows slender dimensions of the bridge cross section. Thus proven sizes built for the existing bridges can be applied.

The concept is based on mooring systems for floating offshore structures with dimensions within the range of proven technology where experience on fabrication and welding technology is available.

## ACKNOWLEDGEMENTS

Cooperation with Dr. Techn. Olav Olsen and Snøhetta AS is most apreciated. The work is partly funded by Norwegian Research Council and Reinertsen AS.

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# SHIP IMPACTS ON THE FLOATING PONTOONS SUPPORTING A MULTIPLE SPAN SUSPENSION BRIDGE

by

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## ABSTRACT

*Under contract with Norwegian Public Roads Administration (NPRA) a three-span suspension bridge has been developed for crossing the Sognefjord on the west coast of Norway where the fjord is 3700 m wide and 1250 m deep. The two main bridge towers situated in the fjord are supported by large floating concrete pontoons. The fjord constitutes a main sailing route for a large number of cruise ships, a variety of cargo ships as well a high speed passengers crafts, and the pontoons might be subjected to impact by large ships at high speeds if they for one reason are out of control. NPRA has provided a risk analysis defining "design impacts" representing impacts with a return period of* $10^4$ *years. The abstract outlines the bridge and the impact cases, and describes the series of analyses that was performed to evaluate the response and the consequences for the bridge. The impact loads are shown to be very large and the resulting vibrations of the superstructure are significant. Principles to absorb the impact loads are discussed.*

## THE BRIDGE

The Norwegian Public Roads Administration (NPRA) has started the work to study if it is technologically feasible to replace ferry crossing of the 8 widest fjords on the west coast of Norway by fixed links.

As a case representing the technological challenges of such crossings the Sognefjord has been selected for feasibility studies. At the location of the bridge crossing the Sognefjord is 3700 m wide and up to 1250 m deep with the water depth increasing very abruptly from the shore on both sides.

A three-span suspension bridge supported partly on floating pontoons, as illustrated in Fig.1 and Fig. 2 is one of the concepts that have been studied for this crossing. The bridge is described in more detail in [1]. The width of each main span is 1234 m. The bridge deck is a conventional orthotropic steel box deck with a width of 18.3 m and a depth of 3.25 m.

The towers are made of steel and consist of four inclined legs each with outer dimensions of 5 x 5 m. These inclined legs form an A-shaped tower seen both in-line and perpendicular to the bridge axis giving enough stiffness and strength to resist the unbalanced forces from the situation when each span is unequally loaded from traffic.

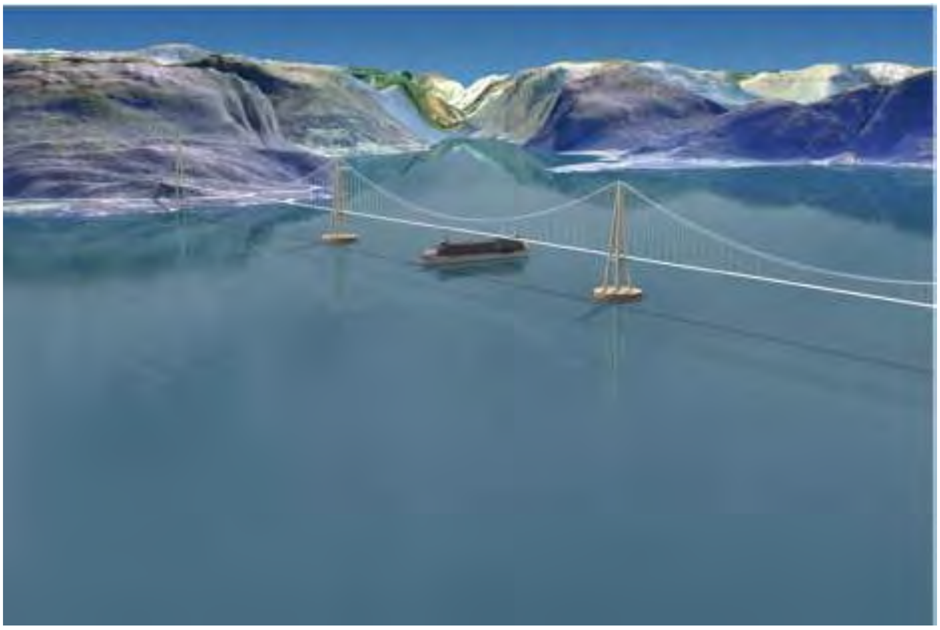


*Figure 1. The three-span suspension bridge crossing Sognefjorden*

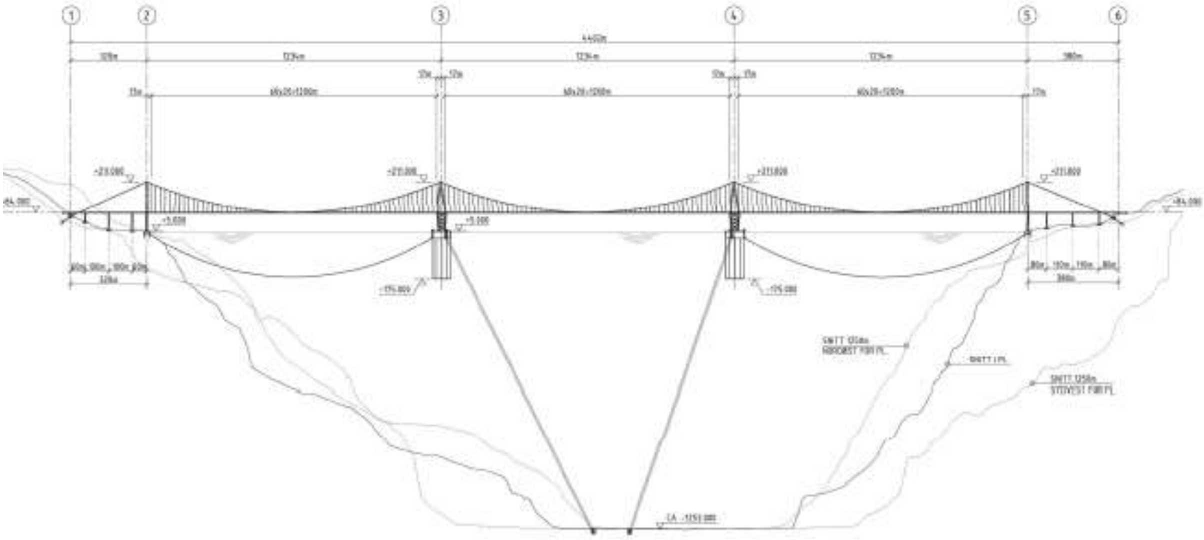


*Figure 2. Elevation of the bridge with anchor system*

The two main mid bridge towers are supported on huge concrete pontoons, see Fig. 3. The pontoons have a free board of 7 m and a draft of 175 m and an equivalent diameter of about 78 m. They consist of 9 circular cylindrical cells in the lower part where the design is governed by external hydro-static water pressure and a circular cell supported by a ring truss and a system of internal walls in the upper part to cope with the high, local forces from potential ship impact. The void spaces between the truss walls are filled with light weight aggregate concrete to provide enough strength in case of a hit between the truss diagonals. The 45m lower parts of the pontoons are filled with heavy solid ballast of olivine giving a large metacentric height and stiffness against tilt of 259 x$10^3$ MNm/rad. The masses of one pontoon including the added mass from water and the mass of the bridge superstructure acting on one pontoon is 0,86 x $10^6$ t of which the mass of the superstructure makes up about 3% only. The rotational inertia in the tilting mode is 3.9 x $10^9$ $tm^2$.

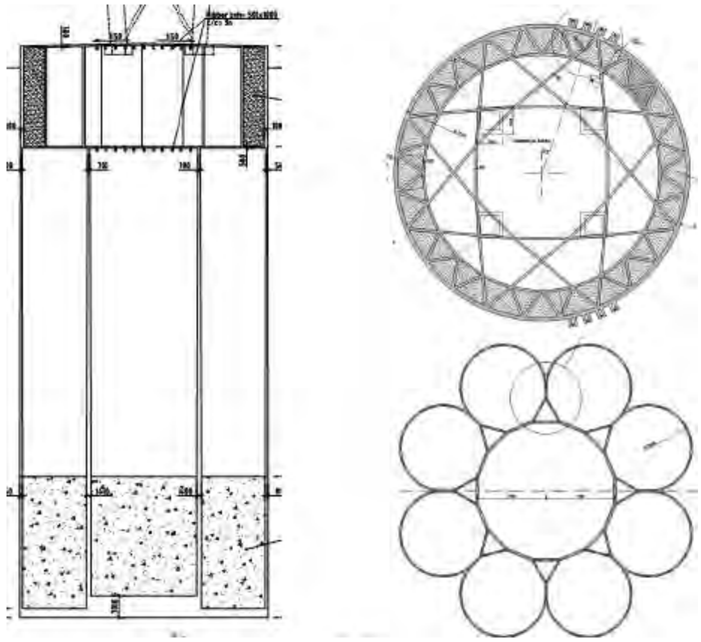


*Figure 3. The pontoons*

The pontoons are anchored to the bottom of the fjord and partly to the shore, as seen on Fig. 2. The anchor system has a total linearized horizontal stiffness perpendicular to the bridge axis of about 2 MN/m. In the direction of the bridge alignment, the bridge deck gives by far the largest contribution to the horizontal stiffness.

## THE SHIP IMPACT

NPRA has subcontracted a risk analysis of possible ship impacts on the bridge [2]. The bridge deck has a free clearance over the whole width of the fjord of more than 70 m, so the possibility of any ship impact on the superstructure has been disregarded. According to NPRA's requirements accidental load cases can be set aside if their probability of occurrence is lower than $10^{-4}$ per year. To represent an impact on the pontoons on this probability level the risk analysis defined a ship for use in the design checks with the following particulars:

Length: 208 m
Breadth: 29 m
Draft: 8.8 m
Displacement: 31456 tonnes
Speed: 17.7 knots (9.1 m/s)

Including added hydrodynamic mass this gives an impact energy of 1565 MNm.

For a head-on impact load indentation curves were given as shown in Fig. 4 of which the curve for the 40.000 DWT ship has been taken as representative for the present case. It is assumed that for a centric head-on impact the total kinetic energy is transferred to the struc-ture. When the impact is eccentric, the ship will slide on the pontoon and only a part of the kinetic energy will be transferred to the structure. Based on the information given in the risk analysis the following three impact cases were thus defined:

1. A centric head-on impact perpendicular to the bridge axis
2. A centric head-on impact in the direction of the main sailing route, giving a skew impact angle relative to the bridge axis of 78º.
3. An eccentric impact with the same direction as impact case 2 with, but with an eccentricity of 26 m and an energy impact transferred to the structure of 66 % of the incoming kinetic energy.

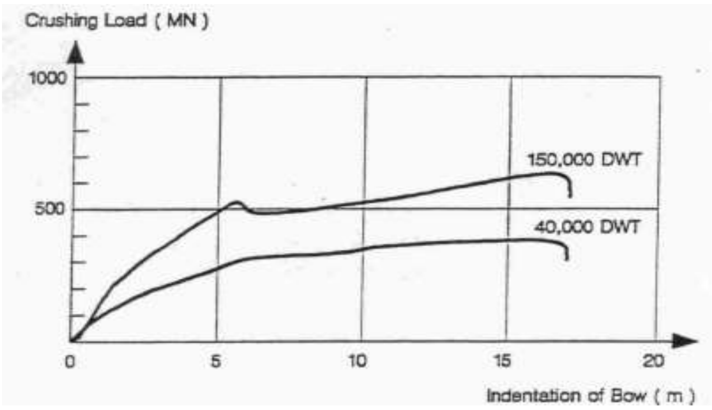



*Figure 4. Load-indentation curves*

Representative load-indentation curves were not known for the eccentric impact. Furthermore, the ship will rotate under and after the impact and the added hydrodynamic mass will be time varying and a very complex function of velocity, retardation and geometry of both the pontoon and the ship. In addition no analytical program was available for the analysis of such a complex impact. Thus, this impact case was estimated by long-hand calculations based on very crude assumptions.

## THE SHIP IMPACT ANALYSES

### General

The available version of the RM Bridge program [3] that was used for the global analysis of the bridge was not designed to simulate the short impact phase. The analysis of the total response to the ship impact was thus made in two steps.

Firstly, the initial value problem was solved by a time integration in the non-linear computer program USFOS [4], that is widely used to analyze e.g. ship impacts on offshore jacket structures. The local dynamics of the superstructure are not modeled here and are thus neglected in the very short period of time of the impact. The time history of the impact force is among the result of this analysis.

This impact load is in the second phase put on the pontoon in the global RM Bridge model by which the total dynamic response of the bridge is analyzed.

### The USFOS-analysis

The USFOS model consists of a vertical cylinder representing the pontoon, and a horizontal element representing the ship, see Fig.5. The pontoon is supported by a vertical spring simulating the buoyancy, a rotational spring at the mass center, a horizontal spring representing the stiffness properties of the anchoring system and a spring representing the bridge deck stiffness. The impact properties are simulated by a nonlinear spring between the ship element the pontoon element and the nonlinear spring properties are digitized from Fig. 4. A structural damping of 0.5% of critical is used. All other energy dissipation is through the strongly non-linear behavior of the spring.

*Figure 5. USFOS model*

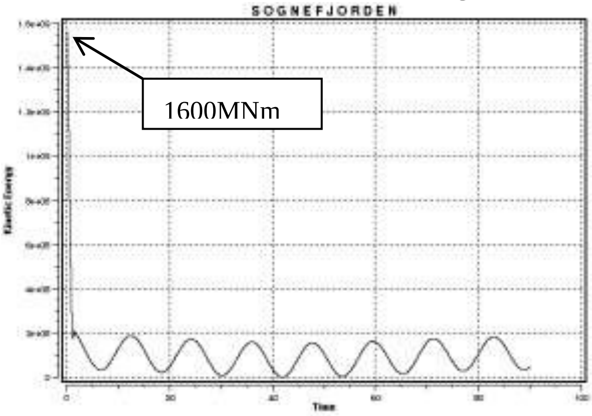
The ship is given an initial velocity of 9.1m/s and USFOS solves the nonlinear dynamic problem in the time domain. When the ship has lost its momentum the nonlinear spring unloads linearly and rapidly, and the ship element and pontoon element separates. The difference between the loading and unloading curve represents a significant hysteresis expressing the energy being absorbed by the plastic deformations in the ship. The analysis is run for 90s.

The load-displacement curve from the analysis is given in Figure 7. The maximum force during the impact is calculated to 340MN and occur at t=1.2 s after initial contact. The impact lasts 1.5 s and the bow is compressed by 6.0 m.

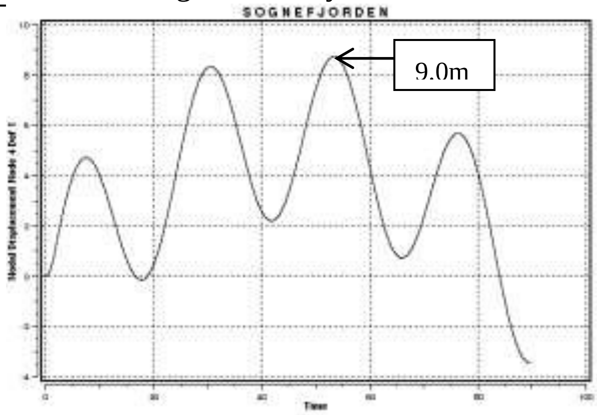
Figure 6 shows time history plots of the first 90s after initial contact between the ship and pontoon. As can be seen the total kinetic energy of the system drops rapidly from about 1600 MNm to 200 MNm. The kinetic energy of the ship is mainly used for deforming the bow, while the remaining kinetic energy initiates motions of the pontoon as seen on the right in this figure. The maximum displacement at waterline is 9.0m and at bridge deck level 11.5m. The retardation of the ship is calculated to 9 $m/s^2$.

**The analysis by the global model, RM Bridge**

A full 3D analysis model has been established in RM Bridge [3] based on beam theory covering non-linear effects as cable element formulation (according to catenary theory), geometric stiffness and large deformations. The analysis is performed in time domain. Figure 7 shows the structural model and the load function from USFOS.




Kinetic energy.




Displacement in waterline.

*Figure 6 Time history plots for the first 90s. Left: Total kinetic energy in the system. Right: pontoon displacement at waterline.*

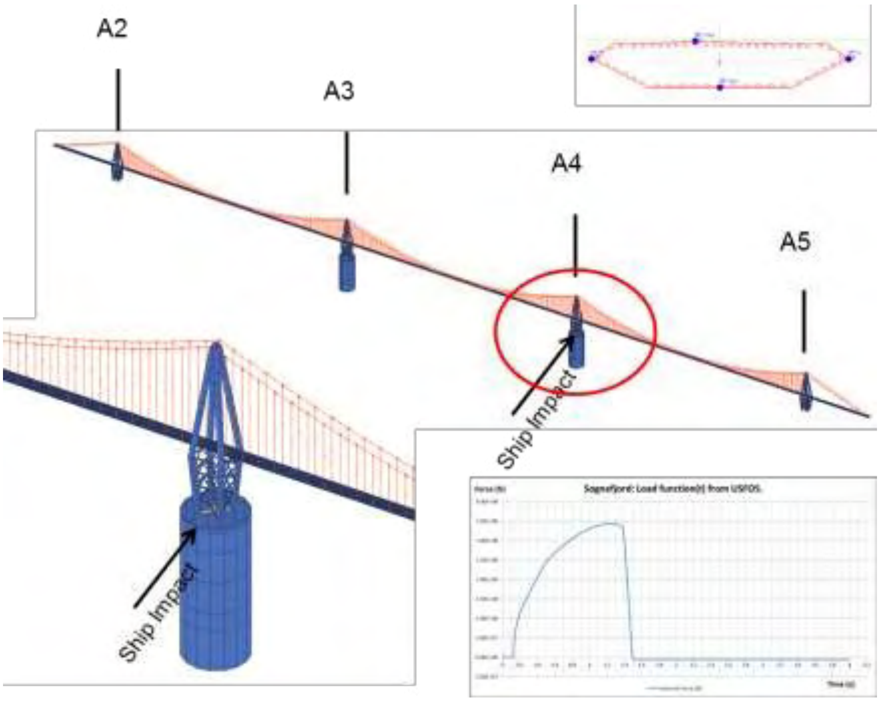



Figure 7. Analysis model in RM Bridge and Load function from USFOS

Initially, the correct loaded geometry of the suspension bridge based on the permanent loads has been established by shortening the main and hanger cable system using an optimization tool within RM Bridge to obtain correct stress-free cable lengths. The permanent loads together with the response due to the ship impact are included to set up the correct geometric stiffness as a function of time.

Three different damping ratios taking hydrodynamic and structural damping as average numbers are investigated initially. In these analysis, the Rayleigh damping method is used where the 1. and the 10. lateral vibration modes were used to define the damping level. For the damping level deemed most likely, 1% and 2% for the two modes respectively, some typical results are shown below.

Figure 8 shows a max motion of 9.2 m in the bridge deck at axis A4 (impact axis). Figure 9 shows a max acceleration of 1.7 m/s$^2$ in the bridge deck at axis A4 (impact axis). The 4 points are located at top of bridge deck in section at axis A3, A4, mid-span between A3/A4 and in mid-span A4/A5.

Figure 10 shows the distribution of the axial stresses along the bridge deck in 4 points. The max axial stress is 728 Mpa on the bridge deck left side. These are very local hot-spot values, and minor modifications reduce these stresses to acceptable values.

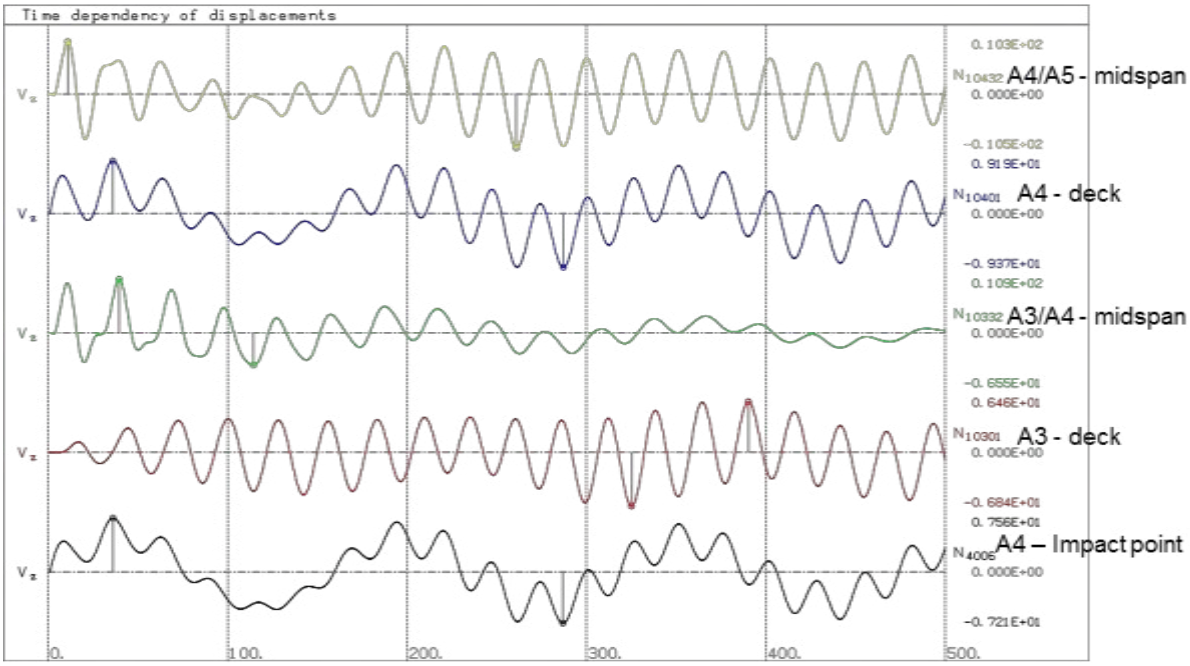



*Figure 8: Lateral motion [m] at Impact point and in 4 deck points along the bridge.*

**The principle of conservation of momentum**

In order to get an idea of the response and the impact forces very early in the design process long-hand calculations were made based on the principles of conservation of momentum. The results of the calculations were later on used to verify the USFOS-results and also to give some crude estimates of the response to eccentric loading.

For the centric impact the structure was modeled as a system with two degrees of freedom, with the stiffness and mass properties as defined above, see Fig. 11.

For the calculations the following assumptions were made:

1. The impact is so short and the displacements of the pontoon are so small that the forces in the anchor system may be neglected.
2. The impact is fully plastic, i.e. the velocity of the ship and the pontoon is the same in the point of contact at the end of the impact.

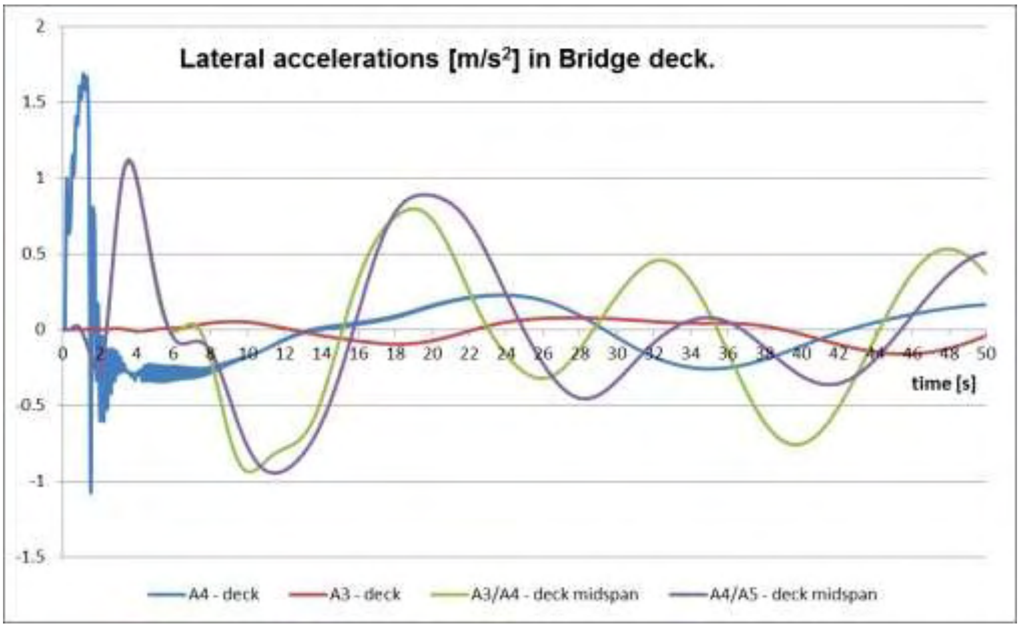



*Figure 9: Lateral acceleration in 4 deck points.*

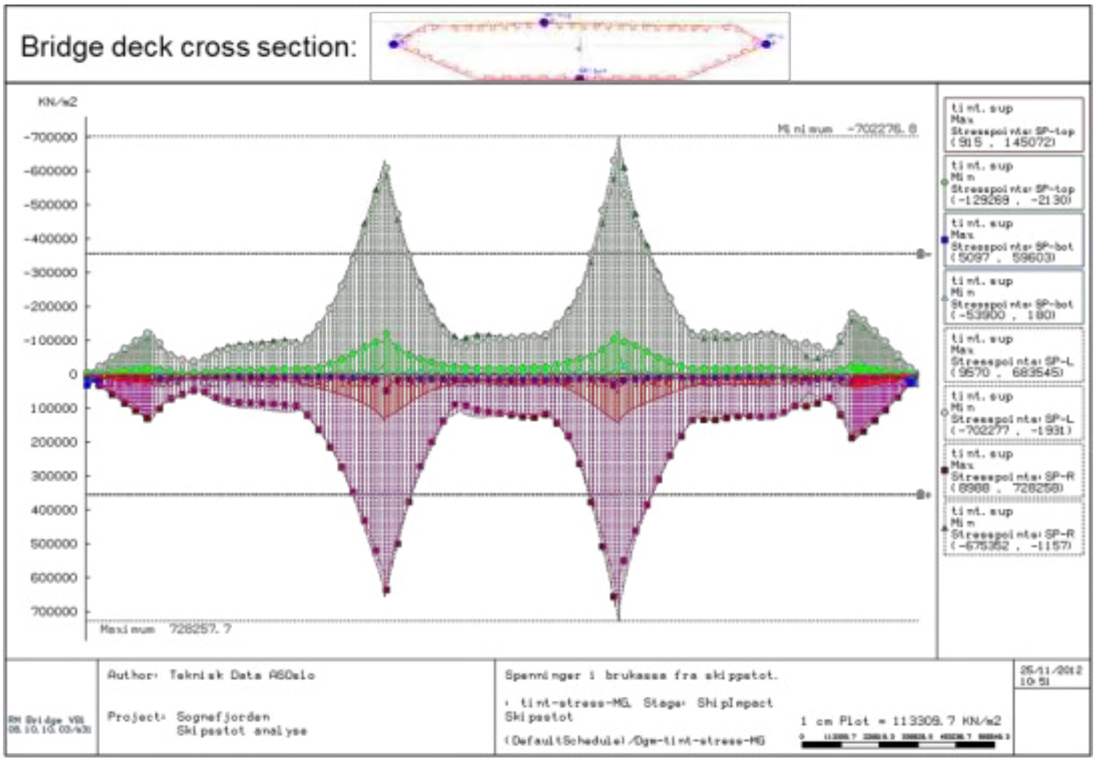



*Figure 10: Normal stresses in 4 points in the bridge girder*

The conservation of momentum then gives:

Linear momentum:

$$m_1 \times v_1 = m_1(v_2 + a \times \varphi) + m_2 \times v_2$$

Moment of momentum:

$$m_1 \times v_1 \times a = I_{m2} \times \varphi + m_1 \times v_2 \times a$$

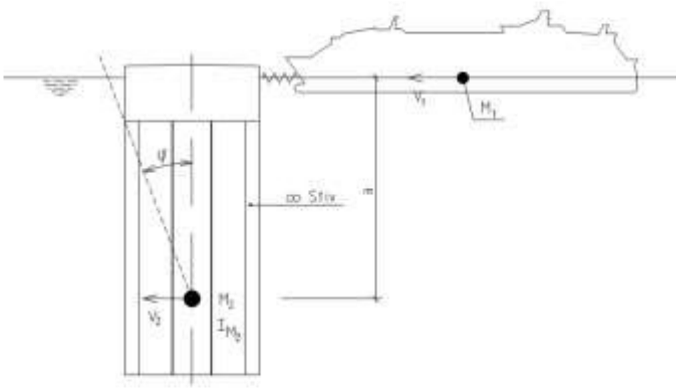


*Figure 11. Simplified 2DOF ship impact model*

where $m_1$and $m_2$ are the masses of the ship and the structure, $I_{m2}$ is the rotational inertia of the structure included the added hydrodynamic mass, $v_1$ is the impact velocity of the ship, $v_2$ and φ are the horizontal and angular velocity of the structure at the end of the impact. Further, a is the distance from the water surface to the center of gravity of the structure including the added hydrodynamic mass.

For the present case this gives the following velocities of the pontoon at the end of the impact:

$v_2 = 0.21$ m/s
$\varphi = 8.6 \times 10^{-3}$ rad/s

From this it may be seen that the kinetic energy at the end of the impact is only about 13% of the incoming kinetic energy. The rest or about 1362 MNm is dissipated by plastic defor-mations in the ship. From the load-indentation curve it may then be found that the maximum impact force is about 330 MN with an indentation of about 7 m.

It may further be found that the horizontal vibration period (in surge) is $T_1 = 170$ s and in tilt, $T_2 = 24$ s. Adding the maxima of the two modes is then a good approximation, giving a maximum horizontal displacement in the water line of 9.4 m and at the deck level of 11,7 m.

It is seen that the above results compare excellently with the USFOS results.

For the eccentric impact the same principles as above are used, now also including rotation about the vertical axis of the pontoon. It is assumed that the difference between the incoming and resulting kinetic energy of the ship equals the energy that is impacting the pontoon, ie 66% of the incoming energy. This gives a ship velocity after the impact of 5.3 m/s. The principles of conservation of momentum then gives a maximum rotation of the pontoon of 1.6º when all dynamics of the superstructure is neglected.

## DESIGN AGAINST SHIP IMPACT

The bridge towers and the bridge deck have been checked in the limit state of progressive collapse for the resulting forces. The results show that the capacities at the most highly stressed spots are near to be fully utilized.

The response from the global model also shows that the dynamic response of the super-structure is pretty vivid and that it builds up over time. It will therefore be important to see if there are methods to increase the damping of the system, for instance by mechanical dampers in the cylinders of the pontoons, by providing external, vertical fins at the periphery of the pontoon or by utilizing the damping in the anchor system.

The impact force of about 350 MN on the pontoon may hit very locally from a level of 20 m below the water surface and to the top of the pontoon. As shown by Fig. 3 the upper part of the pontoon is made very rigid and with high both local and global strength. The shown design has enough strength to resist the impact force.

The duration of the impact is about 1.5 seconds, and the retardation of the ship is close to 0.9 g. It is a question whether this is acceptable for the ship, its crew and the passengers. If not, measures should be taken to reduce the impact force and the retardations of the ship e.g. by establishing restrictions to the allowable speed with which ships can pass the bridge, and/or by mounting more flexible impact absorbing fenders on the pontoons in the areas where ship impact can take place.

## ACKNOWLEDGEMENTS

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# VARIOUS SFT CONCEPTS FOR CROSSING WIDE AND DEEP FJORDS

by

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## ABSTRACT

*In a dialog-based competition organized by the Norwegian Public Roads Administration (NPRA) during the winter and spring 2012, the group consisting of Cowi AS, Aas-Jakobsen AS, Johs Holt AS, NGI and Skanska AS conceived and developed various SFT alternatives for crossing the very wide and extremely deep Sognefjord on the west coast of Norway. In the paper the ideas and principles behind the concepts are described and the special challenges for the various concepts are highlighted. The pros and cons for the different concepts and the conditions under which their special characteristics make the different alternatives to viable and interesting options are discussed.*

## BACKGROUND

The Norwegian Public Roads Administration (NPRA) has started the work to study if it is techno-logically feasible to replace ferry crossing of the 8 widest fjords on the west coast of Norway by fixed links. As a case representing the technological challenges of such crossings the Sognefjorden has been selected for feasibility studies. At the chosen location of the crossing the Sognefjorden is 3700 m wide and up to 1300 m deep with the water depth increasing very abruptly from the shore on both sides. In the early phases of this development work the group consisting of Cowi AS, Aas-Jakobsen AS, Johs Holt AS, NGI and Skanska AS conceived and developed various SFT alternatives for this crossing. A summary of these concept developments are given in this paper.

So far no submerged floating tunnel (SFT) has been built for infrastructure purposes anywhere in the world. For decades however, development work has been going on in many countries. In Norway the by far most advanced development work has been on the Høgsfjord crossing [1]; a work which was terminated some years ago for political reasons.

An SFT consists of one or more tubes providing space for the traffic, anchoring elements if necessary with foundations and abutments to the shore. Performed studies both in Norway and internationally have mainly been based on anchoring lines to the sea bottom or by pontoons penetrating the waterline combined with a shape of the tubes that gives sufficient stiffness and strength in the horizontal direction, see Fig 1, which shows the two principles as proposed for the Høgsfjord-crossing.


*Figure 1. Anchoring principles*

As a conclusion of the Høgsfjord studies, anchors to the seabed would be the preferred concept, and most Norwegian studies have therefore been based on this principle. For the Sognefjord crossing with its extreme water depth this conclusion should obviously be challenged.

As important here is the fact that the fjord constitutes a main sailing route for a large number of cruise ships, a variety of cargo ships as well as high speed passenger crafts. Therefore, pontoons in the fjord might be subjected to impact by large ships at high speeds if these for one reason are out of control. This situation on the other hand calls for a solution where pontoons are avoided. As a result the group then proposed concepts based on both principles for further examination.

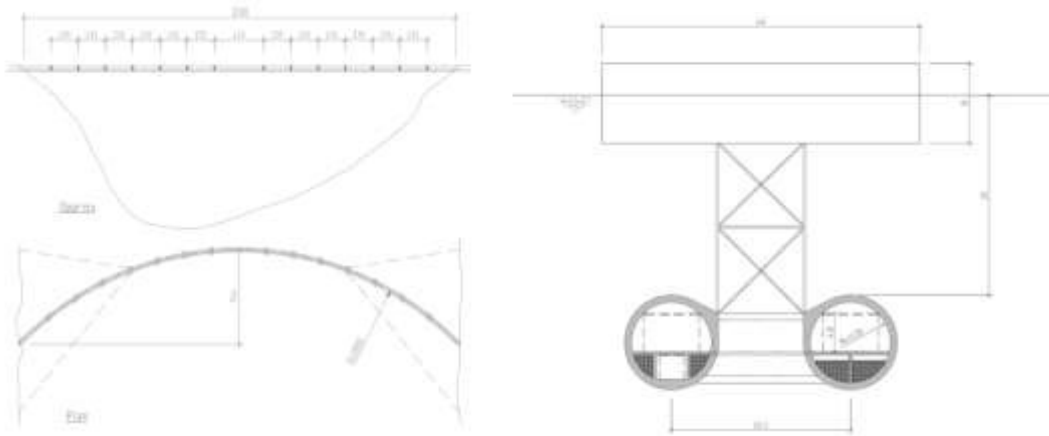
Regarding shape of the tubes multisided cross-sections have in many cases been favored internationally in order to tailor make the internal space to the traffic demands and to the need for dead weight in order to balance the buoyancy. In Norway, low traffic demands have promoted the use of a simple circular tube allowing space for one traffic lane in each direction. For the present study, it was required that the bridge should accommodate two lanes in each direction. Thus, in order to avoid too large net buoyancy two parallel circular tubes and one rectangular tube were proposed for further evaluation.

This work presented is based on the experience from several concept studies, some of which are summarized in [2,3].

## SFT WITH PONTOONS

### The concept

Figure 2 shows the concept based on floating pontoons. For the alternative with two parallel tubes these are connected at a distance of 235 m allowing escapeways between the two tubes of less than 250 m which is the requirement from NPRA. Forming the tubes as an arch in the horizontal plane, the two tubes with these interconnections form a "vierendel" type of curved beam giving a high stiffness in the horizontal plane. This arch then provides the necessary



a. Plan and side view b. Cross-section

*Figure 2. Proposed SFT with pontoons*

stiffness and strength to resist the current, wind, and wave forces in the horizontal direction. The optimal rise to span length ratio of an arch with respect to buckling when subjected to uniform external pressure is about 0.3. In order to reduce the length of the arch a ratio of 0.2 was chosen in the present case. This reduces the critical load from the optimum by about 3% only. A center distance between the two tubes of 22,.5 m was proposed and found to give sufficient stiffness of the arch when the interconnections had a length of about 30 m. A large flexibility exists here to increase the stiffness if found necessary: the distance between the tubes can be increased along the whole length of the tube or locally towards the abutments, and the distance between the interconnections can be reduced. The tubes have an outer diameter of 11.9 m and a wall thickness of 0.9 m.

In the vertical plane the necessary stiffness is provided by the water plane area of the pontoons. The pontoons are located at the same places as the connections between the tubes. In the middle of the fjord one pontoon is however left out to give space for the ship traffic lane which according to the requirements should be at least 400 m wide.

The pontoons have a length of about 40 m, a width of 25 m and a draft of about 9 m. They have a bow-like shape in both ends and are oriented along the main direction of the fjord.

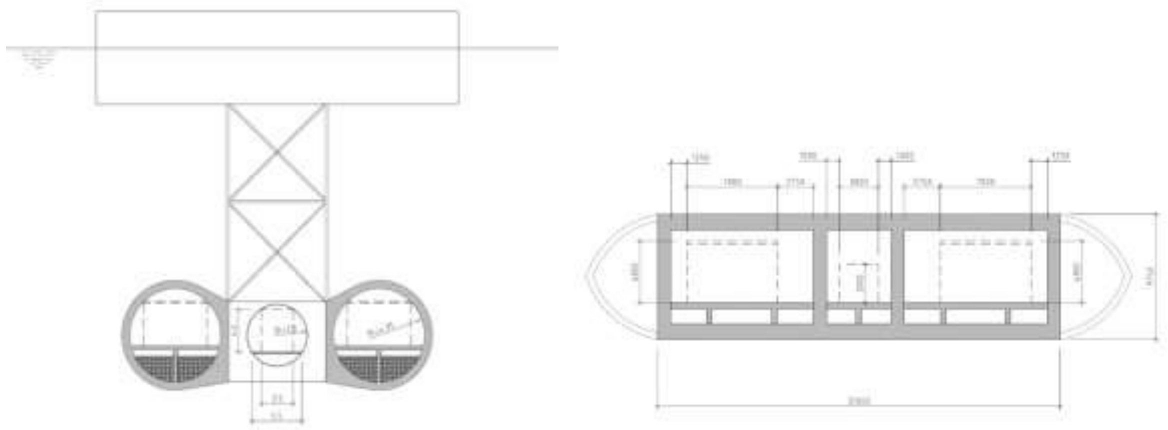
Both the tubes and the pontoons could be made of steel or concrete. In the present proposal the tubes and the interconnections were made by concrete whereas the pontoons and their connections to the tubes were made by steel.

It was a requirement from NPRA that the SFT should provide space also for a separate walkway. This is on Fig.2b proposed to be located in one of the tubes underneath the slab carrying the car traffic. A challenge here was to provide a connection allowing smooth passage by wheel chairs from this level to the traffic lanes and vice versa. This will require ramps with a gradient not larger than 1:20 and landings for rest, implying that the width of the interconnections between the tubes must be increased from 30 m to about 45 m.

As an alternative the walkway could be located in a separate tube, for instance in steel, placed in the free space between the two main tubes, se Fig. 3a.

In the project basis the largest draft of the passing ships was specified to 20 m, so no ship impact on the tubes was considered when the tube was located at a larger depth.

Figure 3b shows the dimensions of a possible rectangular tube. One of the negative aspects of the rectangular form is that it increases significantly the current forces. It will also increase the vertical components of the wave forces, but the significant waves at the actual location is wind-induced waves locally, and they have too short periods to give significant forces at the depth where the tubes are placed. In order to reduce the current forces, "nosed" elements could be mounted on each side of the tube, as indicated on Fig.3b.



a. Walkway in separate tube b. Rectangular cross-section

*Figure 3. Alternative cross-sections*

The double tube and the single, rectangular tube have different merits. The double tube was judged to be better with respect to safety such as dropped objects, explosions and impact from submarines, but was more complex with respect to carrying the global loads in the tubes and also from a con-struction point of view. All together, the group selected the double tube as the preferred one.

**Design challenges**

The proposed SFT has to resist the following special load situations that come in addition to the usual loads from dead weight, buoyancy and traffic:

Buckling as an arch
Ship impact
Vortex induced vibrations (VIV)
First order wave forces
Slowly varying second order wave forces

In the present case the vertical loads have been balanced as much as possible such that the varying loads due to barnacle, water absorption, traffic etc. give about the same net downward weight and net upward buoyancy if all act in the same direction.

*Buckling*

The safety factor against buckling in the horizontal plane is about 30 for wind and current loads. If it in addition very conservatively is assumed that wave forces act simultaneously and in phase on all the pontoons the safety against buckling is still larger than 7. As described earlier it is very easy to increase the stiffness of the tube and thus the safety against buckling, if found necessary.

The lowest buckling mode in the vertical plane has a safety factor of about 10 and involves dis-placements primarily in the midspan area where the pontoon is omitted.

*Ship impact*

The possible ship impact on the pontoons presents a big challenge. According to NPRA's requirements accidental loads should be taken into consideration if their annual probability of occurrence is $10^{-4}$ or higher. The kinetic energy of such a ship at the Sognefjorden crossing would be in the order of 1500 MNm. It was judged that it would be impossible to design an SFT to resist such an impact. A possible solution would then be to design the connection between the pontoon and the tube as a weak link. Figure 2b shows a proposal for such a weak link as a slender steel truss. This truss has to resist the wave and current forces with normal safety factors both with respect to ultimate loads and fatigue. It also has to resist ship impact loads from more "normal" impact cases, e.g. with a return period in the order of 50-100 years. Furthermore, it must be documented that the weak link will break before it will induce forces in the tubes larger than their capacity with sufficient safety margins. Due to the difference between the lower characteristic yield strength and the upper characteristic ultimate strength of the steel as well as load and material partial safety factors, the upper possible capacity of the truss is higher than the design loads by a factor of 3 or more.

The tubes have been checked for the forces induced by the upper possible load capacity of the weak link. A relatively high prestressing level will be necessary to avoid membrane tension in the tube walls for this case. Membrane tension should preferably not be allowed in order to avoid heavy water ingress into the tubes. Again, this situation can be improved by increasing the distance between the tubes and/or by reducing the spacing between the interconnections.

Finally, the tubes have to survive without damage when one of the pontoons is lost. The most critical situation occurs when one of the pontoons near the ship lane is lost, since this will give the largest unsupported span length of the tubes. This situation will also give large demands for longi-tudinal prestressing in the tubes to avoid water ingress into the tubes.

*Vortex induced vibrations (VIV)*

From the design basis it is estimated that the current speed at the depth of the tubes is about v= 1,0 m/s. The first vibrational mode in the vertical direction has an eigenfrequency of f = 0.0451 Hz. This gives a reduced velocity of $v_r$ = v/fxD = 1.0 / (0.0451 x 11.9) = 1.9. This is less than 2.0, which is a limit for the onset of vortex induced transverse vibrations, see Ref [4]. It may thus be expected that cross flow VIV may not present any problem.

For the rectangular cross-section the reduced velocity is estimated to about 3.5, indicating that transverse VIV might be a significant challenge for this shape of the tube.

The first horizontal mode for the double tube alternative has an eigenfrequency of $f_1$ = 0.0117 Hz. This gives a reduced velocity of 1.0/(0.0117 x 11.9) = 7.2. In-line VIV may take place if the reduced velocity is in the range 1 to about 4. The highest mode that gives reduced velocities in this range has a frequency of $f_{12}$ = 0.0631 Hz. Then the reduced velocity is 1.3. According to [4] the maximum amplitude is then below 0.04 x D = 0.04 x 11.9 = 0.5 m, which gives a maximum acceleration of about 0.075 m/s$^2$. This was deemed acceptable.

For a circular cylinder galloping due to current is no issue. However, when two circular tubes act in tandem and the one is located in the wake of the other, galloping might be a problem.

Both this issue and the effect of interference between two cylinders on the vortex induced vibrations should be investigated much more in depth, possibly by model tests, to examine whether these effects may give rise to unacceptable displacements.

*The wave forces*

The dominating waves in the fjord are due to local wind in the fjord basin. According to the design basis the characteristics for a 100 years storm at the site is $H_s$ = 2.3 m and $T_p$ = 4.8 m. The first order effects of such waves will be negligible at the depth where the tube is located. Besides, their period is so short that only very high modes will be excited. Rough estimates show that these waves will not give significant response of the structure. Furthermore, incoming remains from the storm in the outside ocean, estimated at the crossing to be $H_s$ = 0.1 m and $T_p$ ~ 13-14 s, are not expected to give response of any significance.

Slowly varying wave forces of even relatively moderate magnitude may be a threat to such slender structures if resonance can occur. The origin for such wave effects may primarily be due to 2.order wave drift forces and internal waves due to layering in the water from salinity differences.

As a rule of thumb wave drift forces are not expected to give rise to problematic responses if the eigenperiods of the structure are about 30 s or lower.

Based on the results of the work on the Høgsfjord-project it may be assumed that not either internal waves will give rise to significant responses if all eigenperiods of the structure are below about 40 s. Furthermore, due to the nature of the salinity differences these will primarily occur in the summer season where the other wave effects are moderate.

It may then very preliminary be assumed that these two effects will not present feasibility problems to the proposed concept. This of course has to be verified through more advanced investigations at a later stage.

Figure 4 shows how the SFT concept with pontoons may look like.

*Figure 4. The SFT with pontoons*

## SFT WITH TETHERS

### The concept

Here, the tube has positive net buoyancy and is anchored to the sea bottom by tethers that always are in tension. In order to give sufficient horizontal stiffness the tethers are inclined, as shown on Fig. 5. Due to its length of 3700 m the allowable horizontal displacements can be relatively large, so the need for having relatively stiff horizontal anchoring in the transverse direction comes primarily from the requirement to limit the dynamic response to waves and current.

The same cross-sections as shown for the pontoon alternative may also be used for this concept. Based on earlier studies the spacing of the tethers may be in the order of about 250 m. For water depths of 450 m the lower eigenperiod has then been found to be about 5 s. Possible resonance problem with slowly varying wave forces is thus no issue for this concept. The increase in the water depth to 1300 m will tend to increase the fundamental period. As stated above earlier studies have been based on single circular cylindrical tubes. With the cross-sections of Fig. 3 the stiffness will increase, which tends to reduce the fundamental period.

The tethers will be made of steel or alternatively by polyester mooring lines. Steel mooring may be made by steel ropes possibly in combination with chains or by steel tubes. Since the tethers are inclined they will in principle have a sag due to its net weight. This may reduce significantly the
stiffness of the anchoring system. If more detailed analyses show that this gives unfortunate effects, steel tubes may be tailormade so that their weight balances the buoyancy for a certain density of the seawater and quantity of barnacle.

### Design challenges

With all the varying loads such as barnacle, water absorption, traffic and forces from waves and current, the tether forces must be tensile, in order to avoid slack and snapping. This gives a require-ment to the minimum level of pretension in the anchor lines. Together with reversed actions from waves and current when no barnacle, water absorption or traffic are acting this gives relatively large uplift forces on the foundations on the seabed.

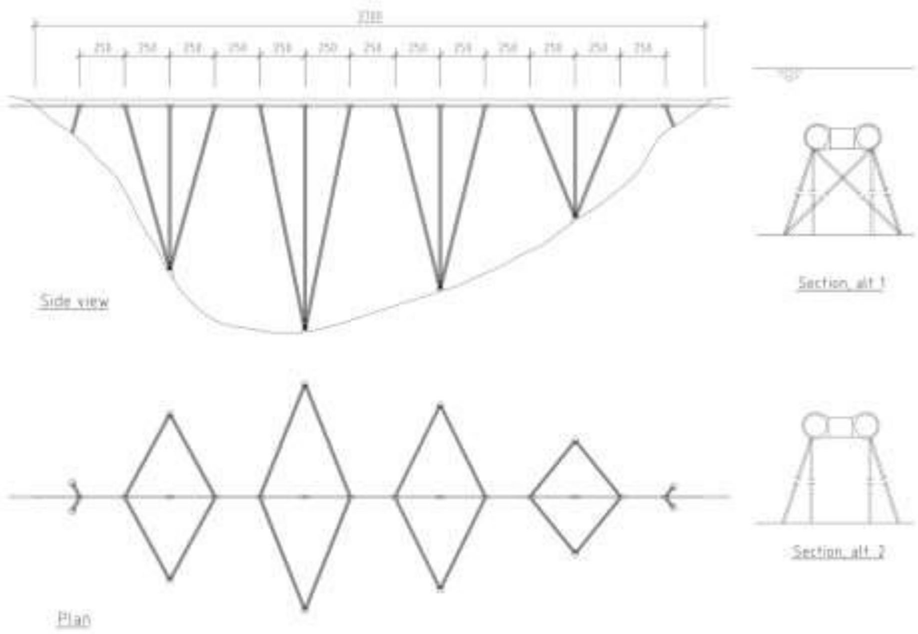



*Figure 5. Principle of SFT with tethers*

Furthermore, the stiffness of the anchor lines will vary significantly with the tension in the anchor lines. The pretension of the anchor lines must also be large enough to avoid potential instability problems known as Mathieu-instability due to variations in the stiffness.

The number of anchor lines in each anchoring point must be large enough to allow at least one anchor line to be out of action either due to failure or during replacement. Several anchor lines interfering with each other may give rise to galloping-like instability [5]. This will govern the center-distance between the anchor lines. When selecting this distance one should also make it large enough to minimize the likelihood that more lines would be destroyed in accidental cases such as a possible hit by submarines or when being hooked by a trawling gear.

In the Sognefjord case the foundation of the anchoring system was deemed to present a significant challenge. No information on the condition on the sea bottom was available. We assumed that the steepest parts of the bottom were more or less bare rock, and that the deeper parts consisted of sediments of clay. In between it was expected that the slopes were more or less rock-strewn. In areas with bare rocks it was conceived that the foundation could be made by rock tunnels driven from the surface and that anchoring chains could be mounted and fixed to the outer parts of the rock tunnel walls by concreting. In clay with very moderate slopes suction anchors as have been used in similar water depths for anchoring of offshore structures was deemed to be a well proven and feasible solution. In the rock-strewn parts however, no obvious way was found to establish a reliable foundation. It was thus concluded that the concept must be so flexible that all anchoring points could be located outside the rock-strewn slopes.

For the reasons described above and also due to the fact that the foundations were considered to be both complicated and expensive, it was proposed to join several anchors in one foundation and thus minimizing the number of foundations points, se Fig. 5. To be able to cope with the need to avoid foundations in the slopes with rock non-symmetric anchor systems may be the result.

## COMPARISON OF THE TWO CONCEPTS

In favour of the pontoon concept, the following could be listed:

- Eliminates uncertainties with respect to the subsea soil conditions
- No costly and risky foundations or anchoring points at the seabed in deep waters
- Less vulnerable to underwater land-slides
- Less vulnerable to collision by submarines
- No tethers that may be subjected to instability phenomena
- Less subjected to excitation by earthquakes

On the other hand the following points may be advocated in favour of the tether concept:

- No restrictions to ship traffic and represents no collision risk for ships
- No visible parts above water
- Easier installation of additional measures, such as tethers, to cope with possible unforeseen behaviour
- Eliminates slowly varying dynamic response
- Less excited by first order wave forces
- Less excited by wind and current
- Probably less complicated dynamic response including less torsional excitations
- Safer and less costly installation of tube since the tethers can be used as mooring systems during installation
- May have both straight and curved horizontal alignment

This then supports the conclusion from the Høgsfjord project cited at the beginning of this paper that the tether concept would generally be the preferred solution. Pontoons may be preferable in cases where the water depth is very large. Where the breaking point is in terms of water depth cannot be stated generally and would depend on many other parameters at the actual site, such as wave conditions, ship traffic, soil conditions, potential for underwater landslides etc.

## ALTERNATIVE TETHER CONCEPT

As an alternative to tethers to the seabed, the concept shown on Fig. 6 was proposed as a way to both eliminate the risk for ship collisions and the uncertainties related to the seabed conditions. The tube has here a net positive buoyancy and is kept in place by inverted suspension cables that are fixed to the rocky parts of the slopes close to the shore. The cables and the tubes are connected by hangers, and the concept works very much as an inverted suspension bridge. This concept was not verified any further than proposing the idea, which seems to be worthwhile pursuing for crossings with deep water, like Sognefjorden.

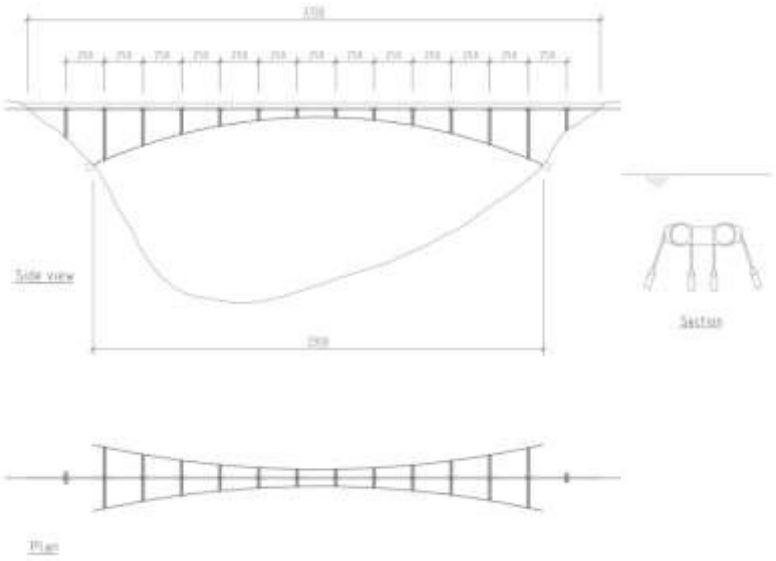


*Figure 6. Alternative tether concept*

## ACKNOWLEDGEMENTS

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# SUB SEA TUNNELS TO OIL FIELD DEVELOPMENTS IN NORTHERN NORWAY
## TBM-tunnelling at 300m water depth in sedimentary rock

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## ABSTRACT

*The oil industry suffered a severe set-back as result of the accident in the Gulf of Mexico in 2009, which showed clearly the public awareness and environmental aspects related to such accidents. A concept based on subsea tunnels and caverns has been developed for an offshore oil field in Northern Norway as alternative to traditional development strategies for offshore oil fields. The concept includes 2 or 3 parallel tunnels descending from an onshore terminal facility to a base station at the low point. From the base station, parallel tunnels will be bored by TBMs to the location of large production caverns that need to be established for drilling and operation facilities. The concept has been found to be feasible and viable compared to traditional development concepts, and the associated cost and time schedules to be competitive for fields in the range of 30 km from the shore. Several challenges need to be tackled for a robust solution, but these are all considered being within the range of technology development. The concept reduces the environmental risk to a minimum, and improves the utilization of the oil field compared with traditional solutions.*

*The concept will however require further development of tunnelling methods and particularly TBM tunnelling with 300 m hydrostatic water head in difficult ground conditions, as the major challenge. This will require development by the TBM suppliers in terms of robust probing and pre-grouting facilities on the TBM for being able to investigate ahead of the tunnel face and to do a satisfactory pre-grouting to stop potential high pressure water inflow. Other issues that need to be further developed are the great demands on productivity and completion time of a field development project like this. Scheduling will require the TBMs to operate at high progress rates, around the clock and for several years, and with several machines at the same time. Thus, there is a requirement on robust and stable machines that need to be fulfilled. A project like this will offer suppliers possibilities for equipment development and breaking new frontiers on the technology level. This article presents the evaluations conducted to investigate the feasibility of such tunnels in the prevailing ground conditions in Northern Norway including time- and costs estimates.*

## INTRODUCTION AND BACKGROUND

Internationally, subsea tunnels are most commonly used for roads and railways and to some extent for pipelines. Recent projects in Norway are the Oslofjord and Bømlafjord tunnels from

2000, with lengths of 7.2 and 7.9 km respectively, and the Eiksund tunnel from 2007 with a length of 7.8 km. The latter is so far the deepest of these tunnels, with a maximum depth below sea of 287 m. The last to be completed was the Atlanterhavs tunnel in 2009 with a length of approx. 7.5km. Plans for the future in Norway include both longer and deeper tunnels, i.e. the Rogfast tunnel with a length of 24 km at depth down to about 400 m below sea level is currently in the planning process. The Hareid - Sula tunnel is a potential future subsea tunnel intended to reach 630 m depth with a length of 17 km. The excavation method for all these projects has been D&B, although TBM has been evaluated as alternative for some, and is now being evaluated also for the Rogfast tunnel. Internationally, the Channel Tunnel with a total length of 51 km was completed in 1994, and the Seikan Tunnel in Japan with a total length of 53.0 km was completed in 1988. Subsea tunnels for road and rail will normally be excavated from both sides, thereby doubling the excavation speed compared to tunnels for petroleum purposes. Other subsea tunnelling projects such as the Øresund Belt, the crossing of Bosporus and others employ the TBM technology for tunnelling, but at shallow water depth and with shallow overburden. These are concepts being excavated according to a significant different approach than those in Norway, and are not comparable.

In 1978, initial studies on tunnelling for oil and gas production were started in Norway by a group of consultants and contractors. The group selected an area offshore Troms in Northern Norway as the basis for their concept study. Due to the rich fishing grounds, the Troms area was at that time the most controversial area in Norway for oil and gas exploitation. In 1984, pre-feasibility studies were performed for Statoil by the Petromine Company on the use of tunnels to the Troll field, about 48 km offshore (Sørheim, 1985). In 1986 the Petromine Company further developed technology to drill and produce oil and gas from tunnels (Holestøl & Palmstrøm, 1987).

Another concept that was developed at almost the same time and for the same purpose was the project named "Troll i fjell". The companies behind this concept study were the general contractor Astrup Høyer AS and the consulting company Grøner which formed the HAG Group. "Troll" is the name of a gas field located in the North Sea some 65km west of the shore approach at the coast close to Bergen. The concept included elements very much similar to this with 3 parallel tunnels leading to the shore approach and a system of tunnels branching off to production tunnels. (Herre, 1985)

Tunnels have, however, never been used for this purpose neither in Norway, nor in any other country as far as we know. Tunnels have had a more conventional use in the oil and gas industry, as landfall tunnels for oil and gas pipelines. Previous plans of subsea tunnels for petroleum development have been suffering two major showstoppers:

- Long construction schedule, resulting in higher project costs than for conventional solutions.
- Safety aspects, escape and evacuation in addition to gas explosions and blowout handling, considered to be the major challenges.

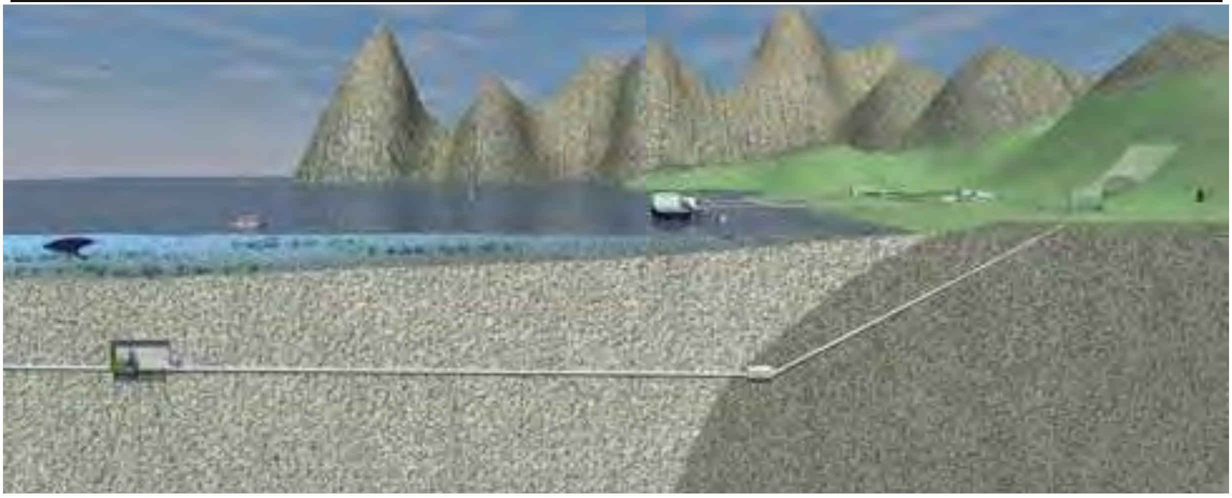

*Figure 1. Acona concept with subsea tunnels and directional drilling from caverns*

Since the early 1980s there has been considerable development in underground excavation technology and risk handling. In a recent study carried out by Acona Wellpro (a Norwegian consulting company within petroleum and risk management) in co-operation with NTNU (the Norwegian University of Science and Technology) and SINTEF (research institute located at NTNU) the potential of utilizing subsea tunnels today for the Norwegian Continental Shelf has been found very promising (Vassmyr et.al, 2011). In this study it is concluded that there are no immediate showstoppers related to the Acona Subsea Tunnel Concept, and that the concept of subsea tunnels, combined with directional drilled production wells could be a realistic alternative for future petroleum activities for coastal areas, e.g. 25 - 50 km from shore.

## ENGINEERING GEOLOGICAL CONDITIONS FOR THE CASE AREA

As base case, a prospective oilfield located in the so called Lofoten-Vesterålen area in Northern Norway has been chosen. An approximately 27 km long potential tunnel off the coast of Northern Norway as sketched in Figure 2 would be required to reach the actual location. Such a tunnel will start and have its first 7 - 9 km in Precambrian, crystalline basement rocks such as gneiss and gabbro, representing the main onshore bedrock of Norway. The outer 18 - 20 km will be in Cretaceous bedrock.

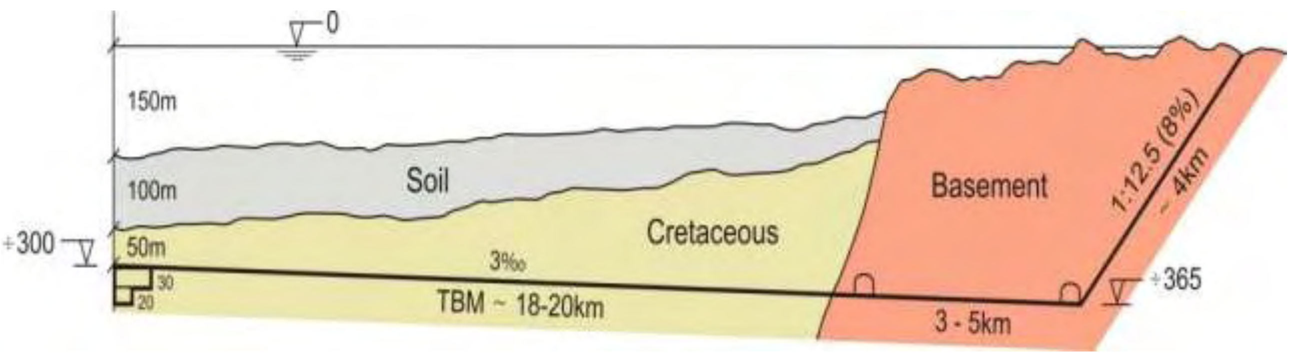


*Figure 2. Schematic sketch of "Base case" for the study.*

The crystalline bedrock onshore has high to very high rock strength (UCS 100 - 200 MPa, even up to 300 MPa and more in some places). Tunnel excavation in such bedrock is normally routine procedure, although fault zones, particularly under water, may represent challenges that need to be appropriately handled. The remaining around 18 - 20km of the tunnel is to be excavated in moderately to poorly consolidated shale, mudstone, siltstone and sandstone mainly of late Cretaceous age (65 - 99 my). These rocks are estimated to have UCS values less than 60 MPa. Based on shallow seismic profiling and drilling it has been found that the Cretaceous rock quality at tunnel level vary considerably, ranging between very schistose and heavily fractured claystone, see Figure 3 and less schistose and less fractured, marly siltstone.

As shown in Figure 2 the sea depth of the base case is up to 150 m, and the thickness of Quaternary sediments is about 100 m. With a rock cover at the outer end of 50 m and with access to the caverns at the top, the outer end of the tunnel will be at level -300 m. As a reference, the current recommendation concerning minimum rock cover for Norwegian subsea road tunnels is 50 m. With the first part of the tunnel system in crystalline bedrock at gradient 1:12.5, and the major part of the tunnels at gradient 0.3% from the base station as sketched in Figure 2, the lowest level of the system will be at approx. -365 m. This is about 70 m deeper than the deepest Norwegian subsea tunnel excavated so far (Eiksund, -287 m) and slightly less than for the planned Rogfast tunnel (approx. -400 m).

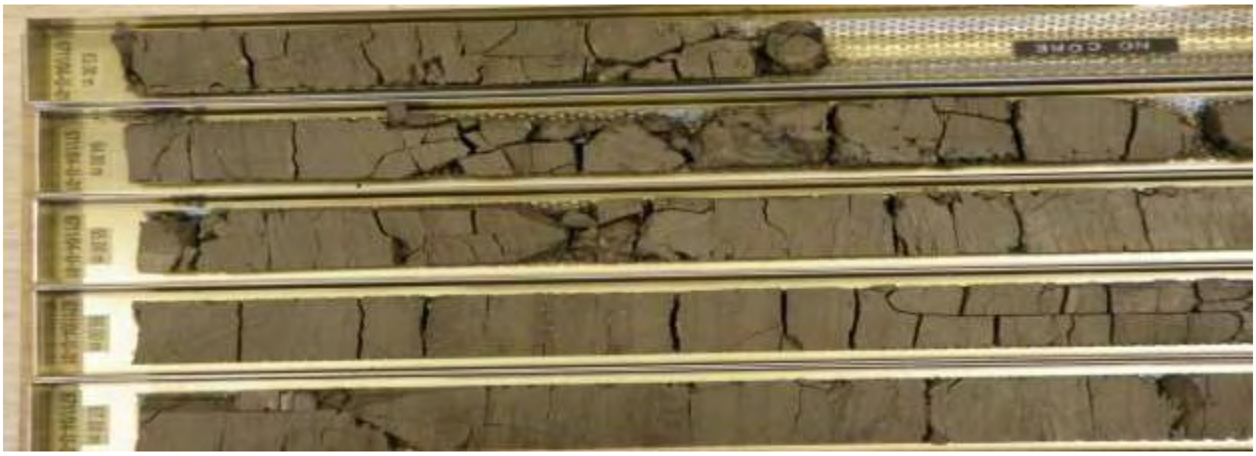

*Figure 3. Drill cores illustrating Cretaceous rock mass quality of base case.*

Ridge push related to the Mid-Atlantic ridge governs to a great extent the regional rock stress situation offshore Norway (Fejerskov, 1993), and the largest horizontal stress (representing major principal stress) offshore is oriented mainly NW-SE; i.e. almost perpendicular to the coast line and parallel with the tunnel axis. The tunnel will most likely have to intersect several distinct faults in the Cretaceous layers. Major faulting has also to be expected at the boundary between old, crystalline rocks close to the shore and the much younger offshore rocks. Based on experience from completed shore approach tunnels further south (pipeline tunnels for Oseberg and Troll), major faults are also likely to be encountered in the crystalline rocks before reaching the young, sedimentary bedrock. The risk of shallow gas influx is to be taken into account particularly in sandstone layers and at faults crossing the tunnel alignment.

**CONCEPT LAYOUT**

Due to escape and evacuation requirements, as well as ventilation and efficiency of construction, the concept is designed with a minimum of two parallel tunnels; one transportation tunnel for typical container size loads, and one escape/evacuation tunnel which is also used for transport of personnel. For both tunnels it is suggested using rail transportation from the base station to the production caverns. All transport of hydrocarbons and utilities will be undertaken in the transport tunnel, while redundant power supply and independent ventilation for the escape system will be installed for the escape/evacuation tunnel. An alternative solution is to construct 3 tunnels as shown in Figure 4. The third tunnel is then used as a relief tunnel for any gas emissions in abnormal situations. By being able to fully isolate one cavern, any gas from leaks or blowouts could be directed to the relief tunnel. Thus, the transportation tunnel would be open for normalization of an incident or for utilizing another cavern for relief well drilling. The 3-tunnels concept has higher cost, but on most other aspects is superior to a 2-tunnels solution. One main advantage of the 3-tunnels solution is that giving preference to the excavation of e.g. the evacuation tunnel, it will provide an early access to the caverns, enabling excavation of the caverns to start as early as possible. The evacuation tunnel may act as a pilot tunnel and also be used for pre-grouting of the rock mass next to the transport tunnels. Further, the 3-tunnels solution provides the availability of three working faces which will give significant flexibility. The concept is less sensitive to possible incidents that could hamper the progress and advance of the tunnelling work. If the advance is slowed down in one tunnel, there are still two more tunnelling faces to work on. A 3-tunnels solution provides a better situation as far as safety is concerned during construction and operation. The 3-tunnels solution separates the different functionalities of the facility and different activities can be allocated to each of the tunnels. By choosing the same cross sections for all tunnels in a 3-tunnels solution, there will be three TBMs which are identical with respect to size, spare parts, maintenance etc.

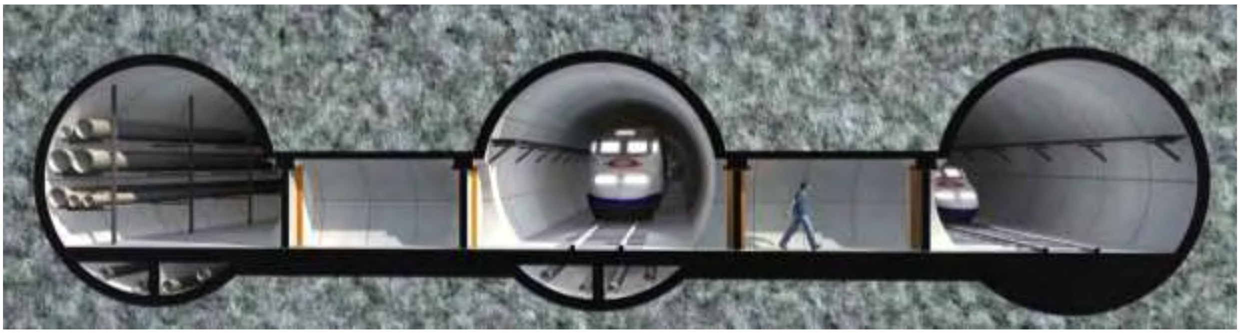

*Figure 4. Option with 3 tunnels (outer diameters: 6.6 m).*

## EXCAVATION AND EQUIPMENT SPECIFICATIONS

### Tunnels onshore by D&B

As shown in Figure 1, the first 7 - 9 km of the project discussed in this paper is intended to be excavated in crystalline bedrock; whilst the remaining 18 - 20 km will be in Cretaceous rocks. Drill and blast excavation is the method planned to be employed for the crystalline bedrock. In Norway, several thousands of kilometres of tunnels have been successfully excavated by D&B in similar rock conditions.  Norwegian subsea tunnels so far have all been excavated based on this method. Excavation for one of the subsea road tunnels is illustrated in Figure 5.

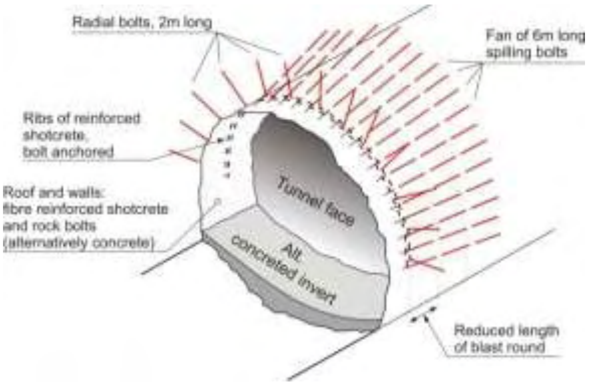


*Figure 5. Drill and blast excavation (left side) in the Frøya subsea road tunnel, combination of rock bolts, shotcrete, invert lining and reinforced ribs for rock support (right side).*

Thus there is a vast basis of experience from D&B excavation in crystalline rocks in Norway. Weekly advance rate for D&B of up to 175 m (Sauda and Kjøsnesfjord) has recently been achieved in good quality rock, and at Spitsbergen (Svea Mine), weekly advance rate of up to 150 m was achieved in Calcareous schist and sandstone oriented with an unfavourable angle relatively to the tunnel axis. Excavation by D&B is considered to be the best alternative also for crossing the major fault between crystalline and Cretaceous rocks (see Figure 2). D&B excavation provides a good overview of the conditions at the tunnel face and great flexibility in applying various types of equipment at the tunnel face, which significantly improves the basis for decision and performance of rock support and grouting ahead of the tunnel face.

Concrete lining in the crystalline basement is intended to be installed only in limited sections where weakness zones/faulted rock intersect the tunnel. As an alternative to concrete lining, a combination of steel fibre shotcrete, rock bolts and reinforced shotcrete ribs as shown in Figure 6 are planned to be used.

**TBM tunnelling offshore**

After having entered the Cretaceous bedrock, TBM excavation has been judged as being the most viable tunnelling method since it is much faster in weak rock than D&B. An additional factor is related to the requirement for ventilation which is much less for a TBM. D&B tunnelling would require an extensive ventilation system to clear the tunnel for gases caused by the blasting as well as exhaust gases from loading and transpoprt. It would also cause long stops in the tunnelling process to allow evacuation of harmful gases. Thus, in this very long tunnel, TBM has been applied in the base case evaluation for excavation beyond the boundary between crystalline and Cretaceous rocks. In some cases TBMs have also been used for excavating tunnels to offshore coal fields. One such case is Cape Breton at Nova Scotia, where a 7.6 m diameter tunnel was successfully excavated in sedimentary rocks including sections of gassy ground at a depth of 200 m below seabed, 3.5 km offshore (Yuen et.al., 1987).

Thus, TBMs have previously been used for excavation of subsea tunnels longer than those discussed in this report, and in rock mass conditions of similar type. This in fact supports the idea that TBM excavation of the outer 20 km is feasible. Although, tunnelling in the complex conditions of the Cretaceous bedrock around 300 m below sea level has to be regarded as a challenging task. As discussed above, shielded TBMs would be required specifically designed for the rock mass conditions of this particular project. Contingencies need to be established to handle unforeseen ground. The main challenges are expected to be:

1) Very weak and unstable rock mass, including running ground, swelling rocks etc.
2) Stress related problems (squeezing etc.)
3) Large water ingress/high pressure
4) Gas pockets/shallow gas
5) Mixed face conditions

Due to the difficult ground conditions that are expected to be encountered in combination with the great depth of tunnelling and the strict time schedules that tunnelling needs to fulfil, the technical specifications for the TBMs to be employed will be comprehensive. As per today, TBMs may not have been applied in similar situations and with such specifications yet, so this project is likely to push the state of the art of TBM tunnelling forward.  To cope with the difficult and variable rock mass conditions, including different varieties of shale and sandstone, squeezing and swelling conditions as well as possibly gassy ground, we do not think that the type of machine to be applied is a standard design at present, although, double shielded TBMs similar to the one shown in Figure 6 are believed to be the most realistic equipment. It is expected that the TBMs to be deployed for a challenging and unique project like this will be designed specifically to cope with the ground conditions that are found based on thorough site investigations. The viability of the project is actually relying on the fact that TBM tunnelling takes place with high advance rates for 3 machines simultaneously over a significant period of time and through challenging ground conditions. The possibility of operating the TBMs in a kind of EPB mode to be able to close the face in case of running ground should appear, may also be specified. In such cases it would need to operate in closed mode against approx. 300 m of water head, representing quite a substantial improvement compared to the current status.

*Figure 6. Double shielded Robbins TBM with the possibility of probing ahead of tunnel face and pregrouting*

Until the mid-1990s, approximately 260 km of tunnels (mainly hydropower tunnels) were excavated by TBMs in Norway. This was mainly in typical hard rock, by so-called Main Beam machines, but internationally TBMs are most commonly used in soft rock and soil. TBMs have been used in several subsea tunnels, such as the Channel tunnel between England and France (49.2 km long with 2 tubes each of diameter 8.5 m and one of diameter 5.7 m) and the Great Belt tunnel in Denmark (7.9 km long with two tubes of diameter 8.5 m). These tunnels are both located in Cretaceous rocks, although quite different from those of the base case discussed in this report (mainly chalk marl and chalk, respectively). In the Channel tunnel, the tunnelling conditions were very favourable, and world record performance was achieved (best week 428 m, best month 1719 m and monthly average 873 m).

The time scheduling of the project described here requires high utilisation and capacity by the TBMs to be competitive and viable for the oil industry. For the particular project that we have looked at in this case, the following parameters were defined for the tunnels: length= 22 000 m, TBM diameter = 6,6m, DRI = 65, CLI = 20, angle of rock mass planes of weakness = 0 degrees and spacing between planes of weakness = 7.5 cm. The results from the Fullprof software applying the NTNU Prognosis model are: Advance rate 283 m/week, and excavation cost excl. rock support 12 666 NOK/m. This is based on working 24/7 during 50 weeks per year. The figure which is of most interest here is the advance rate of 283 m/week. This is by far 'World record speed', however this is an advance rate which shall be undertaken throughout 22 000 m of tunnelling at 3 TBM faces, and simultaneously. To allow this project being viable, it is strictly required that these machines are robust and able to operate at this speed throughout the entire construction phase.

Detailed investigation of rock mass conditions for tunnelling has not yet been carried out for the base case, and details concerning the factors listed above are therefore not fully known. Cores as shown in Figure 3 indicate that the Cretaceous rock mass is mainly quite schistose, intensely jointed and partly of very poor quality. Further investigations of the rock mass quality, and the extent of running, swelling and squeezing ground, as well as presence of fault zones etc. are required. One main challenge for the excavation of deep tunnels under the ocean like in this case, is the potential of large water ingress with high pressure. Tunnelling may become particularly difficult when high pressure water ingress is encountered in faults and poorly consolidated ground. Severe instability during excavation has however been dealt with in tunnels before, including subsea and TBM tunnels. For instance, in the subsea Atlanterhavs tunnel which was opened for traffic in December 2009, large water ingress (up to 500 l/min in one single probe hole) occurred in a major fault zone with soil-like conditions. This incident was successfully handled and the inflow sealed 250 m below sea level. In all subsea tunnelling, systematic probe drilling ahead of the tunnel face using percussive drilling and/or core drilling when needed is one important contingency to secure safe tunnelling. Continuous and systematic probe drilling of several holes from the tunnel face will be required as illustrated in Figure 7. This has been standard procedure in all subsea tunnels in Norway and represents proven technology. For very high water pressures in soil like conditions, the tunnel excavation has to be protected against water and debris inflow, and considerations need to be taken to install "blow-out" preventer at the drilling equipment.

The need for water sealing has to be based on the results from probe drilling, and is carried out when the inflow of the probe drill hole exceeds a pre-set limit. Sealing is preferably carried out as pre-grouting with cement and high grout pressure (up to 10 MPa). The procedure, as shown in Figure 7, represents proven technology, also for TBM tunnels. Most likely, pre-grouting will be needed for a considerable part of the tunnel, also in crystalline bedrock.

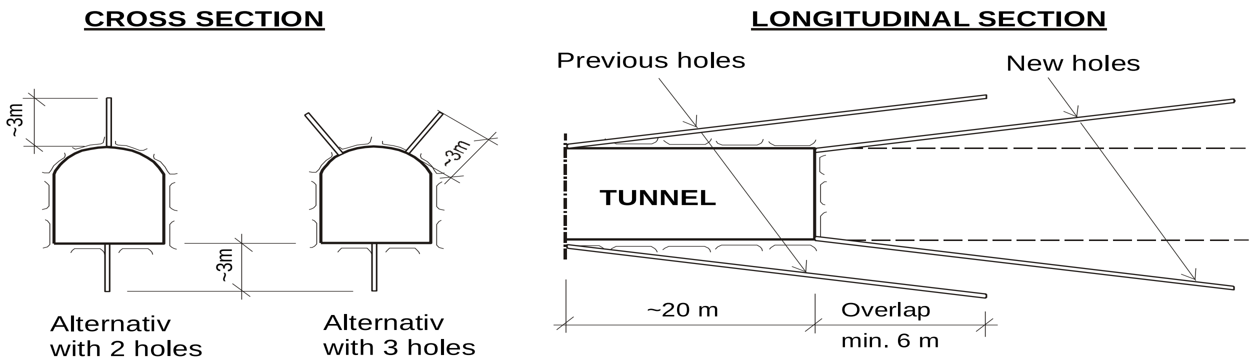


*Figure 7. Principle sketch of probe drilling and pre-grouting ahead of the tunnel face.*

The tunnels act as drained structures and relevant stop criteria for grouting will be established based on the acceptable water inflow level to the tunnels. An adequate pumping and water discharge system therefore needs to be designed and installed. Consequently, any TBM to be involved in a project like this would have to be equipped with drilling units that can secure probing ahead of the tunnel face, and pre-grouting if needed. The probe holes may need to be drilled up to 30 - 35 m ahead of the tunnel face, angled slightly out from the tunnel axis. Therefore, a well designed set-up of drilling units needs to be developed. To reduce the loss of time, probe-drilling simultaneously with advancing the TBM will be required. For appropriate pre-grouting the possibility of drilling through the cutterhead to enable grouting of the rock volume of the coming tunnel cross section would also be required.

Regarding rock support, a combination of fibre reinforced shotcrete and rock bolting is considered being sufficient support in long sections of the tunnel in the crystalline basement. Experience from subsea tunnelling in Norway demonstrates that, if the thickness of the sprayed concrete is above a minimum of 60 - 70 mm, and high quality concrete is used (C45), degradation of the shotcrete is slow. In the sedimentary rocks of the outer parts of the tunnels, continuous lining installed as pre-fabricated segments is expected to be required for the complete tunnel length. The harsh environment with saline water requires rock bolts to be fully grouted and double corrosion protected, and design of the concrete mix for sprayed concrete, segments and structural purposes will take the difficult environment into account too. A well designed logistic system is crucial for achieving a high advance rate in TBM tunnelling. Particular focus must address the discharge of muck from the tunnel and supplying lining segments and cutters to the tunnel face. In critical situations, transport of support measures and grout needs to be prioritized to stabilize the tunnel at the tunnel face.

*Excavation of caverns* in the base case; a minimum of two caverns with a height of approx. 30 m (drainage sump not included) are to be excavated at the end of the TBM tunnels. Cavern span (width) and length will be approx. 15 m and 75 m, respectively. Larger caverns than those being planned have previously been successfully excavated in a wide range of ground conditions. The planned dimensions may, however, still represent a considerable challenge in weak, sedimentary rocks as expected in this case. The main engineering challenges and risk elements associated with excavation of the offshore caverns are: a) Major faults, b) Very weak/unstable rock mass, running ground, swelling, squeezing etc., c) Large water ingress/high water pressure, d) Methane/explosion risk and e) Cuttability of rock mass.

The best alternative for excavation of the caverns is considered to be roadheaders as shown in Figure 8. Excavation with roadheader is in most cases slower than drilling and blasting, but may reach in the range of 30 $m^3$/hour even in rock types with UCS up to 100 MPa (Copur et.al., 1998). Roadheaders have the advantage of great flexibility, and several units may be used simultaneously to achieve satisfactory excavation rate. It also has the advantage that ventilation requirement is less extensive than for D&B. In any case, excavation of 30 m high caverns has to be carried out in a stepwise manner with a top heading followed by 2 - 3 benches. Methane is a potential problem here too, but can be dealt with by appropriate monitoring and ventilation.


*Figure 8. Roadheader for mechanical excavation (example: Voest Alpine).*

## CONCLUSIONS

Cost and time estimates for the base case discussed in this paper have been made based on the extensive experience from Norwegian tunnelling. Special focus has been on the subsea tunnels that have been completed in Norway. For the base case discussed in this paper, the total cost for excavating the tunnels and caverns has been estimated to 8,750 MNOK (approx. USD 1,500 mill.), and the construction time to about 210 weeks. This does not include time for geo-investigations. Based on detailed analysis it is concluded that the base case discussed in this paper is feasible to construct. No significant showstoppers have been identified. There are however several factors that will require close attention, such as:

- Potential large water inflow.
- Potential adverse behaviour of young sedimentary rocks (stability, squeezing, swelling etc.).
- The risk of encountering shallow gas during investigation and excavation.
- Detection of potential gas leakage during operation.
- Ensuring high state of readiness in case of potential blowout, fire or explosion.

Regarding construction cost and operation economy, the tunnel concept for the base case has been found to be competitive with conventional alternatives for oil field development. Based on these results for the project in Norway, it is believed that the tunnel concept may be interesting also for other parts of the world where the possibility of petroleum development from the sea floor is restricted due to environmental or climatic reasons.

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# TUNNELING ROGFAST WITH TBM AT 390 M BELOW SEA LEVEL

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## ABSTRACT AND INTRODUCTION

*The Rogfast tunnel, 25.5 km long and a depth of 390 m will be the deepest subsea tunnel worldwide. Whether the twin tube subsea tunnel will be built with drill&blast or by the use of TBM has still to be decided, its length and depth exceeds the current experience base in Norway and internationally.*

*The detailed geology is uncertain at present, but investigations carried out so far, including refraction seismic profiles and core drilling, indicate that the most difficult stretch of the tunnelling to overcome in terms of geology will be East of the island Kvitsøy and along the northern half of the alignment between Kvitsøy and Arsvågen. The ground investigations have indicated numerous weakness zones in this section of the tunnel. Investigations so far do not indicate weakness zones to the same extent in the southern half of the alignment between Harestad and Kvitsøy, although weakness zones are also identified here, particularly a pronounced fault between Randaberg and Alstein. An exit tunnel to Kvitsøy branches off midway from the main tunnels and tunnelling of this part is expected to be relatively straight forward with respect to the geological conditions to be met.*

*An advantage of TBM tunnelling is considered to be the industrial working method and the possibility of achieving high progress rates also in poor rock mass. For a long tunnel with many weakness zones, as is expected for the Rogfast tunnel, the construction time constitute a significant factor for the choice of method. The length of the Rogfast tunnel motivates the larger investment that TBM poses. Another significant factor in the choice of construction method will be the confidence in the method and the risk related to the method in terms of progress and cost.*

*TBM tunnelling would likely result in a reduction of the risks during construction compared to its alternative of drill&blast. This is to a large extent due to that in adverse ground conditions a TBM of this size provides increased precaution with respect to dealing with water ingress and instability of poor rock mass quality. A TBM that can work in closed mode with counter pressure against the tunnel face provides also an extra measure to deal with very poor rock mass quality. The type of TBM chosen will never release the need of rock mass grouting in the same way and content as would have been done if the tunnel was excavated using drill&blast. This article presents the evaluation of the geological conditions of the Rogfast subsea tunnel and the pros and cons related to TBM and drill&blast as two alternative construction methods in the expected*

*geological circumstances. It is based on an initial study of the use of TBM on the project carried out for the Norwegian Road Authorities. [1] (COWI 2012)*

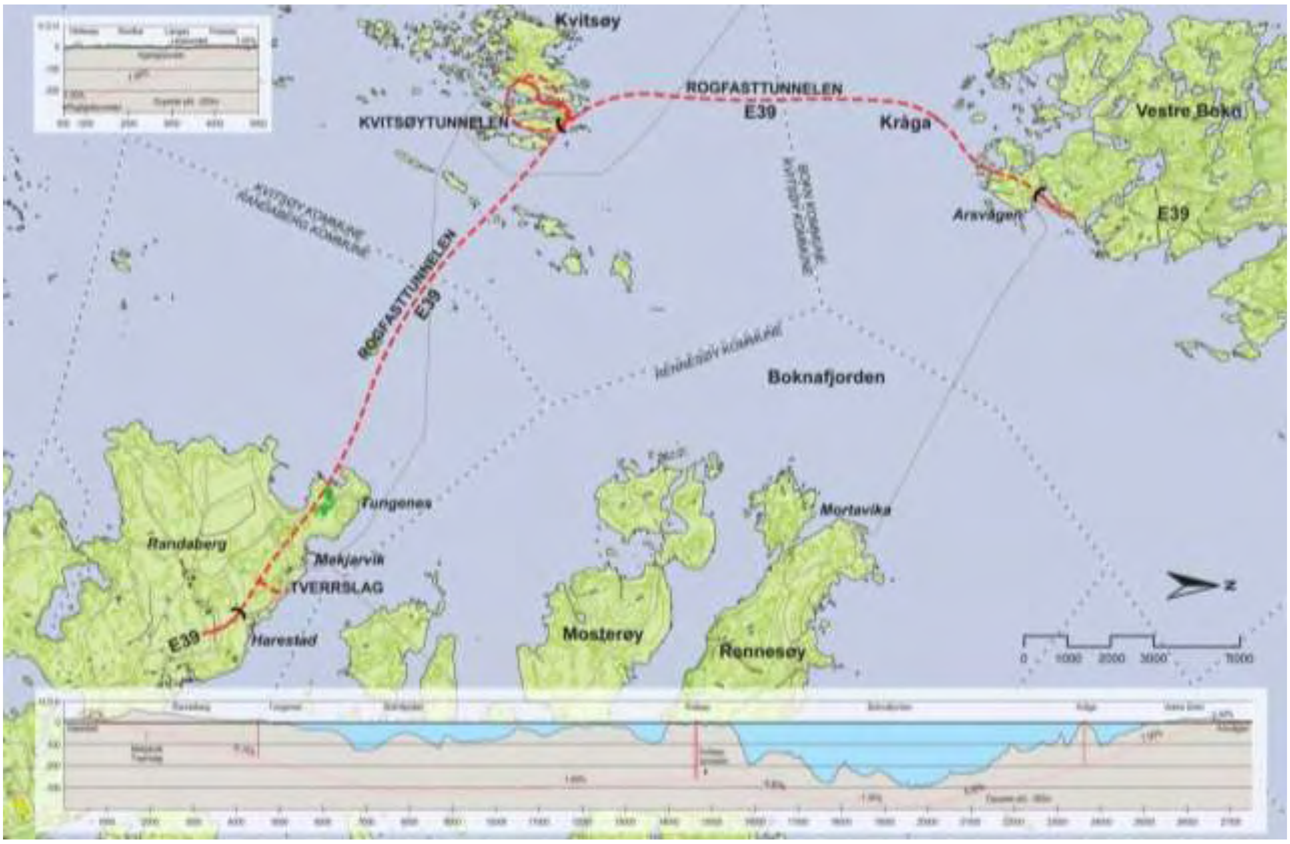


*Figure 1 Rogfast, plan and longitudinal profile.*

## GEOLOGICAL CONDITIONS

The regional geology in the vicinity of the Rogfast tunnel consists of a Precambrian autochthonous basement, observed at the northern tunnel entrance of the Rogfast tunnel. The basement has several overlying thrust sheets and nappes. Table 1 shows the major tectono-stratigraphic units expected to be encountered along the tunnel, and Figure 2 compiles geological map.

*Table 1. Tectono-stratigraphic units in the vicinity of the Rogfast tunnel [3] (NGU, 2011)*

| | | |
|---|---|---|
| Hardangerfjord Nappe | Torvastad and Visnes groups | Middle to Upper Ordovician supracrustal cover of the Karmøy Ophiolite built of greenstones, pillow- and breccia-lavas, tuffs, quartz-rich black shales, diabase dykes, etc. |
| | Karmøy Ophiolite | Lower Ordovician intrusive mafic and ultra-mafic rocks |
| Storheia and Boknafjorden nappes | | Proterozoic granitic gneisses, amphibolites and mica schists with some lenses of marble |
| Viste Thrust Sheet | | Mainly composed by the Cambrian – Ordovician quartz-rich Ryfylke Schists |
| Precambrian autochthonous basement | | Granitic to dioritic gneissic basement, with some bodies of gabbro and covered by thin Cambro-Silurian phyllites. |

As shown on Figure 2 the geological conditions along the tunnel alignment trace varies. At Randaberg, close to the southern entrance of the tunnel, the dominating rock type is Ryfylke Schist, consisting of phyllite and mica schist. Gabbro (Karmøy Ophiolite) was found in connection to core drilling at Alstein, an island between Randaberg and Kvitsøy. At Kvitsøy the dominating rock types are greenstone and green schist from the Torvastad and Visnes groups. At Vestre Bokn granitic and dioritic gneisses (Precambrian) are observed at Vestre Bokn, as well as phyllite (Ryfylke Schist). Further Figure 2 shows that there is still uncertainty when it comes to the rock type in a rather large area between Kvitsøy and Vestre Bokn. This causes uncertainty in the assumptions regarding the distribution of rock types along the tunnel alignment.

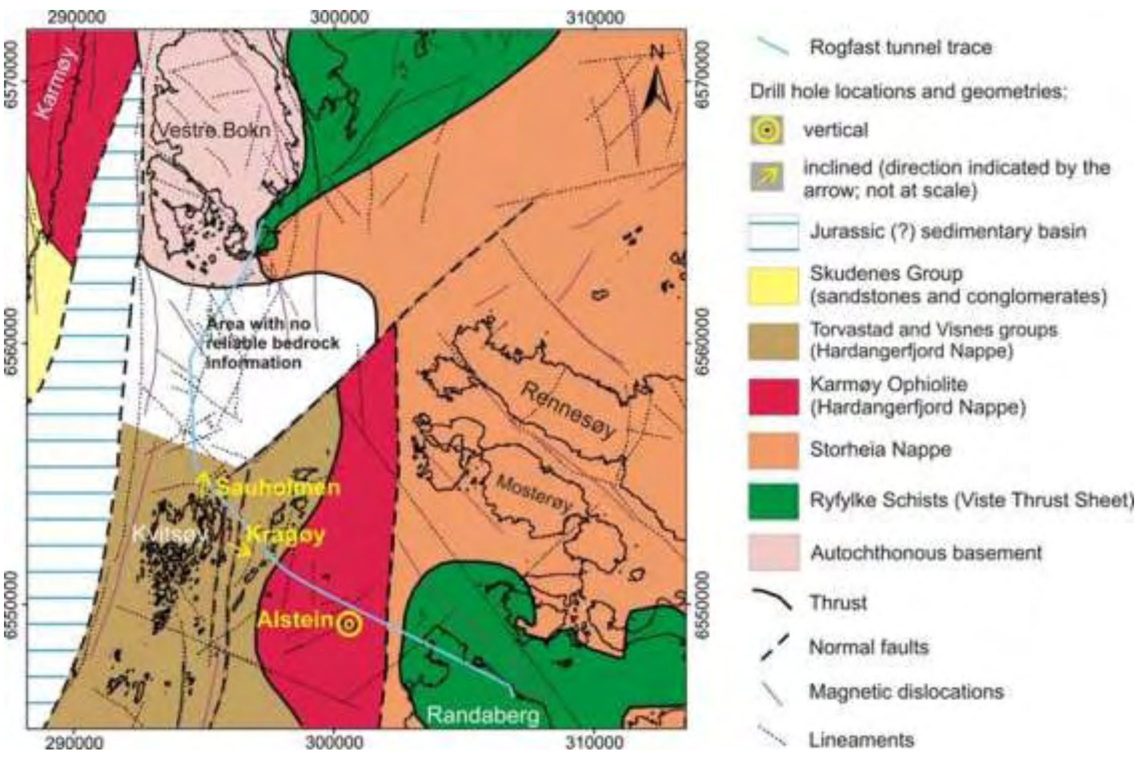


*Figure 2. Geological map of the region [3] (NGU 2011). Tunnel alignment shown as blue line*

The transition between two nappes are often an approximately even rock type boundary, and it is expected that the rock mass in the transition zone is heavily jointed compared to the fracturing in the surrounding rock mass. Aligning the tunnel in such a transition zone, or intercepting a transition zone several times is expected to be associated with significant support works. Table 2 shows the estimated distribution of rock types along the Rogfast tunnel alignment. It is emphasized that there is a great uncertainty connected to this estimate. The uncertainty is associated with; 1) distribution of rock types and 2) locations of transitions between the sheet and nappes along the tunnel alignment. The location of several weakness zones and fault zones are relatively well known from the refraction seismic investigations carried out along the alignment.

*Table 2. Estimated distribution of rock types in length (m) and percentages*

| Profile from - | To | Length (m) | Rock type I | Rock type II | Rock type III | Rock type IV | Rock type V |
|---|---|---|---|---|---|---|---|
| 1 110 | 6 500 | 5 390 | 5 390 | | | | |
| 6 500 | 7 000 | 500 | | | | 500 | |
| 7 000 | 14 000 | 7 000 | | 7 000 | | | |
| 14 000 | 18 500 | 4 500 | | | 4 500 | | |
| 18 500 | 24 500 | 6 000 | | 6 000 | | | |
| 24 500 | 25 000 | 500 | | | | 500 | |
| 25 000 | 25 500 | 500 | 500 | | | | |
| 25 500 | 26 530 | 1 030 | | | | | 1 030 |
| | SUM (m) | 25 420 | 5 890 | 13 000 | 4 500 | 1 000 | 1 030 |
| | | % of length | 23 | 51 | 18 | 4 | 4 |

Descriptions of the different rock types in table 2 are from NGU report 2011.034, where:

| | |
|---|---|
| Rock type I= | Ryfylke Schist – phyllites and mica schist, Cambrian – Ordovician quartz-rich schist |
| Rock type II= | Karmøy Ophiolite – gabbro, intrusive mafic and ultramafic rocks – Lower Ordovician |
| Rock type III= | Karmøy Ophiolite – greenstone, green schist, mica/chlorite shcist, tuff, breccia-lava, black shale and diabase dykes, Torvastad and Visnes groups – Middle to Upper Ordovician |
| Rock type IV= | Storheia and Boknafjnorden nappes – granitic gneisses, amphibolites and mica schists with some lenses of marble – Proterozoic |
| Rock type V= | Autochthonous basement - Granitic to dioritic gneissic basement, with some bodies of gabbro and covered by thin phyllites – Precambrian |

## ROCK MASS CLASSIFICATION

Results from refraction seismic carried out is used to estimate Q-values. Table 3 shows the distribution of Q-values covered by seismic profiles (16 340 m) and for the entire tunnel. The distribution is given both in meters and percent. It also shows correlation used to estimate Q-values.

*Table 3. Distribution of Q-values for areas covered by seismic profiles and for the entire tunnel, given in meters and percent*

| Seismic velocity (m/s) | Q-value | Length along tunnel trace (m) | | Share in % | |
|---|---|---|---|---|---|
| | | Section with seismic profiles | For the entire tunnel | Section with seismic profiles | For the entire tunnel |
| 5500-6000 | "10-40" | 3 950 | 5 020 | 24.2 | 19.7 |
| 5000-5500 | "4-10" | 8 430 | 14 740 | 51.6 | 58.0 |
| 4500-5000 | "1-4" | 1 530 | 3 000 | 9.4 | 11.8 |
| 4000-4500 | "0.4-1" | 880 | 880 | 5.4 | 3.5 |
| 4000-3600 | "0.1-0.4" | 440 | 670 | 2.7 | 2.6 |
| 3200-3600 | "0.04-0.1" | 290 | 290 | 1.8 | 1.1 |
| 2800-3200 | "0.01-0.04" | 400 | 400 | 2.4 | 1.6 |
| 2500-2800 | "0.004-0.01" | 270 | 270 | 1.7 | 1.1 |
| 2000-2500 | "0.001-0.004" | 150 | 150 | 0.9 | 0.6 |
| | | 16 340 | 25 420 | 100 | 100 |

## GEOLOGICAL CHALLENGES EXCAVATING THE ROGFAST TUNNEL

Crossing the Boknafjord with a subsea tunnel will bring along geological and hydrogeological challenges. Refraction seismic along the tunnel alignment and core drillings from Kvitsøy have revealed that tunnelling through very poor to extremely poor rock mass quality must be expected for approximately 10 % of the tunnel (Q-value<1). In addition, rapid changes of rock mass quality, which can provide challenges for the construction, must be expected. Poor rock mass quality can usually be handled, but if high water inflow also is encountered in the weakness zones it will be challenging. At the lowest parts of the tunnel alignment, the depth below sea level is 390 m, thus water pressure can reach 39 bar. A combination of high water inflow and very poor rock mass quality can give problems related to total stability of the tunnel. And in worst case collapse of the tunnel can occur. Geologically it is the highest risk to encounter water inflow in the distal part of fault zones. A systematical approach including probe drilling ahead of the tunnel face and pre-grouting must therefore be carried out prior to entering into the weakness zones. Additional investigations are planned and in summer 2013 directional core drilling (ca. 700 m) from Bokn will be carried out to investigate the geological conditions (rock type and quality) to ensure that the rock cover is sufficient. In addition, it is planned to carry out 4 core drilling holes from boreship between Kvitsøy and Bokn. The boreholes will be located in an attempt to drill through major weakness zones identified through the refraction seismic in the deepest part of the tunnel.

## TUNNELLING THE ROGFAST TUNNEL USING DRILL AND BLAST METHOD

For the Rogfast tunnel the initial plan has been to utilize drill&blast as the construction method. The pre-phase cross section design which is based on drill&blast shows two alternative sizes being as follows; 1) The base version has a width of T9.5 m including an ordinary 1 m shoulder width on both sides and 2) The alternative cross section based on risk and safety analysis has its right hand shoulder width increased to 2 m, equivalent to T10.5 m, see Figure 3 below.

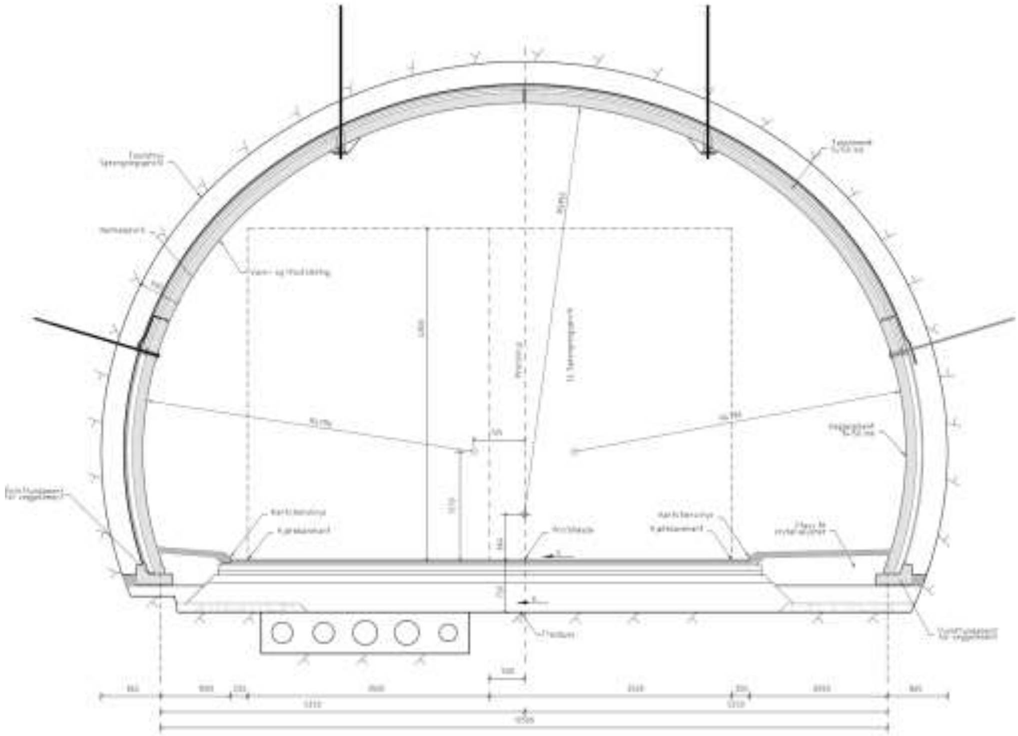

*Figure 3.* *Alternative cross section, with a 2 m wide right hand side shoulder, equivalent to a cross section of T10.5 m*

Tunnelling by use of drill&blast may engage as many as up to eight tunnel faces, each with separate drill jumbos and full set-up of equipment being used simultaneously. It will be possible to operate two drill jumbos from both Harestad and Arsvågen, all in the direction toward the junction at the exit tunnel to Kvitsøy. At the same time an option could be to establish a similar sized crew, equipment and resources to drive the tunnels from the junction in the direction of Arsvågen and Harestad. The latter requires the side tunnel being constructed initially from Kvitsøy to the junction at the main tunnel. The critical path for tunnelling works would therefore include the side tunnel from Kvitsøy and tunneling the 13.8 km long tunnel section from the junction towards Årsvågen.

## TUNNELLING THE ROGFAST TUNNEL USING TBM-METHOD

The TBM cross section is primarily presented to get an idea of the TBM-diameter that is found necessary for the Rogfast tunnel, and secondarily to compare the cross-sectional data at a base level. The final cross-section will be established when entering into the detailed design phase. Both a twin bored tunnel and a double deck tunnel alternative was investigated, but the double deck alternative was disregarded as the twin bored tunnel showed to be superior on a number of parameters, and thus being the most advantageous cross section for the project. The following essential requirements have been set forward to form the basis for drafting the twin TBM bored cross section:

- Base solution holds 2 traffic lanes and 1 m shoulders to reach width of T9.5 m.
- Alternative cross section is based on the requirement to extend the width of the right hand shoulder by 1 m to 2 m.
- Height of the traffic lanes is 4.8 m
- Space for road substructure of 0.75 m, but the final solution is a detail design task.
- Installations in all TBM options are assumed to be placed within the circular cross section.

The resulting cross section of the twin TBM bored tunnel is shown in Figure 3 below. The cross section layout is quite similar to the cross section of drill&blast tunnel. A TBM diameter of app. 12.4 m is anticipated for construction of a tunnel lining with an outer diameter of app. 12.2 m.

It is assumed that installation of concrete segmental lining is typically limited to weakness zones where the rock mass quality is too poor to be supported by traditional rock bolting and sprayed concrete. This means that in general the interior of the tunnel will be dominated by conventional water and frost protection, which for the Rogfast tunnel is anticipated to be self-standing precast concrete segments. Thus for the tunnel users, the tunnel walls and roof will appear quite uniform. To be able to optimize the cross section it was decided to apply the base design above.

Enlargement of the tunnel for other functions such as niches, cross passages and slow traffic lane need to be created following the passing of the TBM tunnelling. It must be noted that in case a segmental lining has been installed excavation of niches and slow traffic lane cannot be excavated without dismantling the lining itself. Thus, the location of such enlarged cross sections need to be well planned with regards to the expected ground conditions that would call for the need of segmental lining. During the tunnelling phase the situation may arise that segmental lining is required in areas not expected due to the occurrence of adverse ground which would then lead to an adjustment of the location of the niches and cross sectional enlargements.

In the market today there are several different varieties of TBM's which are suitable for particular project circumstances. For the Rogfast tunnel the TBM may have to be operated in either in open mode or closed mode pending on the varying ground conditions. In the following the most important geological challenges that need to be dealt with for the TBM are listed:

- Static water pressure up to a maximum of 390 m
- A weakness zone east of Kvitsøy with a total width of more than 60 m. Approximately 30 m of this weakness zone is expected to have Q-values below 0.003.
- Phyllite, greenstone and slate is experienced as favorable rock types for TBM, approx. 40%
- Grouting is expected  in rock types such as metagabbro and gneiss, corresponding to 40%
- Variation in rock mass quality may occur over short distances.
- A total of 83 zones of poor rock mass quality (Q <1) with a total length of 2660 m length constitutes approximately 10 % of the tunnel length.
- 42 of the 83 zones have estimated Q-value between 0.1 and 1, total length 1550 m

- 41 of the 83 zones have estimated Q-value below 0.1, total length 1110 m
- Swelling clay is expected in weakness zones (measured 0.28 MPa = moderately active clay).
- Parts of the tunnel may have low in-situ stress levels which can cause a risk of major leakage
- The tunnel will intersect regional faults where the distal part may be water-bearing

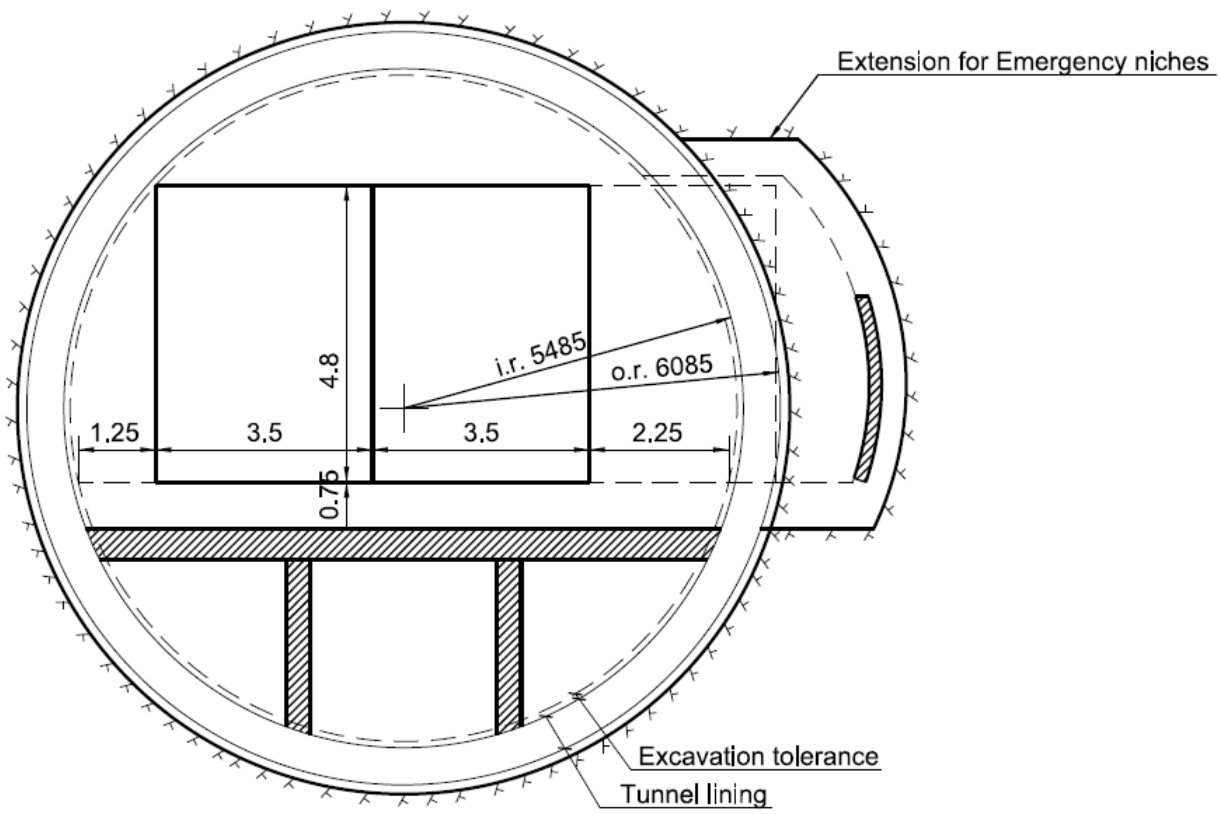


*Figure 4.* *Cross section for TBM*

This means that a TBM for the Rogfast project must be able to handle the rock mass quality for a variety of conditions and it should be a machine that is uniquely specified for this project. It is recognized that these features may not all be developed to a proven technology at present. The concept of TBM-tunnelling of the project and relevant specification will the following:

A. Probe drilling ahead of the tunnel face with different configurations of length, direction, etc. in relation to the expected rock mass quality, zones, etc. and for verification of the rock overburden. Probe holes at the tunnel face with length up to 50 m, and collaring within and outside the tunnel contour with 360 degree coverage. Probe holes upward for control of possible low rock cover or horizontal/ sub-horizontal horizons along the tunnel alignment (or below if this is necessary).
B. Core drilling in front of the tunnel face to reach a capacity up to 200 m in length.
C. Drill holes for grouting ahead of the tunnel face with length up to 25 m and the same coverage as specified for probe holes (see point A above). Perform rock mass grouting ahead of tunnel face with equipment that is able to work simultaneously with three parallel injection lines
D. An additional measure against water inflow at the face or instability is to park the TBM face in EPB mode (closed front) with back pressure of 15 bar, and to operate with in closed mode up to 10 bar. Such technology may be developed to handle higher water pressure heads in the

future. Cutter head to be sealed to reduce the possibility that poor rock or water flushes through the head and enters into the tunnel from the tunnel face.

E. Switch quickly and efficiently between construction with and without concrete segments. In difficult ground conditions with high water pressure and/or poor rock mass quality the TBM can advance with the installation of concrete segmental lining. Alternatively, in some cases, heavy support can be applied behind the TBM shield applying bolts and sprayed concrete after having performed comprehensive grouting at the tunnel face.
F. In ordinary rock as phyllite and slate the experience with TBM operation is beneficial as the machine is expected to run without concrete segmental lining and thus virtually acting as in 'Open-beam' mode. Behind the TBM shield the rock mass will be supported using conventional methods such as rock bolts and sprayed concrete.
G. As a TBM is a large-scale survey hole and to allow the geologists to interpret the geological conditions based on the collected data and parameters it is important to store all such data as including trust, power consumption, RPM and gross progress in 'real time'.
H. The tunnel face shall be drained to keep water pressure acting at the tunnel below predefined values. Adequate pumping capacity at the TBM to be installed to enable drainage of the tunnel. Cutters to be replaceable behind the cutter head such that it is not necessary to move the cutter head to change the cutters.
I. An air lock to allow manual interventions under compressed air into the cutterhead e.g. for cutter replacement etc. in case this is urgently required in a zone of adverse ground.

Based on the assessment that has been made, it concludes that the type of machine best fit would be a dual-shield machine, equipped with EPB capacities, i.e. a so-called "dual-mode TBM". This type of machine can be operated with and without segmental lining to fit the actual ground conditions as they appear. Furthermore, it is possible to close the face as an EPB-TBM according to the demanded water pressure given above. It is also required that the machine can be converted from the application of conveyor belt in open mode to 'screw-conveyor' in closed mode. This is meant to be a redundancy in case a satisfactory injection at the tunnel face is not achieved. The main principle is that water control is obtained through rock mass grouting as the first line of defense.

Such a machine would be able to ensure that the tunnel construction takes place with a great deal of flexibility to handle different types of rock conditions and holds necessary capacity to conduct a stable, safe and acceptable tunnelling progress.

TBM tunnelling using a machine that itself has merely no-flexibility with respect to allowing other equipment to get to the face constitutes the most negative factor with the method. This, however, can be handled by specifying that the TBM holds equipment to allow different types of activities to be carried out especially for probe drilling and grouting. To prevent instability at the tunnel face the possibility to pressurize the tunnel face, i.e. EPB capacity is required.

A double shield TBM has the possibility of placing concrete segmental lining to provide an additional safety when negotiating weakness zones and difficult ground for a successful implementation. These arguments are the basis for the proposal of a "dual-mode TBM" double shield and EPB capacity as described. Segmental lining should be installed in due time before the TBM's will encounter expected known weakness zones in order to ensure that the lining will be able to operate at its design level of up to 10 bar counter pressure through the weakness zone. Similarly, the segmental lining will be maintained to a certain distance after the weakness zone has been encountered. It is recognized that the current status of technology implies that such a "dual-mode TBM" that has been specified here is not fully water tight as some leakage may occur through the joint of the telescopic shield and the gripper shoe opening, while operating in closed mode with counter pressure against the soil and ground water pressure acting on the segmental lining. This is a weak point that could develop during the construction phase, whilst research amongst TBM suppliers is undertaken that hopefully could improve the problem in the future.

For a project like the Rogfast tunnel it will be required to use a number of TBM's in order to ensure sufficient progress, an acceptable construction period and flexibility. Assuming a twin bored tunnel, it is assessed that the most beneficial number of TBM's to be used for excavating the tunnel would be 4 and logically 2 TBM's launched from Randaberg and 2 TBM's from Bokn. The exit tunnel to Kvitsøy, including the junction with the main tunnel situated around halfway through the main tunnel, should then be excavated by the use of drill&blast techniques. The TBM's would then meet either at the junction, or in case of problems encountered by one of the TBM's, one of the other TBM drives could be extended beyond the junction to meet the delayed TBM.

Fewer or more TBM's have also been considered, but reducing the number of TBM's to 2 would require each to do 25 km which is considered an increased project schedule risk. On the other hand, employing as many as 6-8 TBM's would be complicated as some of the TBM's would need to be assembled and launched from a chamber located in the junction with the tunnel from Kvitsøy.

## COMPARING CONSTRUCTION TIME SCHEDULE AND COST ESTIMATES

Tunnelling with drill&blast implies the following estimated advance rates. 34 m per week in sections with ground conditions defined as 'good rock mass'. The estimates suggest an advance rate of less than 10m per week in weakness zones. Based on a shift arrangement with 6 working days per week and working two shifts each with a length of 10 hours per day an average excavation progress, or weekly advance rate would be 27.1 m throughout the Rogfast tunnel using drill&blast considering the expected geological conditions along the current planned tunnel route.

The critical path is differentiated into two independent tunnel portions. Tunneling an entrance from Kvitsøy can be calculated to approximately 100 weeks assuming roughly 4 km tunnel length with 40 m advance rate per week expecting more favorable ground conditions than in the main tunnel. On average approximately 2.7 km can be excavated in the same period of time for the main tunnel from Bokn (critical path), and then followed by tunnelling from the junction in the direction of Randaberg with the same average advance rate. This will take 205 weeks in addition to the aforementioned 100 weeks for the exit tunnel totaling 305 weeks. It is worth to notify that 2 parallel tunnel faces being constructed by drill&blast may not be operated fully independently. This is due to elements such as HSE and mucking which will cause an impact on the neighboring tunnel tube even though both tubes have complete and separate resources. Further transport through the cross connections and the exit to Kvitsøy suggest that additional time is required to obtain reliable advance rates. An optimistic approach by applying an attributing 15 % delay because of this has been chosen. The construction time would then arrive at a total of 361 weeks, or roughly equivalent to 6.7 years.

Construction of niches, slow traffic lane and cross passages is not included in the prognosis model that has been used. However, it has been assumed in this comparison that these tunnel elements can be established behind tunnel faces and without having any significant impact on the progress relating to advancing the tunnel. Using drill&blast it is reasonable to believe that the contractor will establish niches simultaneously as advancing the tunnel without hampering the progress, while cross passages can be established in separate operations. Applying TBM it would be logical to do all this work in separate operations, constructing e.g. niches in a good distance behind tunnel face and cross passages when the main tunnels are completed. The estimates for TBM construction suggest that it is expected to be an average advance rate in those parts of the tunnel that has 'good rock mass' of 75 m per week. In weakness zones, however, the average advance rate is significantly lower and is at about 13 m per week. Based on the current assumptions for the project an average advance rate the Rogfast tunnel using TBM would be 51.8 m per week. Construction of the TBM alternative involves 13.8 km tunnelling from Bokn to the junction with an average advance rate of 51.8 m per week provides a construction time of 265 weeks or nearly 5.1 years. For simplification of the geological conditions it was assumed that they appear similar for the two sections of the tunnel, and that there is conventional tunnelling involved in this alternative.

The above indicates that TBM construction time will be roughly 1.6 years faster compared to drill&blast. It should be mentioned that mobilization time, i.e. time for ordering, manufacturing and supply of TBM's are not included in this comparison.

A cost comparison of the excavation cost of the Rogfast tunnel by drill&blast and TBM tunneling has been made. The comparison is based on NTNU prognosis models for the respective tunneling methods. The base cost includes the main elements such as: excavation, rock support and grouting probe drilling, etc. The cost includes also capital costs associated with the acquisition and depreciation of capital equipment (the TBM), crew costs, in addition to necessary supplies. Including the cost of a continuous water and frost protection in addition to the estimated base cost of excavation work the TBM is 25-30% more expensive than drill&blast. This is mainly due to the more costly segmental lining to be installed for the TBM in mentioned weakness zones and the concrete structures that will be installed throughout the tunnel length in the TBM alternative. Support of weakness zones in drill&blast is based on reinforced sprayed concrete ribs.

**COMPARISON OF RISKS AND ENVIRONMENTAL ASPECTS**

A rough assessment of the main construction risks have been carried out in order to compare between the two alternative construction methods drill&blast vs. TBM. Some of the main results from the risk assessment are as follows:

The TBM is assessed to be better fit to handle geological variations and adverse geological conditions in identified weakness zones compared to drill&blast. This is due to the fact that installation of the segmental lining can be planned prior to entering into expected weakness zones. A TBM type which can excavate in closed or semi-closed mode with counter pressure against the soil and water pressure at the tunnel face provides an additional measure for safe tunnelling through expected weakness zones. In any case the TBM in closed or semi-closed mode should be used in combination with modern grouting techniques. The latter to be applied ahead of the TBM face prior to entering into weakness zones, and in exactly the same manner as would have been applied for drill&blast, and shall be the first line of defense in any tunnelling method subsea.

In unidentified weakness zones the situation will be different, although the probability for encountering such zones will be low as it is assumed that additional ground investigations will be carried out in the future to establish a reliable and robust geological model prior to commencing construction. This would include seismic profiles and core drilling from land and from ship. Further the capability of doing probe drilling ahead of face need to be mandatory and equally the same regardless whether TBM or drill&blast is chosen.

In case the TBM enters unexpectedly into such zones no lining would have been pre-installed and the TBM will have severe problems to immediately mobilize a counter pressure at the tunnel face. In such situations the TBM shield will provide initial support and to some extent prevent a collapse and excessive inflow of water. If the situation gets out of control the consequence could be loss of tunnel and loss of TBM, it takes longer time to recover the situation than recovering a drill&blast.

In order to reduce the consequence of such as losing a tunnel the possibility should be considered to postpone the construction of the cross passages until the main tunnels have been completed. This precaution will reduce the risk of flooding both tunnel tubes if uncontrolled large amounts of inflow water takes place and recovery can take place from the parallel tunnel e.g. through grouting or freezing or similar other measures to be employed. The main difference in terms of environmental aspects if the tunnel excavation is by TBM or drill&blast is that the larger TBM cross section will produce approximately 17% more muck compared to the amount of rock generated by drill&blast. Further the muck produced by TBM will include a larger content of fine materials than by drill& blast excavation. The possibility of placing such material in mass deposits at sea will be at a risk for washout of fine materials unless the rock dumps are built using special measures.

## CONCLUSIONS

The evaluations that have been made and presented in this article indicate that TBM is a possible and feasible tunnelling method for the Rogfast tunnel. It is the opinion of the authors that the project in the future until detailed design phases are completed should include TBM as an alternative method in line with drill&blast. This must be reflected for example in additional ground investigations. As a comparison the railway project from Oslo to Ski was developed with two alternative tunnelling methods in parallel until one of the two clearly stood out as being more beneficial. This appears to be a process that can be a possible path to follow for the Rogfast tunnel also.

It appears that a twin tube tunnel solution is the most optimum solution for TBM excavation. This has several advantages over a single-tube tunnel with traffic areas in a double deck solution. Moreover this solution represents the minimum deviation from the base solution with drill&blast. The type of TBM best fit for the Rogfast project would be a dual-shield machine, equipped with EPB capacities, i.e. a so-called "dual-mode TBM". This type of machine can be operated with and without segmental lining to fit the actual ground conditions as they appear. The project requirements demand future research and development to be done by the machine suppliers as it is not proven technology today.

The cost of the TBM excavation is considered to be approximately 25-30 % more expensive than the cost of drill&blast.

A significant advantage of TBM excavation is consequently considered to be the industrial work procedure and the possibility of relatively high progress rates even in poor rock mass quality. For a long tunnel with many weakness zones like Rogfast the construction period is a significant factor for the selection of method. The length of the Rogfast tunnel motivates the large investment that TBM poses. Based on NTNU's prognosis model it is estimated that TBM excavation could double the advance rates compared to drill&blast excavation. TBM has a total construction time which is 1.6 years shorter than for drill&blast. A prolonged mobilization is required for TBM equipment compared to conventional equipment, which is not recognized in the assessment of the construction time.

A significant factor when selecting the excavation method will be confidence in the method and the risks which are linked to each method in relation to progress and cost. It is considered that TBM can result in a reduction of the scheduling risk compared to drill&blast. This is mainly due to that TBM's are associated with reduced risk in adverse ground conditions compared to drill&blast excavating weakness zones. The benefits of TBM may only materialize if the machines are delivered according to the specifications that require special equipment to be installed. This will probably be perceived as strict and set machine suppliers in a technological challenge, but it is necessary that these specifications are met and that they do not compromise on these. A successful implementation of the Rogfast tunnel with TBM will depend on these requirements being satisfied.

In case of catastrophic events the TBM tunnelling could be a disadvantage, since the repair and recommencing of drilling is more difficult with a TBM than in the case of drill&blast.

Considering the length of the Rogfast tunnel the geological surveys are sparse - large stretches are not investigated. Further investigations are required resulting in more accurate mapping of weak-ness zones to improve the geological model. The investigations should be designed with particular focus on testing to determine parameters relevant for TBM excavation and risk mitigation.

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# EFFECTS OF TOPOGRAPHY ON GUSTY WIND ACTION FOR LONG-SPAN SUSPENSION BRIDGES

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## ABSTRACT

*It is well known that the topography of a mountainous region may create strong chaotic winds with large-scale turbulent eddies, increasing the risk of structural damage and accidents. The objective of this paper is to present the results of a case study emphasising the topographical effects on wind structure and to discuss potential implications for design action and aeroelastic performance of long-span suspension bridges. The available information is summarised and outlined, and the current techniques used in quantifying the wind structure and wind-induced effects are revisited. The case dealt with is an experiment carried out in the seventies at the Sotra Bridge located in the Norwegian archipelago west of Bergen. This case reveals the environmental variability and complexity that are expected to be found at many Norwegian strait crossings.*

*The findings of this study indicate that the topographical structure may significantly influence the design specifications of long-span bridges. The inhomogeneity and non-stationarity inherent in a topographically shaped wind field can result in varying mean wind and turbulence characteristics, as well as varying wind direction, which increases the randomness in the resulting wind action. For the suspension bridges currently under planning, with extreme spatial-temporal structural characteristics, a reformulation of the aeroelastic model is required to account for the non-stationary and inhomogeneous structure of the wind field.*

## INTRODUCTION

The design of long-span suspension bridges is based partly on wind tunnel studies and partly on computational models. In the past, wind tunnel studies have mainly concentrated on assessing aerodynamic stability and the derivation of force coefficients to be applied in computational response analysis. These studies are traditionally based on applications of section models (see for instance, Simiu and Miyata, 2006), which are scaled rigid models of bridge deck sections supported by elastic springs. The section model is usually mounted perpendicular to the air stream in the wind tunnel and tested in uniform smooth (low turbulence) airflow. Even though the similarity of the bridge deck geometry and individual natural frequencies of particular still air structural modes and corresponding structural damping ratios can be obtained, the similarity of

wind-induced dynamic forces is not necessarily assured in such simulations. It has also been argued that the shortcoming of the sectional model technique is transferred to the computational models. In the numerical modelling of the bridge system, the wind is traditionally assumed to be blowing perpendicular to the bridge deck. Furthermore, the wind is approximated as a uniform stream in which the turbulence components are modelled as a homogeneous stationary random process (Jain et al., 1996; Katsuchi et al., 1999; Simiu and Miyate, 2006; Øiseth, et al., 2010 and 2011). These modelling approximations may be justifiable for "short span" bridges in smooth terrain, but do not necessarily hold for the ultra-long-span suspension bridges now under serious consideration in Norway.
The potential effects of complex terrain on wind-induced behaviour were recognised long ago. Such effects have been studied in wind tunnels using terrain models (see, for instance, Davenport and King, 1990). In addition, the applications of computational fluid dynamics (CFD) to study local topographical wind effects are found useful (Cochran and Derickson, 2011). Full-scale studies are of key importance for calibration of the physical and computational models, as well as for their verification and validation.

An interesting topographical wind tunnel study described by Davenport and King (1990) deals with a Norwegian bridge project. A large topographic model was constructed at a scale of 1:2625 covering an area in the wind tunnel of 83 $m^2$, corresponding to 575 $km^2$ in full scale. The model was produced in modular form and could be tested for four different wind directions judged to be significant for the location of the study bridge. Figure 1 shows the topographic model in the wind tunnel, looking downwind from the North Sea.

*Figure 1: Topographical model of the Norwegian West Coast. View downwind from the North Sea (Davenport and King, 1990).*

**THE SOTRA EXPERIMENT**

An important high-quality full-scale experiment was initiated by NTNU/SINTEF as early as in the 1970s. The study is described in detail in a report co-authored by Jensen and Hjorth-Hansen (1978). The selected study site was in the archipelago approximately 50 km west of Bergen, where the Sotra Bridge crosses the strait between Knarrevik on Litlesotra and Drotningsvik on the mainland. The archipelago consists of variably sized rocky islands, forming a barrier between the North Atlantic Ocean and the mainland of Norway (see Figure 2). Overall the area has been characterised aerodynamically as intermittently rough (Jensen and Hjorth-Hansen, 1978). A rather long narrow sound, roughly 600 m wide, dominates the neighbouring topography of the bridge. In this area the wind direction is predominantly south-west; in other words, the wind is frequently blowing along the strait approximately perpendicular to the bridge's longitudinal direction. Under such conditions it is expected that the surface roughness for the bridge site can be quantified as smooth (see Figure 3).


*Figure 2: The location of the Sotra Bridge in the archipelago west of Bergen. The size of the area displayed is approximately 35 x 30 km$^2$. Map from Statens kartverk: www.norgedigitalt.no.*

The total length of the Sotra Bridge is 1236 m; the suspension bridge is 768 m, with a main span of 468 m. The maximum height of the bridge deck above sea level is 57 m. The concrete deck of the suspension bridge is supported by a space frame girder, 5 m high and 10 m wide, suspended by cables running over the top of 103 m high concrete towers (see Figure 3). The typical width of truss members is about 0.2 m. The member profiles are either round or I-shaped. The deck carries two lanes, each 7.5 m wide, with an 0.8 m wide footpath on each side. When the Sotra Bridge was opened for traffic in 1971, it was the longest suspension bridge in Norway, but by 2013 it is the 9$^{th}$ longest. Due to its location and the surrounding topography, the traffic over the bridge is vulnerable to strong winds from the north and the south.

*Figure 3: View southwards from the Sotra Bridge. During the test run discussed in the article, the wind was blowing from the south. The photo was taken on the bridge at the location of the white sonic anemometer (see Figure 3) (photo from Øiseth et al. 2013).*

***The measurement system and applied data***

The Sotra experiment was conducted in 1975. Then simultaneous measurements of atmospheric turbulence, strain response and response acceleration were carried out at the bridge. The measurements were reported by Jensen and Hjorth-Hansen (1978), and some of the data have been processed and published (see, for instance, Jensen, 1978; Kristensen and Jensen, 1979; Kristensen et al, 1981; Øiset et al. 2013). In this article we will discuss only the turbulence data recorded from one particular sensor configuration.

The turbulence measurements were carried out using three-dimensional acoustic anemometer-thermometers from Kaijo Denki (Jensen and Hjorth-Hansen, 1978). Three instruments of this type were deployed, identified below as the green (G), red (R) and white sonic anemometer (W). The wind sensors were mounted on poles placed at the south edge of the bridge, 3.32 m above the bridge deck. Hence the height of the wind sensors above sea level was approximately 60 m. The sonic anemometers were distributed along the bridge as shown in Figure 3. As can be seen, the green sonic is located at the centre of the mid-span of the bridge, the red sonic 68.4 m east of the green sonic, and the white sonic 136.8 m from the green sonic, also to the east.

The wind was blowing from the south during the applied test run, with the wind sensors located at the windward edge of the bridge deck. In order to investigate whether the sonic wind sensors were disturbed by the bridge itself, three cup anemometers were mounted on the windward side at heights of 2.35, 3.10 and 4.30 m above the bridge deck to measure the mean wind distribution. It was concluded that the decrease in mean wind velocity above the bridge deck is small, indicating that the sonic anemometers record undisturbed flow (Jensen and Hjorth-Hansen, 1978).


*Figure 4: Photo of the Sotra Bridge viewed from the north. The inserted points G, R and W represent the configuration of the sonic anemometers used in the test run. The anemometers are referred to as green, red and white sonic in the text. Photo: Norwegian Public Roads Administration.*

### *Characteristics of the wind field*

The time series selected for this study were recorded during the night (starting at 01:18 local time). The wind was blowing from south, the temperature was 10°C, relative humidity 95%, cloud cover 8/8, and there were no disturbances due to traffic on the bridge. The duration of the time series was about 13.6 minutes. The sampling rate was 10 Hz. The wind velocity was fairly stationary during this period, and the mean wind speed was at its maximum in the current run. The time series recorded by the green, red and white sonics are shown in Figure 5. These time series have been transformed into a Cartesian coordinate system where the x-axis is horizontal and pointing in the direction of the mean wind vector, perpendicular to the longitudinal axis of the bridge; the y-axis is horizontal pointing along the bridge axis towards east; and the z-axis is vertical. The corresponding turbulence velocities are identified as *u*-, *v*- and *w*-components, commonly termed the along-wind component, the cross-wind component and the vertical component. The basic one-point statistics are displayed in Table 1.

Apparently there is not a great difference between the nine time series displayed in Figure 5, with the exception of the mean wind velocity characteristic for the along-wind components. On the other hand, the values given in Table 1 reveal an interesting distinction between the three observation sites.

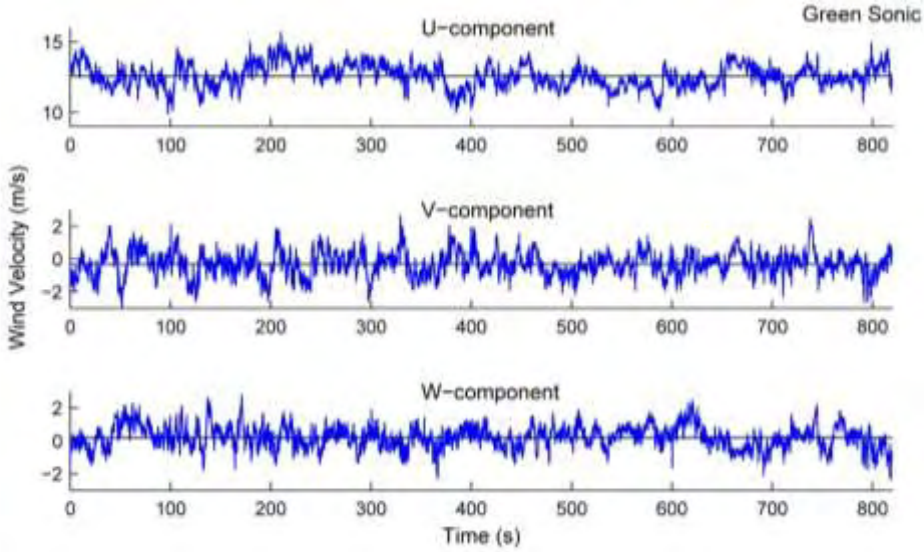
*Table 1: One-point statistics of the wind velocity components at the Sotra Bridge during the applied sensor configuration and the test run.*

| *Quantity* | *Symbol (Unit)* | *Green sonic* | *Red Sonic* | *White Sonic* |
|---|---|---|---|---|
| Mean wind velocity | $\bar{U}$ (m/s) | 12.59 | 12.16 | 11.72 |
| Mean of cross-wind component | $\bar{V}$ (m/s) | -0.31 | -0.21 | -0.27 |
| Mean of vertical component | $\bar{W}$ (m/s) | 0.21 | 0.07 | 0.07 |
| Turbulence intensity | $I_u = \sigma_u/\bar{U}$ | 0.066 | 0.069 | 0.079 |
| | $I_v = \sigma_v/\bar{U}$ | 0.060 | 0.058 | 0.067 |
| | $I_w = \sigma_w/\bar{U}$ | 0.055 | 0.055 | 0.058 |
| Covariance (one-point) | $\sigma_{uv}$ (m/s)$^2$ | -0.073 | -0.103 | -0.254 |
| | $\sigma_{uw}$ (m/s)$^2$ | -0.060 | -0.095 | 0.035 |
| | $\sigma_{vw}$ (m/s)$^2$ | -0.009 | 0.039 | -0.084 |

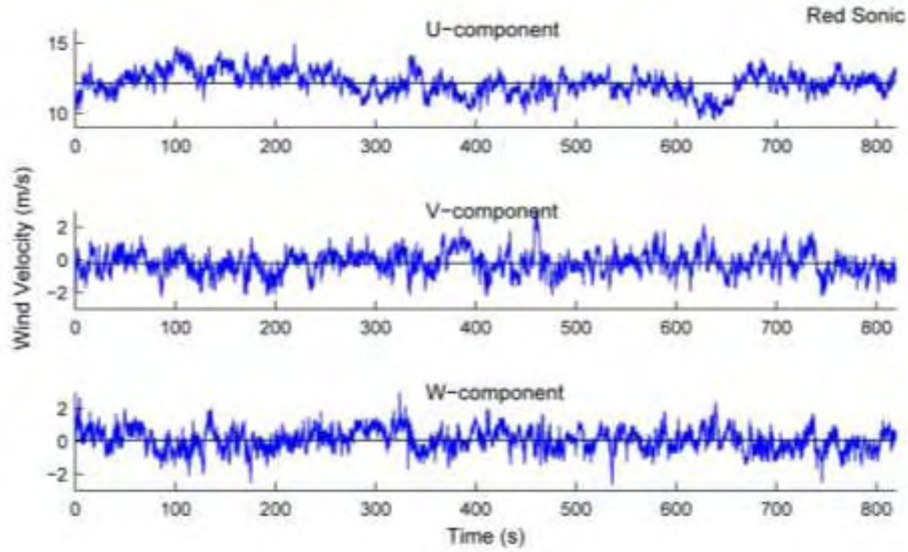
The statistics in the table above show that the highest mean velocity was recorded by the green sonic in the middle of the sound; the recorded mean velocity is lower at the red sonic, while the mean wind velocity is lowest at the white sonic closest to the shore. It is worth noting that the difference is not all that great (less than 10%), but persistent.

The turbulence intensities at the green sonic, $I_n$, $n \in \{u, v, w\}$, in the horizontal along-wind, horizontal across-wind, and vertical directions, respectively, are all slightly different, revealing that the commonly accepted isotropic turbulence model (see, for instance, Kristensen and Jensen, 1979; Mann, 1994) is no more than a fair approximation in this case. The turbulence intensity in the along-wind direction, $I_u$, increases significantly closer to the shore. It is seen from the table that this increase is up to 20% for the case considered. The turbulence intensity $I_v$ seems to decrease slightly at the red sonic relative to its value at the green sonic, while it is highest at the white sonic closest to the shore. The turbulence intensity $I_w$ remains unchanged from the green sonic to the red sonic, while its value becomes significantly higher at the white sonic.

The covariance of the horizontal and vertical components, $\sigma_{uw}$, is proportional to the Reynolds stress and is, hence, related to the surface roughness at the site. For a stable mean wind profile, with the velocity increasing with height, the Reynolds stress is negative and is commonly used as a measure of the surface roughness. Considering the values obtained at the green and red sonics, the covariance is negative with the lowest value at the red sonic; accordingly, it follows that the 'apparent' surface roughness is higher at the red sonic than at the green one. When we come to the white sonic, the covariance is positive. In other words, the wind profile does not behave in the same way at the white sonic as it does at the location of the red and the green sonic; i.e. at the white sonic the mean wind velocity decreases locally with increased height.




(a)




(b)

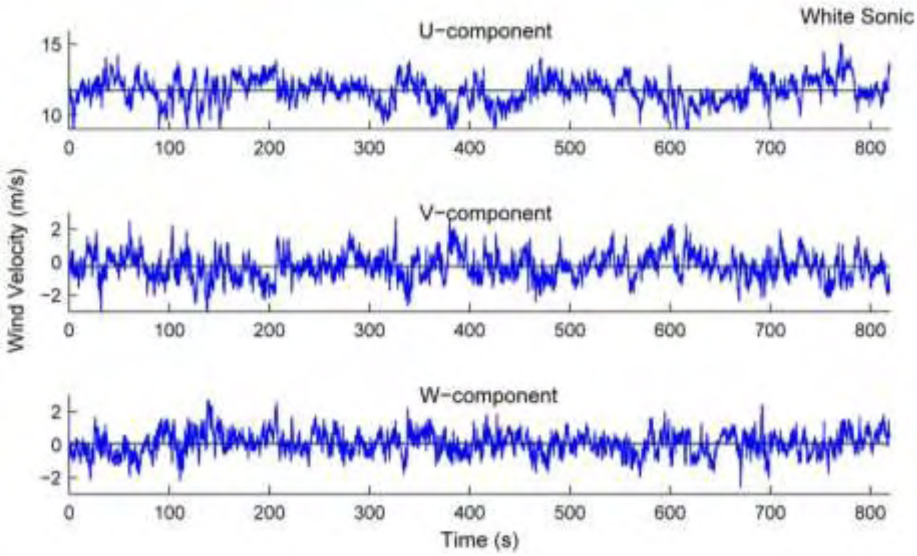



*Figure 5: Measured wind velocities at the mid-span of the Sotra Bridge: (a) the green sonic anemometer; (b) the red sonic anemometer; and (c) the white sonic anemometer. See Figure 4 for anemometer locations.*

The quantification of the wind structure given above clearly reveals an influence from the topography of the sound. In fact, the wind is behaving similar to a channel flow. The mean wind is highest in the middle of the strait but decreases towards the shore, while the turbulence is lowest in the middle and increases towards the shore. Furthermore, the increase in turbulence in the transition zone, between over-sea and over-land conditions, results in an "instable" wind profile. Last but not least, the effects of the coastline seem to stretch far out into the sound and have noticeable influence on the turbulence structure recorded by the red sonic anemometer.

***Spectral properties***

The estimated one-point auto- and cross-spectral densities of the *u*, *v*, and *w* components, measured by the green, red and white sonics, respectively, are presented by Øiseth et al. (2013). The main finding is that the auto-spectral densities of the turbulence components recorded by the green and the red sonic behave as expected. Some discrepancies were observed in the results for the white sonic, which can be interpreted to be a consequence of the topographical influences observed above.

The two-point cross-spectral densities are exemplified in Figure 6 for the *v*- and *w*-component, i.e. the cross-wind and the vertical component. The co-spectra of the *vw*- and *wv*-component for the red and white sonic reveal symmetric behaviour, while the difference in the corresponding quad-spectra is not very great. This behaviour can be visualised by a rotor (vortex) with negative spin about the *x*-axis, reflecting the effects induced by the coastal roughness.

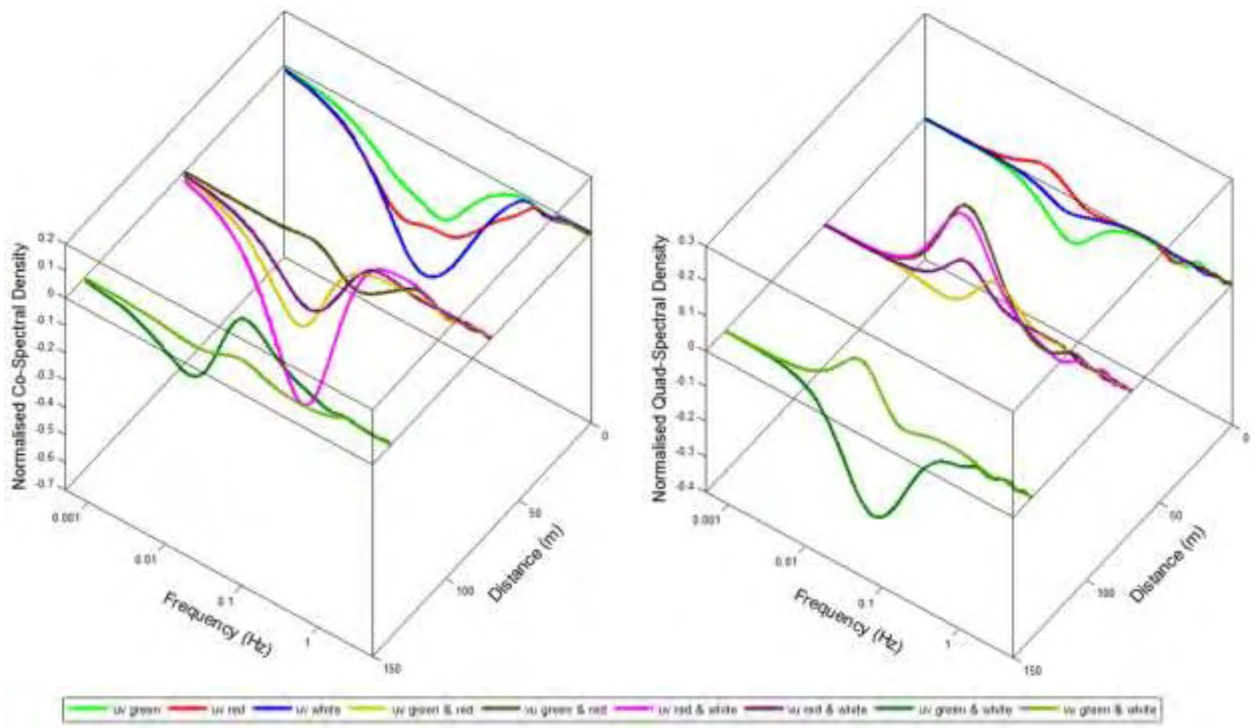



*Figure 6: Cross-spectral density of v- and w-component at the red and white sonic: (a) co-spectral density; and (b) quad-spectral density.*

It is worth noting that these effects are much weaker in the cross-spectra between the same components obtained from the green and the red sonics, which also is a clear indicator of the topographical influences.

It should be mentioned that a similar rotor (vortex with negative spin about the *y*-axis) is visible in the *uw*-components as expected and is stronger closer to the coast than further away; but somewhat more complex behaviour is revealed by the quad-spectrum in this case than seen in Figure 6.

## DISCUSSION AND CONCLUSIONS

The above presentation reveals clearly that the local wind structure at the Sotra Bridge is significantly influenced by the surrounding local topography. Furthermore, it seems clear that the shape of the strait results in channelling effects, which leads to increased turbulence when approaching the coast line, while the mean wind is highest in the middle of the sound. These effects are not found to have significant influence on the structural behaviour of the Sotra Bridge (Øiseth et al., 2013). However, the same is not necessarily the case for an ultra-long-span suspension bridge. For such bridges the natural periods of the structural systems are longer, which implies that the cross-spectral densities become more important in the analysis: not only the *uw*-spectra (see Jones et al. 1992; Øiseth et al. 2013), but also the *uv*-spectra. It is predicted that this will result in a travelling wave pattern along the bridge axis and in the cables in addition to the vibrational response obtained using the current analysis procedures.

The main findings of the study presented and our recommendations based on these findings are the following:

- The Sotra experiment reveals a more complex wind structure than applied in the analysis of long-span suspension bridges today.
- The observed inhomogeneity of the wind field at strait crossings should be accounted for in design provisions.
- For ultra-long suspension bridges, non-stationarity of the wind field may play a role; the standardised 10 min reference period needs to be increased due to the exceptionally long natural periods of such structures.
- Full 3D modelling of the aeroelastic system is needed for ultra-long suspension bridges, and a comprehensive 3D description of the wind field should be included.
- Further studies, especially full-scale studies, into the behaviour of the wind field in complex terrain at strait crossings are required to meet the challenges of future bridge projects.

The above recommendations call for a reformulation of the computational modelling procedures for ultra-long suspension bridges. These models will be the subject of a follow-up article.

## ACKNOWLEDGEMENTS

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# IZMIT BAY BRIDGE - SEISMIC DESIGN

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## ABSTRACT

*The Izmit Bay Suspension Bridge will carry the new Gebze-Orhangazi-Bursa-Izmir motorway across the Sea of Marmara at the Bay of Izmit in northern Turkey. The bridge is located in a highly seismic zone and is arranged as a three span continuous suspension bridge having a total length of 2682m (566m-1550m-566m).*

*The deck is 30.1m wide, is shaped as a closed steel box girder having a depth of 4.75m and is carrying three lanes of highway traffic in each direction. The main cables consist of 110-112 prefabricated parallel wire strands each having 127 galvanized wires with a nominal diameter of 5.91mm. The 241.85m tall H-shaped towers are fabricated in steel due to demands for high seismic loads and short construction time. Closed box sections are used for legs and cross beams. The North anchor block is located on land and the South anchor block on reclaimed land. The tower foundations are located at water depth of 40m and placed on a 3m thick gravel bed to provide a controlled sliding surface thereby limiting the maximum interface shear friction during high amplitude earthquakes.*

*Seismic non-linear Time History Analyses (THA) are used in order to analyse the behaviour of e.g. the tower foundations, where sliding of the foundations is allowed to reduce the seismic impact to the bridge structure. The articulation system includes a buffer system with non-linear characteristics and thus also requires non-linear analysis to model the intended behaviour. The combination of THA and local sub-models being part of the global analysis model (GAM) makes it possible to carry out local seismic design based on results obtained by running only the GAM - i.e. there is no need for manually extracting force combinations from the GAM and apply them as boundary conditions to separate models.*

## 1. INTRODUCTION

The Izmit Bay Bridge is located approx. 50 km to the east of Istanbul, Turkey. It is part of a new major infrastructure project - the 420 km long Gebze-Orhangazi-Bursa-Izmir highway - and it will take the highway across the Sea of Marmara at the Bay of Izmit.

The bridge has a main span of 1550 m which will be the World's 4th longest suspension span. Apart from being an important structure the bridge design and construction also face two major challenges: extremely high seismic load due to the location in a highly seismic zone and a construction period of only 38 months. This paper mainly deals with a description of the bridge and design issues related to the seismic design.

## 2. BACKGROUND

KGM (Ministry of Transportation) have been planning for Izmit Bay Bridge since the 1990s and tender documents for the EPC (engineering, procurement and construction) contract were issued by the concessionaire OTOYOL/NÖMAYG Joint Venture in May 2010. Proposals were submitted in September 2010 and the contract was awarded to IHI infrastructure Systems Co., Japan. Detailed design started in September 2011, preparatory site works in September 2012, permanent site works in January 2013 and the bridge shall be completed by 2016.

COWI has prepared tender design and is currently finalising the detailed design for IHI.

## 3. BRIDGE CONCEPT

The bridge is arranged as a three span continuous suspension bridge having a total length of 2682m (566m-1550m-566m) and a navigational clearance of 64.3x1000m, see Figure 1. The 566m long side spans are with a ratio of 0.37 within the common range of 0.35-0.40 for side span/main span ratios. Simply supported spans of 120m and 105m forming transitions between the suspension bridge and the adjacent approach viaducts are included.

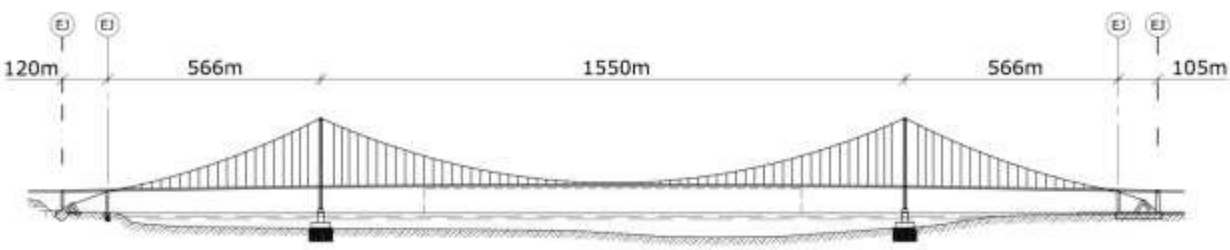


*Figure 1: Elevation of Izmit Bay Bridge*

In addition to saddles at the tower tops and anchor blocks the main cables are supported by deviation saddles where they pass over the side span piers at the end of the side spans, see Figure 2 and 5. Low-lying main cable anchorages are attractive both in terms of aesthetics and construction cost but to ensure that the bridge deck is not too flexible deviation saddles at the side span piers are required. This conceptual feature is shared with other suspension bridges, e.g. the Little Belt Bridge (Denmark) completed in 1970.

*Figure 2: Deck end at side span pier, deviation saddles (left), hydraulic buffers connected to side span pier (right)*

The bridge deck is continuous throughout its length of 2682m with expansion joints only at the ends. This arrangement, first incorporated in the late 1990s in the Great Belt East Bridge (Denmark) and High Coast Bridge (Sweden), avoids the need for additional structurally complex and high maintenance expansion joints and bearings where the deck passes the towers. Lateral bearings are the only bearings provided at the tower/deck interface.
Longitudinal deck movements are controlled by hydraulic buffers which block fast movements arising from fast acting loads like traffic passing the bridge, braking and wind gust but yield to slow movements from slow acting loads (mainly temperature changes and mean wind pressure). Thereby especially the frequent movements caused by traffic passing the bridge are eliminated which otherwise would cause accelerated deterioration of bridge bearings and expansion joints. The buffers connect the deck to the top of the side span pier, see Figure 2.

In non-seismic situations the buffers further act as end stops at the end of the movement range limiting the bridge deck movements at the side span pier to ±1.3m which is manageable by the expansion joints. The end stop shall be able to transmit a static end stop force of 23.5MN in ULS see the load-displacement diagram in Figure 3 (left).

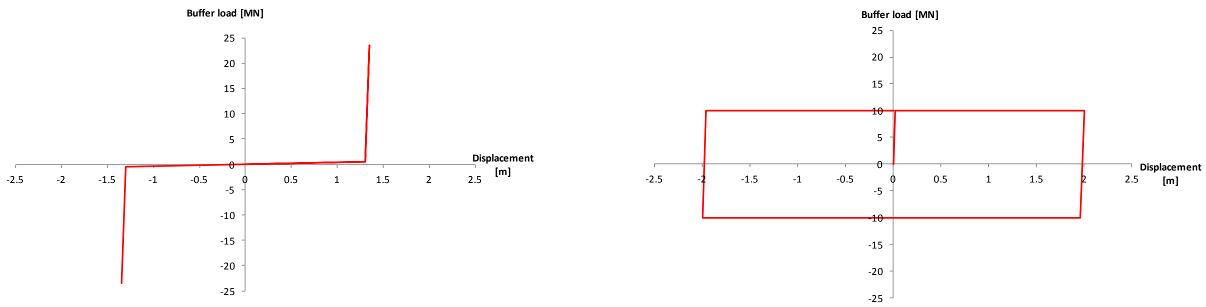


*Figure 3: Load-displacement diagrams, non-seismic situations (left), severe seismic event (right)*

During the most severe seismic events the longitudinal movements may exceed the limit of the end stop at ±1.3m and the end stops are disengaged. The theoretical load-displacement diagram during seismic events, see Figure 3 (right), shows how the buffers act as a lock-up device/damper for fast acting loads. They act as a "rigid element" as long as the force is below the threshold value of 10MN but when the 10MN is exceeded a pressure control valve open allowing the pistons to move while maintaining a constant load of 10MN.

The deck is 30.1m wide and arranged as a closed steel box girder having a depth of 4.75m, see Figure 4. It carries three traffic lanes each 3.65m wide in each direction plus hard shoulders and cantilevering walkways for operational and maintenance use. Trussed diaphragms at 5 m centres are used for minimum weight. The 14mm deck plate is stiffened by 360mm deep longitudinal closed trough stiffeners. The deck spans 25m between hanger cables. The roadway surfacing is a two-layer polymer modified mastic asphalt of 60mm total thickness.

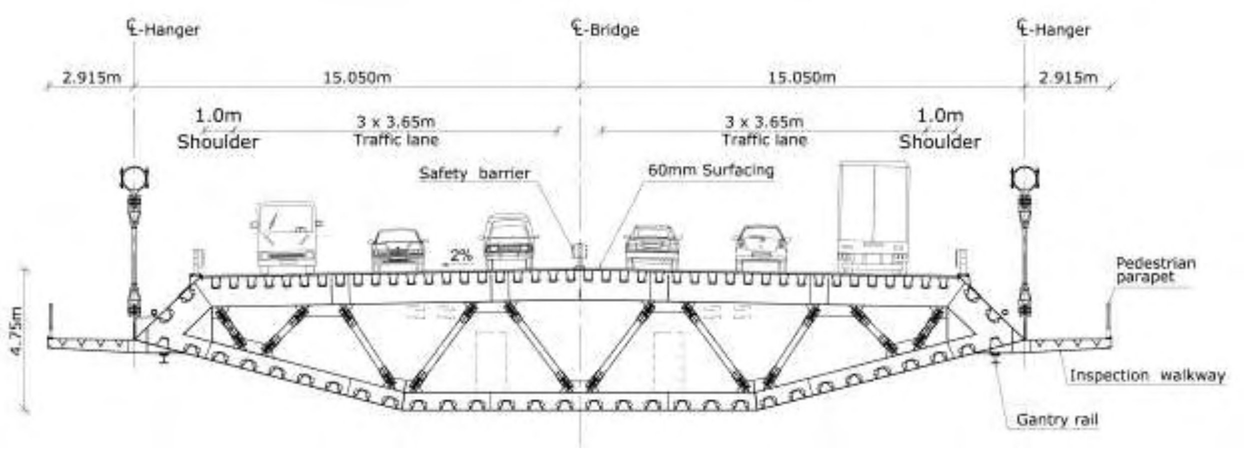


*Figure 4: Bridge deck arranged as a closed steel box girder*

The sag to span ratio 1:9 meet the demands of total quantity optimization (in particular the main cable quantity), the structural performance under traffic and wind load and the overall aesthetics of the bridge. The wire strength is 1760MPa. The main cable layout has been chosen with emphasis on achieving optimal structural quality, while at the same time reducing the construction time to a minimum. The cables are therefore formed by 110 (112 in side spans) prefabricated parallel wire strands (PPWS) each having 127 galvanized 5.91mm dia wires. The diameter of the completed main span cable after compaction is 781mm (assuming 20% air voids). The main cables are protected by S-formed wrapping wire, paint and a dehumidification system.

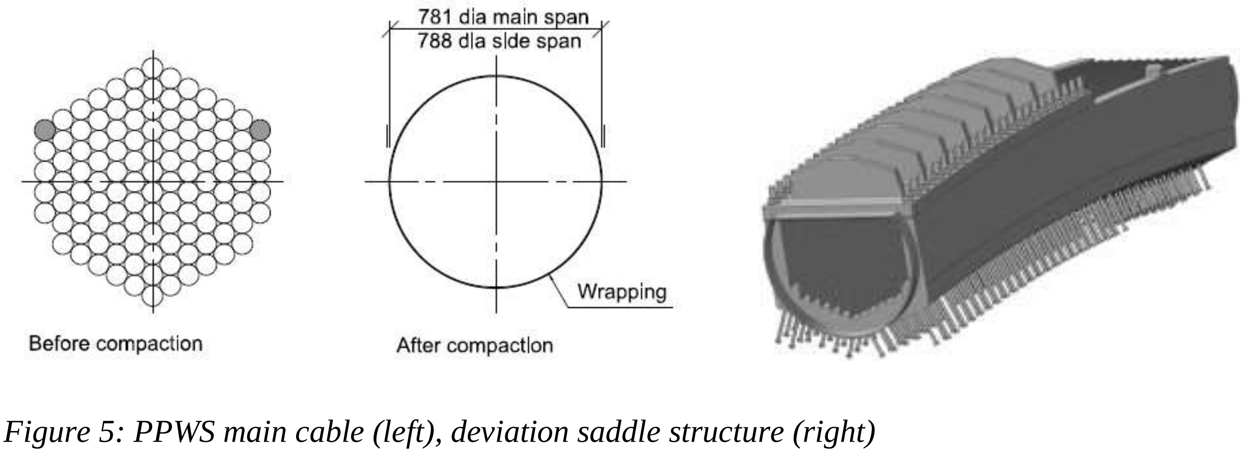


*Figure 5: PPWS main cable (left), deviation saddle structure (right)*

The H-shaped towers are fabricated from steel due to demands for high seismic loads and short construction time, have a total height of 241.85m from base to top of cable saddles and are provided with two cross beams, see Figure 6. Closed single-cell box cross-sections are used for legs and cross beams. The overall dimensions of the tower legs are 7x8m at the base and 7x7m at the top. The leg taper is in the longitudinal direction of the bridge. The outer plates of the tower legs and cross beams are stiffened with flats and transverse diaphragms in the legs are provided at spacing of 1.75-3.67m. Light steel towers are technically the most preferable solution in regions with high seismic loadings. In particular, in the transverse direction the steel frame structure of legs and cross beams is ideal providing the strength and stiffness needed to resist wind loads while also being flexible enough to avoid excessive seismic shear loads, which would be the result for a heavy and rigid concrete structure. A concrete tower alternative was studied during tender design but was deemed unsuitable and uneconomical for this reason.

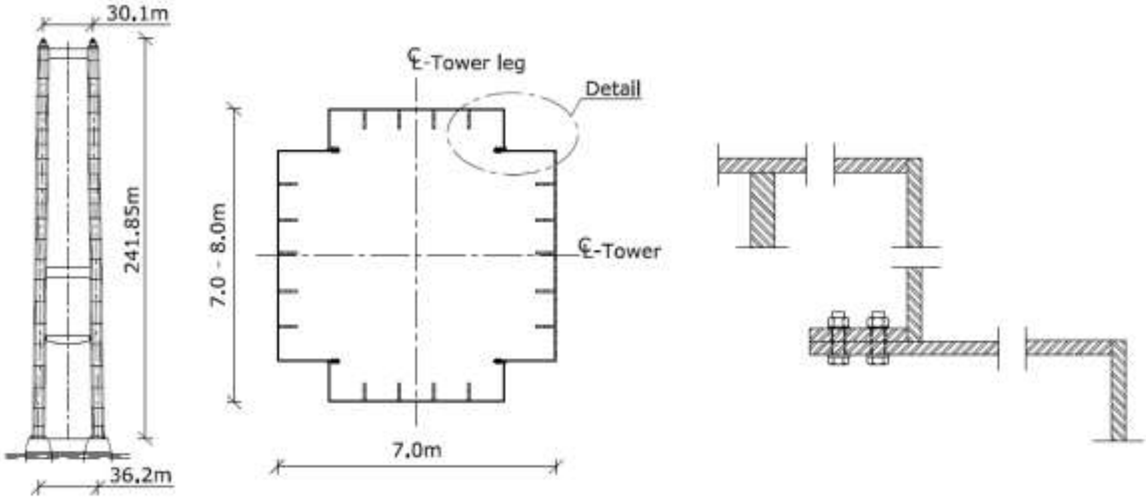


*Figure 6: Tower elevation (left), tower leg cross-section (middle), detail (right)*

The tower foundations are located at water depth of 40m. The soil below the caissons is improved by 2m dia. steel pile inclusions to a depth of about 35m below the seabed providing the required bearing capacity and reducing potential risk of liquefaction during earthquakes. A 3m thick gravel bed separates the improved soil and the reinforced concrete caisson providing a controlled sliding surface thereby limiting the maximum interface shear friction during high amplitude earthquakes (base isolation). Consequently the caisson will start to slide when the horizontal force exceeds the friction force which can be developed between the caisson bottom slab and the gravel bed surface. The sliding resistance is taken as 0.7 times the vertical load on the interface, corresponding to a gravel bed friction angle of 35deg. The gravel bed provides sufficient friction capacity for ship impact loads.

The prefabricated caisson is composed of a lower part consisting of a cellular base with an overall area of 54x67m and a height of 15m. The upper part comprises two cylindrical composite steel/concrete shafts with outer diameter of 16m and a wall thickness of 1.2m.

The anchor blocks are gravity based structures. The North anchor block is founded directly on rock while the South anchor block is supported on dense sand.

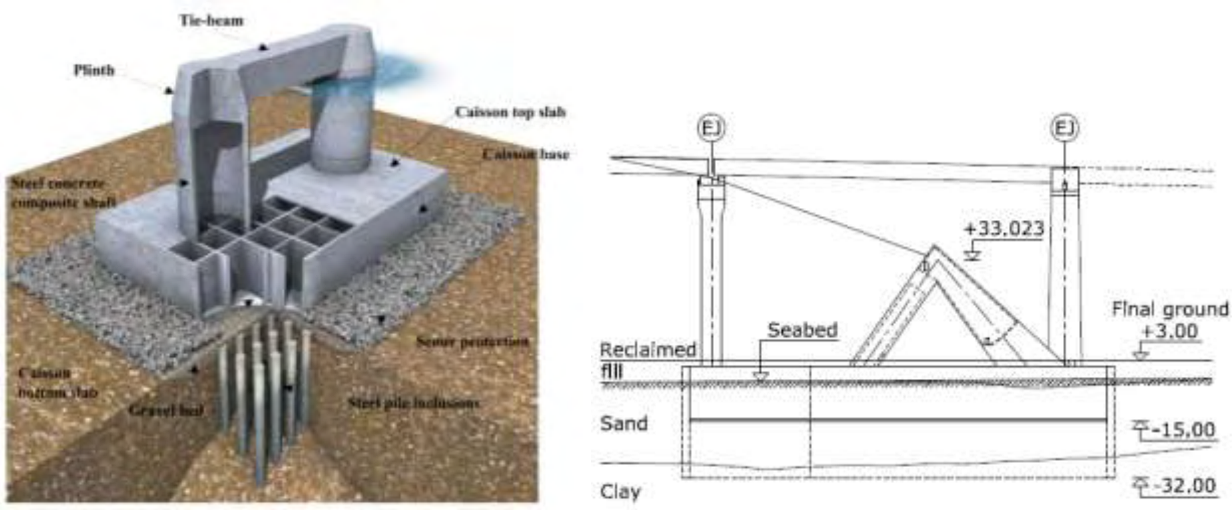


Figure 7: Tower caisson (left), South anchor block (right).

The design is generally carried out according to Eurocodes and site specific requirements.

## 4. GEOLOGY AND SEISMIC DESIGN

Izmit Bay is the eastern continuation of the Marmara Sea and is primarily shaped by the movements along the North Anatolian Fault (NAF), see Figure 8.

The NAF is approximately 1600 km long and extends from Karliova in eastern Turkey to the Aegean Sea in the west. It is a major right lateral strike slip fault that forms the tectonic boundary between the Eurasian Plate and the Anatolian block of the African Plate. The Anatolian block is being extruded westward along the North and East Anatolian Fault zones. The NAF zone forms a narrow band that splays into three strands in the eastern Marmara Sea region. The northern strand occupies the Izmit Bay and projects across the bridge alignment, representing the greatest seismogenic hazard source in the site area.


*Figure 8: Simplified tectonic map of the eastern Mediterranean area, bridge site orange circle*

The design earthquake ground motion is defined for three return periods as follows:

- Functional Evaluation Earthquake (FEE) - return period 150 years
- Safety Evaluation Earthquake (SEE) - return period 1000 years
- No Collapse Earthquake (NCE) - return period 2475 years

Design of the bridge shall be based on the performance criteria in Table 1.

*Table 1: Seismic performance criteria*

| **Seismic Event** | **Ground Motion Return Period** | **Performance Level** | **Damage Performance Level** |
|---|---|---|---|
| Functional Evaluation Earthquake (FEE) | 150 years | Immediate Access | No Damage |
| Safety Evaluation Earthquake (SEE) | 1000 years | Limited Access | Repairable Damage |
| No Collapse Earthquake (NCE) | 2500 years | - | Significant Damage |

The Service Performance Levels are defined as follows:

- Immediate Access - Full access to normal traffic is available almost immediately following the earthquake.
- Limited Access - Limited access is possible within days of the earthquake. Full service is restorable within months.

The Damage Performance Levels are defined as follows:

- Minimal Damage - Essentially elastic performance. Minor inelastic response may occur, and post-earthquake damage is limited to cracking of concrete and minor yielding of steel components. Permanent offsets associated with plastic hinging or with non-linear foundation behavior are not apparent.
- Repairable Damage - Damage that can be repaired without compromising the required service level. Inelastic response is acceptable, including concrete cracking, reinforcement yielding, local minor spalling of cover concrete, and yielding of steel components where intended as part of the design strategy. The extent of damage is to be sufficiently limited such that the structure can be restored essentially to its pre-earthquake condition without replacement of reinforcing bars of primary structural members. Repairs will not require structure closure to traffic.
- Significant Damage - Damage that does not cause collapse of any span or part of the structure, nor lead to the loss of the ability of primary support members to sustain gravity loads. Permanent offsets may occur and damage consisting of cracking, yielding, and major spalling of concrete may require bridge closure. Reinstatement of the structure may require extensive repairs and potentially reconstruction of bridge components.

The seismic verification is based on dynamic time history analyses (response history analyses) considering non-linear behaviour of soil springs and seismic isolation systems (fuse at interface between tower caisson and gravel bed and buffers between deck and top of side span piers).

The appropriate input displacement time histories at foundation levels are determined from the design time histories at competent soil/rock interface. The inelastic time history analyses shall account for different input motion at each of the four foundations (due to different site response because of different soil conditions) and also wave passage effects. The bridge foundations shall be subjected to the full three-dimensional displacement time series, including also the vertical components.

In summary the advantages of carrying out time history analyses compared to modal analyses are:

- Non linear behaviour of soil springs can be applied
- Non-linear behaviour of soil fuse below tower foundations can be modelled
- Non linear behaviour of hydraulic buffers between deck and side span pier can be modelled
- Permanent displacement at soil fuse can be determined
- Different seismic time histories can be applied at each foundation
- Wave passage effects can be included

The calculation work is substantial: 7 sets of earthquake time histories at each of the 4 foundations, for each of the 3 design earthquake return periods (FEE, SEE, NCE), each comprising 3 components (2 horizontal and 1 vertical) - in total 252 analyses. However, calculations were distributed over multiple CPU cores so a complete 4500 step time history run for all 252 analyses could still be completed within 1-2 days.

## 5. GLOBAL ANALYSIS MODEL INCLUDING LOCAL SUB-MODELS

The global analysis model (GAM) is created using the general computer aided structural design and analysis system IBDAS (Integrated Bridge Design and Analysis System), developed by COWI. IBDAS is based on 3D parametric solid modelling and provides procedures for fully integrated design and analysis of load bearing structures. An overall presentation of the GAM is given in Figure 9. The GAM comprises both the suspension bridge and the transition spans located next to the approach viaducts.

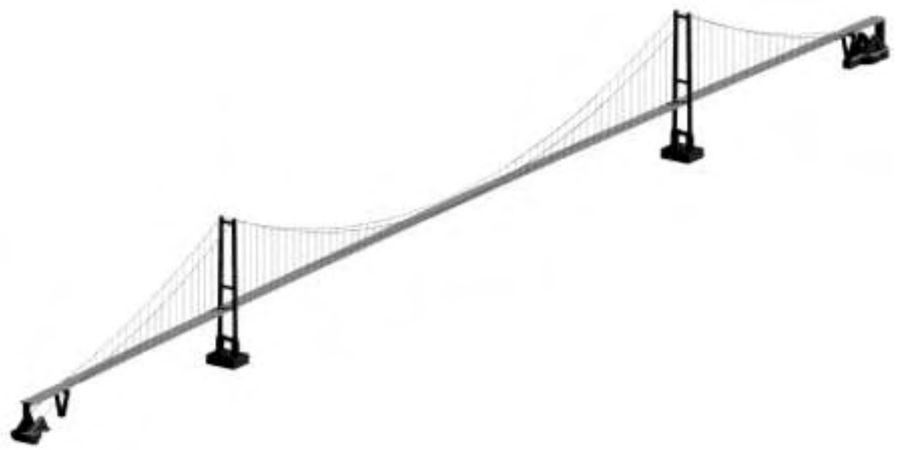

*Figure 9: Global analysis model (GAM) prepared in IBDAS*

The overall behaviour (dynamic response for developing displacement and force demand) of the bridge is analysed by the GAM. For this purpose the substructure elements are modelled in a simplified way in the GAM. However, more comprehensive sub-models are prepared for tower foundations (shell elements), anchorages (shell and solid elements) and side span piers (solid elements) for subsequent detailed capacity checks.  These sub-models can be an integrated part of the GAM, see Figure 10. It is possible to perform detailed structural verification directly in the GAM for a particular substructure element by activating the relevant sub-model while keeping all other substructure elements simplified. The same can be done for stress calculations in parts of the steel tower and the deck (shell elements) e.g. by using a detailed sub-model for the relevant part of the steel tower while keeping all the substructure elements simplified.

The local sub-models for the anchorages, tower foundations and side span piers are thereby used to determine crack widths, stresses and strains and deformations etc. and the local sub-models in towers and deck are used to determine stresses. As the local sub-models are an integrated part of the GAM, forces from the GAM are transferred directly to the sub-models which automatically ensure that the load input to the sub-models is correct. The individual sub-models can be activated on at a time in order to reduce analysis time for the GAM.

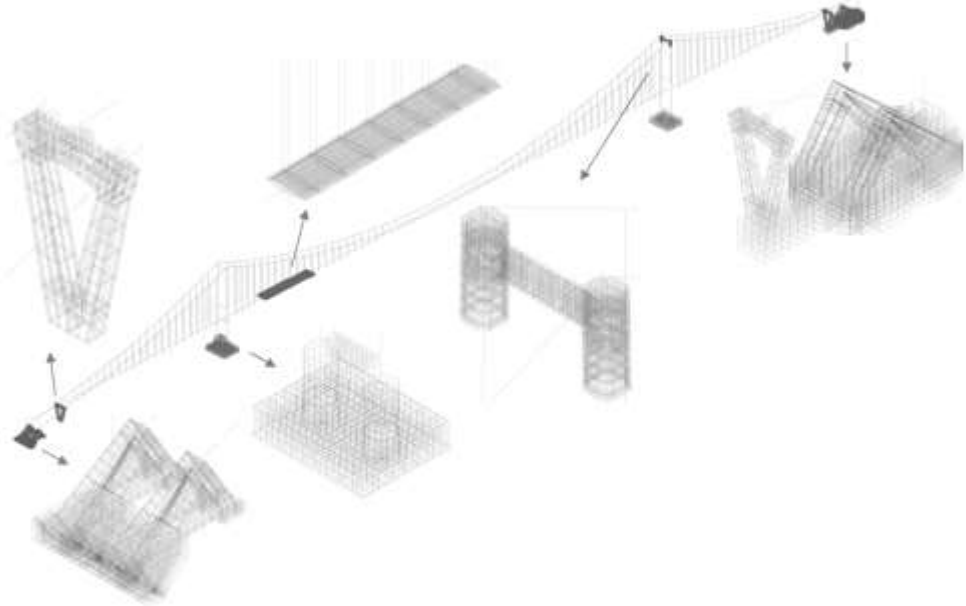

*Figure 10: Global analysis model (GAM) including local sub-models*

Utilization ratios can thereby be determined in the local sub-models for all load effects in SLS, ULS as well as for seismic response determined by time history analyses.

In conclusions the advantages of incorporating local sub-models in the GAM are:

- Boundary conditions are automatically correct as there is no manual transfer of forces from the global to the local sub-model (the local structural model is an integrated sub-model in the GAM)
- Geometrical changes made to the GAM are automatically included in the local sub-model
- Loads and load combinations are taken from a common data base in the GAM
- Design verification can be completed directly, without moving data
- Non-linearities in the GAM are automatically included
- Full non-linear time history analyses can be performed on the local sub-model when it is included in the GAM

## 6. GAM INCLUDING LOCAL TOWER LEG AND CROSS BEAM MODEL

The local sub-model (shell elements) is made directly in the GAM (beam elements). The local sub-model is made by exchanging parts of the relatively simple beam model with a much more detailed shell model. The local sub-model model is built up by shell elements according to the actual geometry of the structure. All skin plates, main vertical stiffeners and diaphragm plates are included in the shell model. The boundary conditions of the shell model are given by connecting the last node of the beam model to the nodes at the edges of the top and bottom part of the shell model through a master-slave coupling, see Figure 11 (left). It should be noted that the local sub-model is incorporated as a symmetric model in the GAM (i.e. all of the cross beam together with parts of the two legs) in order to avoid asymmetry in stiffness and thereby asymmetry in results from the GAM.

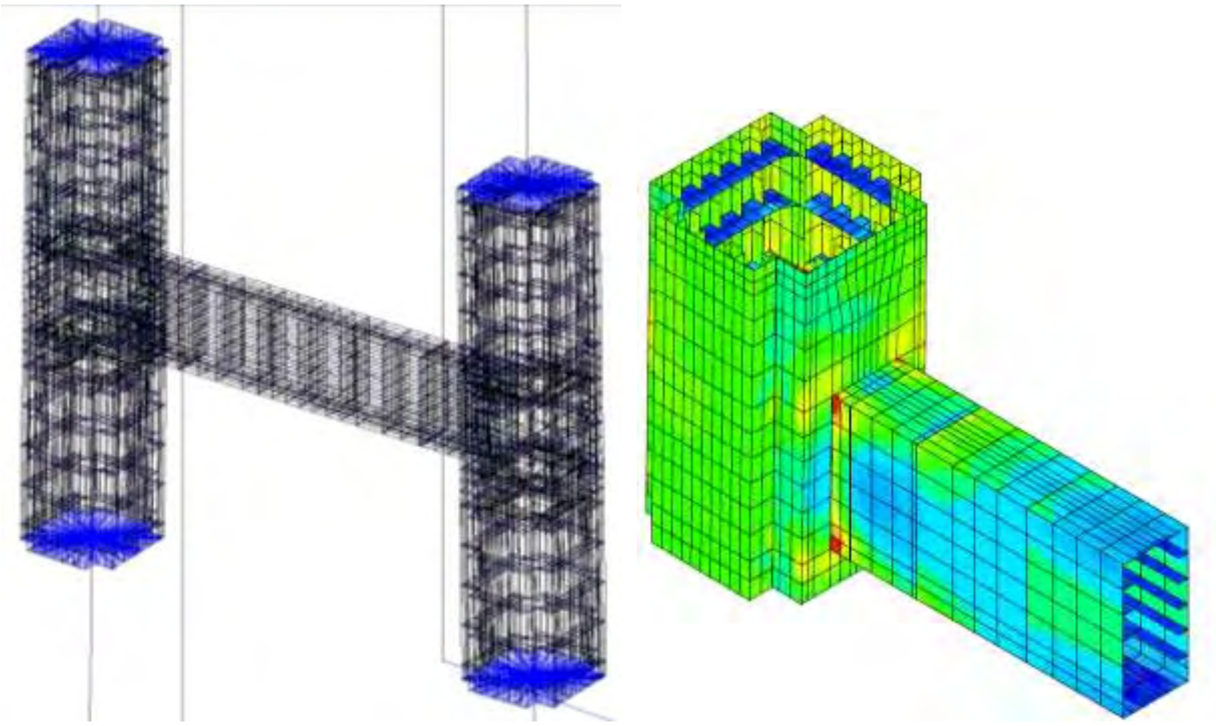

*Figure 11: Coupling of lower cross beam shell model to global beam model (left), ULS envelope plot of von Mises stresses for the lower cross beam connection to the tower leg (right)*

Live loads are individually optimized to produce the maximum load effect (stress) in each calculation point (gauss point). This means that two adjacent stress values on an envelope plot may not be coexisting but rather be a result of e.g. different wind directions or different locations of traffic loads (however, it should be noted that the von Mises stresses given in the result plots in each calculation point are based on coexisting values of $\sigma_s$, $\sigma_y$, and $\tau$). The plot in Figure 11 (right) shows an example of an ULS envelope plot of von Mises stresses for the lower cross beam connection to the tower leg. Such plots are useful in getting an overview of the utilisation of the structure. They are also useful for seismic calculations as one plot can cover the envelope of all time steps during a time history. Further, calculations can be carried out for all 7 time histories, then maximum von Mises stresses for the 7 time histories for each gauss point can be averaged and finally be shown in one plot similar to the plot shown in Figure 11. Again, such a plot is useful in getting a simple overview of the utilisation of the structure.

The important benefit of incorporating the local sub-model into the GAM is that maximum stresses are determined in all gauss points in the local sub-model and thereby a complete verification of the bridge structure is carried out using maximum stresses in all gauss points. Had the local sub-model been established as a separate model outside the GAM it would be necessary to apply set of sectional forces from the GAM as boundary conditions to the separate model and carry out separate analyses of the part of the bridge structure covered by the separate model in another program. This is known to be time consuming and it would from a practical point of view only be possible to carry out verifications for a limited number of combinations of sectional forces. I.e. the local sub-model method presents a solution to what would normally be a huge data handling task with inherent issues of quality assurance and interface control.

## 7. CONCLUSION

This paper gives a technical description of the main structural elements for Izmit Bay Bridge in Turkey.

The seismic verification is based on dynamic displacement time history analyses (response history analyses) considering non-linear behaviour of soil springs and seismic isolation systems (fuse at interface between tower caissons and gravel bed and buffers between deck and top of side span piers).

The detailed design work carried out for Izmit Bay Bridge allows the following conclusion to be drawn: the combination of time history analyses and local sub-models being part of the global analysis model makes it possible to carry out local seismic design based on results obtained by running the global analysis model only - i.e. there is no need for manually extracting force combinations from the global analysis model and apply them as boundary conditions to separate models.

## 8. ACKNOWLEDGEMENT

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# MODERN GEODYNAMIC ACTIVITY DIAGNOSTICS OF THE TERRITORIES CROSSED BY STRAITS WITH THE TRAFFIC CONSTRUCTIONS

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## ABSTRACT

*In this paper the influence of modern geodynamic processes and phenomena on transport facilities crossing straits, fjords, rivers and other water bodies is considered. The experimental data of parameters and types of modern geodynamic movements of the earth's crust are given. Linear and cyclical current geodynamic movements cause additional transition strain and loads in buildings that could lead to their destruction. A list of investigations required to select favorable sections for accommodation of the predicted transitions and develop sustainability measures are recommended.*

## 1 INTRODUCTION

The most important feature of the present stage of human civilization is an intense increase in the frequency and severity of natural and man-made disasters. The current trend of rapidly increasing complexity of the artificial structures and the lack of study of the impact they will have on processes and phenomena in the rock mass of the upper crust contribute to this effect. Unique combination of uncertainties related to technical objects and poorly understood natural systems greatly increases the risk of accidents. Examples of such large incidents are the disaster dam St Francis (USA, 1928), the Nurek hydropower plant (USSR, 1983), the Chernobyl (USSR, 1986), the Sayno-Shusheskaya power plant (Russia, 2009), and the Fukushima nuclear power plant (Japan, 2011).

Currently planned bridges over rivers and straits, including the Sognefjord crossing in Norway, belong to such structures with failure consequences which are comparable with the above. At the same time, they are placed a priori in the complex tectonic setting, inherent in rivers, straits and fjords. Typically, these areas are characterized by a high level of modern geodynamic movements affecting the long-term stability and security structures.

Therefore, regardless of the option selected for crossing bodies of water, the construction of its facilities should be performed within the parameters of modern geodynamic movements.

## 2 TECTONIC SETTINGS CROSSING

Bays, fjords and rivers are in origin confined to major fault zones in the rock mass, accompanied by numerous fissures of subordinate zones. As a result of the secondary structure that occurs in a hierarchical block of rock mass, some of these zones become boundaries of secondary structures, where current geodynamic movements are concentrated. According to the presently available experimental data, these movements can be up to 2-3 times the background value.

Bridges, that are located in the zone of active tectonic structures, may experience additional stress caused by high tectonic movements and deformations. They may take the form of displacement of pillars, bridge abutments and other forms. Figure 1 shows the tectonic structure of the area near the bridge over the Trans-Siberian railway in the city of Yekaterinburg. This bridge was destroyed in the final stages of construction [1].

In these conditions, stability and safety of the proposed bridge requires a detailed study of the tectonic structure of the surrounding territory mapping faults and the diagnostics of their activity.


*Figure 1 - The tectonic structure of the territory near the bridge in the city of Yekaterinburg*

## 3 MODERN GEODYNAMIC ACTIVITIES

Experimental research has identified two types of modern geodynamic movements - linear (creep) and cyclic. Linear movements occur in the form of reciprocal movements of adjacent building blocks of rock mass with a relatively constant speed and direction over a long period of time comparable to the lifetime of the object [2, 3]. Cyclical movements are composed of multiple alternating movements with different frequencies and amplitudes that occur in cycles [3, 4].

Linear movements can be either natural environments caused by tectonic movements along the boundaries of structural blocks, or man-made, caused by redistribution of stresses and strains in the rock mass under the influence of mining, extraction of groundwater resources and other factors. Linear values of displacements ranging from 0.5 mm / year for natural and up to 200 mm / year for man-made displacements are recorded by monitoring instruments [5].

Short-cyclical movements have a wide range of frequencies with cycle times of 30-60 seconds to 1 hour, several hours, days or more. Maximum vertical displacement reaches 85-100 mm, 50-65 mm horizontal, and the maximum strain tension-compression induced shifts may reach$1.2 \cdot 10^{-3}$, the incline $2.5 \cdot 10^{-3}$.

The combined impact of both types of geodynamic movements gives an array of rock and surface constant mobility, which is a natural form of existence of the geological environment (Fig. 2). Under their influence in the rock mass, which has a hierarchical block structure, dangerous geomechanical processes take place: degradation, secondary structuring, transition to the thixotropic state and concentration of geodynamic movements in the boundary areas of secondary structural blocks.

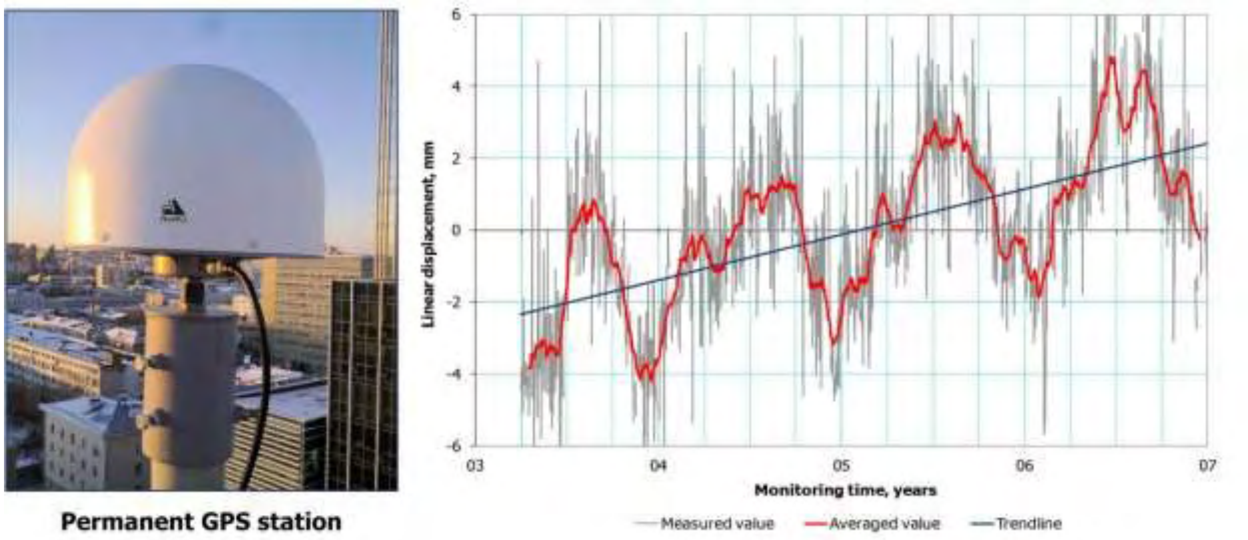


*Figure 2. Trend and cyclical geodynamic movement on the monitoring results for the permanent GPS station in Yekaterinburg*

Processes of destruction and secondary structuring are interconnected and inherent in the hierarchical block mobile medium with alternating stresses. Experimental research has shown that in these conditions secondary structures are developed in the form of consolidated volumes temporarily preserving the integrity of a particular time and the relative properties of the continuous medium. As the stress state is changed, the destruction of secondary structures units is consolidated to form new blocks. The lifetime of the consolidated units depends on many factors, including the structural features of the array, the parameters of the state of stress gradients and their variability. Moreover, the boundaries of the consolidated temporarily blocks are formed by structural defects of various ranks and in some cases may arise due to the destruction of the existing units. The borders of temporarily consolidated units are areas of concentration of modern geodynamic movements. The level of linear and short-cyclical movements in border areas is 2-3 times higher than the background movements, and in the inner regions of consolidated units it is reduced to 0.5 of the background movements.

In addition, tectonic disturbances that serve as joints between adjacent blocks, always have a certain thickness, depending usually on their rank. It can vary from a few centimeters to several meters and in major faults it can reach tens and hundreds of meters. Structure, strength and deformation properties of the rock mass in tectonic disturbance zones are very different from that of rocks in blocks. They are usually highly fractured, often degraded to a clay state, resulting in an array having low strength and deformation characteristics, although some rock samples based on these indicators may differ only slightly from those of the blocks.

## 4 THE IMPACT OF MODERN GEODYNAMIC MOVEMENTS ON THE TRANSITION

Subsurface objects, in the influence zone of active faults, are in potential danger of destruction from the effects of additional loads caused by modern geodynamic movements. [6] In accordance with the above laws and their distribution, the main danger for the designed structures are faults, dividing the area into structural blocks.

Catastrophic failure mechanisms of concrete structures depend on the prevalence of, or a combination of negative factors influencing on the structure, generated by geodynamic movements. Linear geodynamic motions and deformations are affecting structures by continually developing displacements and strains in the rock mass. When reaching the load and strain limit values for the structure, it can be destroyed.

In the case of cyclical movements the geodynamic destruction mechanism may create diverse and more catastrophic situations for the object . With the direct effects of the cyclic motion the object can be broken either by exceeding the allowable stresses and strains, or cyclic fatigue may occur in structural elements.

In the event of a tectonic disturbance in the rock’s thixotropic condition caused by cyclical movements, the object can be destroyed due to uneven deformation caused by thixotropy. In these cases, the destruction of the object can take place either by exceeding the allowable stress and strain from the combined effect of cyclic loads and loads caused by thixotropy, or from fatigue effects caused by the combined action of cyclic loads and thixotropy.

An example of the destruction of the bridge over the Trans-Siberian railway in the city of Yekaterinburg, caused by the combined effect of linear and cyclical geodynamic movements, is shown in Fig. 3.

*Figure 3. The destruction of the bridge in the city of Yekaterinburg*

**5 ESSENTIAL DIAGNOSTIC STUDIES**

Stability and the safe operation of a number of engineering structures, especially bridges across major water barriers require a careful study of the natural conditions of their building and operation site. Notable accidents and abnormal processes that have taken place on bridge crossings during the period from Takomsk (U.S. 1940.) to Volgograd (Russia, 2010.), indicate that the influence of natural factors studied is not exhaustive. This is especially true for the impact of modern geodynamic movements of bridges.

At the design stage major decisions need to be based on accurate information about the tectonic structure, parameters, geodynamic activity and its distribution in the construction area. To do this, along with the traditional engineering standard investigations, geological research should be carried out with special studies of the tectonic structure of the rock mass and the parameters of modern geodynamic movements (Fig. 4).

Experience has shown that general information about the tectonic structure may be provided by use of geophysical rock exploration methods [8]. A general idea of the tectonic structure of the area can be obtained using geoelectrical methods. This may be helpful in determining the location and direction of the strike of faults, their size and some information about the state of the rock.

Spectral seismic methods may be used to further refining the parameters of faults, the detection of sliding surfaces, fissured areas etc. down to depths of 200 - 300 meters below the surface. Detailing the structure of the upper part of the rock mass to depths of 20 - 30 meters where the foudation support for structures can be obtained, may be carried out by GPR sensing methods.

Linear movements are determined by changes in the spatial coordinate increments (vectors) $\Delta X$, $\Delta Y$, $\Delta Z$ between points of geodetic networks or specially equipped observation station frames, made at intervals between repeated cycles of measurements (Fig. 4). The use of complex satellite geodesy GPS and GLONASS can define displacement between points of geodynamic monitoring networks up to 1-3 mm in the single mode. With long-term continuous monitoring measurements the accuracy may be 0.1 mm / year [9]. The resulting deformation of instrumental observation intervals, using mathematical tools of continuum mechanics, can be converted into a tensor representation of the deformation field with display of the main components of the strain tensor [2]. If it is necessary to determine the magnitude and direction of linear movements, ground control frames supporting the observation station and providing absolute positioning of the global network of IGS may be used. Their spatial position is determined in a dynamic frame ITRF.

Defining the parameters of modern short-cyclical movements - their amplitudes and periods are also monitored using satellite geodesy systems by the method proposed in [3]. To perform instrumental measurements standing (permanent) GPS / GLONASS stations are used and specially designed frames provide temporary observation stations which accumulate satellite data over a long period - from a few hours to several years. The readability of received

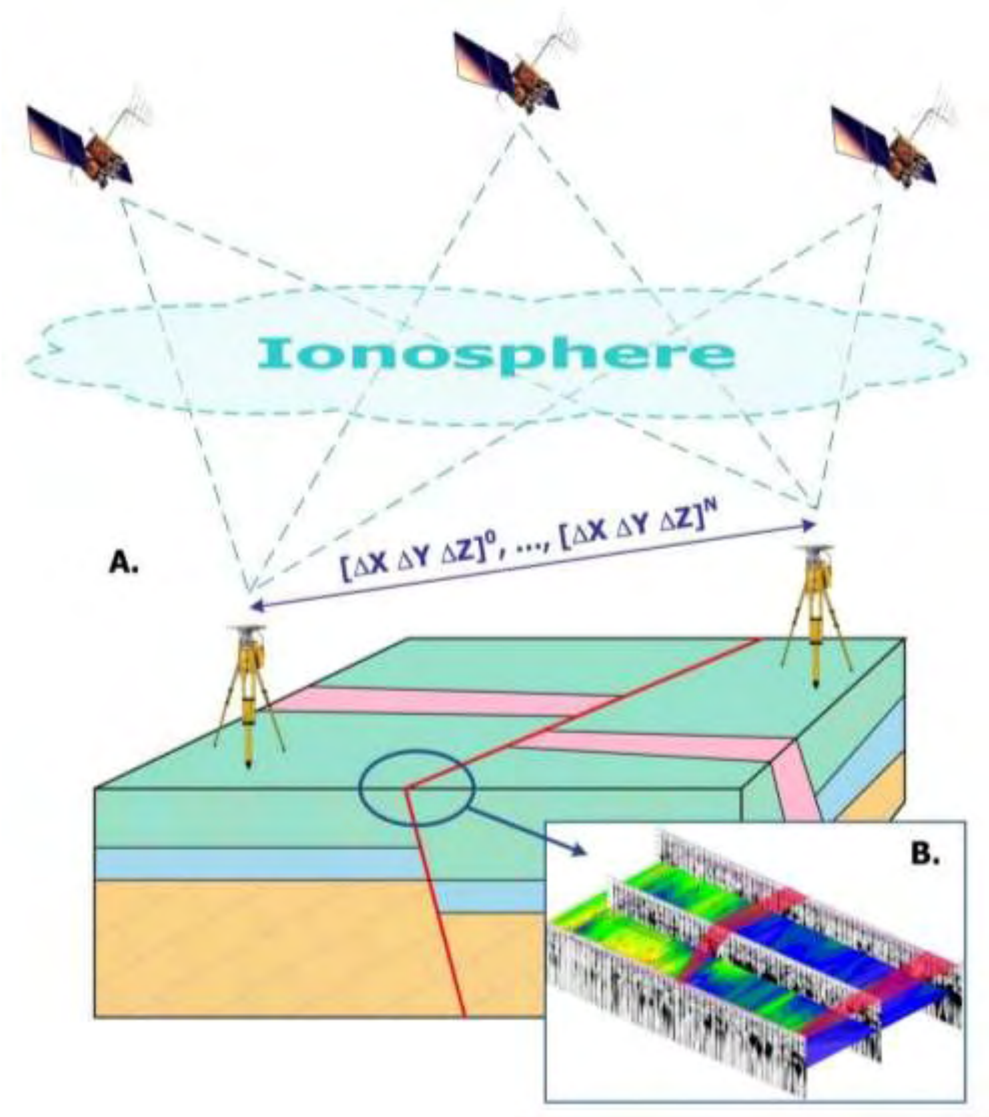


*Figure 4 - Methodology of modern geodynamic movements (A) and the identification of active tectonic structures (B)*

information about the parameters of the cyclic strains varies widely - from 10 to 1 • $10^{-5}$ Hz, with measurements, depending on the methodology used and are available in real-time and in post-processing form. The data, collected in real time, can then be reprocessed to reveal the laws of space-time fluctuations of the deformation field, setting the amplitude and frequency of cyclical movements.

Thus, the organization and conduct of instrumental parameter measurements in the monitoring mode of modern geodynamic activity - linear and cyclical, provides data on the amplitude and velocity of displacements and frequency of short-period strain. This can also be confirmed by the activity of the identified faults, based on a prediction of possible geodynamic activity of the rock mass, which has been the basis for design of engineering structures for a long period of time. Subsequent geodynamic monitoring during operation allows information to be obtained on the rates of growth of strains in foundations and major building structures, and if necessary, to develop measures to ensure the safe operation of the facility, and in the case of critical strain – to make a quick decision to stop the operation and evacuate the dangerous object.

## 6 CONCLUSION

The results of basic research of modern geodynamic movements made in recent years, as well as previous experience of operating bridges and other complex scale objects, shows that their safety depends on compliance with their design factors, among which an important role is played by the geodynamic movements.

Construction of unique bridges are invariably associated with getting into the influence zone of tectonic disturbances, possessing geodynamic activities, which in turn may be the source and cause of loss of stability.

Accident prevention may be based on the diagnostic results of modern geodynamic monitoring by choosing favorable construction sites or a constructive solution neutralizing the negative effects of geodynamic movements.

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# CROSSING OF WIDE FJORDS BY DOUBLE WALLED SUBMERGED FLOATING TUNNEL (SFT)

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## ABSTRACT

*This paper outlines a concept for crossing wide fjords by using a Submerged Floating Tunnel (SFT), more specifically the crossing of the Sognefjord in Western Norway. The SFT is shaped as a horizontal arch with a radius 3000 meters and length 4000 meters, and is fixed to landfalls on both sides of the fjord. Tension legs to the seabed vertically stabilize the SFT.*

*The tunnel cross section is a rectangular precast/ pre-stressed concrete box girder divided into two runs, each with two lanes of traffic, and an intermediate run for a pedestrian walkway and escape route. The cross section is double walled with bulkheads along the base, walls and ceiling. The outer cell structure is utilized as ballast tanks. The bulkheads will prevent possible leakage from filling large volumes.*

*The SFT is planned for construction in a dry dock in appropriate lengths. The structural elements are towed to a tranquil fjord to be assembled. The assembly is carried out with the SFT buoyant. When the assembly is completed, the SFT is ballasted, submerged and stabilized vertically by temporary pontoons. The assembly is then towed to the site, where the construction of the landfalls, temporary and permanent mooring system is established.*

*Buckling loads and natural periods of the arch structure are presented together with the results from global analysis with beam elements. A frame element model is presented showing local load distributions in transverse direction.*

## INTRODUCTION

The Coastal High Route E39 stretches from Kristiansand to Trondheim along the west coast of Norway. Between these two towns, the road crosses seven fjords. Several of these fjords are wide and deep and are, in addition, prone to the elements (wind, waves, currents). SFTs can be a cost favourable solution in such places.

This article deals with crossing the Sognefjord with an SFT. The Sognefjord is 3700 m wide at its narrowest point with a depth mid-fjord of approximately 1250 m. The solution may also be relevant for several other fjord crossings on the E39 route.


7

*Figure 1 Possible SFT location at Oppedal - Lavik.*

So far, no SFT has been constructed. The main reason for this may be the uncertainties related to the reliability of this type of structure, and also with the uncertainties related to the load conditions during construction and installation. Great importance has, therefore, been attached to developing a concept with a high degree of certainty in both the construction and operational phase.

## CONCEPT

The SFT has a horizontal arch-shape with a radius of 3000 m and is vertically retained by tension legs to the seabed. The SFT is connected to tunnels in rock at both ends. The length of the SFT is 4000 m. The SFT is submerged 25 meters below the surface in the middle of the fjord and 12 meters close to the landfall. The arch-shaped placement of the SFT enables greater resistance against horizontal loads (currents). The arch geometry also means that constraints from movements due to temperature, creep and shrinkage are more manageable. The use of a tension leg mooring system instead of pontoons reduces wave forces acting on the SFT, and gives no actions caused by tidal.

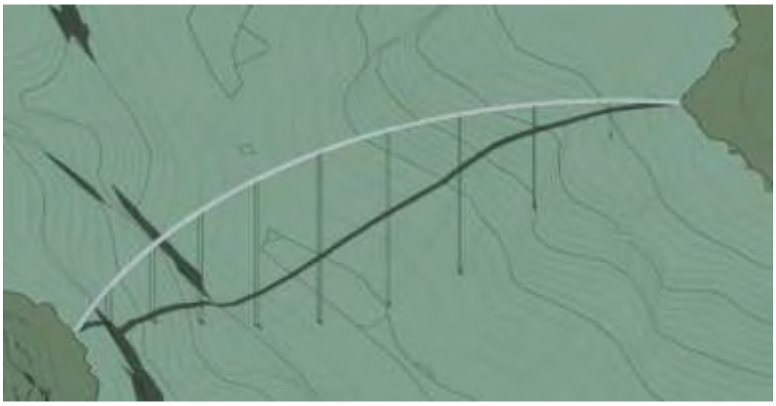

*Figure 2 SFT crossing Sognefjorden*

**Tunnel cross-section**

The cross-section (see figure 3) is a rectangular box section in concrete, post-tensioned by longitudinal and transversal tendons. It has two driving lanes in each direction and a pedestrian and bicycle lane in the middle. This lay-out will ensure evacuation possibilities in the event of traffic accidents or fire. Requirements for space and cross-section area of the SFT are given in [2]. The cross-section is double walled with bulkheads along the base, walls and ceiling. Some of the tanks between the bulkheads are utilized as ballast tanks. The outer walls are designed for the occurring water pressure. The inner walls are designed for explosion/fire loads and will also have the capacity to support water pressure, in the case of the outer wall being damaged by accident loads. The cross bulkhead with a centre distance of approximately 5 m length-wise forms a rigid framework around the cross-section that will carry water pressure and counteract transverse deformations caused by bending of the rectangular section in horizontal curvature (distortion).

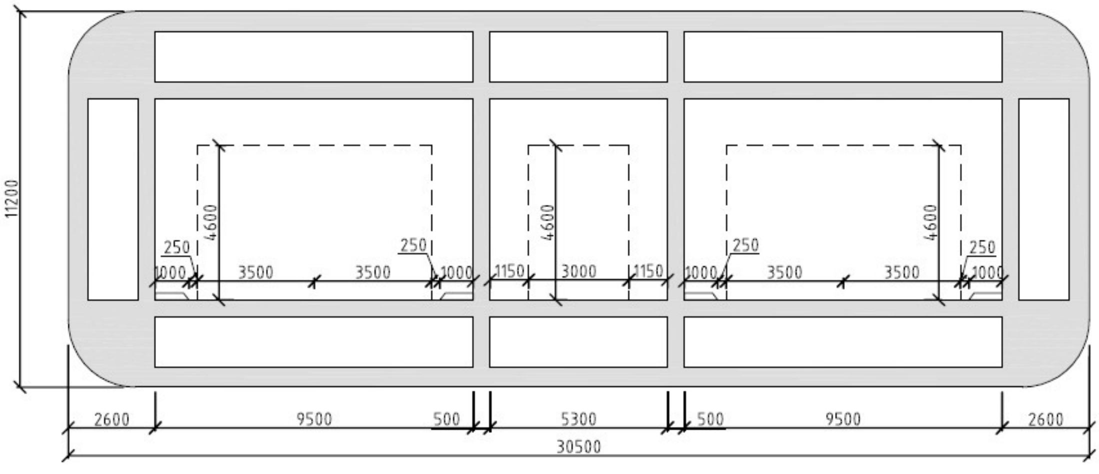


*Figure 3 SFT cross-section*

**Anchoring system**

The tunnel is kept stable vertically with positive buoyancy retained by vertical tension legs attached to the seabed. The distance between the retaining points can vary between 250 and 350 m depending on the topography and soil conditions on the seabed. At least 4 rods are attached to each side of the SFT. The tension rods are made of steel tubes Ø711 and must be treated to avoid corrosion. Attachment of the tension rod to the SFT cross-section is carried out by an anchor beam at the end of the rod. The beam is inserted in a console as shown in figure 4. When ballast water is pumped out, the rods are tensioned. Hydraulic jacks are installed in order to measure and adjust the rod force.

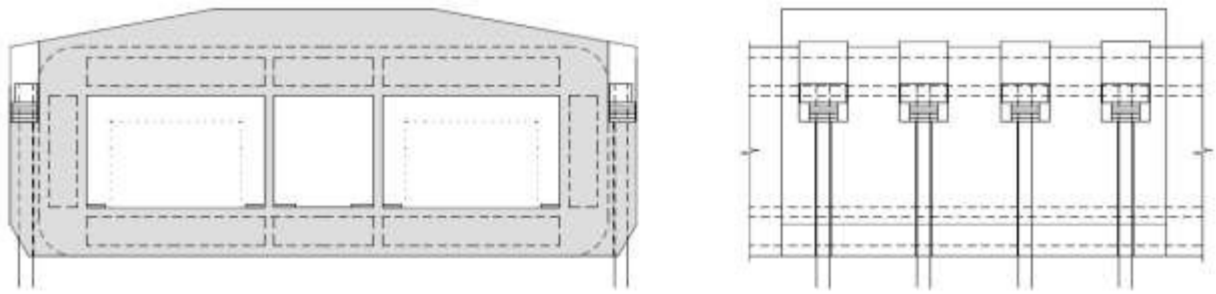

*Figure 4 Fastening tension rods to tunnel cross-section*

The SFT and anchoring system is designed for individually replacing rods, thus allowing the tunnel to remain open for traffic while this work is taking place.

Instead of a tension leg anchoring system a solution with pontoons can be considered as an alternative concept.

**Soil conditions/foundation**

The seabed at mid fjords in the Sognefjord consists of soft normally consolidated clay. Such soil conditions may require piled foundations or suction anchors to take up the tension forces from the SFT. Alternatively the piles or suction anchors can form a foundation for a gravity based anchorage. Rock anchors are used where the seabed is steep and has limited soil cover.

The selection of an optimal foundation method must be made on the basis of accurate mapping of the seabed in the actual anchorage areas and geotechnical soil investigations. Geotechnical and geological conditions must be investigated and documented in detail.

*Figure 5 Gravity based anchorage*

## CONSTRUCTION

### Building in dry dock

The SFT is planned for construction in a dry dock. One possible location is the industrial area at Lutelandet, which is currently under construction with a dry dock and wharfage. Elements with length up to 350 m can be fabricated here. Geometry and weight control are to be performed, and water tightness is tested.

### Towing from dock to a fjord arm for assembly

After fabrication in the dock, the elements are towed to a suitable sheltered fjord, where the SFT-elements can be assembled. The Fuglesetfjord at Bjordal is a suitable location for this procedure. The fjord is narrow, and protected from the environmental loads and has excellent anchoring possibilities. The location is also sheltered from ship traffic.

The splicing of the SFT-elements involves the use of a habitat (dry room/tunnel around the submerged part of the tunnel cross-section). A steel frame is mounted in the moulding joint to prevent movement between the elements in the casting and binder phase. All joints are tested for water tightness.

Once the SFT is fully assembled, it is weighed down at a slight angle to the water surface. Temporary pontoons (barges) are positioned over the SFT as it is gradually lowered, until its entire length is under water.

### Towing and installation at operational depth at the SFT site

The SFT is planned to be transported whilst held submerged with pontoons during the towing to the installation site. It is expected that ten large tugboats will be required to conduct the operation in a safe manner. After the SFT is positioned in a preliminary lowered position, held up by pontoons and temporary mooring systems, it is connected to the tension rods and pulled down to the target depth.

### Construction of landfalls

Landfalls will be established on the Oppedal and Lavik sides of the fjord. This includes:

- Establishment of two two-lane access tunnels in rock that join together at landfalls, as well as an access tunnel to the pedestrian walkway.
- Required concrete structures in order to clamp the SFT in the bedrock.

Geological engineering field studies and investigations will be required in the planning and engineering of the landfalls.

## SAFETY

### Construction safety

The SFT with tension rods to the seafloor is considered to have a high level of structural safety and reliability. Ship traffic will be able to pass unhindered by the SFT. The double cross-section is highly robust against collision damage caused by submarines, sinking ships, trawlers, accidents due to anchors and other possible accident loads. Any water leakage through the outer wall will be limited to filling one tank and the cross section will be able to maintain its functionality. The SFT is also designed to withstand failure of one of the tension rods.

The concrete is expected to have a 100-year service life. Concrete protection systems are installed for increased durability.

### Traffic safety

In the event of traffic accidents, the three separate runs in the SFT allow for evacuation from one run and over to the pedestrian/bicycle area or to the opposite run. Should one of the runs be closed due to accidents or maintenance, the other run can be opened for two-way traffic for short periods.

## STRUCTURAL ANALYSIS

The SFT is designed for the following static loads: dead weight including marine growth on the exterior of the SFT, water absorption, hydrostatic pressure, buoyancy, deformation loads (shrinkage, creep, temperature variations, differential settlements), sea current and traffic loads. In addition, dynamic loads from waves and vortex shedding from currents must be calculated. In the case of an emergency, it is of utmost importance to design for explosion loads, fire, and loss of buoyancy and rod breakage.

In the preliminary design presented in the following, the SFT is modelled as a spatial beam. The distance between the anchor points is set to 300 m.

**Net buoyancy**

Requirements for the calculation of dead weight and buoyancy are given in [1]. Consideration must be given to the variation in water density due to temperature and salinity variations of the water in the fjord, in addition to the SFTs mass and geometry. The minimal net buoyancy to avoid loss of tension in the rods in all load combinations is 100 kN/m. Maximum net buoyancy for unfavourable gravity density values, is calculated to 300 kN/m. Closer to the land connections, net buoyancy can be somewhat reduced compared to these numbers. This reduces the restraining moment in the moorings.

**Sea current and wave loads**

Sea current analyses are based on values given by SINTEF [4]. A designed sea current velocity of 0.7 m/s has been used in the analysis. The current load is calculated from the drag component in Morison's equation and found to be 4 kN/m. Analyses have been performed with uniform current in the fjord, uniform current acting in the mid-half of the fjord width, and analyses with a shear current with uniformly distributed velocity in each direction over each half of the fjord width. The significant wave height in the Sognefjord is 2.34 m with a corresponding wave period of 4.8 s. The maximum wave height is 4.79 m, see ref. [4].

**Traffic loads**

SFT is calculated for traffic loads according to Eurocode 1 Part 2. Uniformly distributed traffic loads for 4 lanes + walkway is 53.7 kN/m, and 1200 kN for concentrated loads. Horizontal braking and acceleration force is 900 kN in longitudinal direction and 225 kN in transverse direction.

**Results, global effects**

In the following, results from the structural analysis are presented for some of the most important load conditions. A diagram of the bending moment along the horizontal axis for maximum net buoyancy is shown in Figure 6. Bending moment from traffic loads is shown in Figure 7. Figure 8 presents the vertical bending moment for a shear current with uniformly distributed velocity in each direction over each half of the fjord width.

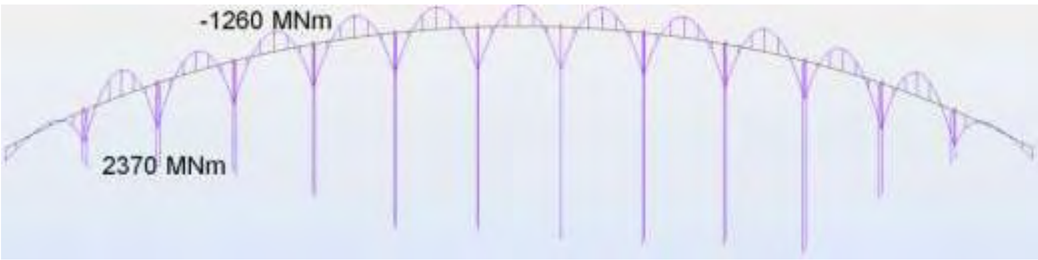


*Figure 6 Horizontal bending moments for maximum net buoyancy of 300 kN/m*


*Figure 7 Horizontal bending moments for traffic loads*

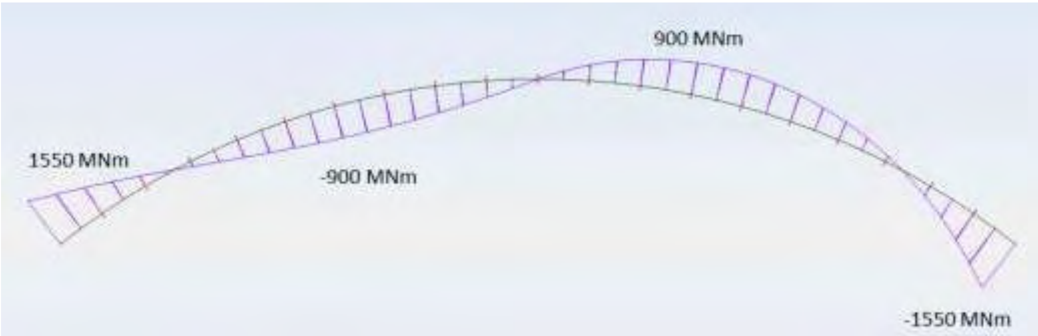


*Figure 8 Vertical bending moments for "shear current"*

Evenly distributed current of 4 kN/m in the fjord gives an axial force in the SFT of approximately 13 MN. This is small compared to the buckling load which is 990 MN for the unsymmetrical buckling mode as shown in figure 9.

*Figure 9 Buckling mode number 1 (in the horizontal plane)*

The eigenperiods of the SFT have been calculated. The first eigenperiod is 52 seconds (sway, horizontal translation). The first eigenperiod in heave (vertical translation) is 6 seconds.

The load combination in Ultimate Limit State which gives largest horizontal bending moments is shown in figure 10.

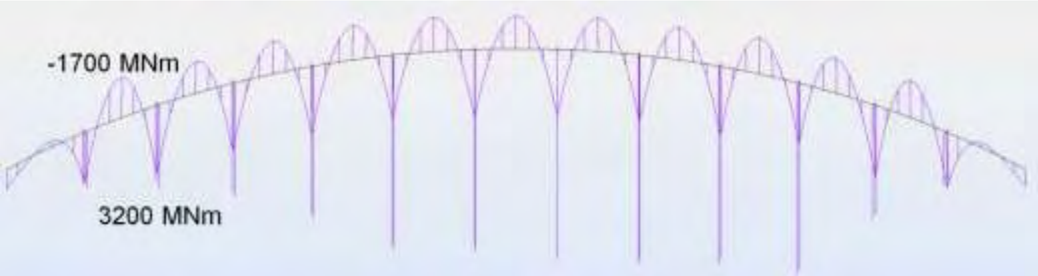


*Figure 10 Horizontal bending moments, ULS*

**Local load effects**

For local analyses, where consideration is given to water pressure and effects that give cross-deformation of the rectangular tunnel cross-section, a frame analysis has been performed, see figure 11.

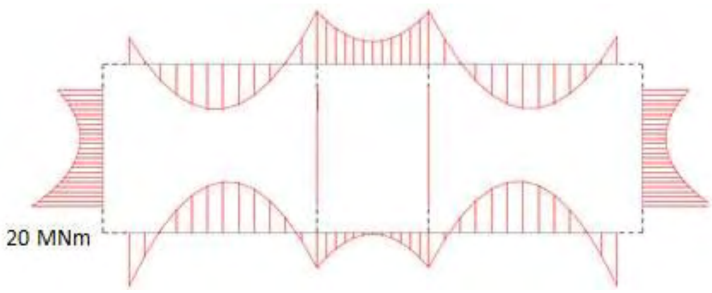


*Figure 11 Frame model* *Moment distributions, ULS*

**CONCLUSIONS**

Based on the performed study it can be concluded that a submerged floating tunnel with rectangular, double walled cross section, vertically stabilized by tension legs, is technically feasible for crossing the Sognefjord.

Great importance has been attached to developing a concept with a high degree of certainty in both the construction and operational phase.

Preliminary analyses indicate that the proposed SFT concept has sufficient capacity for all subjected loads.

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# INTEGRATING RENEWABLE ENERGY INTO BRIDGE CONSTRUCTIONS

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## ABSTRACT

*The possibility of integrating the energy production devices into the bridge structures along the Coastal Highway Route E39 has been investigated. Feasibility studies have been carried out by The Norwegian Public Roads Administration (NPRA), Norconsult, Rambøll and SP Technical Research Institute of Sweden resulting in five reports. The studies assessed the renewable energy potential and studied how bridge structures can be utilised for power generation.*

*The main advantage of integrating energy conversion devices in a bridge structure is the reduction of costs compared to stand-alone devices. The construction could be used as a foundation, a mooring point and provide dry environment for the electrical parts of the devices. Easy access to the production site could also reduce the cost for installation, operation and maintenance. The main challenge would be to find the location with most optimal energy resources.*

*Integrating energy conversion devices in bridge structures may have negative impact on the environment and on the bridge construction. Most of the devices studied would mean additional loads due to forces from wind, wave and tidal current or from their own weight. The potential environmental impacts associated with renewable technologies are the consequences for bird life or marine fauna at the fjord crossing locations, as well as noise and visual impact.*

*Due to the low amount of direct sunlight in Norway it is not found any added value of integrating solar panels to the bridge constructions along E39. The most suitable location for wind energy production is at the Boknafjord crossing, where it seems that integrating ducted augmented wind turbines in the roadway could be the most preferable solution.*

*There is indication for great potential for extracting power from wave and tidal current if the wave and tidal current energy flux density are powerful enough. But lack of energy resource data made it impossible to estimate adequately the expected power production from wave and tidal current. It is expected that the investment cost could be reduced by 40% if the devices could be integrated into the construction. There is need for further study to collecting energy resource data at Boknafjord, Moldefjord and Bjørnafjord in order to predict more adequate the potential for electricity power production.*

*The situation in the energy market has not been considered in this study, but should be taken into account in future studies. Energy prices and subsidised measures for renewable energy are vital factors when determining the viability of renewable energy production.*

## BACKGROUND

The E39 on the west coast of Norway connects Kristiansand in the south to Trondheim in central Norway. The Ministry of Transport and Communications has given the project "Coastal Highway Route E39" a mandate to assess technological solutions for fjord crossings. The project has also been mandated to investigate how the bridge constructions can be exploited to produce energy from wind, sun, waves and tidal currents. The project consists of four components:

- Social impacts
- Fjord crossings
- Energy
- Implementation strategies and types of contracts

Norway is a substantial producer of renewable energy, due to abundant hydropower resources (more than 99% of mainland Norway's electricity production is from hydropower plants). Even so, there is still vast untapped potential in wind, tidal, and wave power. Producing electricity from additional renewable energy sources will reduce the Norwegian carbon footprint and contribute to the Norwegian government's goal of carbon neutrality by 2050.

One of the challenges for renewable energy is to develop technologies that are cost competitive with non-renewable energy. In addition, there are technical challenges associated with large-scale exploitation of energy sources. Using the construction as part of the energy-producing facility can probably reduce the establishment costs and thus increase the potential for the development of renewable power plants.

## OBJECTIVE

The main objective has been to investigate the possibilities of using bridge constructions and submerged floating tunnels to increase the potential for producing renewable energy. Two feasibility studies have been carried out by Rambøll, Norconsult and SP Technical Research Institute of Sweden. The purpose of these studies was to investigate the potential for producing energy by integrating devices into the road infrastructure on fjord crossings and to explore the technical challenges and opportunities.

## FEASIBILITY STUDY

A literature survey was carried out earlier to identify the technological solutions currently available for renewable energy. Completion of this survey, based on mapping "state of the art" technologies, was part of the feasibility study. The suitable technologies were mapped and complementary information was collected from different technology providers. This provided information about established knowledge, research and projects that are in progress or already realised. The study was divided into two parts:

- Feasibility study for renewable energy from wind and sun
- Feasibility study for renewable energy from waves/current and tidal energy

Access to energy resource data is essential when investigating the potential for energy production from each source of energy. This data is also necessary in order to evaluate which of the energy production alternatives would be most suitable for each fjord crossing. Both the literature survey and the work of gathering the available data were part of the feasibility studies. The feasibility studies have been published as separate NPRA reports [1], [2], [3], [4] and [5].

The following crossing locations have been considered in the feasibility study:

- Halsafjord (Kanestraum – Halsa)
- Moldefjorden (Vestnes – Molde)
- Storfjord (Festøy – Solavågen)
- Voldafjord (Volda – Folkestad)
- Nordfjord (Anda – Lote)
- Sognefjord (Opedal – Lavik)
- Bjørnafjord (Sandvikvågen – Halhjem)
- Boknafjord (Mortavika – Arsvågen)

Two alternatives have been proposed for crossing the Bjørnafjord, one short and one long.

The study investigated the impact the additional loads from each proposed technology might have on the bridge construction. However, there is a need for further studies to estimate the exact impact on the construction. This is especially important for long bridge constructions that face the challenge of withstanding horizontal loads from wind, currents and waves.

As with all energy resources, renewable energy resources may also affect the environment. For the time being we have few real world examples and limited experience from bridge constructions equipped with power production devices. The primary challenges are noise and visual impact, the effect on the marine environment and marine fauna or bird life.

## WIND ENERGY

The physical overall wind power potential in Norway is estimated to be thousands of TWh/year, but a great deal of the potential is not available for exploitation because of environmental and economic circumstances [8]. A good commercial wind site should have an average wind speed in the region of 7 – 11 m/s [9]. It is common for wind turbines to start producing power at 3 – 4 m/s [10]. Full load hours vary from almost 1000 hours in the Nordfjord to more than 3000 hours in the Boknafjord, making these two fjords the most suitable locations for wind power [3]. The average wind speed at 50 metres height for the different crossings varies from 8 m/s above the Boknafjord to 5 m/s above the Nordfjord [11].

Onshore wind technology is relatively mature and well developed, while offshore technology is still under development. The offshore technologies are more of interest for this project, since bridges located in the fjords would face similar environmental challenges, such as bottom-fixed foundations due to the great depths. There are three main types of wind turbines that could be of interest for the E39 project:

- Horizontal Axis Wind Turbines (HAWT)
- Vertical Axis Wind Turbines (VAWT)
- Ducted Augmented Wind Turbines (DAWT)

The improvement of DAWT-technology will continue to be in focus, as there are many advantages that could be expected from this type of turbine compared to the traditional horizontal turbines. A number of ideas have been launched for how to combine wind turbines and bridge constructions, for example placing the turbines on the extended pontoons of a floating bridge [1] or integrating DAWTs in the roadway [5].

The estimated power production for the crossing locations varies from 0.3 – 2.6 GWh/km/year for suspension bridges and 0.3 – 30 GWh/km/year for floating bridges. The greatest potential for power production is at the Boknafjord crossing. Table 1 shows the estimated energy production at the Boknafjord crossing (assuming 8.4 km length [5]).

*Table 2. Estimated energy production from wind power*

| Turbine type | Floating bridge [GWh/km/year] | Floating bridge [GWh/year] |
|---|---|---|
| HAWT (small) [1] | 2.7 | 23 |
| HAWT (large) [3] | 30 | 252 |
| VAWT [3] | - | - |
| DAWT [5] | 1.4 | 12 |

The highest value for floating bridges is based on installation of HAWTs with a diameter of 101 m placed every 250 m bridge [3]. This alternative will subject the bridge to extreme additional loads, which is considered unrealistic at this stage. For suspension bridges, DAWTs integrated in the roadway seem to be the best alternative [5]. However, suspension bridge is not possible at Boknafjord crossing. For detailed calculations, see references [1], [3], [5] and [7].

For most of the suggested technologies, the added value and synergies gained by integrating wind turbines in the construction is expected to be low. The costs of integration will most likely be increased due to necessary modification and increased loads. However, there is an exception. Using DAWTs integrated in the construction could be more cost effective and has great potential for power production [5]. Other options that might be viable are to attach turbines as shovels around the pillars or installing VAWTs in the zone near shore [1].

If wind turbines are to be integrated in a bridge construction it is important that this is taken into account in the design phase of the project.

## SOLAR ENERGY

Typical solar irradiance in Norway is 600 – 1000 kWh/m2/year [12], which corresponds to about 1500 times the total annual energy consumption. The potential for exploiting this energy is vast, but little direct sunlight limits the actual power production potential. However, there is much indirect, diffuse sunlight that might lead to some production.

There are two main types of solar energy conversion technology. Solar thermal technology uses the sun for heating purposes by placing collectors (e.g. water-filled tubes) in direct sunlight, while photovoltaic (PV) panels convert solar radiance into electrical energy. Thin-film panels have lower efficiency than crystalline silicon panels, but thin-film panels can utilise both direct irradiance and diffuse radiation [3].

PV panels would be more suitable than solar thermal installations, since these fjord crossings are located in sparsely settled areas; the heat generated would have to transported long distances, which is not a viable option. Electricity, on the other hand, can easily be connected to the grid and transferred to a remote point of use. Thus such restrictions do not apply for PV panels. However, solar water heating could be used for tap water heating, defrosting and snow melting that may be needed on or near the bridge [2].

Table 2 presents the estimated power production for different mounting alternatives of the solar panels. The calculations have been carried out by SP and are based on at 45° tilt angle for optimal utilisation of the irradiance. The assumption is that there is no large variance for the estimated power production at the different fjord crossing locations.

*Table 3. Estimated energy production solar power*

| Type of mounting | Suspension bridge [GWh/km/year] | Floating bridge [GWh/km/year] |
|---|---|---|
| **Side mounted** | 0.6 | 0.6 |
| **Roof mounted** | 1.9 | 1.7 |
| **Mounted on wires** | 8.4 | - |
| **Mounted on pylons** | 0.3 per pylon | - |

Solar panels mounted on the wires of a suspension bridge give the greatest potential for power production. However, this alternative would most likely increase the additional wind loads on the bridge structure and therefore is not recommended. A solar power plant could be easily installed on a bridge with relatively small added value. The solar panels could be mounted directly on the construction without using any other mounting systems if this is planned in the design phase. This would reduce the total costs. For detailed calculations, see references [1], [2], [3] and [7].

Solar panels on the bridges along the E39 are unlikely to be feasible as production systems for delivery to the grid based on the today's solar power technologies and market. If the panels are mounted on the sides, as a roof, on the pylons and on the wires, the energy production would be in the acceptable range. The panels mounted on wires would account for around 75% of the total production [2], but since this is the mounting alternative that gives the greatest additional load, all-round mounting is not an optimal solution. However, pilot installations could be of interest in order to gain new knowledge through experience and to contribute to the further development of solar panels.

## WAVE ENERGY

Wind-generated waves are a huge, largely untapped energy resource, and the potential for extracting energy from waves is considerable. Waves have a very high energy concentration and are a natural store of kinetic energy that can travel thousands of kilometres with little energy loss [15]. Wave power is one of the most abundant energy sources on Earth. It is estimated that the contribution of wave energy to the Norwegian energy portfolio could reach between 12 to 30 TWh per year [14].

Waves vary with time and location. Wave energy scatter diagrams typically contain information about annual distribution of the significant wave heights ($H_s$ in metres) and peak wave period ($T_p$ in seconds). Data from at least one year is required to ensure that seasonal differences in the energy flux are accounted for.

It is impossible to do exact calculations of the expected power production, as there is not sufficient wave statistic data from the fjord crossing locations. However, rough calculations have been carried out by SP based on the rated power and the number of installed WEC devices. The results presented in table 7 are based on the most suitable and applicable devices, listed in [4], which could start producing power when the significant wave height is below one meter. These calculations are not adequate, but they do give an indication of the expected power production (if the significant wave heights are high and long enough, $H_s > 1$ m and $T_p > 3$-4 sec).

The results show that the highest potential for power production is found in the Moldefjord, the Boknafjord and the Bjørnafjord (long crossing). Assuming that the crossing with the greatest wind speed also will have the most powerful waves, the Storfjord crossing could also be a crossing location with a potential for extracting wave energy. It is most likely that the power production would be in the middle or lower range of these intervals. However, it should be expected that the significant wave heights would be more than one metre in the Boknafjord. The results show that there is significant potential for producing energy from waves if the significant wave height is more than one metre. For detailed calculations, see references [1], [4] and [7].

*Table 4. Estimated potential for WEC devices producing energy at $H_s$<1 m and $T_p$<3-4 s.*

| Crossing | Suspension/ floating bridge [GWh/km/year] | Submerged tunnel [GWh/km/year] | Suspension/ floating bridge [GWh/year] | Submerged tunnel [GWh/year] |
|---|---|---|---|---|
| **Halsafjord** | 15-193 | 13-163 | 27-347 | 23-293 |
| **Moldefjord** | **20-235** | **17-200** | **160-1880** | **136-1600** |
| **Storfjord** | 18-218 | 16-185 | 61-741 | 54-629 |
| **Voldafjord** | 16-198 | 13-168 | 32-396 | 26-336 |
| **Nordfjord** | 15-188 | 13-160 | 26-320 | 22-272 |
| **Sognefjord** | 18-221 | 16-188 | 67-843 | 59-717 |
| **Bjørnafjord (short)** | 15-184 | 13-157 | 24-295 | 21-251 |
| **Bjørnafjord (long)** | **20-230** | **16-196** | **114-1320** | **91-1122** |
| **Boknafjord** | **20-236** | **17-201** | **168-1986** | **143-1688** |

The costs of installing WEC devices depend on the device chosen. There are many different types that are already on the market or being developed. A production cost of 1.5 – 3 NOK/kWh is expected, based on experience with the market [1]. The costs for WEC devices would be reduced if they are integrated into the bridge construction. The cost reduction could be expected to be about 40%, which corresponds to a production cost of 0.9 – 1.8 NOK/kwh [15].

There are advantages when combining WEC devices and bridge constructions, especially in terms of cost reductions. The main challenges are the reliability of the energy resource and the additional loads on the structure. It is crucial that the integration of WEC devices be included in an early phase of the bridge design.

## TIDAL ENERGY

The tides, being a result of the periodic variations of the Earth-Moon-Sun system, are more predictable than other renewable energy sources. The predictability of tidal energy is seen to be a major advantage in light of the variations in generation and consumption across the electrical grid.

The theoretical global potential for exploiting the tidal currents for power production has not been calculated precisely, but it is estimated to be in the range of 1000 TWh/year. In 2008 Norway used 228 TWh, and of this the NPRA used around 1 TWh [16]. Two studies of the technical tidal current energy resources of the Norwegian coast have been carried out. Based on these studies, the estimated potential for annual tidal power production is between 0.5 TWh and 1 TWh [14], [17]. In our study, the main focus was on kinetic tidal current energy, which was not considered in the above studies.

Technologies that convert kinetic tidal current energy are referred to as Tidal In-Stream Energy Conversion (TISEC) devices, Marine Current Energy Converters (MCECs) or marine current turbines. The optimum current velocity for traditional technologies is between 1.5 – 3.5 m/s, but several technologies have been developed for velocities down to 0.5 m/s.
Most of the TISEC devices are similar to wind turbines. Some common design principles are presented below; for more technical information, see [4].

- Horizontal axis turbine: similar to vertical axis wind turbine
- Cross-axis turbine: similar to horizontal axis wind turbine
- Enclosed tips turbine: similar to ducted augmented wind turbine
- Oscillation hydrofoil: the passing water generates a lift on the hydrofoil which makes it oscillate

Several ideas have been launched as to how to combine TISEC devices with a bridge construction, for example by mooring the device directly to the bridge or mounting the device directly on a submerged floating tunnel.

There is a great lack of data on currents and tidal current velocity at the sites of interest, which makes it impossible to do exact calculations of the expected tidal power production. The available data indicates an average current velocity of less than 0.25 m/s, with a maximum of 1 – 1.5 m/s [18]. We could not find any data available for Moldefjord, Voldafjord, Nordfjord and Boknafjord. However, a rough estimation has been carried out by SP for these four crossings based on the number of devices and their rated power. The results presented in table 4 are based on the possible number of devices integrated in the construction and their nominal rated power. Only devices that are designed to produce power when the current velocity is below 0.5 m/s are included [4]. This is a rough calculation to give an indication of the expected power production if the tidal current velocities are sufficiently great ($v_{current}$ > 0.5 m/s).

*Table 5. Estimated potential for TISEC devices producing energy at v< 0.5 m/s*

| **Crossing** | **Suspension/ floating bridge [GWh/km/year]** | **Submerged tunnel [GWh/km/year]** | **Suspension/ floating bridge [GWh/year]** | **Submerged tunnel [GWh/year]** |
|---|---|---|---|---|
| **Moldefjord** | **16-105** | **13–89** | **128–840** | **104–712** |
| **Voldafjord** | 13-88 | 11–74 | 26–176 | 22–148 |
| **Nordfjord** | 12-83 | 11–71 | 21–145 | 19–142 |
| **Boknafjord** | **16-105** | **13–89** | **136–893** | **111–757** |

Moldefjord and Boknafjord are the crossings with the highest potential for tidal energy transformation. It is likely that the power production would be in the lower range of these intervals, since the calculations are based on nominal power production, which is usually at vcurrent > 0.5 m/s. Taking this into account, the results still show significant potential for retrieving energy from tidal currents. For detailed calculations, see references [1], [4] and [7].

There are many possible advantages of combining tidal conversion devices with a bridge construction, particularly in terms of cost reduction. The main challenge would be to find the location with sufficient current velocity. The technical challenges are the additional load on the structure and the lack of experience and knowledge about the use of current turbines integrated into an infrastructure. It is crucial that the integration of TISEC devices be included in an early phase of construction design.

In the vicinity of the E39 fjord crossing locations there are several strong tidal currents where it may be possible to produce tidal energy with turbines. Examples of such sites are Skjoldastraumen in Rogaland, Lukksundet in Hordaland, Skodjestraumen near Ålesund and Vestnesstraumen in the Romsdalsfjord. In the most narrow and shallow sections of these courses, the current velocity may be in the range of 2 – 3 m/s. This corresponds to an energy flux of 4 – 14 kW/m2, but the cross-sectional areas are relatively small, limiting the potential amount of produced energy [18].

## CONCLUSION

Power production in connection with bridge constructions poses many challenges. The greatest challenge, and the one that is more or less impossible to overcome, is the limitation imposed by the selection of bridge site location. With this in mind, it is crucial to choose energy production devices which are optimised for the existing conditions at the site. It is also very important to plan the integration of devices in an early design stage of the construction in order to optimise added value.

When considering power production from wind and waves, the Norwegian climate has a great advantage. The strongest winds and most powerful waves occur in the winter, when the national energy demand is at its highest. The opposite applies for solar energy, which means that the energy generated needs to be stored or additional energy sources are needed during the winter months. The tidal currents vary with the phase of the moon, but there are also some seasonal differences. In general the variation in water level is higher during the winter than in the summer, but the extent depends on local conditions [20].

The most suitable locations for wind power production are at the Boknafjord and Storfjord crossings. The wind average speed could reach 7-8 m/s and the full load hours could be more than 3000 hours at Boknafjorden. The added value from integrating wind turbines in the construction is expected to be low for most of the suggested technologies. The costs of integration will most likely be increased due to modification and increased loads. However, there are some exceptions. Integrating Ducted Augmented Wind Turbines (DAWT) in a bridge construction, for example, could be very cost effective and increase the potential for power production.

Due to the limited amount of direct sunlight in Norway, integrating solar panels to the bridge constructions along the E39 does not add value. Solar panels mounted as a roof over the roadway could produce almost 2 GWh/km/year, but the investment costs would be very high. However, it could be of interest to investigate the potential of solar power in the future, since cost reduction and efficiency enhancement are expected for solar power during the next 5 – 10 years.

The energy conversion technology for waves and tidal currents is relatively immature compared to other renewable energy technologies. Increased efficiency and decreased costs are expected within the next 10 – 20 years. Energy conversion technology for wind and solar power are more mature, and improvement of existing technology is expected.

For both wave and tidal currents there is a great lack of energy resources data. It is crucial that further work is focused on collecting energy resources data. The prospects are best for producing wave power, since the investigations into tidal currents at some of the fjord crossing locations have concluded that the current velocities are below 0.25 m/s. There are few energy conversion devices that start producing power from 0.5 m/s and upward. The power output for devices operating in this range is expected to be low. However, harnessing tidal energy by using current turbines is closer to a commercial breakthrough than wave power plants [1].

Lack of energy resource data has made it impossible to estimate adequately the expected power production from waves and tidal currents. SP has made some rough estimates based on the number of devices and their rated power, showing power production in the range of 20-236 GWh/km/year from waves and 16-105 GWh/km/year from tidal currents at the fjord crossing locations Boknafjord, Moldefjord and Bjørnafjord. This is based on the assumption that the significant wave height is greater than one metre and the current velocity is greater than 0.5 m/s. Most likely the potential for power production would be in the lower range of this interval.

It is also expected that investment costs could be reduced by about 40% if the devices could be integrated into the construction. These predictions indicate that there is substantial potential for extracting power from the fjords if the waves and/or the currents are powerful enough. Further study is recommended, in order to collect the energy resource data at Boknafjord, Moldefjord and Bjørnafjord. This would make it possible to estimate more adequately the feasibility of power production from waves and tidal currents.

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# FLOATING BRIDGE WITH HIGH BRIDGE – A CONCEPTUAL PRESENTATION

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## ABSTRACT

*This paper gives an outline summary of a design concept developed for a new generation of floating bridges which will allow ship passage through the floating bridge at any position along the bridge. The floating bridge may also be designed for exposed harsh environment locations.*

*The ship passage described in the paper is based on the use of a high bridge carriageway being accessed from two viaducts, at each side of the high bridge (Figure 1). The high bridge is supported on a horizontal beam structure resting on number of floaters.*

*The floating bridge is anchored to each shore side of the strait by conventional methods, while the ship passage is located at a position being most convenient as a shipping lane. The floating bridge is supported on floaters which will provide buoyancy and stability to meet both stability and safety requirements for the floating bridge. The floating bridge will be designed to absorb and survive predetermined accidental design conditions related to ship collisions.*

*Figure 1 High bridge for ship passage*

*The horizontal beam structure between the two shore side anchoring points will be constructed to absorb dynamic and static forces from both the local environment and from bridge traffic. Along the ship passage this horizontal beam structure is replaced with a subsea, horizontal beam*

*structure, which is structurally connected to the remaining part of the horizontal beam structure above the water line.*

*The design will allow large ship passages to become an integrated part of a floating bridge, e.g. with 400 m sailing width, 70 – 80 m sailing height and 20 – 30 m sailing depth.*

*The length of this type of floating bridge may typically be beneficial with crossing distances exceeding 1 km, and with a length up to 6-10 km.*

*The number of lanes of the carriageway in Norway will typically be 2 to 4, but technically this floating bridge may be designed for e.g. 8 lanes. If required, railway lines for light high speed trains may be designed as an integrated part of the bridge.*

## INTRODUCTION

The topography of Norway keeps the transport, shipping and airline industries busy, but is not necessarily a benefit to an efficient development of the country. The many fjords of Norway have made the Norwegians reliant on the communication by sea, which once was the foundation for Norway becoming a leading country for the maritime industries.

With the rise of oil and gas technology necessary to develop the vast energy reserves on the Norwegian Continental Shelf, the harsh environment of the North Sea became a laboratory for developing offshore technologies and know-how. Some of these technologies and know-how are transferable to other areas, such as floating bridges (Figure 2).

In order to meet the challenges to design floating bridges for extreme crossings, it has been necessary to combine knowhow and experience from the design of offshore structures operating in the exposed environment of the North Sea, efficient ship building practises and innovative, light weight high bridge designs.

The cost of a floating bridge will become competitive through efficient design, and by implementing advanced prefabrication, standardization, modularized design, efficient marine installation and hook-up operations and low maintenance cost.

*Figure 2 Aerial impression of floating bridge concept*

## CASE STUDY

To illustrate the technology of this type of floating bridges, a selected case study for a dual lane bridge is described below, being based on main design criteria for a 2 lane bridge in a typical Norwegian fjord:

- Strait distance: 5,000 m
- High bridge floater located at mid fjord position
- Ship passage: Sailing breadth 400 m, sailing height 70 m, sailing depth 20 m
- Road standard (Nor): S4
- Design life time 100+ years
- Number of lanes 2, plus bicycle/pedestrian lane
- Environment: Norwegian West Coast, partly protected waters, no swells
- Max wave height – 4.7 m at wave period 8.2 secs
- Max surface current – 1.3 m/s
- Dimensioning wind – 40 m/s
- Dimensioning tide - +/- 1.6 m (from mid water)

## GENERAL DESCRIPTION OF FLOATING BRIDGE – CASE STUDY

The horizontal beam structure of the floating bridge will be fixed to both shore sides of the strait, and be supported onto a number of floaters/pontoons, which will secure the vertical position of the floating bridge above the water line. The floaters may be constructed in either steel or concrete, and will be shaped to minimize impact from the local environmental forces.


*Figure 3 Integrated ship passage*

A ship passage may be integrated into the floating bridge where most convenient for the local shipping lanes (figure 3). The ship passage will comprise a high bridge supported on purpose designed high bridge floaters and a horizontal subsea beam structure linking the two high bridge floaters. The carriageway of the high bridge is linked to the viaduct carriageway, while the subsea structure is linked to the horizontal box structures, in order to ensure structural continuity along the whole floating bridge.

The ship passage will be designed in accordance with local ship traffic requirements and mandatory safety assessment and guide lines. The Norwegian regulations and guide lines are currently being revised by the Norwegian Coastal Administration. The current guide lines has assumed a sailing breadth of up to 400 meters with sailing heights will be dependent on local ship traffic.

*Figure 4 Light framework arch bridge*

The ship passage may be built and installed in several ways. One of the options is to build the complete ship passage as a fully integrated structure, being prefabricated at shore, towed to the floating bridge location and installed as separate part of the floating bridge, but structurally hooked up to the remaining part of the bridge.

Recent works has confirmed that light arch bridges will be well suited to for the use as a high bridge across the ship passage (Figure 4). The arch bridges have a favourable weight and wind profile, and work is being carried out to explore the option of implementing framework arch bridge for the ship passage.

The dimensions of the ship passage will be decided from shipping requirements and risk analysis in order to minimize consequences from possible ship collisions with the floating bridge.

*Figure 5 Longitudinal view with floaters*

In order to minimize collision risk in areas with considerable shipping traffic, it will be possible to install several ship passages along the floating bridge, allowing one way shipping lanes, when necessary.

The ship passage may be designed to withstand and survive collision impact from most types of reasonably large ships, such as the largest cruise ships on the market today, - if required. A design collision example may be the consequences from a ship impact peak load of 5,000 metric tonnes.

The viaduct provides access to the high bridge on both sides. The viaduct is supported by columns resting on the horizontal structural box, and will have an inclination in accordance with local regulations.

Small boats and ships may also pass the floating bridge outside the ship passage, as a typical air gap under the continuous, horizontal beam structure of the floating bridge may be 5-8 meters, which may be increased to larger air gaps, such as 10 - 12 meters, to allow small boat passage along most parts of the floating bridge.

In order to meet the long distance of 5,000 meter, or more, the floating bridge will be kept in position by a number of mooring lines in order to avoid any buckling or compression of the horizontal box structure. The number and dimensions of mooring lines will depend on design criterias, decided e.g. by local environmental forces from wind loads, sea currents, waves, traffic loads, redundancy and safety requirements, replacement and maintenance requirements.

A variety of mooring line solutions is possible, depending on local conditions and requirements. Polyester lines have been found especially attractive as these lines are near neutral in sea water and have during the last decade obtained a first class reputation with regards to strength and reliability. Polyester lines with breaking loads exceeding 2,500 tones are common in the offshore industry.

However, the use of wire lines and chains are being further assessed and its use will depend on required flexibility and tension of the mooring lines (Figure 6).

*Figure 6 Mooring lines*

The length of the mooring lines will depend on loads and local water depths. The offshore industries routinely implement mooring of floating structures in water depths of 2 – 3,000 meter. Water depth is not considered to be a limiting factor for use with floating bridges and has negligible effect on total cost. The installed capacity of mooring lines will be typically be 3 x 100%, including safety factors, where each single line has the capacity to meet the design load requirements. During replacement or inspection of one line, there will still be a 2 x 100% tension capacity available.

A variety of reliable, sea bottom anchoring system is available from decades of experience from the offshore industries. This includes e.g. gravity anchors, drag anchors, suction piles, gravity piles, etc.

## PARTICULARS OF FLOATING BRIDGE – CASE STUDY

A summary of the particulars of the floating bridge developed in accordance with the above mentioned case study is given below:

| | |
|---|---|
| • Number of lanes: | 2 |
| • Length of bridge | 5,060 meters |
| • Curvature radius: | 4.750 meters |
| • Distance floaters: | 120 meters |
| • Floater material: | Concrete |
| • Vertical beam dimensions | 16.2 x 5.0 meters |
| • Steel tensile strength: | 355 and 420 MPa |
| • Viaduct angle | 6% |
| • Viaduct support: | Vertical columns, steel |
| • High bridge: | Arch design |
| • High bridge floaters | Concrete |
| • High bridge support | Columns |

- Subsea structure Steel

- Mooring lines Polyester
- Mooring line capacity 3 x 100%
- Anchors Gravity piles

**FABRICATION AND INSTALLATION**

The floating bride will be fabricated in separate modules, which will be assembled and hooked up close to the installation site for the bridge. This covers e.g. the floaters, sections of the horizontal beam structure, the viaduct, the ship passage, etc.

The floaters of the two major existing floating bridges in Norway, have been made in concrete. Such concrete floaters have been found durable and the technology for building them is well proven and straightforward. The concrete floaters may be built either at a shipyard dock or at a purpose built construction dry dock for such structures, prior to being towed to the installation site. The floaters will be actively used throughout the assembly phase for the bridge modules into a complete bridge.

The steel modules will be prefabricated at reputable shipyards in liftable modules, typically up to 400 – 600 tonnes, and will be transported by heavy lift barges to an assembly site near the installation site of the floating bridge. The steel modules will be further prepared at the assembly site, prior to installation and hook – up onto the floaters.

The installation at site will make use of various types of marine operations. It has been an objective to eliminate the use of large heavy lift barges, as these are expensive and has limited flexibility with regards to time frame for performing the contracted lifting operations. During installation of the bridge at its site, maximum use of low cost flat barges and associated ballasting operations will be employed.

**FLOATING BRIDGE – PREFERENCES**

The use of floating bridge becomes especially attractive for long strait crossings, with distances exceeding 1 km. While traditional suspension bridges face a near exponential cost increase for increasing free spans, a floating bridge will have a near linear, and hence moderate, cost increase for increasing lengths.

The main limitation of floating bridges has to date been lack of reliable solutions for ship passage. This paper presents a solution to avoid this limitation, and hence make floating bridges an attractive option for most strait crossings.

For strait crossings less than 1 km traditional land bridges will in general have a competitive edge on functionality even though floating bridges will have a cost in the same order of magnitude. Small, traditional, fixed bridges are proven and well known in terms of construction and ease of ship passage.

For strait crossings between 1 and 2 km, floating bridges will become more attractive and cost competitive compared to e.g. traditional suspension bridges, as the functionality with regards to ship passage is now available for floating bridges.

Suspension bridges with free spans beyond 2 km has yet to be built, but is on the drawing board and has been on the wish list of most bridge engineers for years. Even though technical challenges can be solved for extremely long free spans, a floating bridge with such lengths will remain a very attractive option when cost is essential, and sometimes the only option in terms of budgetary realism.

*Figure 7 Floating bridge with two ship lane passages.*

## ACKNOWLEDGEMENTS

LMG Marin AS is a marine technology and ship design company with offices in Bergen, Norway. LMG Marin has headed the development of floating bridge solutions presented in this paper, but this development has been made possible also due to contributions from other specialist parties, especially;

- High bridge design: Smidt & Ingebrigtsen AS, Bergen, ‘
- Network arch bridge inventor: Dr.ing Per Tveit, professor emeritus
- Marine technology analysis: Marintek, Trondheim
- Structural analysis assistance: TDA AS, Oslo

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# LIFE CYCLE ASSESSMENT OF BRIDGES; ACCOMPLISHMENT AND IMPLEMENTATION

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## ABSTRACT

*Life Cycle Assessment (LCA) is becoming increasingly relevant for infrastructure projects, both as decision support and for reporting the carbon footprint and other environmental impacts of the projects. The Norwegian National Transport Plan 2010 - 2019 states that climate emissions of large infrastructure project shall be reported. This triggers a need for common guidelines and a common methodology for such calculations.*

*This paper encompasses earlier LCA studies on bridges, a common tool for LCA of bridges and implementation of the use of this tool in the bridge planning process. Selected earlier research on LCAs of bridges will be presented in this paper, covering LCAs of several bridge types. A few LCAs on tunnels and roads are also presented. The carbon footprint per $m^2$ trafficable surface area of these various bridges, tunnels and roads indicates that the bridges are the most emission intensive structures, followed by tunnels and then roads. The most important materials in the bridges regarding carbon footprint are concrete, reinforcing and steel.*

*There are very few LCA tools applicable to bridge currently available. BridgeLCA is the most suitable tool particularly for bridges. This tool is developed within a joint project (ETSI) between road administrations and universities in Scandinavia, comprising development of methodologies and tools for LCA, LCC and aesthetics evaluation of bridges. This is applicable at different bridge planning stages, and may hence contribute both to decision making and lifecycle emission reporting. The tool encompasses emissions to several environmental impact categories in addition to global warming (e.g. depletion of the ozone layer).*

*Within the Norwegian Public Road Administration (NPRA) some tests of implementing LCA in the bridge planning process has been carried out as part of the development of a strategy for implementation of LCA in road bridge projects, from the conceptual planning stage to the execution of bridge works.*

## INTRODUCTION

Life Cycle Assessment (LCA) is becoming increasingly relevant for infrastructure projects, both as decision support and for reporting the carbon footprint of projects. The Norwegian National

Transport Plan 2010-2019 states that climate emissions of large infrastructure project shall be reported. This triggers a need for a common methodology and guidelines for such calculations.

This paper presents results from earlier LCA studies on bridges, tunnels and open road segments and the tool *BridgeLCA*[1]. Some of the bridge case studies were performed as a part of the development of this tool, and the rest of the cases are included in this paper to provide a general idea of the magnitude of impacts for bridge, tunnels and roads respectively, and in comparison to each other. *BridgeLCA* is a result from the joint Scandinavian ETSI[2] project and allows for thorough LCAs of bridges. The tool has been developed by the Norwegian partners; Norwegian Public Road Administration (NPRA) and the Norwegian University of Science and Technology (NTNU). The tool is developed for bridges in particular, but with a few further adjustments the tool will also cover tunnels and roads as well.

The NPRA is aiming at implementing LCA in the planning and execution of bridge works, in a consistent and standardised manner. Consecutively, this will also include tunnels and roads.

## METHODOLOGY

LCA applied in *BridgeLCA* and in the case studies are performed in accordance with ISO 14040 [1]. The case road bridges are modeled in the LCA software SimaPro [2], and the LCI database ecoinvent [3] is used for environmental data. For the case studies production data for the materials consumed in high volumes are adjusted to Norwegian technology. In *BridgeLCA* the user can choose between applying generic ecoinvent data or enter national or product specific data. The impact assessment methods applied are ReCiPe [4] (both in case studies and in *BridgeLCA*) and USEtox [5] (in *BridgeLCA*).

The Functional unit for the earlier case studies are *1 $m^2$ trafficable area of a road segment (open road, bridge, and tunnel) through a lifetime of 40 years*. In this paper only the emissions of $CO_2$-equivalents are included in the results for the case studies[3]. The technical lifetime of bridges and tunnels are known to be about 100 years, the lifetime of 40 years applied in this study is based on the assumption that the *functional* lifetime of all the road elements are 40 years. The functional lifetime is applied because it is assumed most likely that traffic patterns (e.g. volume and load) will require adjustments/renewal of road stretches; like widening of traffic lanes, structural enhancement or replacement of bridges, or construction of a new road stretch.

[1] This is the current name of the tool, but it is important to note that further development and implementation of the tool also will include tunnels and roads

[2] Project website: http://etsi.aalto.fi/Etsi3/

[3] Results for other impact categories can be provided at request

**CASE STUDIES; BRIDGES, TUNNELS AND ROADS**

**Bridges**

As a basis for the development of *BridgeLCA*, an LCA of 3 three built bridges in Norway; a steel box girder bridge (Klenevaagen), a concrete box girder bridge (Hillersvika) and a timber arch bridge (Fretheim) was performed [6]. Figure 10 shows the relative impacts to global warming related to bridge parts and life cycle stage for each bridge.

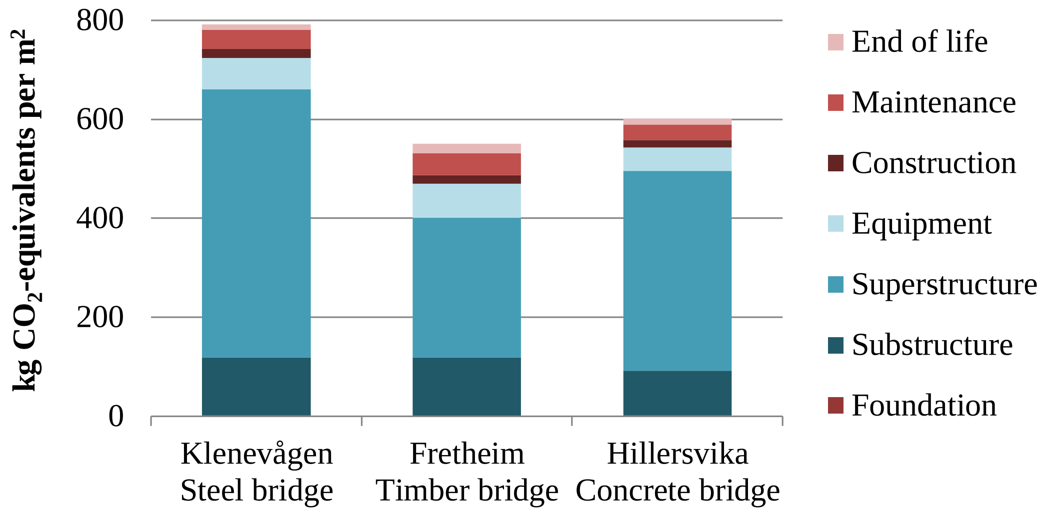


*Figure 10: Emissions of $CO_2$-equivalents per $m^2$ split up in bridge structural parts and life cycle phases*

For all bridges the superstructure dominate the impacts (~50 – 70 %), followed by the substructure (~15 – 20 %). In total the emissions of $CO_2$-equivalents related to material manufacturing spans from 85 % to 90 %.

In connection with the planning of a new road (E6) between Vinstra and Sjoa in Norway, three alternative bridge designs were compared with regards to environmental performance; steel, concrete and timber bridge [7]. Figure 11 shows the emissions of $CO_2$ split up per bridge part and life cycle phase.

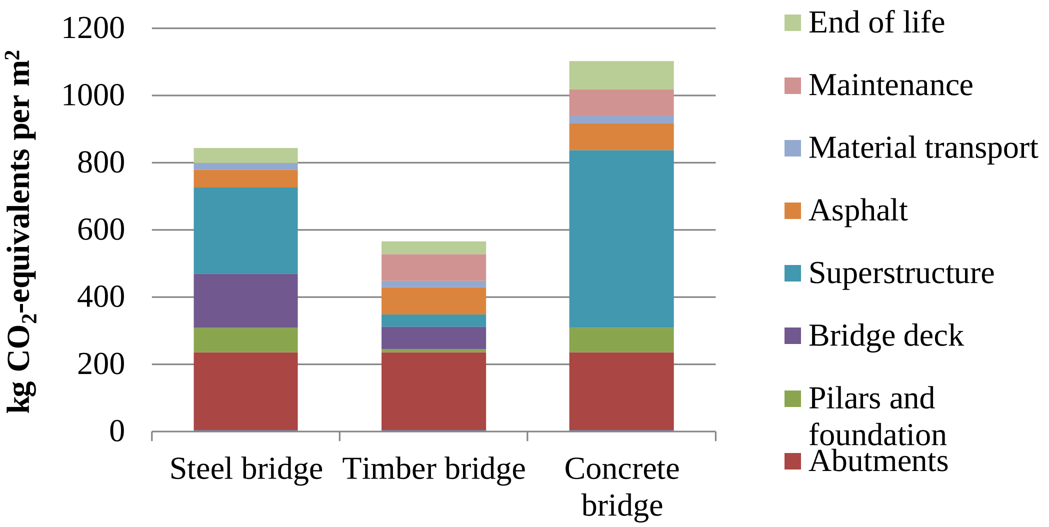


*Figure 11: Emissions of $CO_2$-equivalents per $m^2$ split up in bridge structural parts and life cycle phases*

For these bridges the abutments, pillars and foundation causes a higher share of the total impacts compared to the relative contribution from the corresponding part, substructure, in the case bridges above. For the timber bridge alternative, these parts represent almost half of the impacts.

The paper 'Environmental analysis of bridges in a life cycle perspective' [8] presents results of simplified LCAs of a wide selection of bridges built in Norway. A selection of these results is repeated here, given in Figure 12. Materials included in this study are: concrete, reinforcing, steel, surface treatment of steel, glue laminated timber, timber preservation, sawn timber, copper, asphalt membrane, tack coat and asphalt. The construction phase includes blasting and use of equipment in earth treatment and transport. Operation and maintenance includes lighting and resurfacing.

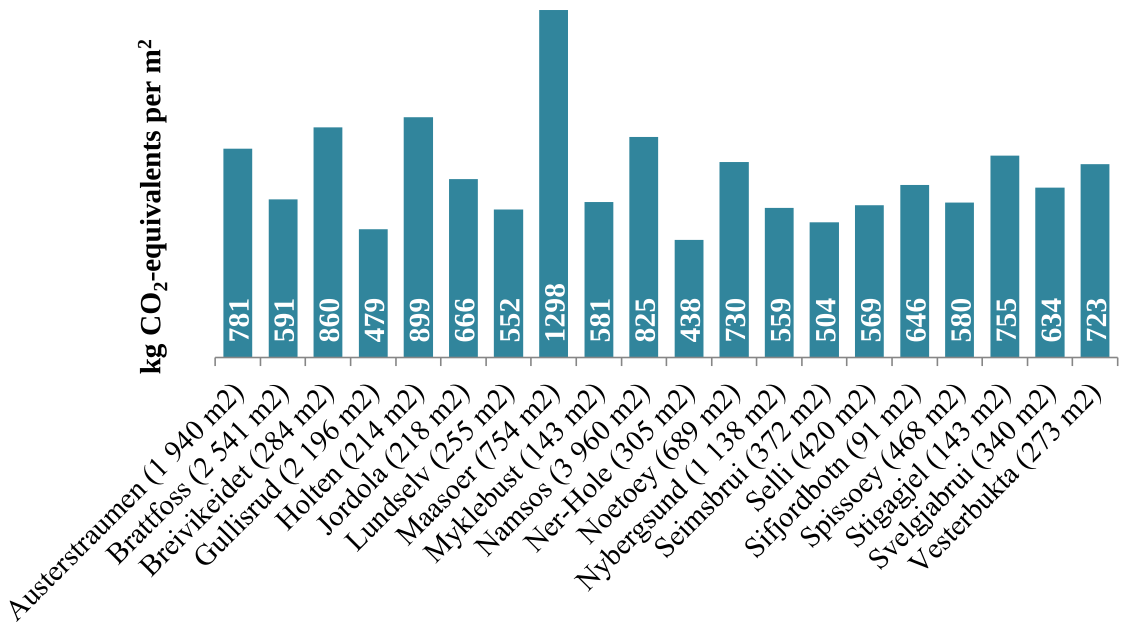

*Figure 12: Results for a selection of built bridges in Norway*

The climate emissions for these bridges spans from 440 to 1 300 kg $CO_2$-eq. Maasoer bridge has especially high emissions (400 kg $CO_2$-eq more than the second most emission intensive bridge) due to the site at a river mouth, exposed to relatively large tidal variations. To protect the timber from the salty seawater the arches consists of unusually high amounts of concrete. In addition, the ground conditions at the site are quite poor; hence additional reinforced concrete elements for extra support are used. Figure 13 below shows the results for the four of the largest bridges, split up per input parameter. Concrete, reinforcing and steel clearly dominates the impacts. This is also the case for the timber bridge, where the gluelam account for only 7 % of the emissions.

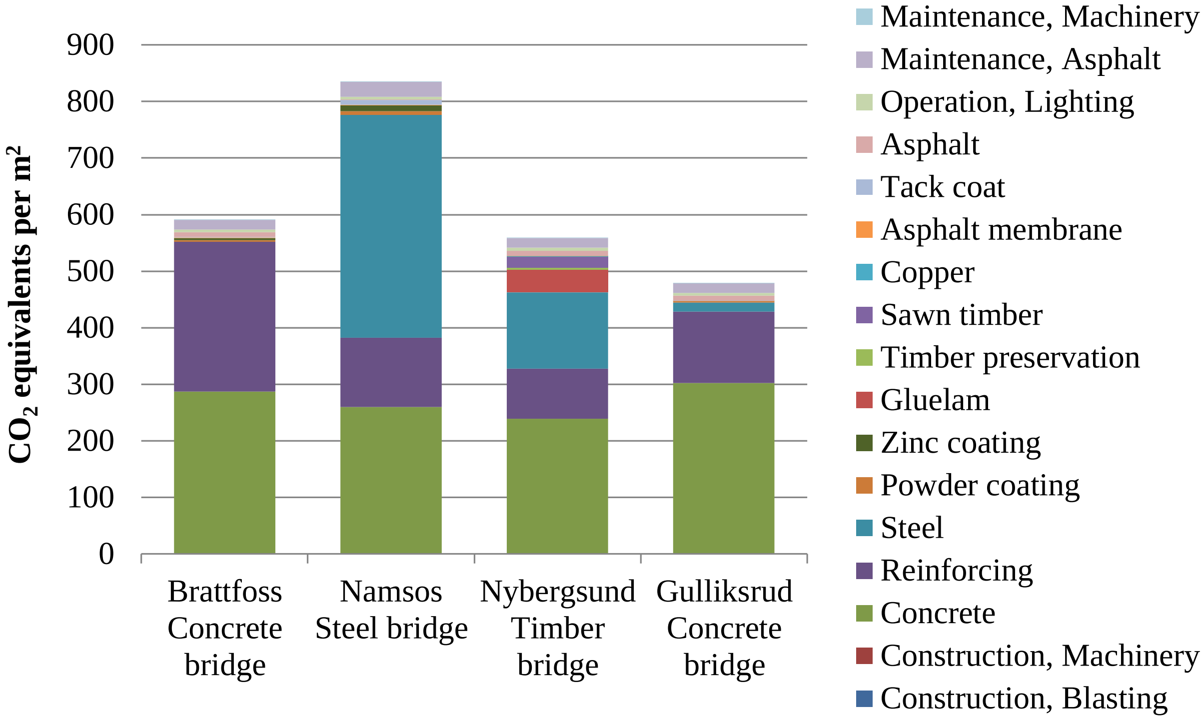


*Figure 13: Results for large bridges split up in input parameters (2 541, 3 960, 1 138 and 2 196 $m^2$ respectively)*

**Tunnels**

Similar calculations are performed for tunnels the results are given in Figure 14. For tunnels, the emissions per $m^2$ trafficable area spans from 200 to 580 $CO_2$-eq.

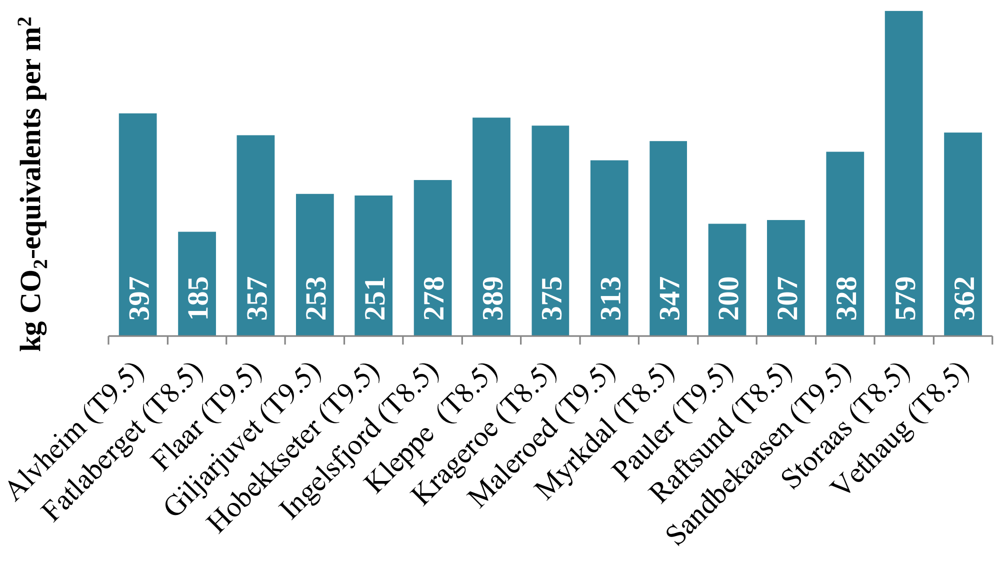


*Figure 14: $CO_2$ emissions for tunnels, per $m^2$ trafficable area*

The impacts vary due to various sizes of portals and requirements for the arch lining (shotcrete or concrete elements) and bolting.

## Roads

For open road sections, the impacts vary a great deal. This is most likely because the calculations are quite course; only a limited amount of input parameters are included and also the uncertainty of the underlying data (both material amounts and road widths) is relatively high. In spite of this, the results are included here in order to get some idea of impacts per trafficable area of open road segments compared to corresponding results for roads and tunnels.

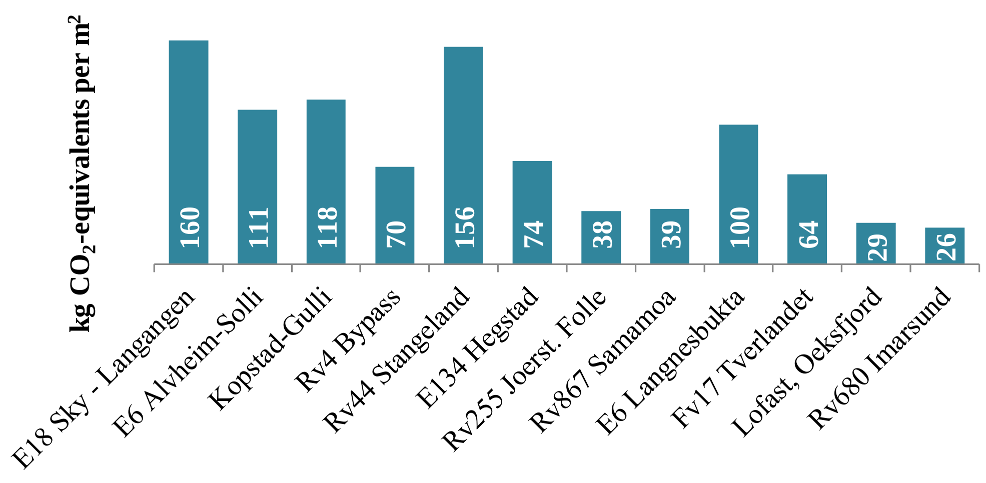


*Figure 15: $CO_2$ emissions for roads, per $m^2$ trafficable area*

**TOOLS FOR PERFORMING LCA OF BRIDGES**

ETSI is a Nordic joint research project started in 2006 which comprise costs, environmental impacts and aesthetics for bridges in a life cycle perspective. The projects aim is to develop tools for assessing life cycle costs, life cycle environmental impacts and evaluation of aesthetics. The LCA tool developed within this project; *BridgeLCA*, is applicable at different planning stages as the level of detail in data inputs are decided by the user. The tool may hence be used both for decision support and lifecycle emission reporting (e.g. carbon footprint). The latest version of the tool for life cycle assessment, *BridgeLCA*, is currently subject to extensive testing both within Scandinavian road administrations and bridge planning consultants. Future version of the tool will also be applicable for tunnels and roads.

Dependent on the detail level of the data entered by the user, *BridgeLCA* offers highly detailed results for the following categories: Climate Change, Ozone depletion, Acidification, Eutrophication, Fossil depletion, Human toxicity (split up in cancer and non-cancer effects), Ecotoxicity and Energy consumption (split up in renewable and non-renewable). The environmental impact results are given in various detail levels:

- Total normalised score, in Person Equivalents (PE)
- Total weighted score, in Person Equivalents (PE). These results are further split up in both input parameters and lifecycle stages
- Total midpoint results for each impact category, in the respective emission-equivalent (e.g. $CO_2$-equivalents for global warming). These results are further split up in both input parameters and lifecycle stages
-

The four lifecycle stages are Material, Construction, Operation, repair and maintenance, and End of life.

As described above, *BridgeLCA* includes a wide range of emissions and energy consumption (aggregated into the 8 impact categories). For simplicity only impacts to Global warming, (i.e. carbon footprints) is included in the following text.

A literature and web search for LCA tools for bridges has been performed. Many LCA tools for buildings are openly available (e.g. BREEAM, BEES, Athena impact estimator for buildings) and some for roads (e.g. PaLATE, ROAD-RES). For bridges in particular, one tool is described in the Master thesis *A Dynamic Life Cycle Assessment Tool for Comparing Bridge Deck Designs* [9], which, as the title indicates, does include bridge decks and not the total bridge structures. At KTH in Sweden, an LCA tool for railway bridges has been developed, by Guangli Du [10]. This was part of a licentiate thesis which were co-operated and partially financed within the ETSI project, and is hence quite similar to *BridgeLCA* in both methodology and structure.

In the knowledge of the authors, there is currently no LCA tool for road bridges available, except for *BridgeLCA*.

## IMPLEMENTATION OF LCA IN BRIDGE PLANNING AND EXECUTION OF BRIDGE WORKS

A strategy for implementing LCA in bridge planning and executing of bridge works is not decided in NPRA so far. In general, it will be used as support in the selection between alternative design concepts and choices related to the construction phase. It will also be used for documentation of climate emissions (carbon footprints) of bridges after they are built.

The choice of bridge concept for one particular site will not be based on environmental performance alone. Other parameters like costs, service life, aesthetic value, and duration of the construction period also affects the choice. Regarding the construction phase, the LCA can be used to identify the material choice causing the least environmental damage, considering both transport distance and production related impacts.

Implementing LCA in bridge planning and construction instigates the need for a uniform methodology. Variations in methodological choices between the actors affecting bridge planning and construction will have low value (if any) to the NPRA, as inconsistency cannot be used as basis for further development of overall policies and calculations of aggregated carbon footprints. Hence, *BridgeLCA* is very suitable for this purpose.

The intended application of *BridgeLCA* is to document the carbon footprint (and other environmental aspects), and potentially also total yearly national emission of bridge construction. A further development of the tool is intended, in order to cover roads and tunnels as well. Estimating the carbon footprints using the tool can further be used to improve or reduce the environmental performance of bridges (and roads and tunnels).

LCA in bridge design and construction works can be implemented at five stages:

1. **Conceptual design stage:** This is covered by EFFEKT [11], another program developed by the NPRA which encompasses whole road projects (a further description of this tool is not included in this paper). However, *BridgeLCA* could be used to improve EFFEKT allowing for it to cover more detailed designs.
2. **Detailed design stage:** The intention here is to predict the carbon footprint of a specific bridge design, and also to compare various bridge designs for the same site. Further the tool allows for minimisation of the carbon footprint by comparing different solutions as an embankment or a subsea tunnel or a road around the bay/fjord. In the design stage one shall decide upon bridge type, materials and construction methods.

3. **In tendering stage:** NPRA has for some years worked for the inclusion of climate emission in the tender and acquisition process, and to establish an acceptable procedure for doing so. This has been tested a few times, also including an obligation for the contractors to report back at the NPRA that the actual climate emissions from construction did not exceed a certain defined limit. One challenge for the NPRA was that there were no procedures or tools for calculating and documenting the climate emissions.
   Another idea was to have the possibility to give economic awards or penalties to companies that reduced or increased the emissions to what they have stated in the tender documents. This might be possible to introduce within current regulations.
4. **Execution of bridge work stage:** The intention here is to assess the carbon footprint related to the construction in more detail. This will include choice of material and material supplier, transport (means and distance) of materials, and alternative processes in the bridge work (for instance optimisation of mass movement, choice of construction machinery etc.).
   Comparison of different material suppliers requires EPD's from each of them, and also inclusion of the transport of the material to the bridge site.
5. **Overall stage:** Aggregate the national climate emission for a certain period of time. This can be used in planning and/or execution of climate emission abatement strategies, both within the entrepreneur firms, the NPRA and nationally.

## CONCLUSIONS

The climate emissions for the case bridges studies here spans from 440 – 1 300 kg $CO_2$-eq, for the tunnels 185 – 580 $CO_2$-eq and for roads 26 – 160 $CO_2$-eq. even though the analyses of tunnels and roads are quite uncertain, this strongly indicates that bridges are most carbon emission intensive, followed by tunnels and roads. For bridges, the firs 6 cases show that material production is the far most important life cycle stage. Both Operation & maintenance and end-of-life contribute less than 5 %, and the construction phase less than 2-3 %. Of the materials in the bridges, concrete, reinforcing and steel are far most important.

Implementation of LCA in NPRA aims at covering all stages, from planning to construction and an aggregated level. This is important to make it possible to both improve and report the overall environmental performance from material production technologies, road work routines, and operation, repair and management activities to the overall infrastructure development in Norway.

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LCA tools mentioned in this text are available and further described at the following websites:

BREEAM:

www.breeam.org/

BEES:

ws680.nist.gov/Bees/(A(8PvbsmVozgEkAAAAYTk4OGZhYjUtNDk5NC00M2UzLTgxODYtYWVkYmQxMmE5Y2I2tGQvrVy1wjtGgFOi8EW9Twp5TSU1))/Default.aspx

Athena impact estimator for buildings:

www.athenasmi.org/our-software-data/impact-estimator/

PaLATE

www.ce.berkeley.edu/~horvath/palate.html

ROAD-RES

www.athenasmi.org/our-software-data/impact-estimator/

# DESIGN AND EXCAVATION OF XIAMEN XIANG′AN SUBSEA TUNNEL

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## ABSTRACT

*The first subsea tunnel in China, the Xiamen Xiang′an subsea tunnel, opened to traffic on April 26th, 2010 and has been operated. The tunneling excavation commenced on September 6th, 2005 and went through on June 13th, 2009. The Xiang′an subsea tunnel was planned to be a high traffic tunnel, consisting of two large traffic tunnels, each with 3 lanes plus a service tunnel. The 6.05 km tunnel is located in partly problematic and challenging ground conditions, including fault zones undersea, shallow sand layer and completed weathered rock mass on shore. The design was based on comprehensive geological investigations. The construction of the tunnel was completed successfully without a single major accident and delay. This paper is to highlight the challenges and achievements in the construction of the Xiang'an subsea tunnel.*

## 1 INTRODUCTION

The first subsea tunnel in China, the Xiamen Xiang′an subsea tunnel, has opened to traffic since April 26th, 2010. The tunneling excavation commenced on September 6th, 2005 and completed on June 13th, 2009. The location of the tunnel is shown in Fig. 1 and Fig. 2. The Xiang′an subsea tunnel was planned to be a high traffic tunnel, consisting of two large traffic tunnels, each with 3 lanes plus a service tunnel. The 6.05 km long tunnel is located in partly problematic and challenging ground conditions, including completely weathered trough undersea, shallow sand layer and completed weathered rock mass onshore. The construction of the tunnel is completed without a single death accident and a major delay. Conventional drill and blast (D&B) method was applied in the tunneling. Under difficult geological conditions, Centre Cross Diagram (CRD) method was employed to cope with the large deformation of the driving tunnels with large span of about 16.5 m.

The tunnel was planned according to the standards of high grade highway with 6 carriageway lanes with the following features [1] [2].

- Design vehicle speed: 80 km/h in main lanes
- Dimension of 3 lanes tunnel: clearance width of 13.5 m and height of 5 m, horse-shoe shape
- Dimension of service tunnel: 6.5 m wide and clearance height of 6m, circular
- 12 cross passages for people and 5 cross passages for traffic
- Design longitudinal gradient of tunnel: 2.9%
- Horizontal curve radius: for tunnel, bridge and approach road: > 700 m; for ramp or ramp

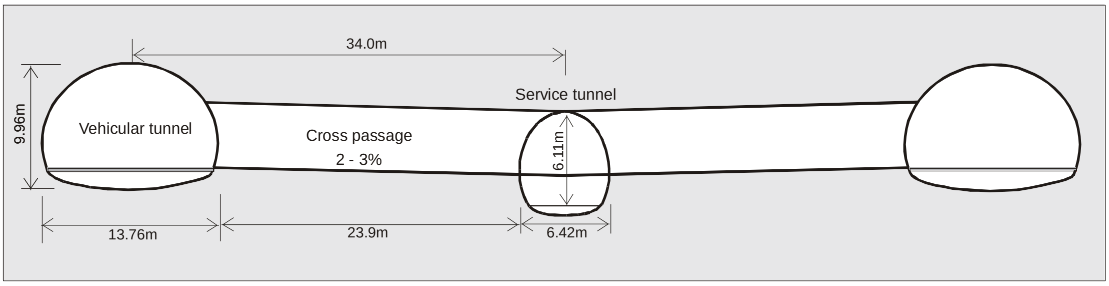
bridge: > 50 m

- Traffic load: at opening: 24000 vehicles per day in both directions; dimensioning load 60000 vehicles with 15000 trucks (i.e. 94000 car units)

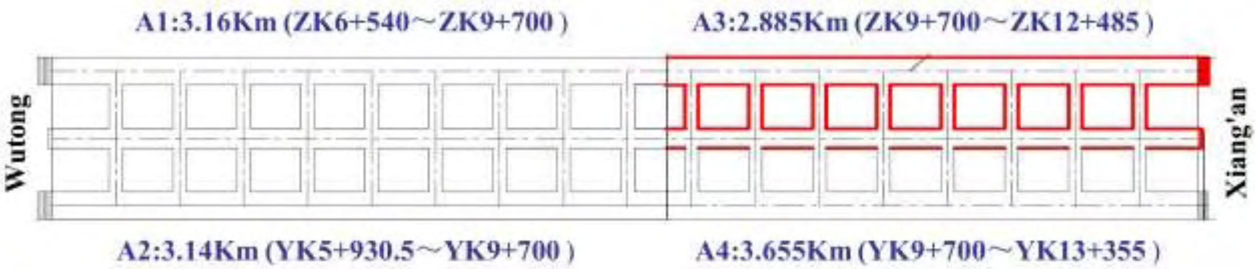

*Fig.1 The Xiamen City, China*


*Fig 2. Location of the Xiang'an Subsea Tunnel, Xiamen*

The cross section of the tunnels and the sketch map of the contract sections are shown in Fig. 3[3]. The service tunnel is introduced as it has important safety aspects and risk reducing elements both for construction and operation. During construction it was used as an exploratory adit to probe ground conditions ahead of main driving tunnels excavation, to provide more work faces to advance the tunnelling and to drain water in the tunnel. During operation, it provides maintenance and safety channel, and to facilitate ventilation.




(a)




(b)

Fig. 3. (a) A cross section of the main tunnels and the service tunnel. (b) Sketch map on contract sections of Xiang'an tunnel[3]

## 2 DESIGN OF THE XIANG′AN SUBSEA TUNNEL

### 2.1 Geological Investigation and Geological Conditions

Compared to conventional tunnels, subsea tunnels are special in several ways. Most of the project area is covered by water. Hence, special techniques and a large amount of investigations need to be applied, and interpretation of the investigation results is more uncertain. The potential of water inflow is indefinite. The consequences of severe water ingress may be disastrous, as the whole tunnel can be quickly filled with water. All water inflows have to be pumped out along the tunnel. The saline character of leakage water has to be considered both for the tunnelling equipment, rock support materials, linings and technical installations. The locations of straits are often formed because faults and weak zones or parts in the bedrock are easier eroded. Thus, the deepest parts of the strait present often the most difficult part tunnel excavation conditions.

For the construction of the Xiang’an subsea tunnels, comprehensive geological investigations were carried out at pre-feasibility, feasibility and detailed design stages from 1998 to 2003 and provided a detailed picture of the ground composition along the tunnel route. The investigations include mainly:

- Geophysical profiles, totalling 136 km.
- 66 vertical core drillings, of these 35 in the sea sections. The deepest hole is 75 m below seabed and the average hole length is approximately 50 m.
- Cross hole seismic measurements between many of the boreholes. In addition, pressure test in 7 holes using 0.9 MPa overpressure.
- Hydro fracture test to measure the in-situ stress.
- Laboratory testing of some rock and soil samples
- The earthquake potential of the area, concluding that the site is located in a stable and stiff plate in between active earthquake faults, i.e. no active faults will cross nearby the tunnel.
- The radioactivity potential of the rock mass has been found to be lower than the recommended threshold level.

Fig. 4 shows the geological profile along the selected tunnel alignment with Wutong entrance in the left side and Xiang’an entrance the right side. The Xiang’an subsea tunnel passes through the bedrock with overburden layers in Quaternary system. The rock is dominated by granitic diorite, biotite granite and vein rocks. Four strata have been identified in terms of weathering with a successive range from completely decomposed/weathered ground (CDG) to highly decomposed granite (HDG), moderately decomposed granite (MDG) and slightly decomposed granite (SDG). The ground along the tunnel may be divided into three classes, A, B and C, with the distribution shown in Fig. 4. The largest extent of completely and strongly decomposed is found on the onshore part and in shallow sea water region. Along the subsea part of the tunnel, the rock masses consist mainly of slightly decomposed rock intersected by some few faults where grooves of completely and highly weathered rock masses occur. The Quaternary overlying stratum includes mainly residual soils from invaded rocks in deep and some alluvial deposits of clay and clayey sand in Pleistocene in the middle, with minor marine deposit of sand, clay and silt on the top. Basement rock, including weathered rocks except for the completely weathered rock, has the minimum friction angel of about $35^{o}$ and Young′s Modulus greater than 1GPa as listed in Table 1[4].

There are two major faults in the region far away from the tunnel. Along the tunnel alignment, four fault/weakness zones were identified by seismic refraction measurements. The plan view of $F_1$ intersecting the tunnels is shown in Fig. 5. Water inflow and poor stability (in faults especially $F_1$ and $F_4$) were expected to be encountered. Field test shows that in the weakly to slightly weathered rock masses, the hydraulic conductivity is low, about $10^{-7}$ m/s, and about $10^{-6}$ m/s in strongly to completely weathered rock mass. The estimated total water inflow into a tunnel is about 50,000 to 70,000 $m^3$/day without water sealing. In-situ stress measurement results indicate the maximum horizontal stress of 3 MPa in NNW-SSE direction. The regional seismic intensity is magnitude grade VII.

Digital strata modeling technique was applied to provide a 3D vision platform of the tunnel with functional data for the construction and the operation[5].

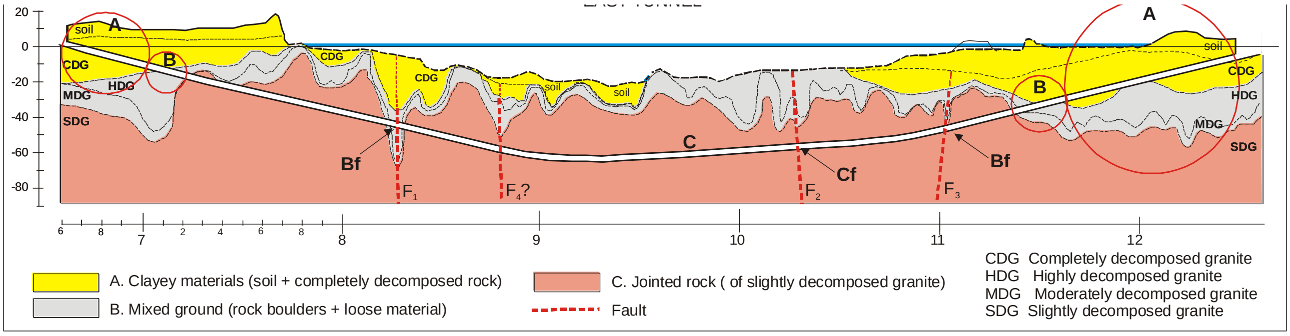



Fig. 4 Geological profile along the Xiang'an subsea tunnel (left) from Wutong to Xiang'an. $F_1$, $F_2$, $F_3$, and $F_4$ indicate the fault zones [3]

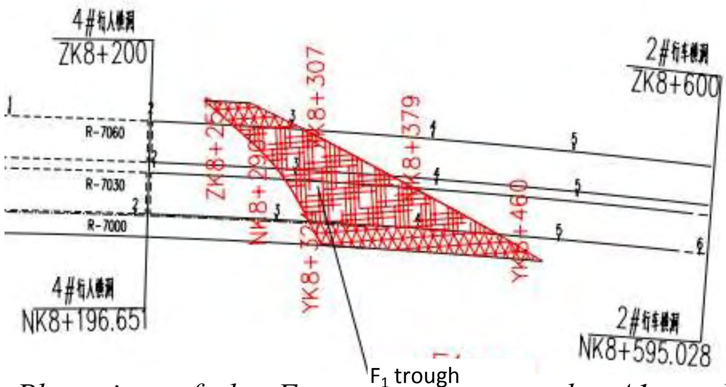



*Fig. 5 Plan view of the $F_1$ trough crossing the A1tunnel (ZK8+250~ZK8+307), service tunnel(NK8+290~NK8+379) and A2 tunnel (YK8+327~YK8+460).* [1]

*Table 1 Mechanical propertyXuanyi of the rock mass classes (corresponding to Fig. 4) [4].*

| Rock type | Young′s modulus /GPa | Poisson ratio | Cohesion /MPa | Friction angel $\varphi$/(°) | Unit weight /(kN • m$^{-3}$) | Permeability /(m • d$^{-1}$) |
|---|---|---|---|---|---|---|
| CDG(W4) | 0.05 | 0.30 | 0.033 | 23 | 19.5 | 6.4x10$^{-2}$ |
| HDG(W3) | 1.00 | 0.30 | 0.200 | 30 | 26.5 | 6.4x10$^{-4}$ |
| MDG(W2) to SDG(W1) | 15.00 | 0.25 | 1.00 | 39 | 26.5 | 6.4x10$^{-6}$ |

### 2.2 Horizontal and vertical alignment

The horizontal alignment is to try to place the tunnel in the solid rock as long as possible and to avoid the intersections with the weathering trough $F_1$ ~ $F_4$ as small as possible. When consider the vertical alignment, the rock cover defined by the distance between the tunnel roof and the rock surface at sea bottom, is a crucial feature with regard to the risks and the feasibility of the tunnel. Many factors decide what can be accepted as minimum rock cover, such as water depth, rock mass quality, the size of the tunnel, the accuracy of the geological investigation. Norwegian subsea tunnel experiences show that 40m are considered as a general minimum cover in poor rock mass conditions such as weakness zones and faults, when the ground information is limited. For the Xiang’an subsea tunnel, the rock cover has been determined based on the geological conditions, case study and stability analyses. It is concluded that a minimum rock cover of greater than 15m is technically feasible [4]. As shown in Fig. 4, the intersection with fault $F_1$ and $F_4$, the rock cover is about 30 m with water depth of about 25 m, and with from 50~100 m. These are in line with what revealed during the excavation.

Two ventilation shafts are designed in the left tunnel 1.31 km to the Wutong entrance, and in the right tunnel 1.235 km from the Xiang’an entrance, with 46m and 52 m heigh, respectively. Both shafts have diameter of 8.3 m after concrete lining. Between the main tunnels, there are 12 cross passages for people and 5 for traffic.

### 2.3 Considerations of the application of the Norwegian experiences

Many Norwegian subsea tunnels are located in similar fair to good rocks having comparable conditions; however, few are in highly weathered troughs. Challenging ground conditions have been encountered in Norwegian tunnels in connection with faults. The slightly decomposed rock masses present mostly favorable conditions for tunnel excavation. With the Norwegian experiences, excavation in completely and highly decomposed rock masses for the subsea part should be done with great care, using available means of investigations ahead of the tunnel, as well as pre-grouting, and special excavation using short rounds and stepwise heavy rock support. These works require close follow-up and monitoring.

Several site visiting consulting services took place in 2003-2007 [3] [6]. The Norwegian subsea tunneling experiences were applied with focus on the weathered troughs in the undersea part. An important issue is the implementation conditions. Given the difference of labor cost specifically, there is a certain gap to fully apply the Norwegian experiences in practice. Consequently, the construction takes more time due to lack of modern effective equipment and the skillful crews. This relates especially to shotcrete rig, drilling rig and grouting rig, in addition to methods for erection of the steel arches, etc.. However, efforts have been made to adapt the experience principles to the practical work conditions. More advanced equipment had been used particularly during excavation of the subsea tunnel sections and skilful workers had been trained.

## 3 EXCAVATION OF THE XIANG'AN TUNNEL

### 3.1 General

Though the Xiang'an tunnel is located in massive granite of generally good quality for tunnel construction, it has many excavation challenges as summarized in Table 2. Various measures have been studied and implemented, as seen in Table 2 as well. To excavate the driving tunnels with large span, special efforts were made to apply a proper excavation method to cope with the large deformation when excavating through the completely weathered rock masses. In the sand layer, continuous concrete wall and pumping were employed to control the seepage. In excavation through the completely weathered troughs under sea part, pre-investigation and pre-grouting were carried out to prevent from potential sea water inflowing. The excavation revealed that the total length of three tunnels intersecting the troughs were about 965 m ($F_1$: 278 m, $F_2$:220 m, $F_3$: 255 m, $F_4$:212.5 m). The deepest sea water was about 30 m with maximum water inflow of 6.2 $m^3$/day.m. Outside of the fault zones or completely weathered rock masses region, the granite was massive and exhibited good qualities for tunnel excavation.

*Table 2 Main excavation difficulties and solutions for the Xiang'an tunnel[3].*

| Type | Main known challenging ground for tunnel excavation | Solutions planned and implemented |
|---|---|---|
| A | **Soft ground** of clayey material, highly weathered rock from both portals. Problemes in permeabile parts | • CRD sectional excavation method with steel arches<br>• Filed monitoring and back analysis<br>• Pre-grouting support in roof<br>• Locking grouting anchor tube with plate<br>• Slow progress around 0.5 – 1.0 m/d |
| B | **Fault zones** to pass beneath the sea bottom. Two of them are expected to have weathered trough consisting of completely and highly weathered rocks, comparable to the conditions in A | • Comprehensive pre-grouting full tunnel face<br>• Grouting curtain<br>• CRD sectional excavation method with steel arches. |
| C | **Sand layers** within the zone of weathering on the Tongan side, permeable and highly friable | • Continuous wall through and around the sand layer and drainage of the sand layer within the wall<br>• Sectional excavation method (CRD, Double Side Walls) with steel arches |

### 3.2 Excavation and support of soft ground

In the onshore sections of the tunnel from both portals, ground belongs to completely and highly weathered rock CDG and SDG as shown in Table 1. The rock has been changed into a clayey material, which loses its strength when being saturated in water. The difficult geological conditions provided the most challenge conditions for tunneling with large cross section of about 170 $m^2$, span over 17m with overburden from 4 m to 25 m. The total tunnel section length with such conditions is about 2.2km from both portals. Among designed excavation methods, such as Full Face, Bench Cut, Centre Diagram (CD), Centre Cross Diagram (CRD), Side Drift Method, the Centre Cross Diagram (CRD) Method was mostly applied in the excavation of the driving tunnels, as shown in Fig. 6. The service tunnel, having a smaller diameter, was driven by the Bench Cut method.

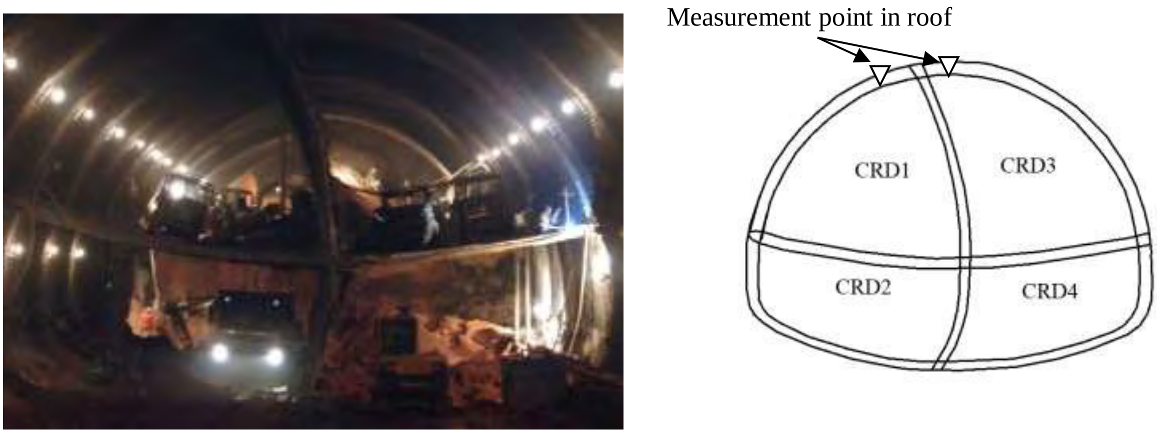



*Fig. 6 CRD method*

During the excavation large deformations at roof, side walls and the vertical cross steel rib wall have taken place, even exceeded the requirement of the design and the relevant National Regulations[7]. Local deformation caused shotcrete cracks and severe deformation of the temporary support. An example of the extraordinary vault settlement recorded is shown in Fig. 7. The total vault settlement of CRD1 is about 620 mm, more than 5 times of the reserved displacement.

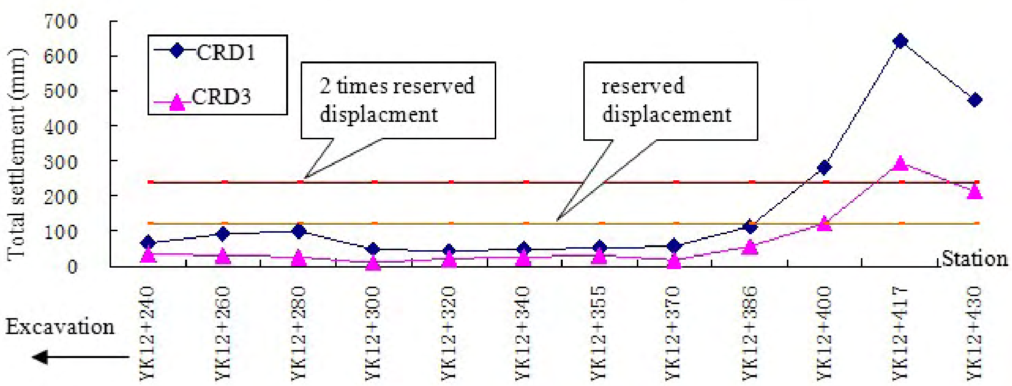
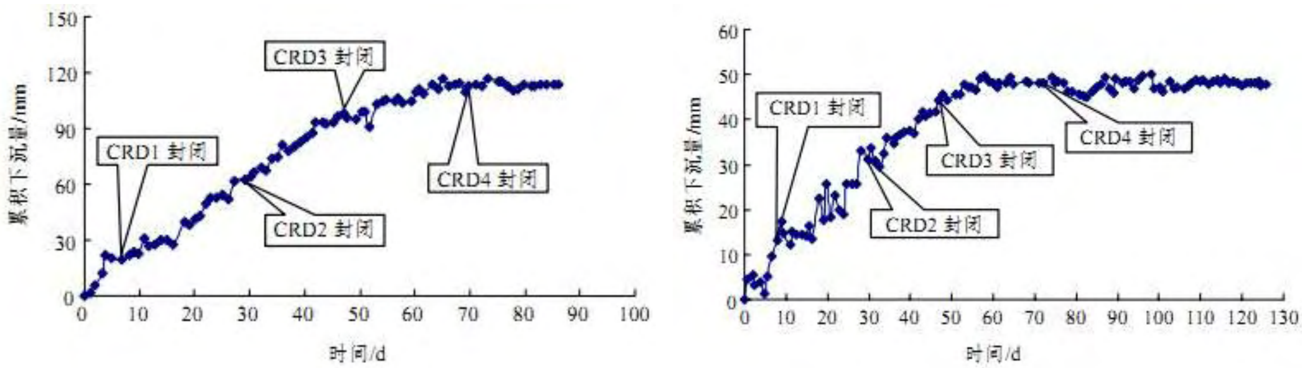



*Fig. 7 Large vault settlement at a tunnel section of Xiang'an side (location of the measurement points of CRD1 and CRD3 shown in Fig.6) [8].*

The monitoring data on deformation of the ground and stress in the support elements have been analyzed by numerical modeling of the excavation and the rock supports. Some measures have been applied to aim to reduce the tunnel deformation in order to ensure the safety of the construction and to meet the design requirement. The measures include dewatering, forepoling with micropilots, enhancement of the initial support stiffness, optimizing the excavation sequence of the CRD four parts, control the heading distance among the CRD parts and the time to complete the whole support at a cross section. The measures and the effects are summarized as follows:

(1)**Drainage**: The tunnel excavation caused loosening of the surrounding rock mass and formed fissures and cracks that provided water channels between the surrounding rock mass and the sand layer connected to the sea water. The highly weathered biotite granite disintegrated intensively with water, and thus the porosity increased. This caused decrease of the critical hydraulic gradient and thus piping took place. On the other hand, the highly weathered granite contained quartz which behaved similar to sand. Under seepage pressure, the seepage channels enlarged gradually and piping occurred too [9].

A very important issue when excavating in the highly weathered rock with fine-grained soil properties is to avoid water to enter into the tunnel changing the ground to be excavated into a muddy ground. This will reduce the risk for mudflow. An appropriate solution is to drain water. This was carried out by drilling pumping wells from ground surface and using vacuum pumper inside the tunnel. Water content thus deceases and the self-stability of the surrounding rock masses increases. The effect of drain is illustrated in Fig. 8, showing the comparison of the vault deformation at two cross sections. The vault settlement after dewatering is less than half of that before dewatering.




(a) CRD1 Vault settlement curve at Station YK12+386, before dewatering

(b) CRD1 Vault settlement curve at Station YK12+340, after dewatering

*Fig. 8 Comparison of the vault settlement of CRD1 of two cross sections before and after drainage at Xiang'an side [7]*

(2) **Forepoling with jetgrouted columns**: The excavation started forepoling in roof using Φ42 mm 3.5 m long injection pipe spaced 0.2 m to 0.3 m with overlap length 1.5 m, see Fig. 9. In the parts with very low stability also the face was reinforced with 6m long jet poles of 56mm diameter. The tube is specially designed with grouting hole of 6~8 mm spacing 10~15 cm. Both pure cement (W:C=1:1~1:1.5) and mixed cement with water glass (W:C=1:1) were used, with grout pressure ranged in 1.0~1.5 MPa, 0.5~1.0 MPa, respectively.

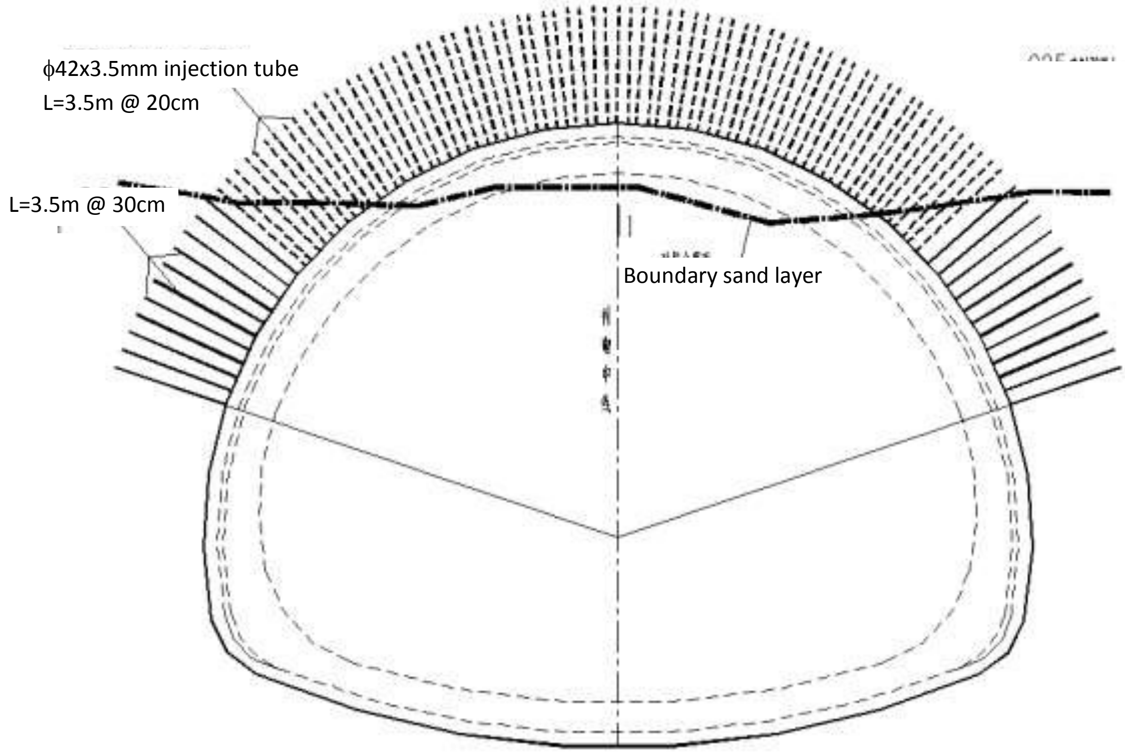



*Fig. 9 Installation of micropilots ahead of excavation*

(3) **Enhance the initial support stiffness:** The initial support was enhanced by increasing the shotcrete thickness from 16 cm to 20 cm, changing the I14 steel arch to I20 steel arch for the crossing wall, and the I20 steel arch to I22b steel arch for the side walls. By this enhancement the vault settlement decreased about 6% ~10%.


*Fig. 10 Steel arch installation in roof and invert, and the soft ground condition.*

(4) **Optimization of the excavation sequence of CRD parts:** Comparison of different excavation sequences of the CRD parts has been investigated. The sequence shown in Fig. 10 indicates the optimum solution that given the lowest roof settlement and side wall deformation, and thus it was implemented through the excavation of all the driving tunnels.

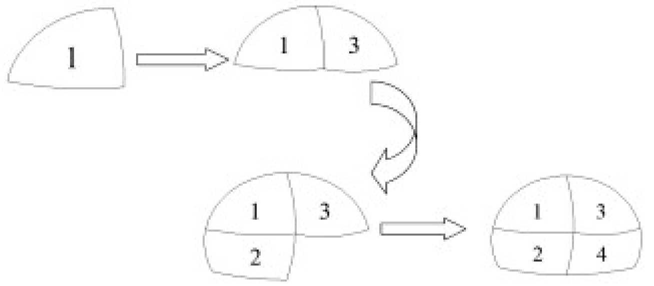



*Fig. 10 Optimum excavation sequence of the CRD parts*

(5) **Control the closure time and distance of parts:** A statistical analysis of the monitoring data of 31 cross sections shows that the distance of neighboring CRD parts tunnel faces should be about 10m and the time to complete all the four CRD parts at a cross section should be less than 40days in order to keep the deformation within a designed range.

(6) **Jetgrouted columns as arch leg**: As shown in Fig. 11, at locations of lower portion of the steel arch of each CRD part, a pair of injection pipes with length of 3.5m and diameter of 42mm was installed the same as that used for forepoling. It aimed to create jetgrouted columns as arch legs to support the arching effect. Both numerical simulation of the effect of the jetgrouted columns and the monitoring data indicated that the vault settlement is reduced about 20% ~ 40% after installation of the injection tubes and grouting. The effect in side wall deformation is however not significant.

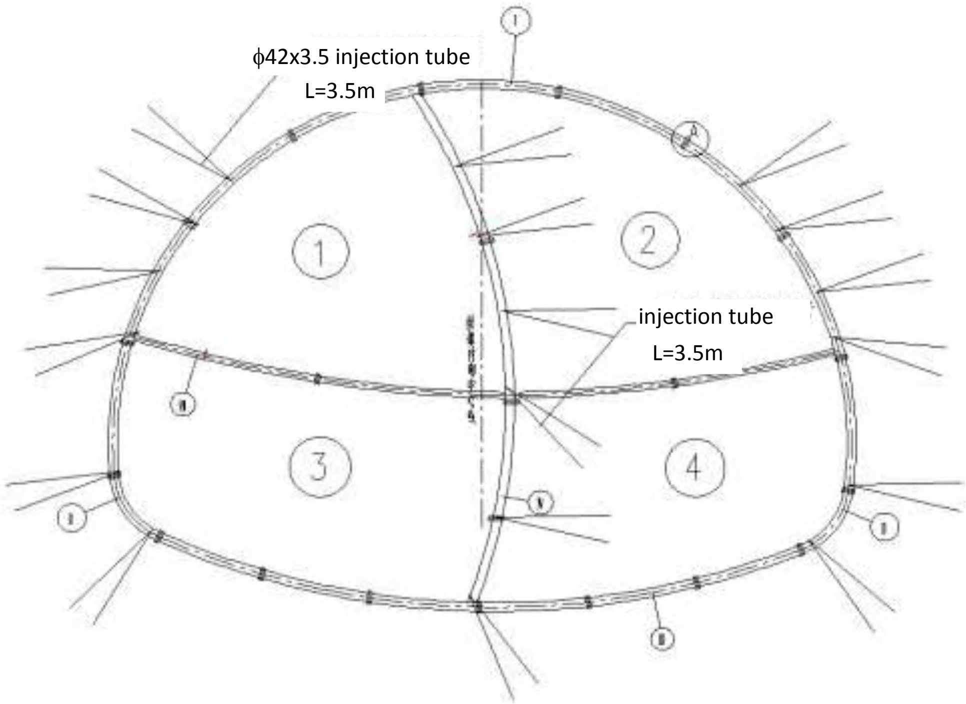



*Fig. 11 Injection tubes (ϕ42mmx3.5m) to form jetgrouted columns*

(7) **Post grouting:** To enhance the reinforcement, post grouting was carried out after the initial support, using Φ42 mm 70 cm pipes at spacing 2 m x 2 m. The spacing was reduced where there was leakage. Grouting stopped when the cement taking was less than 10L/min. At locations where there was continuous roof settlement, post grouting was also used to reinforce the invert within 1.5 m deep.

The effect of the above measure is significant to reduce the deformation and ensure the safety construction as summarized in Table 3. The monthly advance of tunneling is less than 30 m with CRD method at the beginning of the construction. After implementation of the above measures, the average monthly advance is increased to 45 m with maximum 65m and the number of the monitored displacement exceeding the reserved deformation limit reduced significantly.

It is noted that after removing the temporary steel arches in the CRD method, the monitoring data indicated that the deformation increased about 4% to 9%, the safety factor of the support elements reduced about 10%.

*Table 3 Reduction of tunnel displacement by various measures[7].*

| Measures | Roof settlement reduction (%) | Horizontal convergence reduction (%) |
|---|---|---|
| Dewatering | 50 | 30 |
| Injection columns as arch leg | 16~18 | 12 |
| Enhancement of support stiffness | 6~10 | increase |
| Post-grouting | 16~20 | 0 |
| Combination of the above | 50~60 | ~35 |
| Optimum CRD part sequence | 50 | 20 |

### 3.3 Excavation and support of sandy layer

At Xiang'an side the tunnel drives through a sand layer section of 450 m long as shown in a geological profile in Fig. 12. Even it is underlain by a clay layer, seepage and boiling of sand occurred during the excavation, showing the evidence of the connection between the sea water and the sand layer. This threatened the safety construction. Preliminary design of anti-seepage using CCP method (Cement Churning Pile) was evaluated and abandoned concerning its less efficient in water sealing. Instead, continuous concrete walls were set up to prevent the sea water into the sand layer in combination of pumping wells to lower down the ground water level, as shown in Fig. 13. The C25 pure subaqueous concrete wall of 60cm thick was embedded into the highly weathered granite 4.0 ~ 6.0 m deep in order to increase the seepage path. By this measure, the ground water conditions were improved significantly.

Other measures have been used such as forepoling with jetgrouted column, steel arch, shotcrete and post grouting as used in the soft ground on-shore. A daily advance of more than 1.5 m per day was achieved and there was no single cave in or boiling of sand incident occurred afterwards.

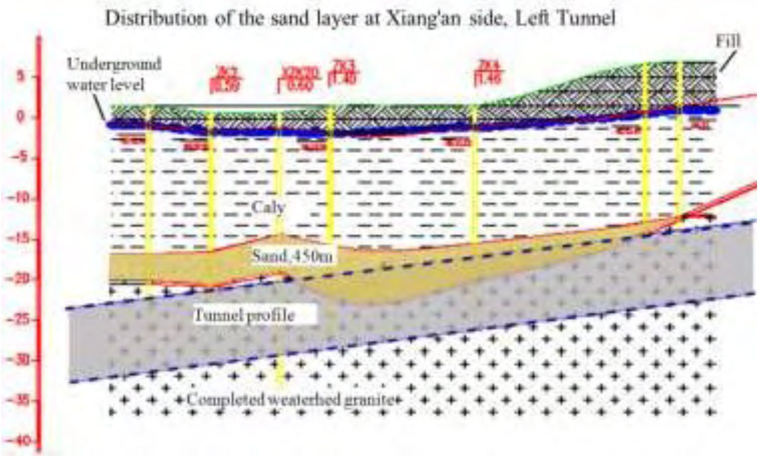



*Fig. 12. Geological profile showing the distribution of the sand layer intersecting the tunnel at Xiang'an side*[1]

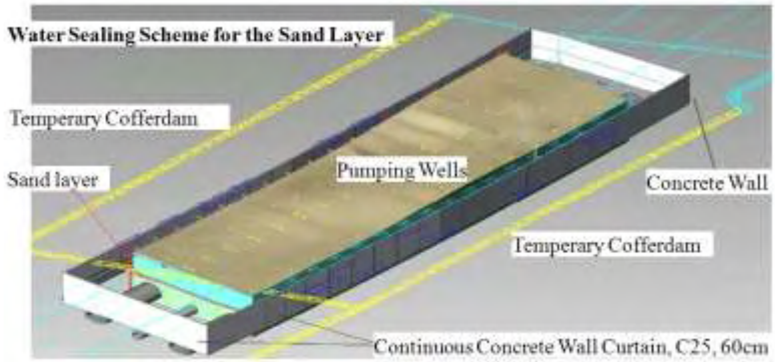



*Fig. 13. Water sealing scheme of the sand layer*[1] [10]

### 3.4 Excavation and support of weathered troughs

The most difficult excavations are expected in the poor stability conditions especially where also water inflow takes place. This happened during tunnelling through the onshore soft ground and the sand layer. Special excavation and rock support methods were used as described above. It becomes more challenge to excavate through the completely weathered troughs/fault zones in the subsea part of the tunnel with potential large amount of water inflow while few details exists on the composition and size or thickness of these faults before excavation. Forecasting of the ground conditions and pre-reinforcement before excavation are crucial important in safe tunnel construction.

As shown in Fig. 4, the geological investigations at design stage predicted steep-dipping fault zones $F_1$~$F_4$. The faults cross the tunnel at angle of 45°~80°. Fig. 5 indicates that the $F_1$ intersects the left tunnel A1, the service tunnel and the right tunnel A2 with a section length of about 70m, 89 m and 134 m, respectively. The bore hole cores indicate that $F_1$ consists of strongly weathered granite with medium to coarse particles, blocky, decomposed to sandy clay with main component of quartz, with high content of kaoline locally[11]. The highly weathered granite has lower strength and lacks of competence with static water pressure in a range of 0.5~0.7MPa.

Every effort was made to excavate through the fault zones with focus on detecting the water bearing structure and pre-grouting. A comprehensive construction procedure was planned as shown in Fig. 14. The $F_1$ trough was firstly and successfully excavated through with three rounds in A1, A2 and the service tunnels.

**Pre-investigation during excavation:**

Various pre-investigations were applied when tunnelling through the fault zones, to cover different ranges ahead of the tunnel in different ways. It is important to make sound judgement on the possible water bearing structures and poor ground conditions based on results gained from the various methods as follows:

- Geophysical methods including TSP with best operation range of 50 m~100 m ahead of the tunnel, Ground Penetrating Radar with forecast range of 20~30 m.
- Exploratory drilling holes including long boreholes of 50m or more and medium long boreholes typically 30m.

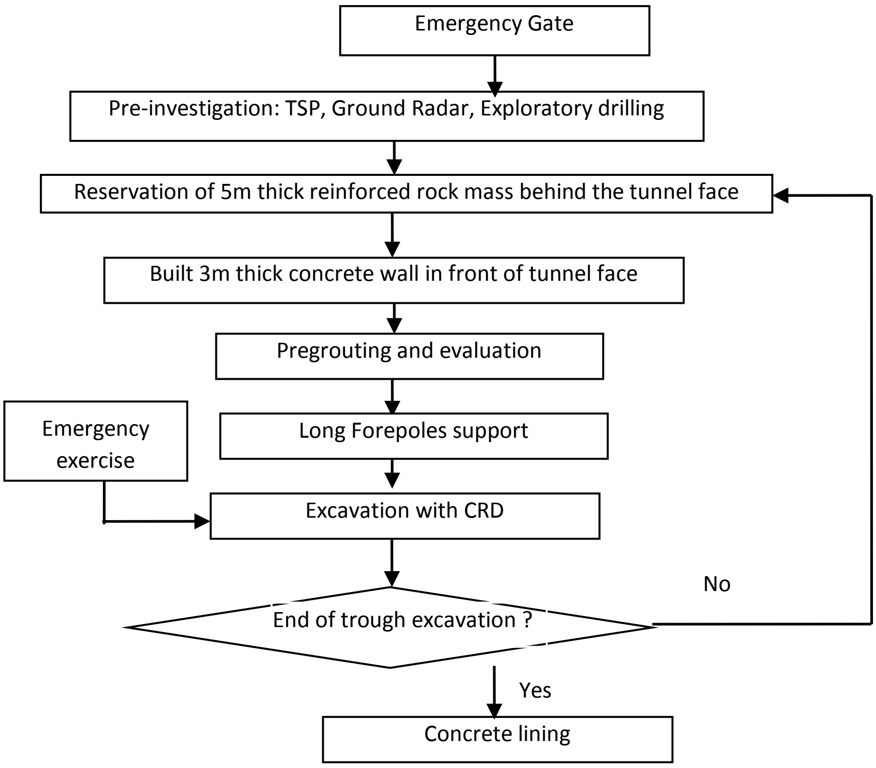



*Fig. 14 Construction procedure when excavation through the fault zones*

To forecast the ground condition of the fault zone $F_1$ TSP was operated in the range of ZK8+231～ZK8+375 in A1 tunnel that supposed to cover the trough. It concluded that the thickness of the $F_1$ crossing the tunnel is about 55 m at ZK8+270～ZK8+325, 15m shorter and 25m behind the presumed size and location from the previous geological investigation. The geological conditions in the trough are similar to what predicted. The trough is composed of highly weathered granite in the centre portion with moderately decomposed granite at the fault boundaries. Ground Radar was employed in a shorter range in 25m from ZK8+258 ~ZK8+283. The results indicated a fractured rock mass zone with potential water leakage or inflow, in line with that from the TSP.

Exploratory drilling was carried out. At ZK8+240, three holes of 90 mm diameter were drilled, SZK1, SZK2 and SZK3 with length of 36 m, 48 m and 50m, respectively, as shown in Fig. 15. At ZK8+270, one hole, SZK4 with length of 56 m was drilled and at ZK8+291, four holes were drilled, SZK5~SZK8, with length of 36 m, 38 m, 38 m and 37 m, respectively. The probe drilling holes indicated the tunnel intersects the trough $F_1$ at an acute angle with left side wall encountered the trough firstly.

The tunnel excavation revealed that the location of the faults is quite close to that determined from the pre-investigation. The $F_1$ trough is in a range of ZK8+275~ZK8+325 in A1 tunnel. Fig. 16 shows the exploratory drilling holes in A2 tunnel and the weathering conditions predicted.

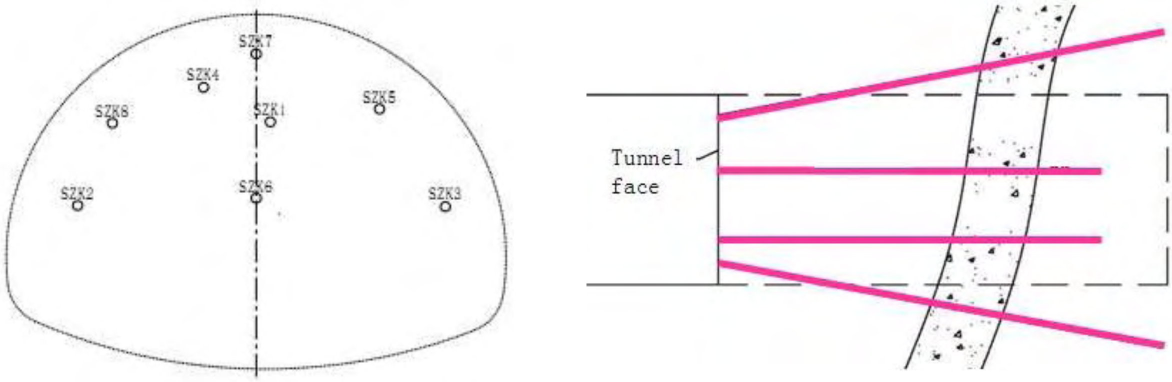



*Fig. 15 Probe driling holes in the left tunnel face at ZK8+240, ZK8+270 and ZK8+291.*

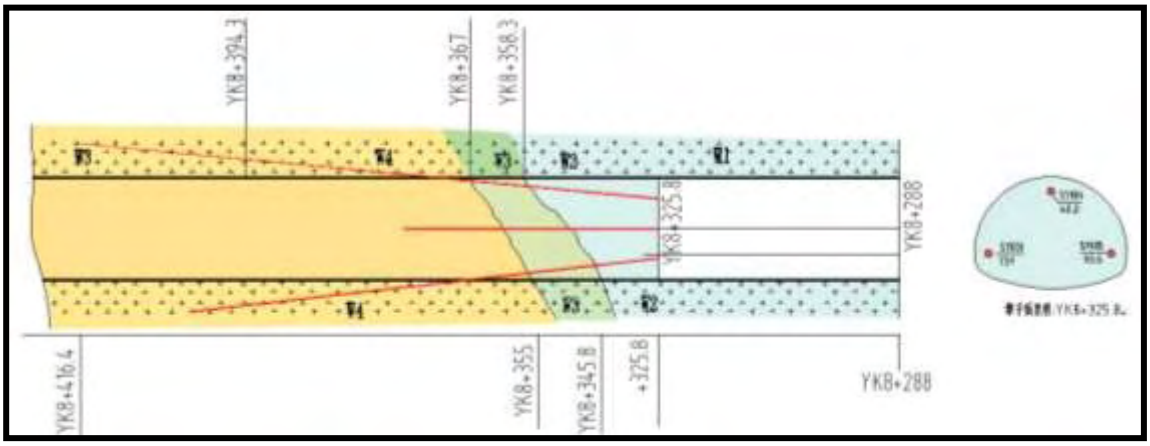
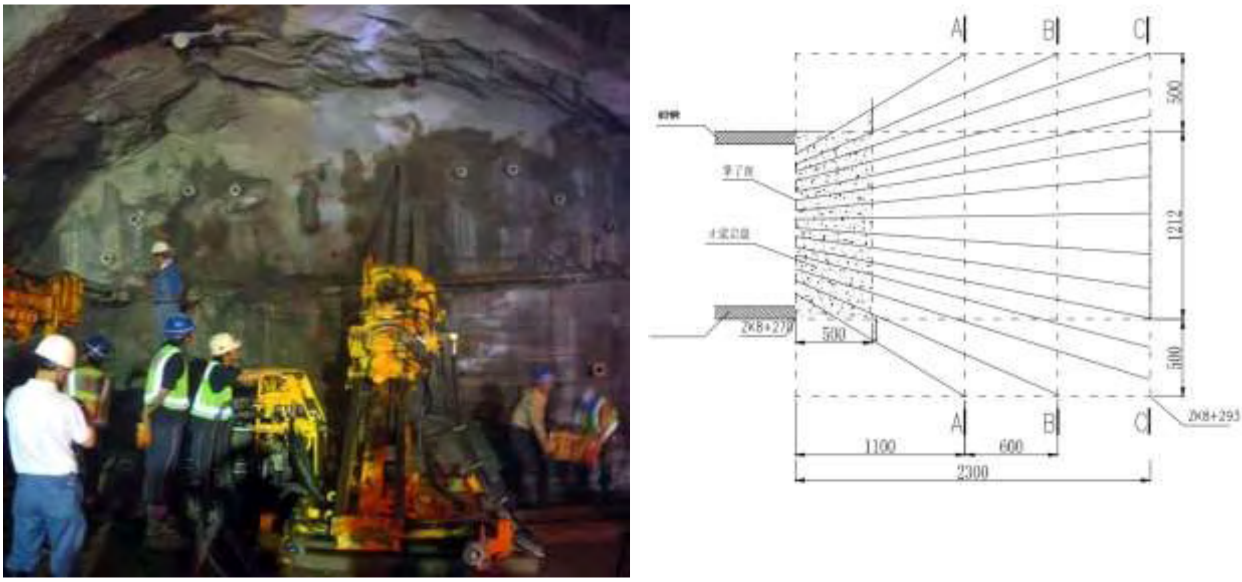



*Fig. 16 $F_1$ trough predicted by exploratory drill holes in A2 tunnel, in a range of YK8+343~ YK8+460 with overburden of 28~35 m and 20~27 m deep sea water.*

**Pre-grouting**

The pre-grouting is necessary to prevent water inflow and to reinforce the fault zone. From the information the probe holes provided, pre-grouting work of the full tunnel face was designed for trough $F_1$ in both A1 and A2 tunnels firstly with respect to holes pattern and lengths, grout mix and pressure, etc.. It started with concreting of the tunnel face. The designed grout holes were reduced according to local ground conditions. The pattern holes and the profile in A1 tunnel were shown in Fig. 17. Total number of 195 holes in Φ90 mm were drilled in the first round. The grouting holes was divided into three groups, A, B and C with drilling and grouting length of 11 m, 17 m and 23 m, respectively. Stage grouting forward with drilling hole was used mainly. The backward stage grouting starting from the bottom of the drilled hole was used in case of water inflowing from the hole or collapse of the hole. In general, grouting started from lower to higher location and from outer to inner. The grouting pressure is about 2.0～4.0 MPa. A maximum pressure of 2.5 MPa was applied for microfine cement plus water glass, and 4 MPa for ordinary cement plus water glass. The grouting speed is at 5~30 L/min. The grout consumption was controlled by 0.3～1.0 $m^3$ per meter. Depended upon the actual conditions, grouting stopped and the hole was sealed when the grouting speed was less than 5 L/min. 28 check holes were drilled. In general the bore hole cores contain the grout material and the containment of grout is higher within 15 m from the surface. Where water inflow exceeded 0.15 L/min/m hole or 3 L/min locally, 10 additional grouting holes were drilled and new check holes performed. This process continued until the check holes satisfied the design requirement. And further excavation can be started. Total three rounds of pre-grouting were executed for the F1 trough in each tunnel.




*(a)* (b)

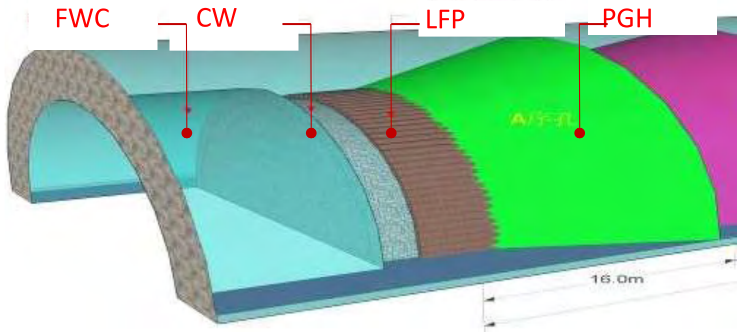
*Fig. 17 (a) The drilling of pre-grouting holes at A2 tunnel. (b) The pattern grout holes profile in group A, B and C, the first round in A1 tunnel.*

In A2 tunnel, for the $F_1$ trough, the grouting of the first round was designed with a total of 235 holes for a full tunnel face grouting at the beginning, and then changed to 184 holes in the upper portion. The contractor had adjusted this to 126 holes according to local ground conditions. 10 check holes are drilled.

**Forepoling and Excavation**

After completion of the pre-grouting each round, in A1, A2 tunnels and the service tunnel, long forepoles with a fan of injected tubes were installed in the roof above and ahead of tunnel at the forepole working chamber, shown in Fig. 18(a). The chamber is a 7.5 m long tunnel section enlarged by 1.7 m in radial direction in the roof. The injection tubes have diameter of 108 mm, δ=6 mm, 25 m long with spacing at 30 cm, angle of 5˚ to the tunnel direction. Microfine cement with W:C=(0.6~0.8):1 was used and grouting stops at pressure of 2~3 MPa. Fig. 18 (b) shows the forepoles in the service tunnel. A fan of jetgrouted column was also applied in the roof with Φ42 mm x 3.5 mm, 3 m long injection pipes the same with that used in soft ground onshore. Normal cement was used with W:C=1: (1.~1.5), grouting pressure 0.5~1.0 MPa.

The tunnel was then excavated with CRD or Benching method with the same rock supports used in the soft ground onshore. Monitoring data indicate that the tunnel deformation is stable during and after excavation of the $F_1$ trough. In A1 tunnel, service tunnel and A2 tunnel, the maximum roof settlement was measured about 62 mm, 34 mm (NK8+305) and 42 mm, respectively.




(a)

(b)

*Fig. 18 (a) Indication of Forepole Working Chamber (FWC), Concrete Wall (CW), Long Forepoles (LFP) and pre-grouting holes (PGH). (b) Forepoling with a half circular fan of jetgrouted tube of Φ108 x 6 mm, 18m in the roof of the service tunnel through $F_1$ trough.*

**Progress**

It took about 8 months to complete the first round of excavation through $F_1$ in A2 tunnel. The pre-investigation and pre-grouting works are time-consuming, about double time of the excavation. After the first round of more or less experimental, pre-grouting of full tunnel face was changed to only in the upper half tunnel face and along the periphery of the lower tunnel portion. In combination with other improvements, the construction time through other weathered troughs was shortened significantly and successfully.

## 4 CONCLUSIONS AND DISCUSSIONS

The design of the subsea tunnel was based on comprehensive geological investigation that ensured the proper location of the tunnel within sound rock formation to a large extent, and also predicted the challenge ground conditions. Studies in excavation methods, rock support measures have been carried out in advance to assist the design.

The excavation of the tunnel was successful overcoming the difficult ground conditions in the soft ground and the sand layer onshore, and the fault zones in the subsea part. The CRD method was essential to excavate large span tunnels in soft ground with intensive rock supports and water seepage control measures. Pre-investigation and pre-grouting were compulsory during excavation of the fault zones in the subsea part to prevent inflowing water into the tunnel from the sea.

A drainage system was designed including water collection basin, water pumpers and pipes. To prevent potential water gushing into the tunnel emergency waterproof steel gates were set at locations before entering into the fault zone $F_1$, which were not used.

Grouting is often time-consuming especially in the ground with unfavorable rock mass conditions. However, with application of modern equipment, grouting mix, and not least, grouting know-how and flexible grouting scheme to match the actual ground conditions, the time for grouting can be significantly reduced. On the other hand, it is necessary and meaningful to make use of low cost labor in the excavation. The construction of the Xiamen Xiangan subsea tunnel achieved a good balance between using advanced equipment with high cost and shorter construction time and using less advanced equipment with cheaper labor and longer construction time. The big surprise is that there was no death accident during the construction of such a big project in about five years. This was also contributed by the comprehensive risk management and safety control program throughout the construction period, which may be beyond the content of this paper and should be presented separately.

**ACKNOWLEDGEMENT**

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# ENVIRONMENTAL FOOTPRINT IN EARLY PLANNING OF COASTAL ROAD SECTIONS

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## ABSTRACT

*Planning a coastal road section involves combining bridges, tunnels and road sections of various types and lengths to connect locations along a route. Completed studies have shown that the impacts in terms of energy use, greenhouse gases and other emissions from construction, maintenance and operation vary greatly between tunnels, bridges and open road sections. Moreover, different bridge and tunnel concepts imply different material uses with respect to quantities and composition, as well as differences in transport demands during construction.*

*There is a need for environmental information regarding planning options for roads in the early assessment stage. This paper shows the potential for use of a modular life-cycle assessment approach to identify hot-spots and environmental trade-offs between planning options. A case study of the proposed submerged floating tunnel concept for the Sognefjord crossing is compared to alternative coastal road concepts involving route alternatives and combinations of bridges, tunnels and road lengths.*

*The findings suggest that alternative crossing concepts can involve considerable material requirements and associated emissions from the construction phase, but still be a viable alternative when considering the full life cycle emissions and the reduction in traffic emissions from reduced travel distance compared to conventional alternatives.*

## BACKGROUND

Route E39 from Kristiansand to Trondheim along the Norwegian west coast is about 1100 km long, goes through six counties, and takes about 20 hours to drive. The western part of Norway has a challenging topography with mountains and not least many long and wide fjord crossings. As a result, the coastal road is twisting, has many tunnels, and not least seven fjord crossings by ferry. In order to reduce travel time and improve communications and road standards, the Norwegian Ministry of Transport and Communications in 2010 initiated a project to investigate the possibility for a ferry-free coastal route, implying several crossings of an unprecedented scale. The estimated reduction in travel time is seven hours, cutting the travel time between Kristiansand and Trondheim to 13 hours.

A ferry-free coastal route will mean rerouting of some sections, construction of new road sections and connections, and upgrading of parts of existing infrastructure. Route E39 consists of mainly two-lane sections and has currently only 19 km of highway.

Environmental footprints, and in particular carbon footprints, of transport infrastructure have received increasing attention in the last years. Studies of infrastructure construction and use have shown that considerable emissions arise from the infrastructure itself, and not only the direct emissions from the vehicles using it. The Norwegian transport authorities have initiated work on common guidelines for assessment of environmental impacts from infrastructure, as well as a harmonization with international standards, and in particular collaboration with Swedish transport authorities (Kjerkol H et al 2010; PCR 2012). A number of studies and tools have been developed to assess the environmental impacts of infrastructure, primarily for road and rail. The Norwegian Public Roads Administration (NPRA) has developed a methodology for estimating energy use and emissions of climate gases from road infrastructure projects (Hammervold 2009). This method is implemented in the NPRA's EFFEKT tool for socio-economic cost-benefit analyses (Vegdirektoratet 2008).

## OBJECTIVE

In much of the same way as for costs, the environmental impacts from road construction is different for an open section, a tunnel or a bridge. Road specifications with respect to speed limits, traffic volumes, number of lanes etc. is also influencing the potential impacts. For road infrastructure planning this implies that evaluation of corridor alternatives should take into consideration the corridor composition with respect to infrastructure types and lengths. Environmental impacts from different infrastructure types do not only vary in the construction phase, but also in the maintenance and operation phase in a long-term perspective. Choice of corridor alternatives will in some cases also have an effect in the use phase if there are significant differences in corridor lengths between alternatives.

The project for a ferry-free coastal road includes several new fjord crossings with different alternatives for each of them. New crossings are not limited to technical solutions for the actual crossing, whether it is tunnel concepts or bridge concepts, but also requires connecting infrastructure on both sides of the crossing.

This paper aims to demonstrate how a modular life-cycle assessment (LCA) approach can be used in early planning of infrastructure to identify environmental hot-spots and trade-offs between route alternatives in a lifetime perspective. Proposed fjord crossing alternatives for Sognefjorden will be used as a case study to test the applicability of the model for early planning and screening.

## METHOD

A sound evaluation process requires transparency and an approach that encompasses all aspects of the proposed alternatives. Life cycle assessment offers a holistic and systematic methodology for assessing and comparing infrastructure projects in terms of potential environmental impacts. By including operation, maintenance and renewal of infrastructure, a more complete picture of total impacts from infrastructure projects is obtained, and potential trade-offs between the construction and operation phase can be identified and addressed. When comparing route alternatives, the direct emissions from the traffic should also be considered, as different routes can have different characteristics and distances, and thereby give differences in traffic emissions. Given the long lifetime of infrastructure, differences in traffic emissions can be significant between route alternatives.

In order to provide credible results, the underlying data and methodology need to be transparent and structured in a meaningful way to enable analysis and identification of important hot-spots.

Better informed decisions can be made if data is organized in a way that facilitates identification of critical components and processes, and understanding of relationships and parameters that are controlling the overall performance. In this paper we describe and apply a modular approach to early screening and planning of route alternatives with regard to environmental performance. By modular approach we mean that the transport and transport infrastructure are divided into modular building blocks and described with inputs and outputs. In this way we are able to separate between:

- Life cycle stages – emissions from construction, operation and maintenance, end-of-life
- Transport system – emissions from infrastructure and traffic
- Infrastructure types – open sections, bridges, tunnels
- Infrastructure components – components making up infrastructure types
- Materials – material composition of different components

Data availability in an early screening and planning phase will be limited and an assessment at this stage will be based on generic data, complemented by estimates from technical descriptions and empirical information from other projects. Later planning and construction phases offer more details and higher precision. However, generic data can be modified according to available information in order to improve the precision level, ensuring transfer of models and environmental information between planning stages. Early screening is useful for early identification of key aspects for environmental performance, potential trade-offs and identification of areas that require more analysis. Technical measures and technology characteristics are detailed in later stages of planning.

## ROAD SECTION TYPES

Road infrastructure can be divided into three main types; open section, tunnel section and bridge section. Within each type there are numerous differences depending on topography and landscape, road class, speed etc. In general, emission levels correspond with the complexity of the infrastructure and the material use. Construction of roads causes emissions in the on-site construction process, but the main impact is associated with production of construction materials. Tunnel and bridge sections include all the same major components as open sections, but do in addition have material intensive components and structures that give a considerably higher environmental footprint than a normal open section. Table 6 shows generic values for greenhouse gas (GHG) emissions, measured in $CO_2$-equivalents ($CO_2e$), associated with construction of different road section types [REF].

*Table 6: GHG emission levels from construction of road sections (Source: MiSA internal data).*

| | Construction [t $CO_2$e/km] | Total emissions, 100 years [t $CO_2$e/km] |
|---|---|---|
| Open section | 184 | 1 020 |
| Tunnel section | 1 430 | 2 230 |
| Bridge section | 6 810 | 7 360 |

The figures in Table 6 show nearly eight times as high emissions from tunnel compared to open section. Higher emission figures for tunnel sections are mainly caused by blasting processes for construction, materials for securing and tunnel portals. Concrete and steel are the dominating materials, and production of these is the major emission source. The tunnel construction is modelled without tunnel liner, which would have increased the emissions considerably due to intensive use of concrete and reinforcing steel.

Bridge sections have even higher emissions than tunnels, and for the same distance, a bridge section has approximately 37 times as high emissions as an open section. As for tunnels, the reason is the large material quantities involved in the construction, and mainly concrete and steel for the structure and foundations.

Both tunnels and bridges are more complex constructions than open sections, and they are designed for a long lifetime. Table 6 also shows total GHG emissions from construction, operation and maintenance/renewal of the infrastructure types. Tunnels and bridges are assumed to have a lifetime of 100 years, while open sections have a shorter lifetime of 40 years and are adjusted accordingly. The big differences in the construction phase become smaller when considering the whole lifetime of tunnels and bridges. However, the differences are still significant, with more than twice and seven times as high emissions for tunnels and bridges, respectively, compared to open sections.

The figures presented in Table 6 are generic figures for two lane roads, and other road classes will give different results, depending on technical characteristics, requirements, traffic loads etc.

## ROAD TRANSPORT

Considerable emissions are associated with construction, operation and maintenance of road infrastructure, and significant reductions can be achieved by optimizing the route and route composition. However, in a life cycle perspective the use of the road is even more important and should be taken into consideration also in the planning of new infrastructure projects. A simple calculation exemplifies the importance of the traffic on the road. Comparing construction, maintenance and operation of infrastructure for a 100 year period, with transport emissions for the same period, shows that infrastructure emissions are 3%, 6% and 21% those of the transport for open section, tunnel and bridge, respectively. Transport emissions are based on an average annual daily traffic (AADT) of 5 000, with a 5% share being trucks, and emission intensities for present vehicle technology. Even if the average emissions intensities for car and truck are reduced by 50%, the transport on the road is still by far dominating compared to infrastructure emissions.

## THE SOGNEFJORD CROSSING – SUBMERGED FLOATING TUNNEL

The fjord crossing between Lavik and Oppedal in the Sognefjord is one of the most challenging parts of a future ferry-free E39 coastal route, and is presently the world’s longest fjord crossing concept with its 3.7 km length. Several concepts have been proposed for this crossing, including suspension bridge, floating bridge and a third concept for a submerged floating tunnel. In this paper we will investigate the concept with a submerged floating tunnel in more detail, and compare the climate related emissions from this concept with conventional road sections and crossing alternatives.

A feasibility study has been carried out for the proposed submerged floating tunnel (SFT) concept by the Reinertsen Olav Olsen Group on assignment from the NPRA (Fjeld 2012). The concept consists of two submerged concrete tunnels fastened by steel shafts to floating pontoons, and by bracings between the tunnels for lateral stability. Landfall sections, partly in rock tunnel, connect the floating tunnel to land. Both tunnel tubes have two driving lanes, whereof only one is used for traffic and the other is intended for emergency stops, maintenance and in general act as a buffer to avoid traffic disruptions. The tunnels are dimensioned with T9.5 profile. A detailed technical description is provided in the feasibility study.

**Materials inventory for submerged floating tunnel concept**

The feasibility study provides estimates for the material use associated with the main structural components of the proposed concept (Fjeld 2012). Material estimates are limited to concrete and steel components in the feasibility study, but in the inventory and results presented here, the system is expanded to include rock tunnel construction for the landfall sections, construction and maintenance (repavement) of driving lanes, and energy use for lighting and ventilation of the tunnels. The inventory therefore includes all main components necessary for comparison with alternative infrastructure. Table 7 shows an overview of the concrete and steel quantities associated with structural components for the SFT concept, as described in the feasibility study. Only material for in-place structural components is included, i.e. material use for temporary elements is not reflected in the table.

*Table 7: Estimated use of concrete and steel for structural elements for the SFT concept (Fjeld 2012).*

| Structural element | Concrete volume [$m^3$] | Reinforcement steel [tons] | Steel for post-tensioning [tons] |
|---|---|---|---|
| Submerged concrete tunnel | 349 144 | 78 471 | 20 960 |
| Landfall elements | 49 668 | 11 128 | 3 364 |
| Steel components | - | 61 482 | - |
| **Total** | **398 812** | **151 081** | **24 324** |

Most of the material use is related to the tunnel sections and the main hull for the traffic tubes. Bracings and brace joints, shafts and shaft bases are other important components for the concrete use. The same elements are important for the steel use due to reinforcement of concrete elements, but in addition there are large amounts of steel needed for the pontoons, and for the shafts connecting the pontoons and the tunnels.

Other potentially important factors that are not described or quantified in the inventory are related to construction operations, most importantly; dock reconstruction for facilitating construction of tunnel sections, transport of tunnel sections from dock to the Sognefjord, on-site assembly of tunnel sections to full crossing, and construction of on-site mooring installations for tunnel sections prior to assembly.

Table 7 provides a material inventory that is merely an estimate and that does not include other materials than concrete and steel for the main structural elements. However, concrete and steel for these elements will be the dominating inputs for the construction even if a detailed inventory comprising all material and energy inputs had been developed, and it therefore provides sufficient information for use in early screening of emissions from infrastructure solutions. The estimated material quantities are regarded as conservative estimates.

**Results for submerged floating tunnel concept**

Figure 16 shows main results for the construction of the submerged floating tunnel concept with regard to climate emissions, and the relative contributions from main structures and their sub-components. Total emissions from construction are estimated to 432 000 t $CO_2$e. The SFT is modelled with three main structures; the submerged concrete tunnel, landfall sections, and structural steel components the anchoring and securing of the tunnel. The most material intensive constructions are the twin tunnel tubes with large quantities of concrete and steel, as described in Table 7. These make up more than 2/3 of the total construction emissions. The second most important structures are the steel pontoons and shaft with more than 20%, while the landfall sections account for approximately 20% of total construction emissions. Construction of driving lanes is also included, but this has a negligible contribution relative to the massive concrete and steel constructions.

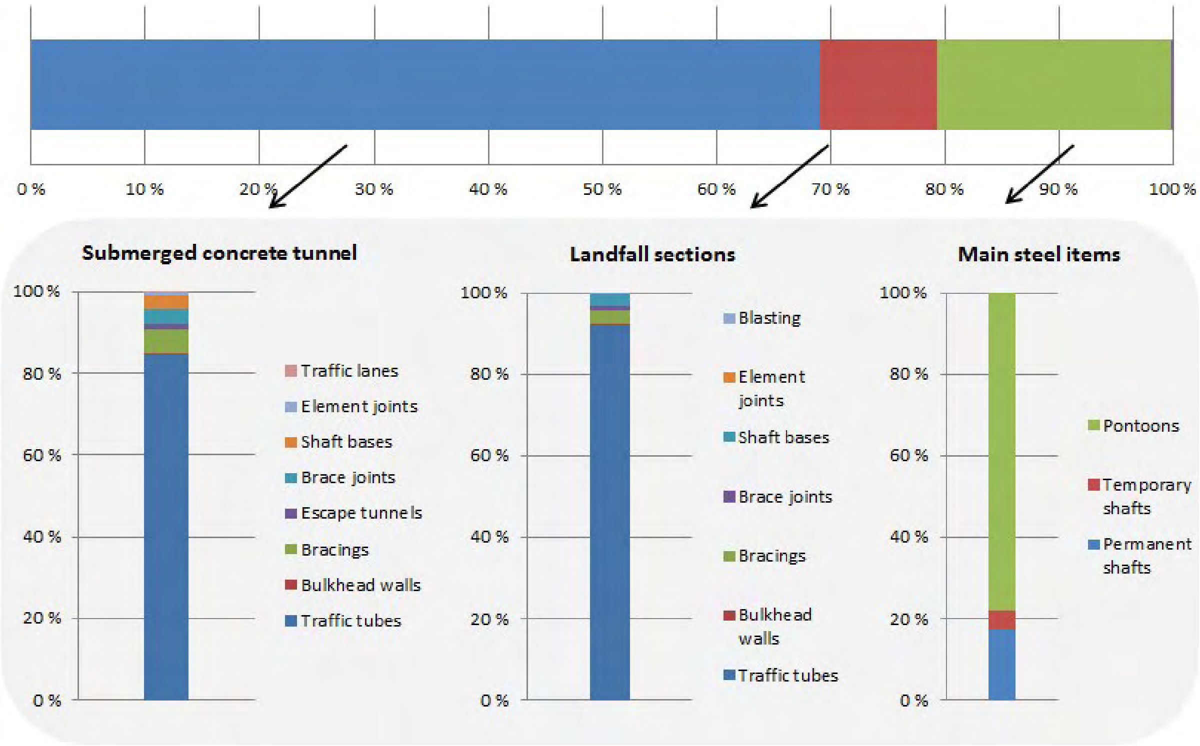


*Figure 16: Main results for climate emissions from construction of the submerged floating tunnel concept.*

Several components like bracings, brace joints, shaft bases etc. contribute to the results, but the main emissions are associated with the traffic tubes and their construction. The inventory is largely based on estimates for concrete and steel use for structural components, and the emissions results are therefore related to production of these materials. For construction of the full SFT crossing concept, approximately 50% of emissions come from production of reinforcement steel, while about 25% come from concrete production and another 25% from high quality construction steel used for post-tensioning. Emission factors are sensitive to different steel types and can potentially shift the total emissions in both directions.

## ENVIRONMENTAL FOOTPRINT FOR ROUTE PLANNING

Early planning of route alternatives involves uncertainty with regard to many aspects, including environmental performance in addition to technical feasibility, economic performance, land use planning etc. In order to make informed decisions about the potential environmental impacts, all relevant aspects need to be taken into consideration. This implies including infrastructure as part of the analysis when assessing environmental impacts.

## Infrastructure is important

Table 6 shows that choice of road section type gives significant variations in emission levels from construction, operation and maintenance of infrastructure. In Table 8 the total emissions are summed up for a 100 year period for 1 km of different infrastructure types. Direct emissions from traffic are also included as described in earlier in the text. All results are based on assessment of current technology, i.e. technology improvements are not accounted for.

*Table 8: Emissions levels and relative differences for infrastructure types and traffic.*

| | Assumed lifetime [yr] | Total emissions 100 yr [t $CO_2$e/km] | Relative [%] |
|---|---|---|---|
| Open section | 40 | 1 020 | 3.5 |
| Tunnel section | 100 | 2 230 | 8 |
| Bridge section | 100 | 7 360 | 26 |
| Submerged floating tunnel concept | 100 | 118 000 | 410 |
| Car transport, direct emissions | - | 28 800 | 100 |

The rightmost column provides relative results with direct emissions from the traffic as the reference value. This shows that for standard road sections the use of the road, i.e. traffic emissions, is the most important component in a life cycle assessment of road transport. But the results show that infrastructure is indeed important for the overall environmental performance of road transport, and that route planning should take into consideration the life cycle emissions when assessing and planning future road transport solutions. Even if traffic emissions are dominating the total emissions, it is far from irrelevant whether the infrastructure consists of tunnels, bridges or open section. Construction, operation and maintenance of a bridge account for one fourth of total emissions in a 100 year period, and must be considered a significant contribution to overall emissions.

The results for the proposed fjord crossing with a submerged floating tunnel demonstrates that this type of large and complex infrastructures imply significant emissions from the construction phase due the large material quantities required. Per kilometre of tunnel, the SFT concept is four times as emission intensive as the emissions from traffic in a 100 year period. This is very high for infrastructure, but the SFT is also a special case. However, Table 8 presents results per km of infrastructure and transport, while the purpose of installations like the SFT is to reduce travel time and travel distance.

## Route planning

The previous sections have presented information about emissions from different infrastructure types and use of the infrastructure. Results per km provide interesting insight, but the results need to be put into the context of route planning. The purpose of fjord crossings is to reduce time and distance for travel by avoiding going around the fjord. This also implies a reduction in the length of the infrastructure sections.

In Table 4 life cycle infrastructure emissions and traffic emissions are added together to provide total transport emissions. In addition, fjord crossing by conventional ferry is included in the table, and this option has the highest total emissions. The high emission figures from conventional ferry are caused by direct emissions from diesel combustion, while construction, operation and maintenance of pier infrastructure and vessels account for only a minor part.

Emissions from ferry transport are sensitive to local conditions that influence size of ferry, number of departures, sailing distance etc.

*Table 4: Total and relative emissions for different transport solutions.*

| | Total emissions 100 yr [t $CO_2$e/km] | Relative [%] |
|---|---|---|
| Open section | 35 400 | 23 |
| Tunnel section | 36 600 | 24 |
| Bridge section | 41 700 | 27 |
| Submerged floating tunnel concept | 153 000 | 100 |
| Ferry transport | 198 000 | 129 |

The figures in 4 indicate that alternative crossing concepts like the proposed submerged floating tunnel for the Sognefjord can be a viable and environmentally sound alternative, depending on the characteristics of the alternative solutions. The SFT concept is preferable to conventional ferry transport and preferable to standard road sections if the driving distance can be reduced by approximately four km per km of SFT crossing. The main benefits come from reduced direct emissions from traffic, demonstrating the importance of including reduced travel distance in the assessments of infrastructure planning.

Figure 17 shows a hypothetical fjord with different route alternatives for going from one side of the fjord to the other. In an early screening of environmental performance, information like that presented in the above tables can be used to assess different alternatives and provide some key information regarding route planning choices from an environmental perspective.

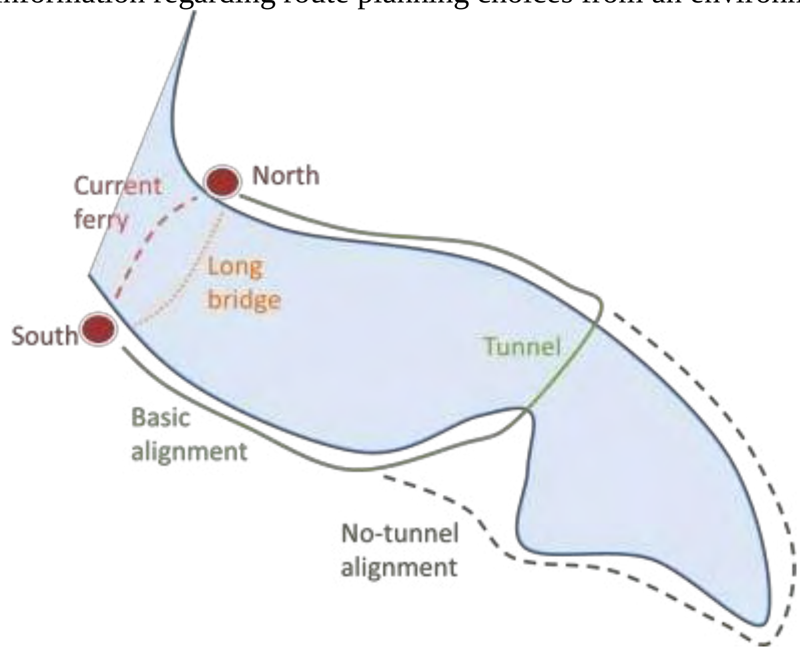


*Figure 17: Route planning alternatives for a fjord crossing.*

Several route alternatives are indicated in Figure 17:

- ❖ Current route alternative: conventional ferry crossing
- ❖ Alternative 1: fjord crossing with a long bridge
- ❖ Alternative 2: road section to a narrower part of the fjord and a tunnel to the other side
- ❖ Alternative 3: road around the fjord

These are only some potential alternatives. The road sections can be comprised of tunnels and open sections depending on the topography, settlements can require road connections, bridges can be located at different locations etc. Information about infrastructure types and traffic loads can be used as a tool for early planning and assessment of different alternatives.

## CONLUSIONS

Results from the proposed SFT concept for crossing the Sognefjord show that there are considerable emissions associated with large and complex infrastructure installations of this type. However, different route planning alternatives need to take into consideration the full life cycle emissions from the infrastructure and the use of the infrastructure. Fjord crossings are designed and constructed to save time and travel distance and will thereby reduce traffic emissions, which are the main emissions source for road transport. Generic data can be used in early screening of alternatives and provide useful information at an early stage. Assessing projects and concepts at an early planning stage implies uncertainty and should preferably be adjusted as far as possible to local conditions.

Direct emissions from traffic are dominating the total emissions in the results presented in this paper since only climate related impacts are considered. Including other impact categories would put more emphasis on other parts of the system and on infrastructure in particular, and is important to keep in mind in an environmental assessment that should preferably go beyond only climate effects.

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# PLANNING OF COMPLEX IMMERSED TUNNELS IN WAVES AND CURRENT

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## ABSTRACT

*Underwater tunnels will be constructed under more and more complex conditions; tunnels will become larger, longer and deeper. This paper demonstrates that complex project conditions can be addressed successfully with good planning by the owner, designer and contractor. Underwater tunnels that seem almost unfeasible to construct can be carried out with an acceptable risk by adding specific studies and investigations as a basis for the design and construction. Planning requires efficient collaboration between experienced designers and contractors.*

*The paper initially describes key issues defining the layout of a tunnel, e.g. traffic requirements, ventilation, fire safety and water tightness. Given the project requirements for tunnel operation, safety, durability and environmental impact, the construction of an immersed tunnel is often governed by the site conditions, such as the geological and hydraulic conditions, soil-parameters, waves and currents etc. The selection of tunnel type (e.g. bored or immersed), with or without membrane (for immersed tunnels), with cast monolithic or segmental elements etc. are choices made by designer/owner based on the impact on the operational risk. Immersed tunnel element length, location of the casting yard and the concrete casting technique will impact the construction cost and time-schedule.*

*This paper specifically elaborates on construction issues related to immersion with a focus on the hydraulic studies as basis for the planning, design and construction of the temporary works and installation of tunnel elements. The paper includes examples from the following recent long, deep, complex immersed tunnels: Øresund, Busan, Bosphorous and Hong Kong - Macao. The paper also shows examples on how to establish loads and movements of tunnel elements during transport and immersion. Issues like weather windows and forecast systems are proved to be useful tools to control project risks. Finally the paper presents future challenges for underwater tunnels where new technologies are needed.*

## INTRODUCTION

In recent years immersed tunnels projects world wide have been constructed under more and more complex conditions. Large diameter TBM and immersed tunnel technology have been pushed to the limits. Soft soil, seismic action and high waves are challenges to be faced by designer and contractor of subsea tunnels in addition to the water pressure.
This paper demonstrates that complex project conditions can be handled successfully with good planning by the owner, designer and contractor. Underwater tunnels that seem almost unfeasible to construct can be carried out with an acceptable risk by adding specific studies and investigations as a basis for the design and construction. Planning requires efficient collaboration between experienced designers and contractors.

## IMMERSED TUNNEL DESIGN PARAMETERS

### Operational requirements

The parameters for determining the cross section layout of a tunnel comes from the requirements from a) traffic volumes, b) ventilation and c) safety, as stated in the European directive for minimum safety requirements in tunnels 0. Tunnel requirements on safety, traffic and ventilations are all linked and shall for each tunnel project be described in the tunnel safety concept. Requirements for immersed tunnels are similar to the requirements for cut-&-cover tunnels, bored tunnels and for drill & blast tunnels. There are however some fundamental differences in the way the requirements are addressed, it is for example much easier to make cross passages for fire escape in an immersed tunnel between two bores, than it is for a bored tunnel in soft soils. For bored tunnels in soft soils cross passages can be made either with freezing or with grouting. The layout of the tunnel depends on the safety concept established from project to project.

For tunnels with large traffic volumes and for tunnels with combined roads and rail, an immersed tunnel solution has an advantage over bored tunnels, because the layout of the immersed tunnel can be tailor made to suit the large number of lanes e.g. Øresund tunnel (Denmark/Sweden) and Fehmarnbelt crossing (Denmark/Germany). For a bored tunnel that combines roads and rails would require large diameter TBM or several bored tubes. The global trend is that bored tunnels with time are becoming larger and larger and are now used under many different conditions.

There is another advantage for immersed tunnels compared to a bored tunnel. The alignment for an immersed tunnel can usually be right below seabed level, whereas a bored tunnel often needs to be located deeper. It also means that the immersed tunnel is exposed to accidental ship impact and hydraulic loads during construction and at permanent stage, and this has to be taken into consideration in the planning and the design. Fig. 1 shows the main differences between the general layout of immersed tunnel and bored tunnels

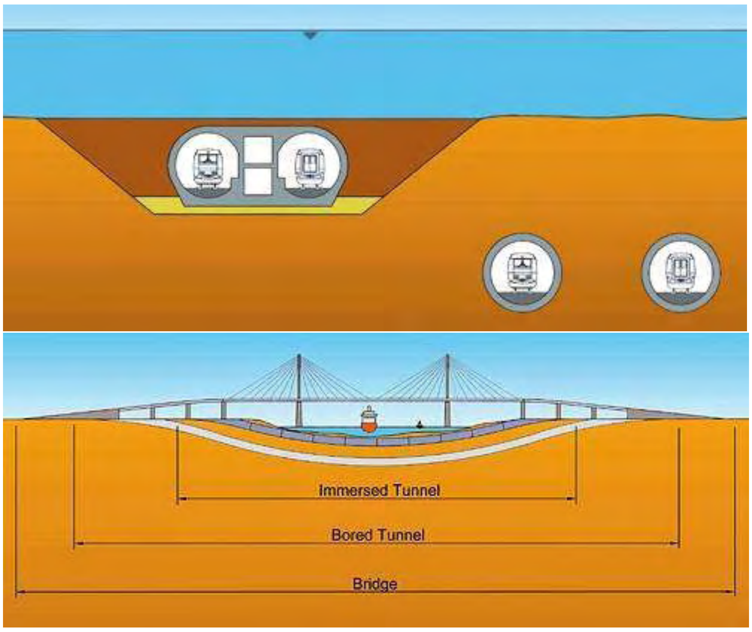


*Fig. 1 Typical comparison of immersed tunnel and bored tunnels (ITA)*

**Structural and durability requirements**

The main structural requirement of a sub-sea tunnel is for the structure to maintain its water tightness throughout the lifetime of the project. A design lifetime of around 100 - 120years has been the norm for recent major sub-sea tunnel projects. The durability of a sub-sea tunnel is directly linked to the ability of the structure to avoid water ingress through cracks and at joints. Immersed tunnels exposed to seawater need to be watertight to avoid chloride (born by the water) reaching the inside of the tunnel and in combination with air, will trigger continued corrosion at locations that can be very difficult to locate and repair.

Immersed tunnel can be categorized into 3 types.

1) Monolithic type Immersed Tunnel
2) Segmental type Immersed Tunnel
3) Sandwich type immersed Tunnel

Monolithic immersed tunnels are constructed with elements that are around 100-140m long. Between the elements are the immersion joints, where the Gina gasket and Omega seal are placed, see Fig. 2. The monolithic tunnel acts a long beam with rotation joints at the immersion joints.

Due to the element length the bending moment and tension from restraints can create through-going cracks in top and bottom slab. It is therefore required to have an external waterproofing membrane and the longitudinal reinforcement can also be significant. The immersed tunnel at Preveza in Greece is constructed as a monolithic tunnel 0.

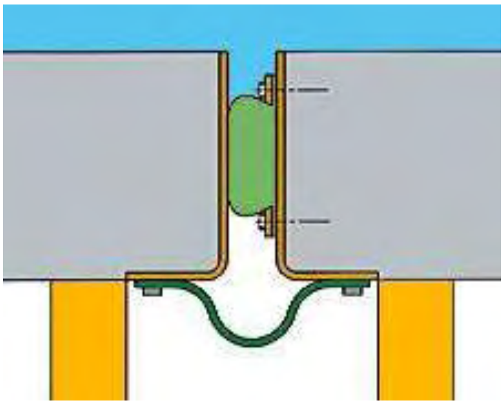

*Fig. 2 Element joint showing Gina and Omega profiles*

To avoid building up large tension forces in the longitudinal direction, the tunnel elements can be divided into smaller segments, hence termed segmental tunnel.

The advantage of the segmental tunnel compared to a monolithic tunnel is to i) avoid a waterproofing membrane, ii) reduce the amount of longitudinal reinforcement and iii) increase the element length up to 160-180m, hereby reduce the number of immersions with associated reductions in cost and time. But the segmental tunnel also has some disadvantages compared to a monolithic tunnel; i) there are many segment joints (5-7 per element) often with complex geometry and ii) the rotation capacity of the segment joints are much smaller than for a segment joint. Therefore segmental tunnels, have so far never been constructed in a high seismic zone.

Sandwich type tunnels are constructed with an outer and inner shell of steel plates. The plates are connected with steel bracing/supports. The space between the outer and inner steel plates are filled with concrete, forming the sandwich structure. The steel and the concrete also form a watertighness barrier. This immersed tunnel type is often used in Japan and in limited scale elsewhere, recently immersed tunnels at Bosphorous in Istanbul and Söderstrømmen in Stockholm 0 have been designed with an outer steel shell.

In any case, the control of settlements is important for all immersed tunnel types. The selection of immersed tunnel type is a complex choice and depends on many factors that are different from project to project. Studies of the advantages and disadvantages of each tunnel can be carried out for a specific project and the choice can be made on this background. In 0 the process of selecting the immersed tunnel type for the HZM is described in more details.

Site and construction conditions

Geological conditions

Site conditions for a sub-sea tunnel strait crossing projects are often the most important factor for the selection of the construction methodology. The deepest immersed tunnel constructed in the world is the Bosphorous tunnel (depth 65m ) for the Istanbul metro, the deepest for road traffic is the Busan - Geoje immersed tunnel in South Korea (depth 45m) 0 and the immersed tunnel with the largest overburden (20m siltation) is the Hong Kong - Zhuhai - Macao (HZM) Immersed tunnel (depth 40m) 0. The geology conditions are suitable for an immersed tunnel at all these locations, a bored tunnel at these three projects would require deeper alignments and/or specially developed TBM to cope with the cross section size and geology.

The cost of dredging can be up to 20% of the total construction cost, for that reason it is important to know the material to excavate. Control of settlements is important to control for the section forces in the tunnel elements and for the joints movements between elements. In case the tunnel is constructed on soft soils, the control of settlements can be achieved by soil-improvement, see photo Fig. 3 or with bearing piles. Recent immersed tunnels (Busan, HZM, Bjørvika and Söderstrømmen) have used various forms of soil-improvement and piles to control settlements.

*Fig. 3 Equipment for ground improvement in Busan*

Seismic conditions

Seismic conditions are often governing in the selection of the joints and immersed tunnel type, to assure water tightness also after a seismic event. As for other types of underground structures the effect of seismic events on an immersed tunnel structures are not as severe as for super structures above ground (like bridges and buildings). Because of the limited predictability of joint movement in areas with high seismic activity, joint designs are often made much more robust in for example Japan, compared to areas with limited seismic activity for example Northern Europe. Tunnel movements can be calculated and analysed as a function of the bedrock movement, depth to bedrock, shear wave velocity, dynamic stiffness of the ground and the stiffness of the tunnel structure. The reliability of such dynamic models is not better than the input to the seismic models.

In some cases the foundation material have a risk of liquefaction, in such cases the immersed tunnel foundation can be supplemented with for example stone columns to increase the permeability and drainage during seismic event. Such solution was applied for the immersed tunnel in Preveza, Greece (2001) 0.

Hydraulic conditions

The transport and immersion operations are critical activities for an immersed tunnel project, if there is large tidal range or large current it has on impact on the equipment and the methods used for immersion. It is therefore important at an early stage to get all the hydraulic parameters established to assure feasibility and the best possible construction methodology.

Often immersed tunnel elements are constructed at one location and then transported a long distance to the immersion location at the alignment. For the Bjørvika tunnel the elements where transported 500km from Bergen on the west-coast of Norway to Oslo, for the Söderstrømmen, Stockholm the elements (i.e. the outer steel shell) where constructed in Estonia, transported by barge across the Baltics and sailed through the canals to the centre of Stockholm where they were immersed. Both projects required precise knowledge about hydraulic conditions along the transportation route in order to control the risk.

In some cases the tunnel elements need to be stored temporarily until immersion of the elements is possible. Maybe the availability of the dry-dock is restricted to a certain period, maybe the land connections of the tunnel is not ready when the tunnel elements are completed in the drydock, maybe the immersions can only be carried out at a certain time of the year (e.g. to avoid typhoons). There are many factors different from project to project, that determines if a storage area is needed or not, and how many tunnel elements, it should be able to keep.

For the Busan Immersed tunnel, South Korea the tunnel alignment is located between two islands exposed to typhoon waves from the Pacific, see site photo Fig. 4. The waves, especially long period waves from distant storms, was a demanding challenge. With good planning, numerical and physical modelling of tunnel element behaviour under mooring, transport and immersion of tunnel and wave forecasting, the operations could be carried out with an acceptable risk for the project. For the permanent stage the immersed tunnel was designed against impact from large waves passing from typhoons 0.

Detailed analysis was carried out by the main-contractor and the consultant, later the studies where further detailed by the specialist immersion contractor and refined after each immersion. A chapter later in this paper shows how the complex marine operations where analysed. Based on the studies the construction methodologies where confirmed to be feasible and later the studies were refined to increase the amount of weather windows, and thereby keep the construction time schedule.

*Fig. 4 Overview of Busan Geoje Offshore tunnel alignment*

## HYDRAULIC LOADS ON TUNNEL ELEMENTS

This chapter is based on the specific conditions governing the Busan-Geoje immersed tunnel project, where the immersion of tunnel elements took place in the period 2007-2010. The installation of the immersed tunnel consists of three processes. The first is the towing of an element (TE) to the immersion site. The second is the immersion at the sea bottom and the third is the backfilling around the element. Physical model results demonstrate that the installation of the TE is heavily dependant on the environmental (sea & current) conditions at site. Small changes in the wave conditions (Hs, Tp, spectrum) or current velocity can bring significant increase in the loads in the wires used for towing and immersion, the bending moments or the movements of the TE when it is close to touch down.

After the immersion at the sea bottom and before the locking fill is placed, the element is vulnerable to sliding due to wave forces. Physical modelling shows that the placing of ballast weight must be carried out as quickly as possible in order to prevent displacement of the tunnel.

### Loads during transportation

The governing conditions for the bending moments (i.e. for longitudinal reinforcement or temporary pre-stressing) and the towing forces during transportation (i.e. for wires and bollards) are the towing speed (V), the current (C) and the wave conditions along the route.

For the Busan project, physical model tests where carried out for a limited number of current and wave directions together with different towing headings as shown in Fig. 5.

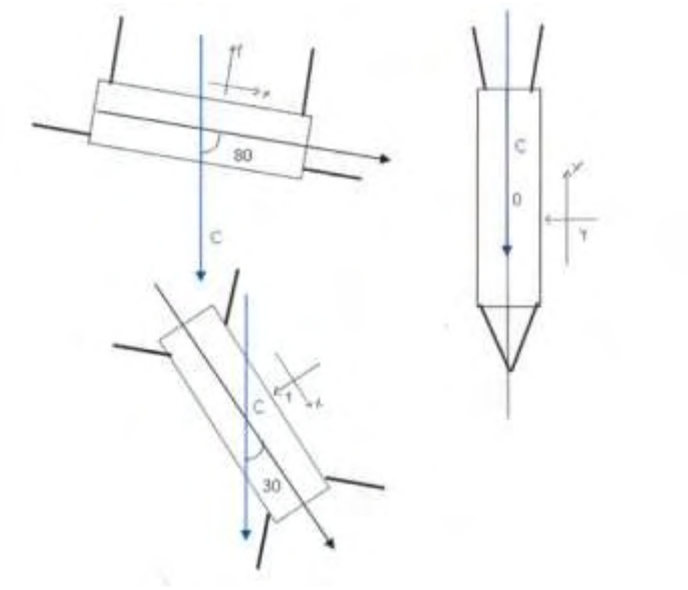


*Fig. 5 Tunnel towing configuration in respect to wave/current directions*

A first analysis of the data, for when only the current is applied, shows that the maximum towing forces and the maximum vertical bending moments increase with the current/towing speed. The presence of the current in combination with the waves alters the kinematics of the waves. When waves are applied, the bending moment Mb increases with wave period Tp until a "saturation" condition is achieved approximately when the Tp corresponds to a wave length that is comparable with the exposed TE surface to the wave. Before reaching saturation, a linear dependency between the Mb and the Tp can be found as follows:

$$M_b(\alpha \cdot H_s) = \alpha \cdot M_b(Hs) \qquad (1)$$

and

$$M_b(wave.spectrum) = \sqrt{M_b(sea.wave)^2 + M_b(swell.wave)^2}$$

Such a relationship can be used to calculate the bending moment for any wave height and wave period combinations given the original results from the physical model. With all data combinations it is possible to make Hs/Tp matrixes showing the wave conditions in which it is possible to carry out towing operations. The go-ahead conditions are dashed in green (Normal conditions), wave conditions beyond the Normal conditions are dashed in yellow and the ones beyond the Survival conditions in red Fig. 6.

| Hs/Tp | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
|---|---|---|---|---|---|---|---|---|---|
| 0.8 | 33.6 | 52.5 | 71.5 | 90.4 | 118.5 | 142.7 | 162.9 | 179.0 | 193.3 |
| 0.7 | 29.4 | 46.0 | 62.5 | 79.1 | 103.7 | 124.9 | 142.5 | 156.6 | 169.1 |
| 0.6 | 25.2 | 39.4 | 53.6 | 67.8 | 88.9 | 107.0 | 122.2 | 134.2 | 145.0 |
| 0.5 | 21.0 | 32.8 | 44.7 | 56.5 | 74.1 | 89.2 | 101.8 | 111.9 | 120.8 |
| 0.4 | 16.8 | 26.3 | 35.7 | 45.2 | 59.2 | 71.4 | 81.4 | 89.5 | 96.6 |
| 0.3 | 12.6 | 19.7 | 26.8 | 33.9 | 44.4 | 53.5 | 61.1 | 67.1 | 72.5 |
| 0.2 | 8.4 | 13.1 | 17.9 | 22.6 | 29.6 | 35.7 | 40.7 | 44.7 | 48.3 |

*Fig. 6 Example of Hs/Tp matrix used to define the conditions for towing.*

Load effects on the tunnel elements from long waves (i.e. swell) are more severe than for shorter waves (i.e. sea) with the same wave height. As there is always a risk that swell waves can occur at the immersion site, it is recommended that these are detected by a reliable weather forecast system before the tunnel element actually arrives at the immersion site.

**Loads during immersion**

The immersion operations start when the TE arrives with the tug-boats at the immersion location and has been turned into position.  The operational wave conditions for immersion depend on the type of immersion equipment that is used, capacities of cast-in items on the elements, wire-configuration, stiffness and ballast overweight. A traditional immersion with 2 immersion rigs each with two pontoons was chosen for the Busan project.

The installation operation was divided into 3 phases for the purpose of studying the forces in the wires, the TE movements, deformation and speed in response to waves and current loads:

- Phase I: Immersion of the Tunnel Element (TE);
- Phase II: TE in Final Position without locking fill;
- Phase III: Placing of locking fill and backfill.

During Phase I, the TE is manoeuvred in the direction of the catch on the previously immersed element and lowered with a maximum speed of 1cm/s. The TE is suspended from two immersion rigs by two suspension wires from each rig. The immersion rigs are moored each by four mooring lines. Additionally two contraction lines and longitudinal lines connect the immersion rig to the TE and the TE to the seabed anchors placed in a way so that the movements are limited.  After that, the TE is lowered onto the gravel bed with a guiding system assuring the immersed element at the primary meets the previous element. At the secondary end the element is adjusted through the External Positioning System. Once the TE is adjusted into its final position (Phase II), placing of the backfill is carried out. An immersion configuration is shown in Fig. 7.

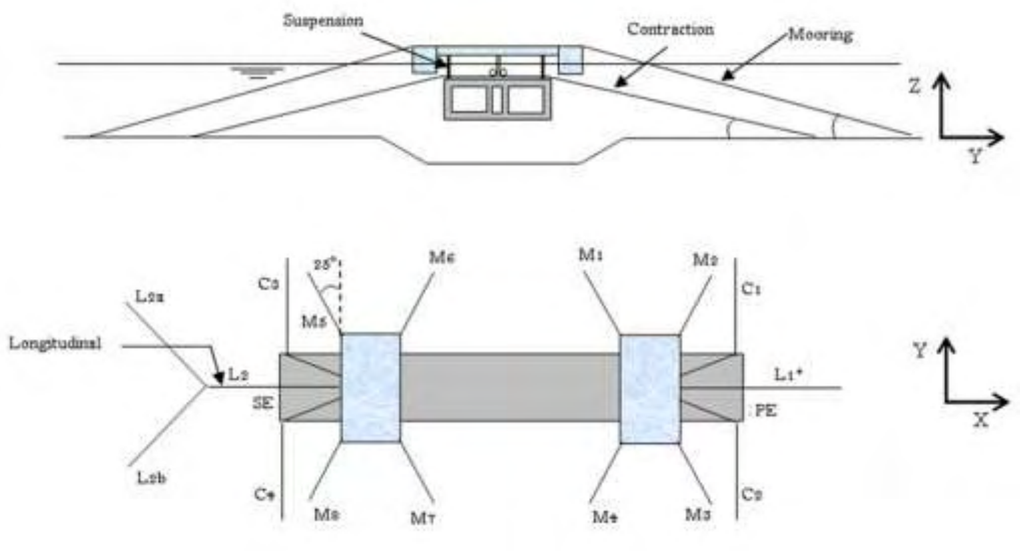


*Fig. 7 Mooring configuration*

Physical and numerical modelling of the above phases was carried out for the Busan project. For the setup of the physical model, all lines used for the immersion were modelled by rigid wires and the correct elasticity/stiffness was modelled by springs. Model tests initially focused on optimizing the configuration of the mooring system in order to reduce the maximum line loads. Subsequently approximately 6,000 numerical simulations were carried out considering a large number of environmental conditions. The numerical model was calibrated on the physical model on the basis of standard deviations of all wire forces and displacements and visual comparison. Motions of the TE and pontoon as well as the line tensions resulting from the incoming irregular waves were calculated and analyzed.

Results of numerical simulations were given in terms of:

- Max loads in the contraction, mooring, longitudinal and suspension lines[4];
- Max TE movements and rotation of the TE;
- Max TE velocity and angular velocity of the TE;
- Max suspension line angle.

Results were presented in a graphical manner through matrixes, one for each swell peak period. The colours inside the matrixes (red, yellow and green) indicate if a certain combination of current, swell/wave leads to a value that is inside or outside the loads and/or movements constraints defined by the contractor. A separate matrix was constructed for each constraint and a summary sheet was made to summarize the final workability conditions. An example of a matrix is shown in Fig. 8.

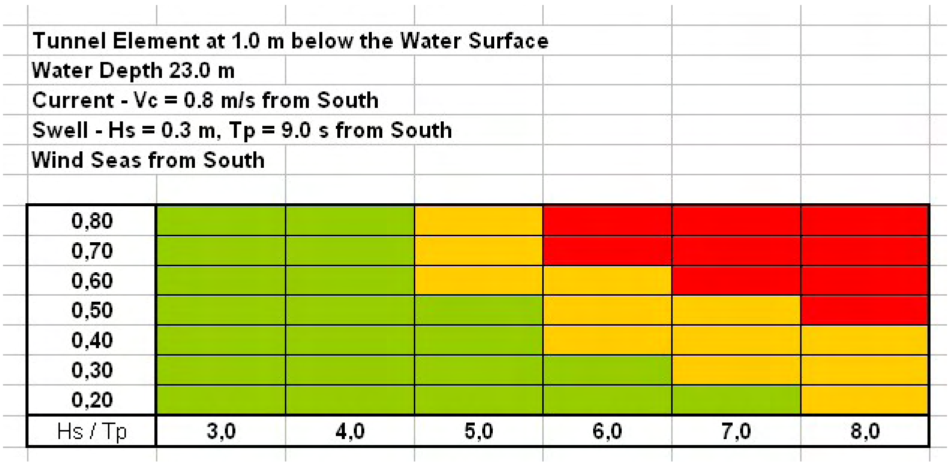


*Fig. 8 Example of immersion workability matrix*

**Loads during backfilling**

When the TE is placed in the final position and before it is fixed by the locking fill, it will be exposed to wave and current forces for which it must be held in position. Time for immersion and securing the element can be up to 36h.

For the Busan project physical model tests were carried out to assess the effects of wave and current loading for 3 cases, see Fig. 9: TE in deep trench; TE in shallow trench; TE on seabed. Tests have been performed with swell waves and current/no-current conditions.

[4] These loads include the pretension.

Analyzing the results it is possible to see that the horizontal force Fh on the TE increases slightly with increasing peak period Tp while on the contrary the vertical force Fv decreases significantly with increasing peak period. There is on average a higher uplifting force when the current is applied while the horizontal force is increased in the non-current situation.

The largest horizontal forces are measured for the test with the TE on the seabed and the lowest for the TE inside the deep trench. This is logical as the deeper inside the seabed the TE is located, the less it will be exposed to the current and waves horizontal force. All three cases show the same trend Fh=f(Tp).

For the vertical force the picture is more complex. The largest vertical force is measured for the shortest period waves when the TE is installed in the deep trench. For higher wave periods, the highest vertical force is measured when the TE is lying of the seabed. The smallest vertical force for all combinations of peak period and significant wave height is measured when the TE is installed in the shallow trench.

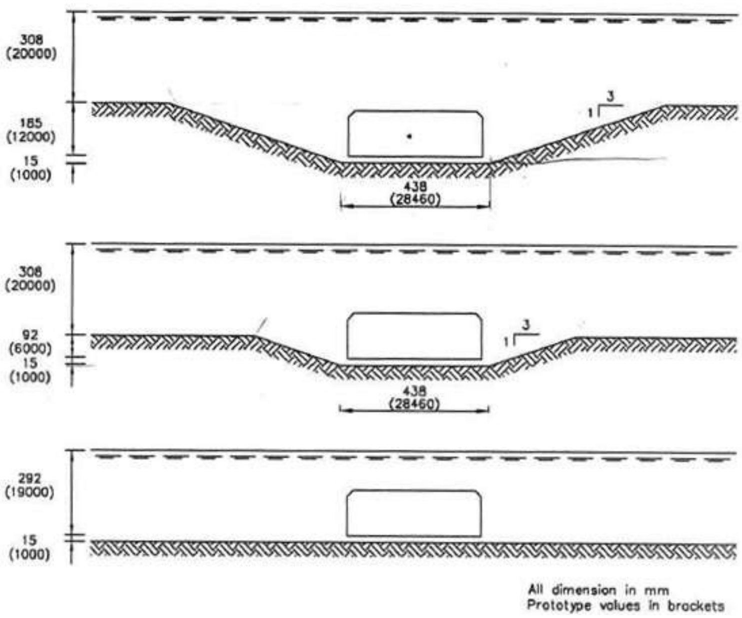


*Fig. 9 TE configuration for backfilling analysis*

Based on the above results it has been possible to assess how critical the process of securing the element by backfill is for the different tunnel-trench configurations and on the basis of this, the meteorological window during which immersion/backfilling is possible.

## CONCLUSION

Complex immersed tunnel projects can be made feasible and executed within planned cost, time and risk. It requires however realistic planning, proper establishment of expectations, experienced specialists involved, good project management and effective collaboration between specialists from the designers, contractors and owners. This papers shows that wave typhoon conditions can be addressed by physical and numerical modelling in combination with practical experience and judgements.

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# DEEP SEA FLOATING FOUNDATIONS FOR STRAIT CROSSINGS

by

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## ABSTRACT

*Recent concept developments concerning floating bridges for the new E39 Fixed Link in Norway has utilised the knowhow and experience from the marine floating structures operating in the North Sea and worldwide in order to find solutions to support and anchor such bridges at extremely deep bridge sites.*

*The paper present the solutions adapted for Sognefjord and Boknafjord crossings, i.e. utilizing stay anchored floaters and tension leg floaters. The paper discusses the advantages and the disadvantages with each solution based on the experiences from the concept development. The paper will also further discuss the possibilities to find optimized solutions for anchoring floating bridges for water depths between 300 and 1300m and demonstrate the feasibility in introducing bridge concepts for such depths.*

## INTRODUCTION

The development of strait crossings using floating structures has a long history in Norway, with the building of the Nordhordland and Bergsøysund floating bridges, see e.g. [1], and the development of the Høgsfjord submerged floating tunnel as the previous culmination point. The development of the E39 fixed link means a new concentration in this area. The goal of the E39 fixed link project is to link the Norwegian cities Kristiansand, Stavanger, Bergen and Trondheim by means of a ferry free highway E39. Currently E39 have altogether 8 ferry connections along the route and the objective is to replace all these with fixed connections, i.e. either bridges or subsea tunnels.

Two of the most challenging crossing will be Sognefjord and Boknafjord, since these sites are 3700 m wide and 1250 m deep (Sognefjord) and 7500 m wide and 600 m deep (Boknafjord). Feasibility studies for these two crossings were launched by the Norwegian Public Roads Administration in 2012. The Sognefjord study was performed by a consultant group consisting of Aas-Jakobsen, Johs. Holt, Cowi, NGI and Skanska, see [2], while the Boknafjord study was performed by the Norwegian Public Roads Administration, see [3]. The proposed concepts for floating bridges are both multispan suspension bridges, for Sognefjord the configuration is 3 x 1234 m spans and for Boknafjord 5 x 1420 m spans. Different approaches have been utilized for deep sea foundation, both cases adopting technology from the oil and gas industry. The paper compares and discusses the advantages and the disadvantages with these foundation concepts.

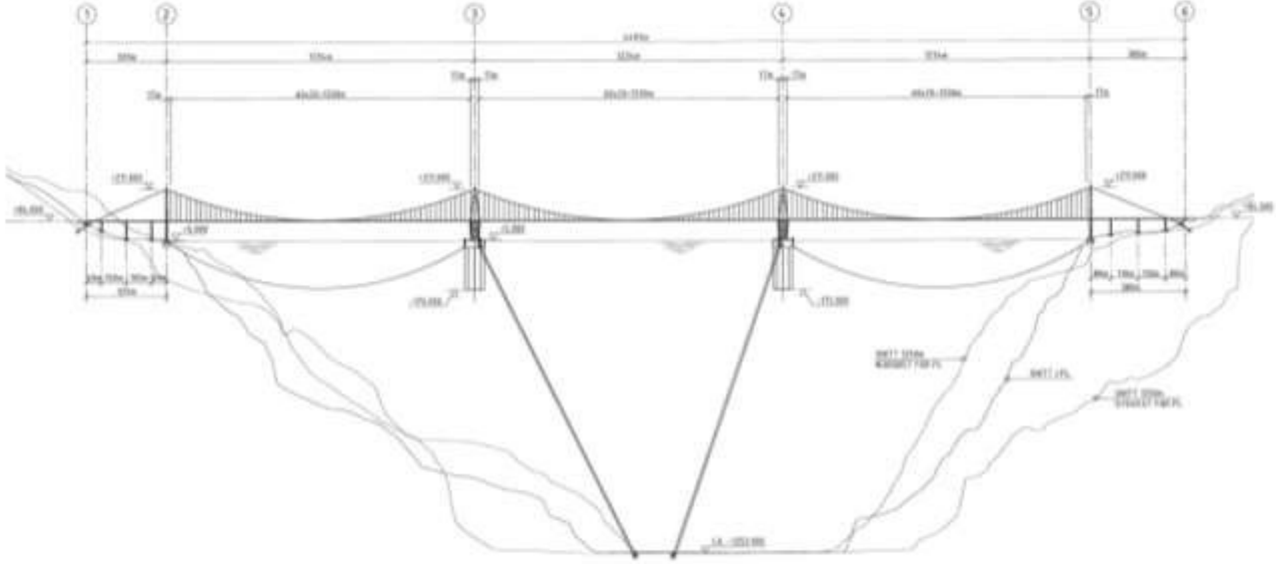


*Figure 1: General View of Sognefjord Concept*

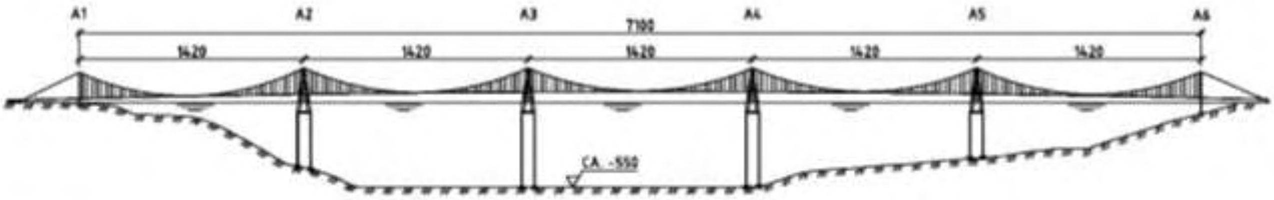


*Figure 2: General View of Boknafjord Concept*

**CONCEPT DESCRIPTION SOGNEFJORD**

The concept proposed for crossing Sognefjord is a floating bridge consisting of a multispan suspension bridge utilizing floating pontoons as foundations for the deep part of the fjord. In order to get a robust foundation in the middle of this deep fjord, two challenges have to be dealt with. Firstly a system must be designed supporting the weight of the suspension bridge beam, the cable and the towers. Secondly the system must be kept reasonably stationary in the middle of the fjord when exposed to variable loadings from wind and waves.

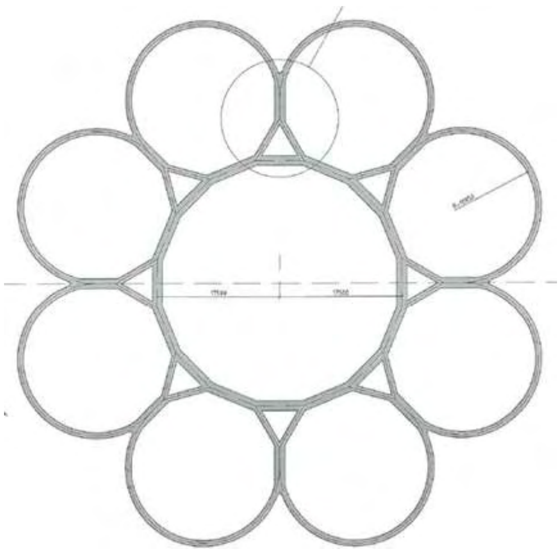

*Fig. 3: Sognefjord Pontoon Section*

The first requirement is met by using two large floating pontoons, providing sufficient buoyancy to support the bridge. Weight of the superstructure for the suggested bridge concept for Sognefjorden comprising pylon, cables and bridge girder is in the range of 210 to 220 MN when steel towers are assumed. This weight is counteracted by a pontoon having a draft of 175 m and a freeboard of 7 m. The floating pontoons are concrete structures with a 9 cell cylindrical configuration with diameters approx. 35 m for the centre cell and 11 m for the outer ones, see Fig. 3. The total concrete volume is 105 000 $m^3$. To ensure global stability, a ballast of 155 000 $m^3$ of olivine aggregate is used in the bottom part. The pontoon have a total displacement of 760 000 $m^3$.

The top 25 m of the pontoon configuration is designed to resist and absorb ship collision impacts by means of several secondary walls and light weight concrete aggregate in-fills. The design of this part is carried out to ensure that a ship collision will not compromise the floating capacity of the pontoon. The challenge will be to absorb the energy for the total system and for the ship to survive the encounter with this huge structure.

For construction of these pontoons, experience from construction of traditional marine Gravity Base Structures, found for instance in the North Sea, is utilized. This includes dry dock casting of base slabs and lower parts of cell walls, and conducting remaining work in a floating condition at a suitable deep water site. All construction phases outlined are common practise used for several structures of even larger size.

The second challenge is solved by an anchorage system providing sufficient lateral stiffness to keep the pontoons stationary in the middle of the fjord. The anchorage system consists of stay cables anchored to the seabed and to the pontoons. The main part of the cables are locked coil steel spiral strands for subsea use with a breaking load of approximately 22MN, see Fig. 4. Composite material could also be used for this part. In the top and bottom part of the cable, marine chains are designed in accordance with common practice. The bottom end of the anchor line is fixed to the seabed by suction anchors (or suction caissons). This type of foundation which is developed for the oil industry, have been installed in many of the deep water oil producing areas around the world. A suction anchor can simply be described as an upside down bucket embedded in the marine sediment. For Sognefjord crossing the suction anchors are located in the middle of the fjord, where the soil surface is relatively even. Clay sediments are assumed to be found in this location. The dimensions of the anchor are calculated to be

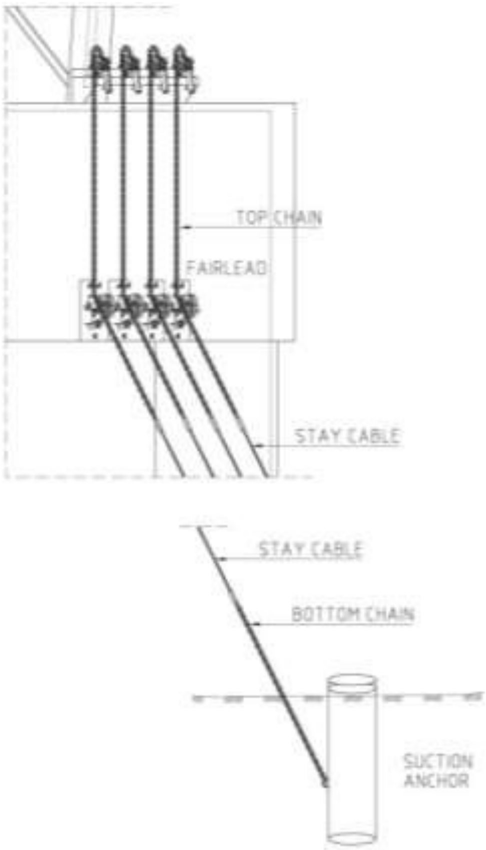


*Fig. 4: Sognefjord Anchorage System*

approximately 18m into the soil and with an outer diameter of 6m. The design of the suction anchor is based on the permanent loading being carried by submerged weight of the anchor, while the fluctuating load is carried by skirt friction and suction.

At the top end, the anchor chains are fixed to the pontoon by using specially designed fairleads attached to the pontoon. The anchor force is controlled by means of adjustable jacks placed on the top deck of the pontoon.

The erection of the anchorage system is performed in accordance with common practice used for similar marine floating structures worldwide. This involves onshore/dry dock production facilities, barge transportation/ working platforms, offshore cranes, ROVs, temporary floating elements etc.

The anchor system is used for achieving lateral stiffness only. The rotational stiffness required is achieved by the floating stability of the pontoon. With a pontoon draft of 175 m, maximum tilt from wind loading (static + dynamic) will be 0.96 degrees.

The total system has a fundamental frequency of 0.006 Hz for lateral excursions. This means that wind loading will be the dominant load as long as the bridge location is protected against long periodic ocean waves. Higher-order components of the local wind generated waves will give some response, but not in the same order as the wind loading. The large total depth of the pontoon prevents any wave induced vertical displacements of the pontoons. Maximum lateral displacement from wind loading of the pontoon is 10.2 m.

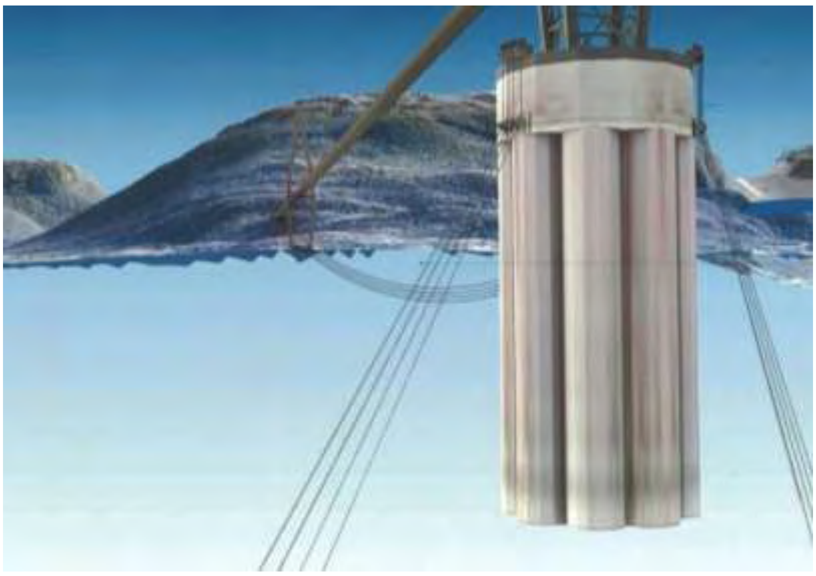

*Fig. 5: Sognefjord Floating Foundation*

The current in the fjord may give rise to vortex shedding effects inducing vibrations of the pontoon both in-line and as cross-flow. For the vibration frequencies found for this concept, such vibrations are not likely to occur.

**CONCEPT DESCRIPTION BOKNAFJORD**

For Boknafjord use of Tension Leg Platform (TLP) technology has been proposed. A TLP can be considered as an inverted pendulum where gravity is replaced by buoyancy. Then the limitation of the supporting structure is the material tensile properties and not compression forces and associated buckling as would be the case for a gravity based structure. The deepest gravity based structure today is the Troll platform which is a giant underwater tower standing at 300 meters water depth in the North Sea.

*Fig. 6: Boknafjord Floating Foundation*

Special features of the tension leg concept is that the heave, pitch and roll motions are prevented. Wave and wind forces on a vertical cylinder will always be greater in the horizontal plane than in the vertical plane. By allowing lateral motions (surge, sway and yaw), the largest environmental loads are largely balanced by inertia rather than by forces that would be typical for a rigid structure.

Wave forces will provide 2$^{nd}$ order slowly varying forces which may cause resonant motions in the horizontal plane. Excitation by wind may occur in the same frequency region and contribute significantly to such motions. Non-linear effects of the waves and hydrodynamic forces can cause resonant motions, known as "ringing". The awareness and knowledge of these effects makes it possible to adjust the resonance frequency and damping of the system to avoid this unwanted response occurring. Bridge structures are often located in a relatively protected inshore environment compared to the harsh climate associated with an oil installation location. In the case of Boknafjorden the significant wave height is calculated to 8.2 meters with an associated mean wave period of 13.4 seconds.

Tethers are always vertical and parallel to the initial position. When the platform moves horizontally there will not be significant rotations about the horizontal axes. Depending on the stiffness ratio between the main cables for the suspension bridge and the tethers, a tower on the platform will almost be vertical in any horizontal position. Horizontal displacements will result in a small vertical displacement termed "surge induced heave" or "set down". Yaw rotations associated with surge will maintain the distance between all anchoring points at seabed and platform for all the tethers. Consequently, no tethers are slackened due to the lateral displacements alone.

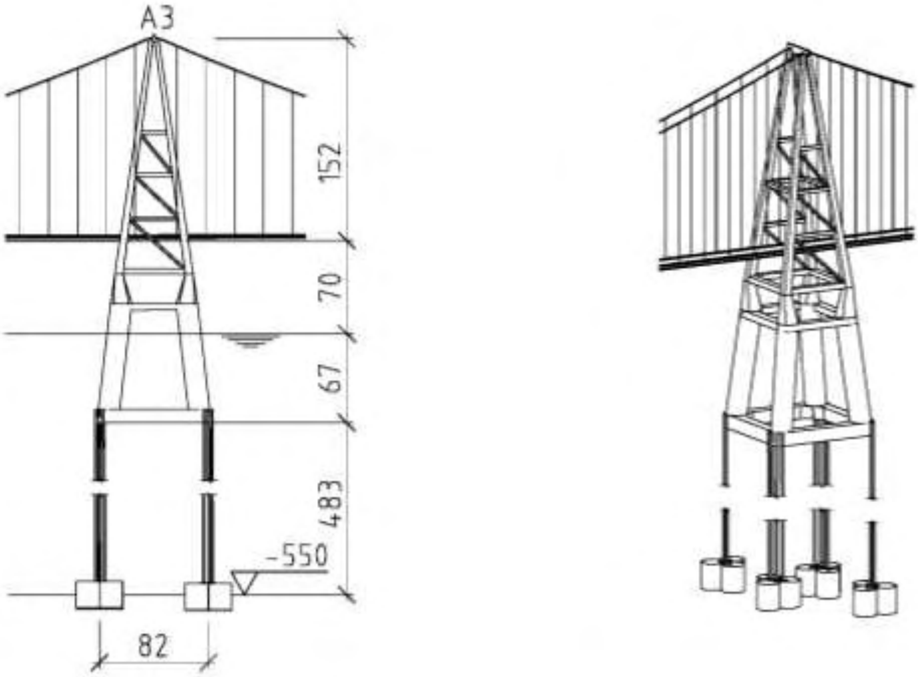


*Fig. 7: Layout of Boknafjord Floating Foundation*

In this feasibility study Snorre A platform was used as base case for proposing a design of the anchorage and hull structure. Snorre A was installed in 1988 and operates at 310 meters water depth. The hull is made of steel. The diameter of the columns is 25 meters and the displacement is 106 000 $m^3$. The distance between the columns is 76 x 76 meters. The draft is 37.5 meters. Total weight is 930 MN. The tension in the tethers is totally 247 MN.

The weight of the superstructure for the suggested bridge concept for Boknafjord comprising pylon, cables and bridge girder is in the range 280 to 300 MN based on a span of 1420m and using steel as the construction material. Weight of the hull is in the range from 160 to 190 MN, again assuming the use of steel. Total weight will consequently be at a maximum of 500 MN (approx.). Applying 80 MN in ballast this results in abt. 300 MN of total tension in the tethers for the self-weight condition. The hull displacement will be 88 000 $m^3$.

It is also an alternative to use concrete in building the hull. The total weight of the concrete hull and steel superstructure will for this case be approx. 800 MN based on data for the Heidrun TLP built in 1995.

Similar to Snorre A four steel tethers in each corner of the TLP is envisaged. The tethers are anchored by four Concrete Foundation Templates (CFTs), see Fig. 8. Since tension forces in the tethers of Boknafjord are in the same order of magnitude as for the Snorre A TLP, a simplification in the study is made in designing the CFTs similar to those used for Snorre A, see [4]. In this case the CFT is designed with 3 individual skirt compartments. The diameter of each compartment is 17m and one CFT covers a total area of 720 $m^2$. To account for seabed topography the skirt penetration depth can be made different for each of the foundations. The cells are closed at the top with a watertight dome structure. The outer skirt walls are extended above the domes forming retaining walls for solid ballast to be placed in the compartments on top of the domes.

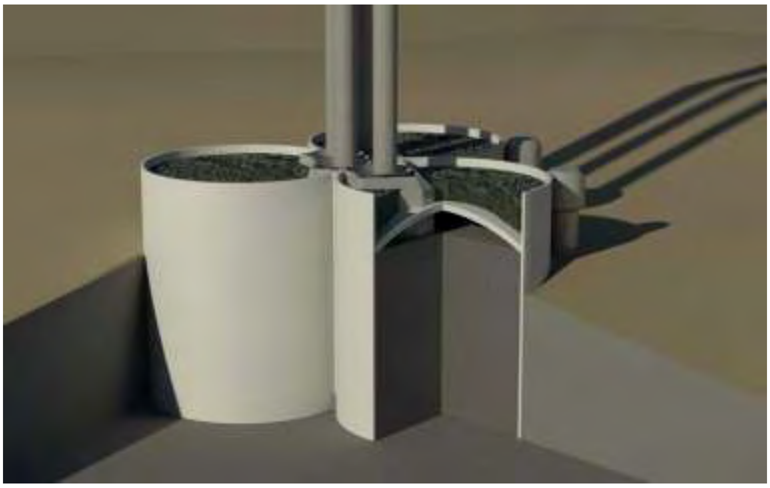

*Fig. 8: Boknafjord Concrete Foundation Templates*

The installation weight for the CFT is approx. 40 MN submerged. Additionally, about 40 MN of ballast material (iron ore and olivine) is placed in the compartments on top of each CFT subsequent to the installation resulting in a total in-place weight of 80 MN.

Strict installation tolerances are required. Two docking piles for each CFT are pre-installed as guides to make sure that the CFTs are installed within the tolerances. The vertical position and the inclination are both controlled by the suction pressure in the skirt compartments. It is assumed that the upper soil layers comprise very soft clay. The average static tension of approx. 75 MN for each CFT is counteracted by the weight of the TLP foundation and additional ballast in the top compartments in order to avoid any long term creep effects in the form of uplift.

In calm weather conditions the soil resistance is not mobilized. The additional cyclic wave and wind induced loads are transferred to the soil partly by skirt friction and suction under the domes. The tension load on each CFT is varying between approximately 0 and 160 MN (design loads) in a 100-year storm situation due to wave and wind induced TLP motions.

The tension load becomes inclined as the TLP is moves horizontally. The CFT may carry a horizontal load component of approximately 20 MN which corresponds to an inclination of 7 degrees in the tethers. For 550 m water depth, this corresponds to an approximate offset of 65 m while response calculations predict an offset in the range of 20-30 m.

The CFTs may be fabricated on a large barge, transported to the site and lift-installed by a heavy lift vessel.

## TYPICAL RESULTS AND FINDINGS

Both bridges will be flexible and robust regarding environmental loads. This also applies to the lifetime aspect. All items that cannot be replaced must be designed to be durable for the entire bridge lifetime without significant maintenance. This typically applies to the anchors, but tethers can also be designed to be durable for the bridge lifetime of minimum 100 years, if experience gained from the two Norwegian TLPs, Snorre A and Heidrun, are extrapolated. Anchor systems for stay cables have generally a lifetime less than 100 years, hence the design must be made to allow for replacements.

If the conditions on the seabed do not allow for a solution utilizing skirt friction and suction, then the dynamic response has to be carried by means gravity in the foundations. Another solution would be to use groups of piles in tension as foundation for the anchors. Norwegian regulations do not at the present time open for such a solution, but it could be a solution which is worthwhile looking at

Although the proposed solutions are based on known technology, a lot of research and development is required before the concept is mature enough for implementation. Similar to the practise in the offshore industry, the solution must be subject to a process verifying all the technical aspects of the concept. Such a process includes in-depth simulations with relevant computer tools and downscaled model experiments.

## COMPARISONS AND RECOMMENDATIONS

Concepts for two major strait crossings have been developed. The challenges in these two projects are both similar and different. For both crossings a multispan suspension bridge concept is utilized. This type of bridge with its inherent flexibility can absorb significant deformations that inevitability will arise when utilizing the concept of floating pontoons as a means of vertical support for the bridge in deep water. None of the cases have revealed fundamental problems related to this usage. The concept of floating deep sea foundations enables crossing wide straits with suspension bridges without stretching the span length beyond the conventional limits. Traditional construction methods used for building suspension bridges may very well also be used for construction of the superstructure for both these bridges.

The Sognefjord crossing is characterized by extreme water depth but rather moderate environmental loading. The Boknafjord crossing has a more moderate water depth but the environmental loading is more extreme. Thus different concepts may be appropriate for the two crossings.

The two concepts utilize principally different techniques to achieve sufficient vertical and rotational stability. For the Boknafjord concept these displacements are restrained by the TLP-concept itself, while for the Sognefjord concept they are marginalized by the depth of the pontoon. This is the reason for the significant difference in the displacements of the pontoon for the two concepts. The large difference in displacement also means a large difference in materials needed for the construction of the pontoons and associated costs. Large potentials for optimization and further development thus exist in these areas. Especially for crossing of fjords of intermediate depths, i.e. depth of 300m to 800 m, it seems interesting to consider an anchorage system which combine vertical tethers and inclined stay cables. The reduced cost of the pontoon must then be compared with the increased cost of introducing additional foundation elements. Based on the performed studies this concept appears more competitive for intermediate depths than for the large depths of Sognefjorden. This will be a focus for further work in this area.

The two different concepts also result in different approaches regarding the foundation technology to be used. A major problem identified for the Sognefjord crossing is the possibility to place foundations at certain locations along the bridge axis. Based on the available information it must be assumed that the steepest parts of the seabed near shore are more or less bare rock, and that the deeper parts consist of clay. In the steeper areas in-between it is assumed this will be covered by boulders from old rock falls and where there are obviously impossible to establish a reliable foundation. This demands a bridge concept to be sufficiently flexible to avoid anchoring points in these areas. Such a demand is not easily met for the Boknafjord concept.

In the need to restrain lateral displacements due to the large environmental loads both concepts balance these by compliance rather than stiffness typical for a rigid structure. The challenge for the Boknafjord concept is to limit the large lateral deflections generated by wind and waves. For the Sognefjord concept, the allowable lateral displacement characteristics are achieved by adjusting the design of the anchorage system. A combination of these two systems may have a large potential for further optimization of the concepts.

## AKNOWLEDGEMENTS

The authors want to thank the Norwegian Public Roads Administration for permitting publication of this paper.

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# DEEP SEA FOUNDATIONS FOR STRAIT CROSSINGS

by

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## ABSTRACT

*A growing interest from bridge owners to cross deep waters with fixed links such as bridges has lead to a renewed interest in deep water foundation knowhow.This paper discusses some available concepts for sea-bed foundation of bridges in water depths between 50 and 300 m. Emphasis is placed on describing key features and challenges related to pile, caisson, jacket and GBS (Gravity Base Structures) type of foundations, both in Norway and worldwide. Examples are discussed such as a proposed foundation design of a high speed railway bridge and a highway bridge in Lake Mjøsa with water depths down to 65 m.*

## INTRODUCTION

As the demand for improved infrastructure increases, new knowledge enables bridges to be built at sites previously considered uneconomic. Experience from offshore projects in the North Sea has provided expert technical knowledge in designing subsea foundations. These principles can also to some extent be applied for projects in lakes.

The main differences between an offshore and lake location is the reduced environmental loads such as wind and waves, which will lead to more economic structures. However, heavy duty offshore equipment like high capacity lifting vessels (HLV’s), large barges, pile hammers, tug boats and yards for pre-assembly of the structure are not readily available. Thus, this must be taken into account in the design and planning process.

Several types of deep sea foundation types have been built. This paper discusses pile- and caisson foundations, GBS and jackets. The main focus is on bridges in lakes and the limitations experienced when bringing offshore technology onshore.

## PILE FOUNDATION

A piled foundation consists of several vertical and inclined free spanning piles tied together at a pile-cap, usually made of concrete. A piled foundation may in principle be based on friction bearing piles or end bearing piles. For friction bearing piles the capacity is limited by the soil conditions, for piles embedded in rock the capacity will be limited by the structural capacity of the piles. A piled foundation is typically effective for water depths between 5 and 50m.

The pile type is chosen based on the design loads, soil conditions and ease of installation. Driven piles are generally preferred, as they are easier to install. Bored piles will be required if the soil is hard (e.g. moraine) and in cases also for large diameter piles due to lack of suitable hammers. Typical diameters are 600 mm and upwards.

For cases when piles are driven to bedrock a pile shoe is added to ease the chiselling. If the bedrock is located too deep, vertical capacity is obtained by friction and the piles are driven sufficiently deep to obtain the required vertical capacity. In the case of friction bearing piles it is good practice to include bearings at the top of the bridge column to allow for later jacking in order to compensate for possible future settlements.

**Tangen High-speed Railway Bridge**

In connection with plans to build a high-speed railway northwards from Oslo towards the town of Hamar, there is a requirement to cross the bay of Tangen at Lake Mjøsa with a 1220 m long bridge, see *Figure 18*.

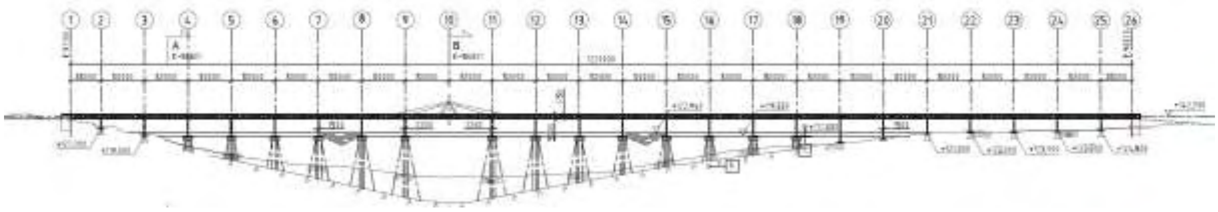

*Figure 18 Tangen Railway Bridge.*

The bridge is planned to consist of 26 axes. Typical span widths are 50 m and the main span is 100 m. Some of the pier foundations will be located on rock at terrain level while 14 axes are planned to be established by piling to bedrock. The pile-caps are is in these cases located below water level of aesthetic reasons. Soil overburden varies from about 0 to 50 m. Preliminary investigations indicate that the upper layer is soft. The upper part of the piles will be free spanning for up to 50 m. Maximum total length of piles will be approx. 90 m.

*Figure 19* shows the foundation layout and analysis model used. For each of the foundation at the main span a total number of 18 steel piles with dimension Ø1410x32 mm were found to be required. Governing design condition was buckling. Reinforced concrete is installed in the upper part of the piles to increase capacity at areas with peak loads and corrosion effects were accounted for by a corrosion allowance, i.e. by adding extra wall thickness to the piles.

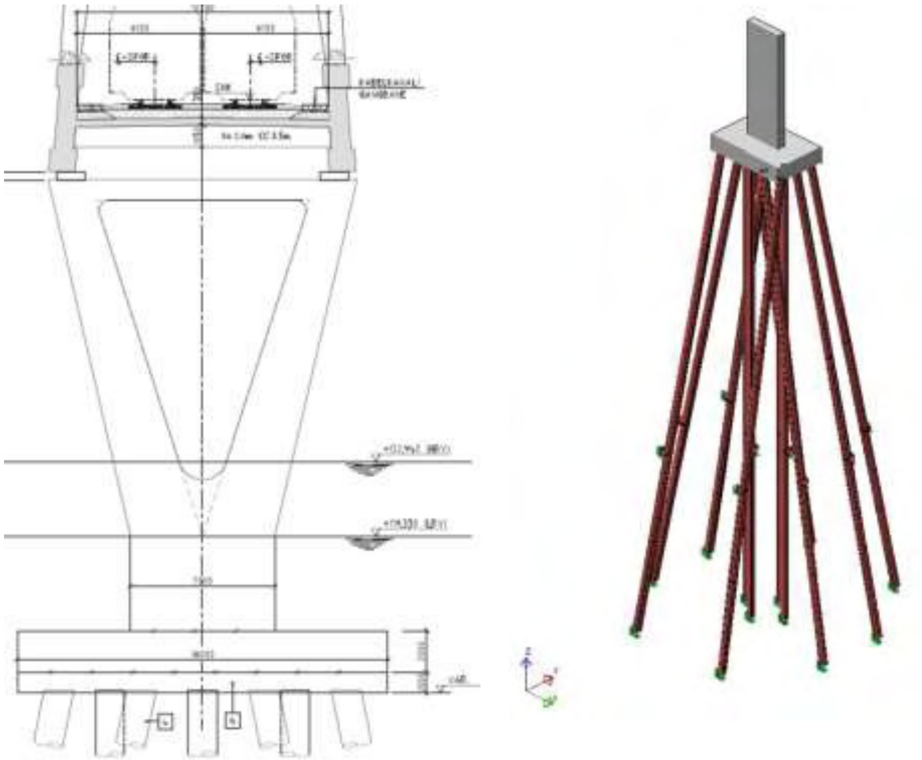

*Figure 19 Tangen Foundation Concept*

**Kveøya Bridge**

Kveøya Bridge is a fjord crossing in the northern part of Norway. The main challenge relating to the foundation of this bridge was the limited soil overburden above bedrock. It was found to be about 8-11 m, while the water depths were ranging from 21 to 31 m. Thus, the slenderness of the piles was high, which indicates that they are susceptible to buckling. To alleviate this, the soil overburden was increased with a material suitable for pile driving. Additionally, as these deposits have not settled over time, as is the case for indigenous mass, the effect of down-drag on the piles needs to be taken into account.

For this bridge it was decided - for simplicity of construction - that the pile-caps should be located entirely above sea level. The gap between the soffit of the pile-cap and the sea was closed by means of a skirt fitted towards the end of the construction period, see Figure 20. This solution is thus implemented for aesthetic reasons only.

*Figure 20 Left: Pile cap before installation of skirts. Right: Increased soil overburden.*

## CAISSON

A caisson is a concrete box placed on bedrock or hard soil with design strength higher than 5MPa (approx.). It might be supported by piles if necessary and is an effective concept between 5m and 40m water depth. It is usually built with ship impact protection barrier. Overturning moments from transverse loads could be resisted by rock anchors or ballasting.

The caisson must be placed on level ground. This could be obtained by underwater levelling or temporary supports before grouting, which will require underwater work. *Table 9* shows a list of Norwegian bridges with caisson foundation.

*Table 9 Examples of Norwegian bridges with caisson foundation.*

| **Project** | **Caisson depth** | **Bridge type** |
|---|---|---|
| Nordhordland Bridge | 30 m | Cable stayed / floating |
| Storda Bridge | 7 and 14 m | Suspension |
| Austevoll | 37 m | Free cantilever |
| Smaalenen | 20m | Cable stayed |
| Eiksund | 9 and 15 m | Free cantilever |
| Straumsund | 14 m | Free cantilever |
| Tverrlandsbrua | 30 m | Free cantilever |

### New Mjøsa Bridge

Conceptual engineering has recently been carried out to establish a concept for crossing lake Mjøsa with a new 4-lane highway (E6) between the cities of Hamar and Lillehammer. Typical span widths are 69 m with a proposed main span of 138 m. The total length of the bridge is approx. 1500 m.  The new bridge will replace an existing highway bridge built around 1980.

Two road alignments are investigated. The southern alternative, which at this point of time is the favoured alternative, has water depths down to 70 m. The soil, in lack of detailed soil surveys, is assumed to be layers of clay, sand and silt down to 100 m below the seabed. A solution consisting of friction piles has thus been chosen as the most beneficial concept for the foundations.

The load bearing pile capacities are determined for a range of pile lengths and pile dimensions varying from 30-40 m of soil embedment and for pile diameters of Ø813, Ø913, Ø1220 and Ø1420 mm. Pile forces are determined from global structural analyses for the various bridge alternatives. Non-linear analyses are performed to verify that load bearing capacities in the soil can be obtained in the piles without these being subjected to unacceptable utilizations due to buckling in the free-spanning part.

A number of bridge alternatives and span widths have been investigated. Generally, a pile dimension of Ø1220x22 mm is found to be suitable. This gives acceptable pile lengths and pile configurations for all alternatives investigated. For water depths more than 40 m (approx.), the reduced buckling capacity of the piles leads to the concept becoming uneconomical. The following measures are evaluated to solve this problem:

- Increasing pile dimensions.
- Introducing local fixations between the piles.
- Introducing steel guides and fixations at top part of piles.

- Introducing concrete caisson guidance at top part of piles.

For this project a concrete caisson solution is introduced, see *Figure 21*. The caisson consists of an outer cylindrical wall with several horizontal slabs at various levels. Between these, pile guides are placed in order to support the piles during installation. The slabs also act as fixation against buckling. The caisson also functions as formwork for dry casting of pile caps and columns.

The caissons are cast partly in dry dock or on a barge and partly whilst floating in the water. The behaviour of the caisson is simulated though nonlinear analyses of the whole system including pile group, caisson and column/tower.

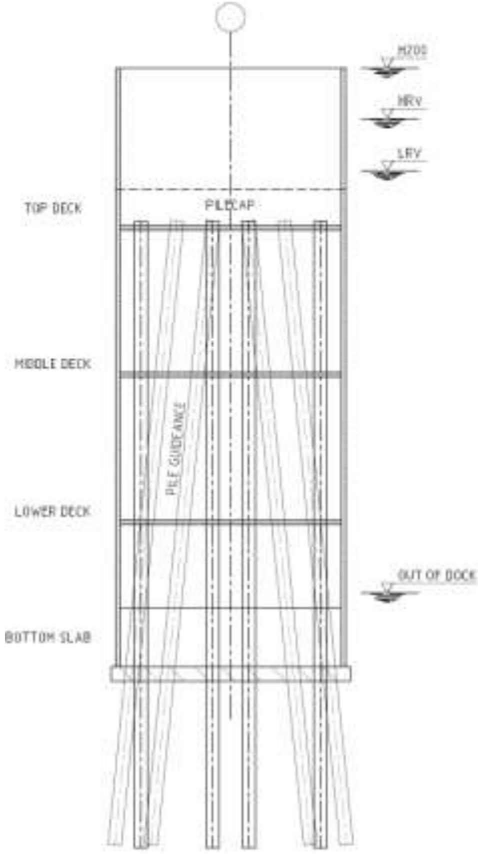


*Figure 21 Caisson for pile fixation.*

**JACKET**

A jacket is a steel space frame with tubular members. It is supported at the seabed by means of piles driven through the legs and/or piles-sleeves located in the base area of the jacket. Jackets are predominantly used offshore for oil or gas production and the piles are normally friction piles. In a few cases piling has been replaced by suction type anchors ("bucket foundation"). Favourable depths for this concept are 40-150 m. Although jackets are predominantly an offshore application, there are also examples where this concept has been used for bridges.

The common method of fabricating and installing a jacket is to build it at a yard and transport it to its intended location in one piece although a concept installing the jacket in more than one piece is a viable alternative for an inshore application. If the structure consists of several pieces to be mated in the case of a lake site, the crane capacity which can realistically be provided at such a location may limit the section size. In most lake areas there are no yard facilities for pre-assembly work. Thus, a temporary site will need to be established. The merit of having a temporary site close to the final bridge location is that smaller parts can be brought to the temporary site and assembled into larger pieces before transportation to its final destination.

Such an approach was used for the Chiapas Bridge in Mexico 0, which was completed in 2003. The steel bridge is supported by jackets and has an overall length of more than 1200 m with individual span lengths of 168 m. Each jacket consists of four straight main legs of steel tubulars (i.e. no batter) with a diameter of 2.8 m and with overall length of up to 85 m. The legs were placed in a rectangular pattern with 10m centre distance in the bridge longitudinal direction and 18m in the transverse direction. The legs were braced and local stiffening was located to the outside. This gave flush insides, which allowed for internal placement of piles. The piles were drilled into the rock, reinforced and concreted.

*Figure 22 Chiapas Bridge 0.*

The jackets were produced at a temporary construction site near the bridge site. After finishing the work on shore the legs were closed and additional buoyancy tanks installed see *Figure 23*. The system was then launched into the lake, floated to site, upended and secured to the seabed.


*Figure 23 Jacket with buoyancy tanks 0.*

**Gravity Base Structure (GBS)**

For bridges with large vertical loads, a GBS might be an option. They can be installed on relatively soft soil as its footprint can easily be increased to accommodate the soil conditions. However, this might require more extensive underwater levelling of the soil to accommodate the large footprint. Casting of tall GBS structures requires that part of the casting needs to be done in a floating condition. Thus, there is need for a sheltered deep water area to complete the construction work, and it requires a sufficiently deep channel for towing to the bridge site. Favourable depths for this concept are 30-300 m.

An example of use is the three main foundations of the Rion-Antirion Bridge in Greece, opened in 2004, 0. In this case the GBS is founded on a leveled and soil strengthened seabed. The water depth is 65 m. The diameter of the base is 90 m, see Figure 24.

When using GBS foundations for bridges it is critical to consider the effects of force transfer to the soil from environmental loads and ship impacts. Alternative concept needs to be jointly considered with respect to structural and soil integrity as well as fabrication/installation costs to arrive at the optimum solution.

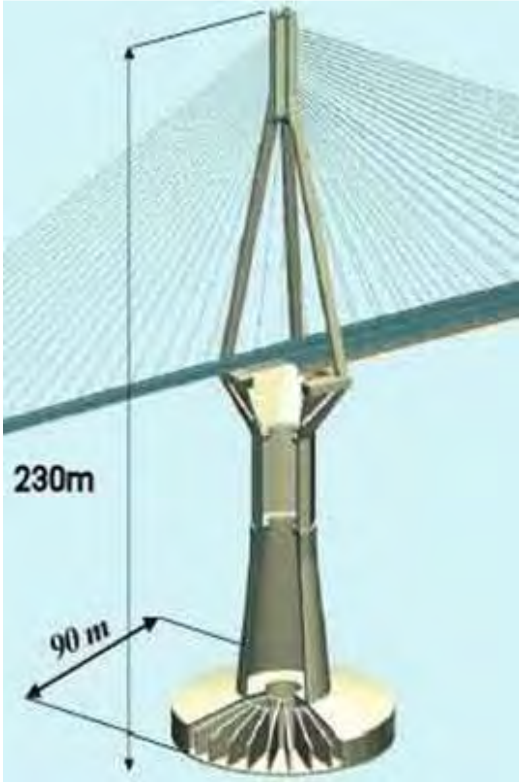


*Figure 24 Main foundations of Rion-Antirion bridge 0.*

**DISCUSSION**

In the process of selecting the best type of foundation type it is important to have access to bathymetric data. Equally important is to have detailed soil condition data at all foundation locations, as these data may limit the number of viable foundation alternatives. Placing caisson type of foundations on the seabed requires the surface to be even and level. This can be achieved by dredging/ planning or alternatively by or in combination with deploying a layer of crushed stones.

Maintenance issues are also important as they impact the life cycle cost of the structure and deep water structures are expensive to repair if it is needed. Thus, maintenance cost should also be addressed during the various phases of design.

Table 10 shows a summary of the key features of the different types of foundation structures.

*Table 10 Key data for different foundation structures.*

| **Structure** | **Relevant water depths [m]** | **Soil conditions** |
|---|---|---|
| Pile foundation | 5-70 | Poor, soft, firm |
| Caisson | 5-40 | Bedrock or hard soil |
| Combined caisson and pile foundation | 5-80 | Medium firm to hard |
| Jacket, piled | 40-150 | Soft to medium firm. Not deep soft material. |
| GBS | 30-300 | Relatively soft to hard, levelled ground, even conditions |

**SUMMARY**

This paper outlines the use of deep water foundations for bridge structures in lakes. Piles, caisson foundations, jackets and GBS are all possible alternatives. The choice will be governed by parameters such as water depth, soil conditions and available construction/installation facilities. Currently pile foundation concepts are the most common and readily available approach, but increased experience with caisson technology makes this a promising alternative for deep water foundations for bridges across lakes.

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# DEVELOPMENT OF A SUBMERGED FLOATING TUNNEL CONCEPT FOR CROSSING THE SOGNEFJORD

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## ABSTRACT

*The infrastructure along the rugged West coast of Norway is presently highly dependent on ferry links. The potential for a ferry-free coastal route is currently investigated by the Norwegian Public Roads Administration (NPRA). This will involve crossing of fjords with more than 400 m water depth and crossing lengths up to 7.5 km. For these fjords the Submerged Floating Tunnel (SFT) is regarded an attractive crossing solution. The applicability of a SFT has recently been proven in a feasibility study for the 3.7 km wide and 1 300 m deep Sognefjord. The study was part of NPRA's "Fjord crossing" project aiming to develop and qualify crossing technologies applicable for the most demanding fjords.*

*The paper presents the crossing solution with its concept features, overall geometry and early phase dimensions. Although the crossing scale and overall dimensions are beyond past experience, it is demonstrated that the concept is technically feasible and can be based on recognized technology. The structural performance of the SFT will be described with particular focus on the predicted dynamic response to current and 2nd order wave effects. The proposed construction and installation methods, largely based on proven offshore techniques, will be described. The paper will also present a resumé of the extensive risk assessment carried out for both the construction and operation phase. Finally, the potential for application to others sites is briefly outlined.*

## INTRODUCTION

The Norwegian Public Roads Administration's (NPRA) is currently exploring the potential for upgrading the coastal highway E39 between Kristiansand and Trondheim. The route runs along the rugged western coast of Norway, and the efficiency of the corridor is greatly hampered by ferry links. There are eight ferry links, most of them are wide and deep fjord crossings, and the ultimate ambition is to replace the ferry crossings with fixed links. Crossing the 3 700 m wide and up to 1 300 m deep Sognefjord is seen as the major technical obstacle towards achieving that goal. Both length and depth exceeds the limitations imposed by traditional bridges or subsea tunnels. This calls for alternative crossing concepts.

As part of NPRA's "Fjord crossing" project aiming to develop and qualify crossing technologies applicable for the most demanding fjords along the coastal route, Reinertsen Olav Olsen Group (ROO) conducted a feasibility study for a submerged floating tunnel concept for a generic crossing over the Sognefjord. A submerged floating tunnel (SFT) has been proposed as a solution for several crossings around the world, but has not yet been realized. Crossing the Sognefjord with a SFT is a technical feasible concept that is independent of water depth. Rather than treating water as a barrier to overcome, the SFT exploits its bearing capability. Crossing the Sognefjord is a state of the art project, and though the overall structural configuration has never been used before, extensive experience from the major offshore concrete platforms gives confidence in the development of the concept. The best endeavors were made to apply well proven and recognized technical solutions, conservative design assumptions and generous design acceptance criteria.

When designing the SFT the main focus areas were:

- Safety
- Constructability
- Calculating response from dynamic loads
- Securing a functional ship impact strategy
- Making reasonable assumptions for calculating the self-weight

*Figure 25: Arial view of the Sognefjord SFT (artistic renderings courtesy of NPRA)*

*Figure 26: Key structural elements (artistic renderings courtesy of NPRA)*

## THE CROSSING CONCEPT

The SFT developed for Sognefjord crossing consists of two concrete tubes interconnected by diagonal concrete bracings. The tubes are supported by 16 steel pontoons connected to the tubes by steel shafts. The horizontal alignment follows a circular arch with a radius of 2 682 m ($f$ : L $\approx$ 1 : 5) and a center line length of 4 083 m. The purpose of the arch shape is to give the SFT sufficient horizontal stiffness, while the pontoons take up variations in traffic loads and self-weight and provides hydrodynamic damping. The vertical alignment is adapted to the ship clearance requirement having a slightly curved, symmetric trajectory with a draft of 20 m within the 400 m wide navigational channel.

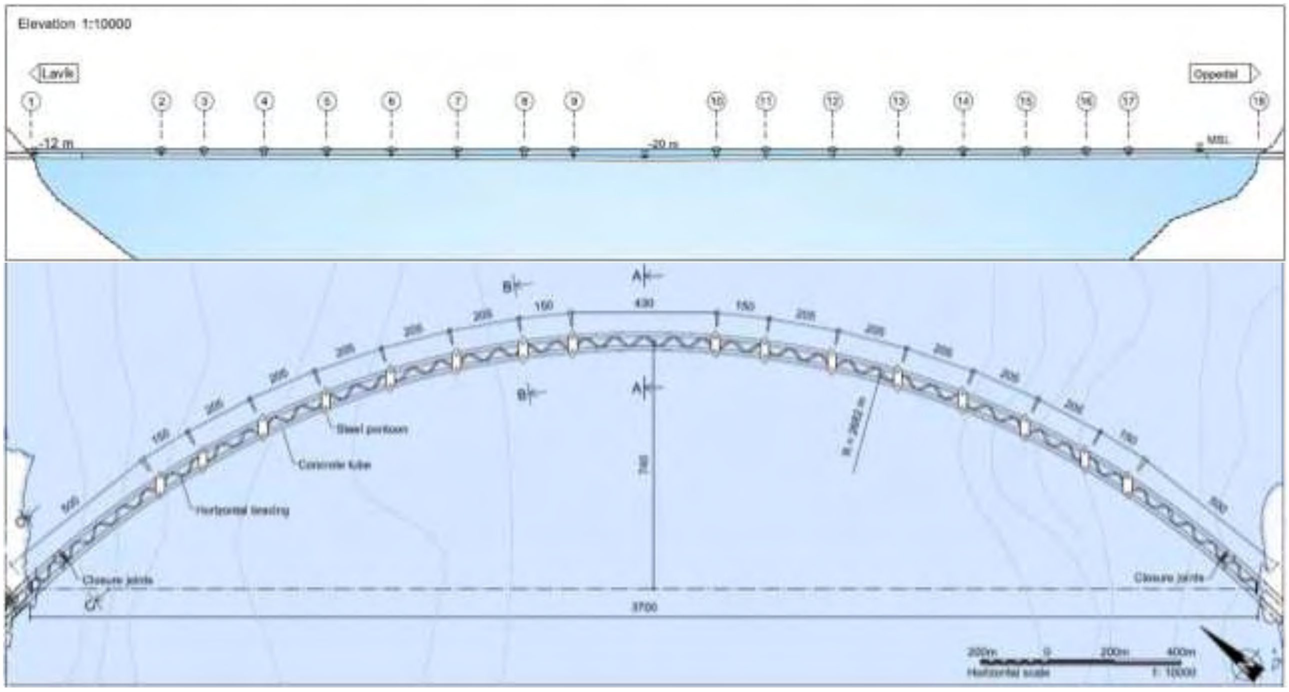



*Figure 27: Elevation and plan of the Sognefjord SFT*

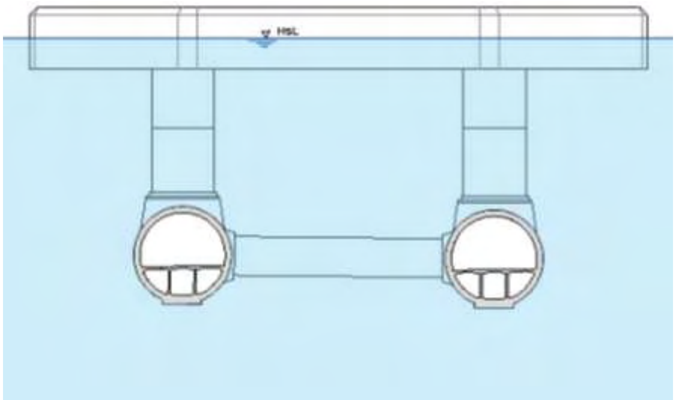
There are several advantages with this concept. Some of the key features are:

- The SFT is located at 20 meters water depth where the environmental loads are reduced to a fraction of the loads at the surface. In an optimization phase, water depth is one of the elements that should be considered as the loads can be further reduced.
- The use of pontoons eliminates the major uncertainty in foundation conditions and makes the concept independent of water depth.
- Robustness with regards to ship impact
- Low roadway gradients

### Concrete tubes and horizontal braces

The concrete tubes have an outer diameter of 12.6 m and a general wall thickness of 0.8 m. The tube is divided internally by a concrete slab to provide an upper main compartment for car traffic. The space above the traffic profile may be used for ventilation fans, traffic signals and sign boards. Below the traffic deck the section is compartmentalized into three non-communicative compartments. The central compartment is designated waster ballasting, whereas the outer compartments are used for solid (concrete) ballast.

Due to restraining moments the tube's cross-section is increased towards the landfalls. At the landfalls adequate bending resistance is obtained by increasing the outer diameter to 14 m and the wall thickness to 1.0 m.



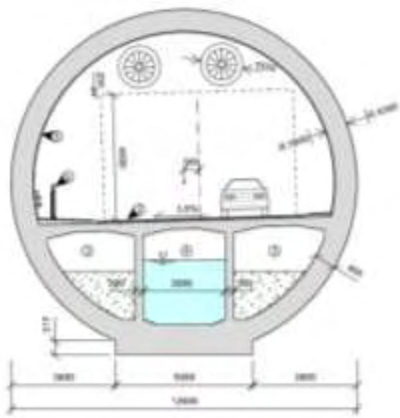


*Figure 28: Tunnel and tube cross-section*

A concrete truss is used to mobilize composite action between the two tubes. The horizontal bracing of the tubes consists of diagonals with a constant inclination of about 40°. The bracing elements are foreseen as circular concrete tubes having an outer diameter of 5 m and 0.5 m wall thickness. Approximately every 500 m the bracing diameter is increased to 8 m to accommodate escape ways and technical installations, while the other diagonals are water filled.

**Landfalls**

The tubes are guided horizontally into the entrance of the land tunnel with its apex 12 m beneath sea level. The land tunnel rises inside the rock with 5 % inclination up to free air. At both ends the tunnel is monolithically joined with the landfall structure without dilatation. Thus the chosen connection requires a minimum of maintenance and inspection and a high degree of durability is obtained. A monolithic joint towards the landfalls attracts restraining moments due to loads induced by tides, distributed vertical loads and wave loads. An optimization of the distance to the 1st pontoon with respect to balancing the moment distribution in the end spans should be carried out when more accurate environmental data is available.

The effect of tidal variation may be relieved by adjustable water ballasting of the pontoons closest to shore. This will require and active ballasting scheme which is not desirable due to operation and maintenance conditions, thus ballasting for tidal effects have been considered not to be beneficial to the SFT.

**Pontoons with integrated ship impact mechanism**

The pontoons have a rectangular box configuration with triangular ends and a tentative water plane area of 1 600 m$^2$ (26 × 80 m) connected to the concrete tube by a cylindrical steel shaft with diameter about 8 m. Steel pontoons are preferred from a construction and installation point of view. Ultimately, the pontoon design will be governed by the adopted ship impact strategy.

The ship traffic in the Sognefjord challenges the crossing concepts, especially the significant traffic in the summer season from large cruise ships. These cruise ships would have tremendous impact energy in case of a collision. To isolate the tunnel from impact overload an impact 'fuse' mechanism is introduced between the pontoon and tunnel. A weak link will also beneficial with respect to damages to the ship. The ship impact mechanism limits the forces that can be transferred to the concrete tubes by ensuring a shear failure in the pontoon shaft. In the ship impact mechanism the shear forces are separated from axial forces and moment to be able to accurately predict the failure load.

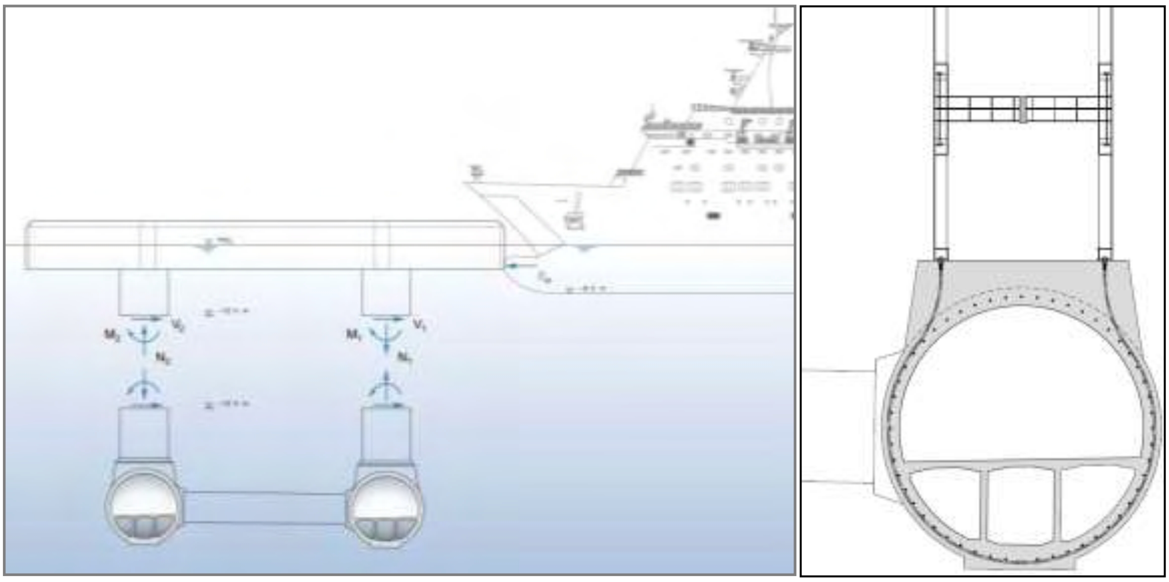


*Figure 29: Ship impact mechanism*

The following preventive measures are implemented to assure full structural integrity of the tunnel in case of an accidental ship impact:

- The tunnel is designed to tolerate loss of one pontoon without losing its structural integrity or suffering other structural damage. By tuning the water ballast it is even possible to have an operable bridge in the period until a new pontoon is in place.
- The pontoons are designed to have two-compartment damage stability, i.e. to have adequate residual stability after damage and flooding of up to two adjacent compartments. Apart from compartmentalization, infill of buoyant material in the outmost chambers is considered.
- A ship impact mechanism, “weak link”, is introduced in the shafts to prevent overstressing of the tunnel structure.
- Introduction of energy absorbing capability, a sliding friction fender has been suggested in the feasibility study. The motivation for adding such a fender is to protect medium size vessels.

To warrant robustness a return period of 100 years was conservatively chosen as design criteria for the weak link. A hundred year return period imposes a design collision energy load in the magnitude of 50 MNm. For design purposes this implies an impact load on the tubes of 50 MNm at ultimate, and a criterion for the weak link to fail for any impact above 50 MNm.

The weak link has to be designed for any ship collision larger than the failure criterion. A ship collision will act as an impulse load. To describe the impulse, the maximum force and the introduction period needs to be determined. Investigations have therefore been made to evaluate types of ship collisions that can occur based on the range of ships sailing in the Sognefjord.

The impulse load corresponding to collision energy of 50 MNm gives a maximum horizontal displacement of as little as 82 mm in the tunnel due to the significant inertia of the tunnel.

## DYNAMIC PERFORMANCE

Resonance issues are dominant for structures whit such high slenderness as the SFT. Viscous effects and wave radiated damping are therefore of great importance in addition to the spatial properties of the loads. An analytical method has been developed to calculate the hydrodynamic wave response. The incentive to develop this method has been to obtain a tailor-made analysis for slender structures with supported boundaries, where all assumptions that go in to the method are fully evaluated with respect to this special issue. The sea state is expressed by a power spectrum, e.g. the JONSWAP spectrum, with a directional distribution. Potential flow theory is used to express the load transfer function, which contains both first order wave forces in addition to slowly varying drift forces. One of the main advantages of this method is its' ability to account for the geometry of the structure, as well as the short crested properties of the sea state, when evaluating the load characteristics. Correlation lengths for some loads effects are illustrated in *Figure 30* (right).

The first order wave loads are greatly reduced due to the location of the SFT at -20 m as illustrated in *Figure 30* (left). This in addition of the high Eigenperiods causes the response shown in *Figure 31* to be mostly related to the slowly varying, second order, wave loads.

Research performed in conjunction to the Langeled pipeline [1] on vortex shedding and the effect of lock-in on circular cylinders have been used to make predictions on the effect of Vortex Induced Vibrations (VIV) on the SFT. Due to uncertainties in these predictions, highly conservative loads have been assumed which causes VIV to be the dimensioning dynamic load. However the maximum amplitude of motion is approximately 0.5 m corresponding to a deflection of $L/8\ 000$.

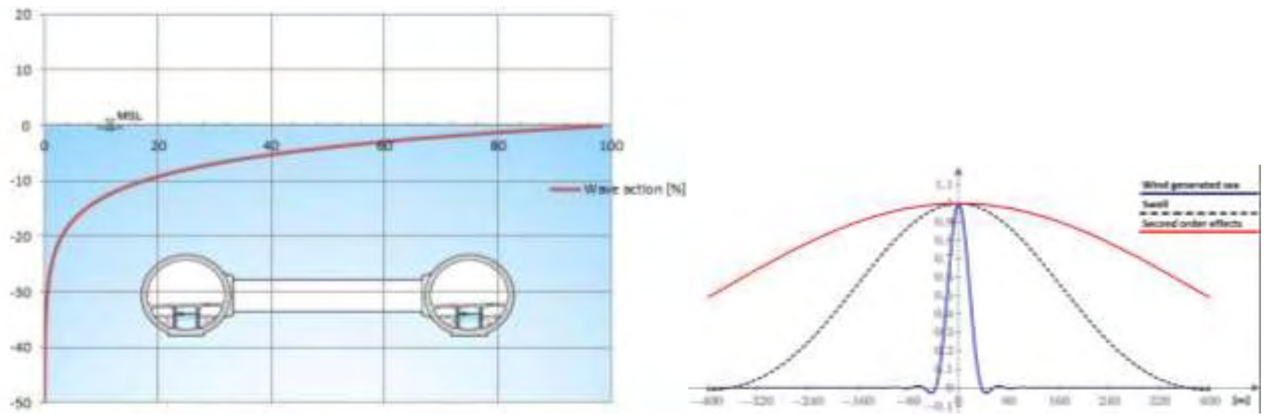



*Figure 30: Water depth vs. normalized wave action (left) and correlation lengths for wave actions (right)*

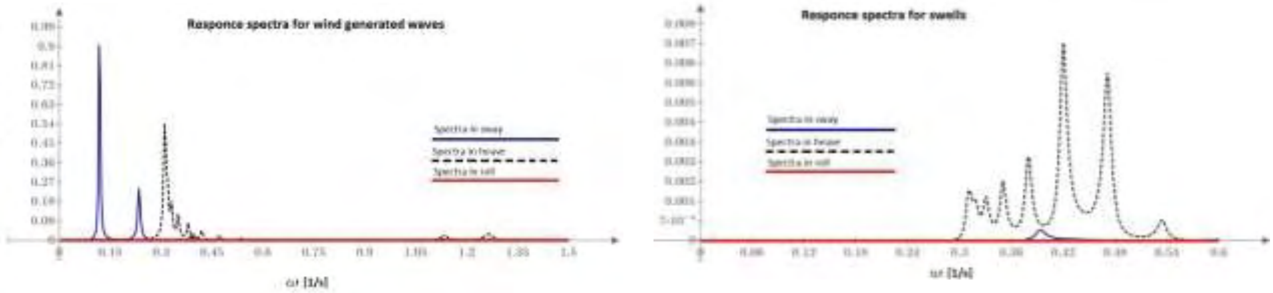



*Figure 31: Response spectra for (left) wind generated waves and (right) swells*

## WEIGHT CALCULATIONS

The dominant static load for the SFT is the self-weight. As the SFT is designed to float as neutral as possible in the water, positive net buoyancy is equally stressful as negative net buoyancy, and this special case is not well reflected in existing regulations. In order to minimize the variance in the net self-weight, rigorous weight control will be executed in the dry dock. When the construction of an element is finished, the dry dock will be filled with water and the freeboard measured. This ensures that the weight of the elements out of dock is accurately known.

Weight control is a well-known part of constructing floating concrete structures. In the years 1973 to 1995 a total of 17 concrete platforms was built on construction sites on the west coast of Norway by Norwegian Contractors and delivered to oilfields in the North Sea. All were towed out with large topsides, and weight control was crucial to the stability during the tow. The Troll A platform is even the largest structure that has been moved by man. Weight control very important for keeping the variance in self-weight low, but this is not regarded to be a feasibility issue due to the extensive experience with this issue in the Norwegian concrete society.

## CONSTRUCTION AND INSTALLATION

The bridge elements of approximately 300 meter will be constructed in a dry dock, preferably somewhere near the crossing site. Hanøytangen is a proposed alternative. Lutelandet is another nearby industrial area that is currently being developed.

The construction will be performed by in situ poured concrete, using formwork placed on rails and in sections length of about 30 m. The base of the cross-section will be casted before the remaining upper part of the tube, hence there will be two longitudinal casting joints at the base, and one cross sectional joint every 30 m. Prestressing in the longitudinal direction will be performed successively with an average cable length approximately 100 m. This procedure is a common method for constructing of submerged tunnel sections.

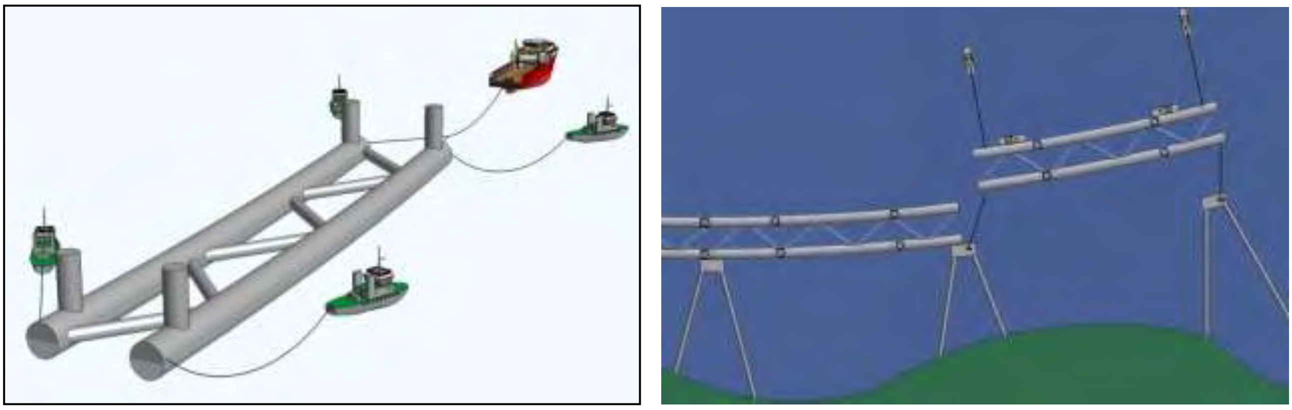


*Figure 32: Tow to crossing location (left) and mooring at assembly site (right)*

After the elements have been towed to the crossing site, three and three elements are jointed together and submerged. These segments are then jointed under water to reduce the environmental loads to the fully assembled bridge string of around 3 600 m. The landfall elements are constructed and installed separately to create a solid support in a highly stressed area. The bridge string is then monolithically jointed to the landfall elements, followed by the installation of the pontoons.

Water tightness is a key requirement to ensuring a high safety level for the SFT. The joints will be made monolithic by having overlapping prestressing as shown in *Figure 33*.

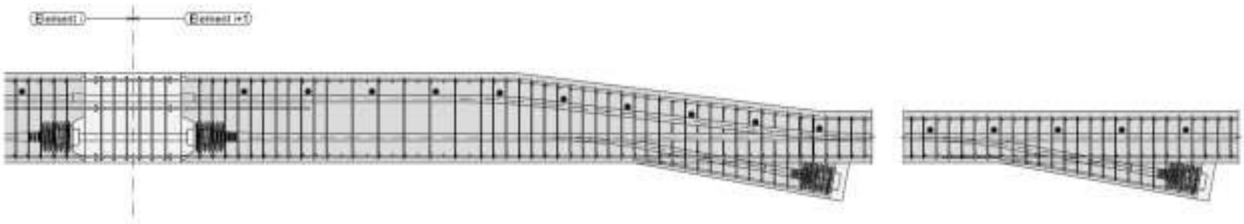


*Figure 33: Joint longitudinal post-tensioning*

The marine operation during the installation of the full length SFT string is considered complex, but manageable. The tunnel will be towed from the temporary mooring/assembly site to the installation site, a distance of about 3 – 4 nautical miles. The towing will be performed by 6 main tugs of about 200 TBP each and 8 assisting tractor tugs of about 50 TBP each. The tractor tugs will be moored to the shafts and will quasi act as thrusters to the tunnel. At the installation site additional holding and moving forces will be applied by a set of winches placed on land at both abutments.

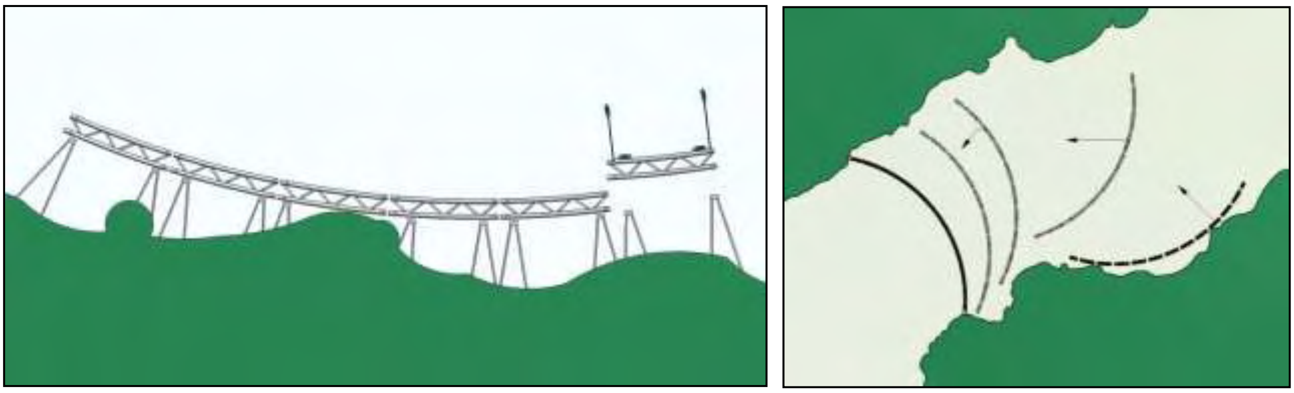


*Figure 34: SFT at crossing site, (a) mooring elements and (b) SFT installation*

## SAFETY

It is an indispensable requirement to the Submerged Floating Tunnel that the safety is at least as good as for other fjord crossings on bridge or in rock tunnel. The best endeavors are made to achieve this goal by application of well proven and recognized technical solutions, conservative design assumptions and generous design acceptance criteria. Main emphasize is given to redundancy and element reliability.

At this early stage of design and planning safety related aspects will be a major design driver. The overall structural concept has never been used before. The dimensions, the complexity, the slenderness, the joints and the marine operations exceed previous experience. Even if each individual element and operation is found to have adequate safety, in principle a hazardous chain of events may be released unless the interaction of elements is adequately understood.

In the feasibility study the main hazards to the bridge have been identified, their possible consequences and their possible mitigating actions. To complete this hazard identification study (HAZID), a work shop was held with an audience of prominent independent experts in all related technologies and in reliability analyses. In conclusion, this HAZID did not reveal any hazard that threatens the technical feasibility of the concept. It was found to be a sound concept for further development.

## COST AND SCHEDULE

The total construction time is estimated to be in the order of 7 to 9 years, and this includes at detail engineering phase of 1,5 years before construction can start. The construction of elements is on the critical path when it comes to construction time, and it is possible to use two docks to reduce the construction time by approximately 2,5 years. Prior to start of the detailed engineering, Front End Engineering and Design (FEED) and tendering must be performed.

The conservative cost estimate for construction of the SFT is 12 BNOK including landfalls but excluding technical installations. For long and deep crossings, the SFT is deemed a highly competitive alternative. Subsequent cost benchmark analyses of the different crossing options will hopefully give more information about the cost efficiency of the SFT.

## APPLICABILITY FOR OTHER E39 CROSSINGS

The length of the Boknafjorden crossing (7500 m) makes the proposed alternative unfeasible. Side anchorage to the sea bed is here necessary in order to control the horizontal forces and movement in the tunnel. A configuration with a single or double straight tube combined with sea bed anchorage is considered as most relevant here.

For the Bjørnafjord, Voldafjord and Storfjord, the proposed alternative is feasible as described for Sognefjorden. Relatively long span and deep water makes the end anchorage together with a double tube arched construction the most relevant configuration of the SFT.

The Nordfjord, Moldefjord and Halsafjord are relative short crossings, combined with shallow water. The proposed solution for the Sognefjord is feasible for these crossings, but a single straight tube with anchorage to the sea bed is here considered the most relevant solution. A single tube arch with end anchorage as for the Sognefjord may also be an option for these crossings.

## REFERENCES

1. Det Norske Veritas. "DNV-RP-F105 – Free Spanning Pipelines". 2006.

# STATE ROUTE 520 FLOATING BRIDGE AND LANDINGS PROJECT SEATTLE, WASHINGTON, USA

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## ABSTRACT

*The State Route 520 (SR 520) Floating Bridge and Landings project replaces the existing floating bridge and pile-supported bridge approaches with entirely new approaches and a new larger floating bridge. This project of replacing the east-west bridge across Lake Washington near Seattle, Washington, USA, is part of a Washington State Department of Transportation (WSDOT) $4.13 billion program for improvements of the east-west transportation link between Interstates 5 and 405. The existing SR 520 floating bridge was first opened for commuter traffic on 28 August 1963. After nearly 50 years, the floating bridge is showing its age. The bridge's pontoons have become vulnerable to windstorms and its pile-supported bridge structures are vulnerable to earthquakes. Additionally, the existing bridge only has two lanes in each direction, no shoulders, and no high-occupancy-vehicle (HOV) lanes. Adding transit/HOV lanes, shoulders, and a bicycle/pedestrian path will provide greater reliability and more options for regional traffic growth.*

*The floating portion of the new bridge is 2,350 m long, consisting of 77 floating pontoon elements that are anchored to the lake bottom by 58 anchors. The anchors are a combination of 45 fluke anchors, 8 gravity anchors, and 5 drilled shaft anchors. The water depth at the bridge site is up to 60 m, and the new roadway surface will be 5.7 m above lake level. The 34.5-m-wide, low-rise portion of the roadway structure consists of half-width precast deck elements assembled together, while the high-rise portion consists of standard WSDOT highway girders with a cast-in-place deck supported on columns and crossheads, which are supported by the floating pontoons. Fifty-eight-m-long steel girder spans transition the floating bridge into fixed bridge structures at each end of the floating bridge.*

*WSDOT is the owner of the bridge and is funding the SR 520 replacement project. Kiewit Infrastructure West Co., with their design/build partners General Construction and Manson Construction and Engineering, is the primary contractor responsible for fabrication and installation of the pontoons; pontoon-supported roadway; and the transition bridge structures. KPFF Consulting Engineers and BergerABAM Inc. are the two lead engineering companies that are part of Kiewit's design/build team.*

## INTRODUCTION

The city of Seattle, Washington, USA, is located in the northwest corner of the continental United States. The city is on the shore of Puget Sound, directly connected to the Pacific Ocean to the west off the coast of the state of Washington. The east part of the city borders Lake Washington, an elevated freshwater lake, which is connected to Puget Sound through the Ballard

Locks. Two major interstate freeways run north-south in this area; Interstate 5 (I-5) passes through the city of Seattle, and I-405 runs north-south and goes through Seattle's neighboring cities of Bellevue and Kirkland on the east side of Lake Washington. A portion of State Route 520 (SR 520) running east-west connects these two interstate freeways.

This paper describes the Washington State Department of Transportation (WSDOT) program to replace the existing floating bridge (also called Evergreen Point Bridge), which will provide significant improvements to this vital east-west transportation link. The existing floating bridge, one of four floating bridges in the state of Washington, was opened for commuter traffic on 28 August 1963 after three years of construction. It was built as a four-lane toll bridge and the total cost of the bridge in 1961 dollars was $21 million (U.S.) ($153 million in 2011 dollars). To pay for this cost, commuters paid a 35-cent toll in each direction until the bridge was paid off in 1979. The toll booths were then converted into bus stops.

The existing bridge is 18.29 m wide, and the floating section of the SR 520 floating bridge is 2.34 km long, making it the longest floating bridge in the world. The bridge is supported by 33 bridge pontoons of varying sizes, the largest being 109 m long, 23 m wide, and 8.54 m tall. The bridge pontoons are held in place by 60-mm- and 70-mm-diameter cables attached to 62 buried anchors weighing 70 tons each. Lake Washington is relatively shallow with the deepest point in the lake being 65 m. Depth under the bridge drawspan is approximately 61 m. The bridge was designed to withstand 80 to 117 kph winds. However, it has been determined that the bridge is vulnerable to wind damage so the bridge is closed to traffic in order to open the drawspan, which relieves pressure on the bridge from wind and wave action during windstorms. The criteria for closing the bridge to traffic and opening the drawspan is 80 kph gusts sustained for 15 minutes. When a 64 kph gust is sustained for 1 minute, a warning alarm alerts crews to come to the bridge for inspection and monitoring.

The bridge is also vulnerable to earthquakes as a significant portion of the west segment of the bridge is supported on large diameter hollow prestressed concrete piles. Based on recent earthquake design evolution and on determination that the region is susceptible to significantly higher earthquake force, these piles were not properly detailed to resist implosion when subjected to the current design level earthquake effects.

The current bridge was designed to carry 65,000 vehicles per day. Today, approximately 115,000 vehicles use the SR 520 floating bridge to cross Lake Washington. Additionally, the existing bridge roadway surface is directly on the pontoons themselves and only has two lanes in each direction, no shoulders, and no high-occupancy-vehicle (HOV) lanes. Adding transit/HOV lanes, shoulders, and a bicycle/pedestrian path will provide greater reliability and more options for regional traffic growth, and the bicycle/pedestrian path will also connect bicycle and pedestrian regional trails on either side of Lake Washington creating additional recreation opportunities.

In summary, replacing the SR 520 floating bridge will have the following benefits of providing

- A safer structure that is resistant to windstorms up to 143 kph and capable of withstanding a 1,000-year return earthquake event.
- Two general-purpose lanes and one transit/HOV lane in each direction.
- Wider, safer shoulders that will allow vehicles to pull over in the case of a breakdown.
- A 4.27-m-wide bicycle and pedestrian path on the north side of the bridge.
- Ability to accommodate future light rail if the region chooses to fund it in the future.
- Improved maintainability through use of an elevated roadway deck; the new bridge will allow continuous maintenance activity without the need for traffic closures.

- Continuous use of this corridor crossing Lake Washington; except for occasional short-term closures, SR 520 will remain open through construction of the new floating bridge.
- Improved transit reliability and travel times.
- Hundreds of jobs throughout western Washington by construction of bridge components in Aberdeen, Tacoma, and Kenmore, Washington.

Figure 1 below shows the existing floating bridge looking to the east, as well as illustrating the challenge of the road surface only being 3.96 m above the lake surface as the bridge is at times overtopped by wind-driven waves requiring bridge closure. The replacement bridge will be located immediately to the north (left in the figure) of the existing bridge, which will allow use of the bridge during construction except for a brief interruption when the tie-in at each end between the existing and new bridge is made.

*Figure 1 – Existing SR 520 Floating Bridge (looking to the east)*

## SR 520 REPLACEMENT PROGRAM DESCRIPTION AND PERMIT PROCESS

Figure 2 gives a broad overview of the SR 520 Program.


*Figure 2 – SR 520 Replacement Program*

As a public program, it is subject to a well-defined and complex process regarding funding and permits. Detailed information about the process and the program is published on WSDOT's web site. A brief overview of the permit process is as follows.

As part of the environmental compliance process for the I-5 to Medina project, WSDOT applied for numerous federal, state, and local permits. This environmental process involved working closely with local, state, and federal agencies throughout the planning and design process on issues, such as wetland and aquatic mitigation, in-water construction, stormwater, and fish passage. All agencies provided valuable input and feedback during the design process, and agency feedback resulted in fewer impacts to natural resources and helped the project team identify appropriate mitigation for impacts that could not be avoided or minimized. The Seattle and Eastside communities were also heavily involved through a public engagement process that further refined the basic features of the I-5 to Medina project through a collaborative urban and sustainable design process. This broader community feedback on comprehensive, livable, and sustainable design methods to be applied to the SR 520 corridor further enhanced the project. The final design report includes topics, such as the project vision of "Nature Meets City," design preferences that will guide design work moving forward. The draft report that was released in September 2012 received over 1,600 public comments that were subsequently considered by WSDOT and the design/build team.

There are four milestones in the I-5 to Medina: Bridge Replacement and HOV Project environmental process. They are

**1. Publish Draft EIS.** In the summer of 2006, WSDOT published a draft environmental impact statement (EIS) describing how the bridge replacement and HOV project could affect the environment in the project area and how WSDOT would avoid, minimize, and mitigate these effects. An EIS is a document that discloses potential effects from a proposed project under the requirements of the National Environmental Policy Act (NEPA) and the State Environmental Policy Act (SEPA). An EIS describes the purpose and need for the project and proposed plans for construction. It also provides comprehensive analysis of project operations and environmental effects.

**2. Publish Supplemental Draft EIS.** In January 2010, WSDOT published the supplemental draft EIS, which builds on the findings of the draft EIS. This is an interim document between the draft EIS and the final EIS. The supplemental draft EIS evaluated new design options proposed through the west side mediation process with involved stakeholders and provides more details about construction effects and mitigation. The supplemental draft EIS presents an important opportunity for the public to formally comment on the I-5 to Medina: Bridge Replacement and HOV Project's effects and have their comments tracked, published, and addressed by the project team in the final EIS.

The SR 520, I-5 to Medina: Bridge Replacement and HOV Project Supplemental Draft EIS presents information about the project to inform citizens about the potential effects of project choices and to assist decision-makers in considering how the project should proceed. It builds on the work of the 2006 draft EIS by evaluating a new set of six-lane alternative design options.

According to the NEPA and SEPA, an agency must prepare a supplemental draft EIS when

- The agency makes substantial changes in the proposed action that are relevant to environmental concerns; or
- There are significant new circumstances or information relevant to environmental concerns and bearing on the proposed action or its impacts.

**3. Publish Final EIS**. In June 2011, WSDOT published the final EIS, which evaluates the preferred alternative and includes a response to comments received on both the draft and supplemental draft EIS documents.

The final EIS compares the Preferred Alternative with the design options analyzed in the supplemental draft EIS, further refines previous analyses based on the Preferred Alternative design, and identifies how WSDOT will avoid, minimize, and mitigate for project effects. The document also takes into account the latest assumptions about other regional transportation and development projects to place the project within a wider context. In the final EIS, WSDOT responded to comments from agencies, involved North American Indian tribes, local jurisdictions, and the public.

**4. FHWA Prepares Record of Decision**. In August 2011, after a review of all of the project information developed, the Federal Highway Administration (FHWA) published the Record of Decision (ROD), a public document that describes the project's course of action and specific mitigation measures. FHWA signed the ROD, finalizing the NEPA environmental process and allowing WSDOT to further design the I-5 to Medina: Bridge Replacement and HOV Project and obtain construction permits.

The ROD also lists many commitments made by WSDOT and FHWA to surrounding communities before, during, and after project construction. These commitments include mitigation for project effects to the environment and neighborhoods, as well as implementing traffic-calming measures in the surrounding area and developing a community construction management plan.

Following several years of coordination, in late March 2012, WSDOT received the final permit necessary to begin construction on the SR 520 floating bridge.

## SR 520 PROGRAM COST AND SCHEDULE

Major transportation projects often require innovative and complex funding solutions. The SR 520 Bridge Replacement and HOV Program, working with the State Governor and the Washington State Legislature, has secured a variety of state and federal funding sources, including tolling the existing floating bridge, to help pay for the SR 520 program. In 2009, the Washington State Legislature set a program budget of $4.65 billion. In October 2012, WSDOT released updated cost estimates that show all project elements – from I-5 in Seattle to SR 202 in Redmond – amounting to $4.128 billion or $522 million less than the program budget. The latest project plans, including the west side preferred alternative, are within this program budget.

Consequently, WSDOT as of February 2013 has funded and is moving forward with four separate major construction projects on the SR 520 corridor.

1. Floating Bridge and Landings Design/Build Project - Constructing a new, safer floating bridge.
2. Pontoon Construction Design/Build Project - Building a new pontoon construction facility (graving dock) and 33 pontoons in Aberdeen, Washington.
3. Eastside Transit and HOV Design/Build Project - Widening and improving transit facilities from Medina to I-405.
4. West Approach Bridge (North) Project - This is a traditional design/bid/build project to construct the north half of the new west approach bridge, connecting six lanes of traffic from the Montlake interchange to the new floating bridge.

The project schedule for the major elements is as follows.

- **Early 2011** - Begin construction of a graving dock in Aberdeen, Washington, for pontoon construction.
- **Early 2012** - Begin pontoon construction in Tacoma, Washington, and roadway deck segment and anchor construction in Kenmore, Washington.
- **Spring 2012** - Begin bridge construction on eastern shore of Lake Washington.
- **Mid-2015** - Target date to open new floating bridge to drivers.
- **Late 2015/Early 2016** - Demolish old SR 520 floating bridge and landings.

## SR 520 BRIDGE - EXISTING VERSUS NEW

In the January 2010 supplemental draft EIS, WSDOT evaluated a new SR 520 floating bridge with a roadway surface that would be approximately 9.14 m above the water. Based on community feedback, the height of the floating bridge roadway surface was lowered to approximately 6.10 m, included in the preferred alternative that was announced in April 2010.

Comparison graphics of the existing floating bridge and the new floating bridge are below.

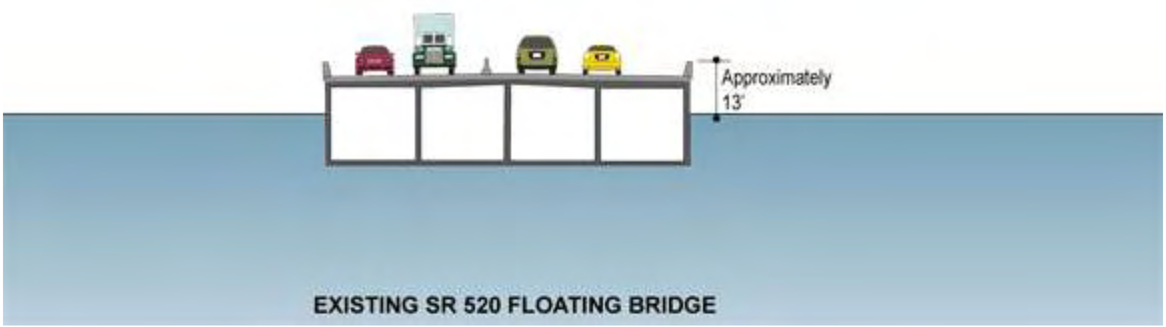


*Figure 3 – Existing SR 520 Floating Bridge, 18.29 m wide (note 13 feet = 3.96 m)*

The current configuration of the existing SR 520 flowing bridge includes

- Two general-purpose lanes in each direction, with a 3.35-m inside lane and a 3.66-m outside lane
- One 0.61-m outside shoulder in each direction
- One 0.30-m inside shoulder in each direction
- One 0.61-m median barrier
- A 0.91-m maintenance sidewalk on the south side of the bridge

As required by legislation and analyzed in the supplemental draft EIS, WSDOT will construct a six-lane SR 520 corridor from I-5 in Seattle to SR 202 in Redmond. The lane configuration across the new SR 520 floating bridge includes

- Two 3.35-m general-purpose lanes in each direction
- One 3.66-m transit/HOV lane in each direction
- One 3.05-m outside shoulder in each direction
- One 1.22-m inside shoulder in each direction
- One 0.61-m median barrier
- A 4.27-m bicycle/pedestrian path on the north side

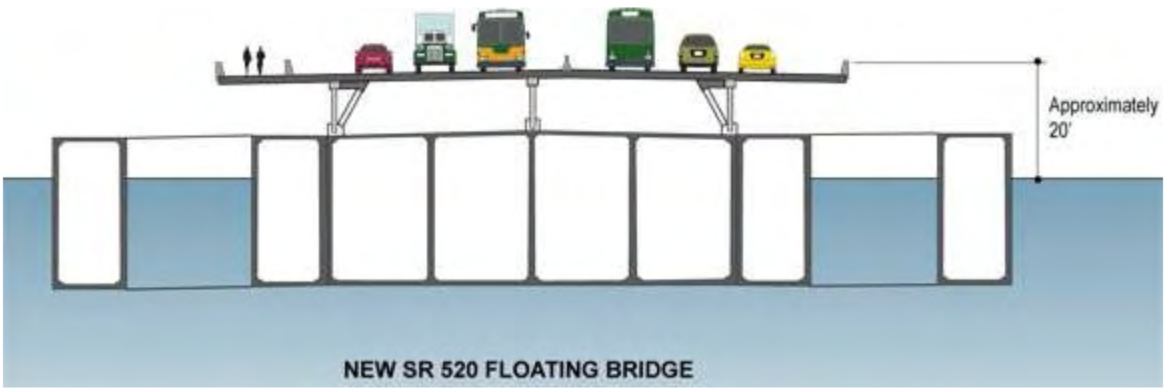


*Figure 4 – New SR 520 Floating Bridge, 35.37 m wide (note 20 feet = 6.10 m)*

The new SR 520 floating bridge will also be designed to accommodate future light rail as it is designed so that additional supplemental pontoons (not part of this project) can be added to the side of the longitudinal pontoons in the future to support the weight of light rail. In addition, all floating bridge superstructure elements, transition spans, and approach structures have been designed to accommodate light rail loading. However, it is anticipated that adding light rail will be a time-consuming process with significant additional permitting efforts, as well as added cost. It is presently not part of this program.

## DESIGN/BUILD CONTRACTING AND SR 520 CONSTRUCTION CONTRACTS

The potential for innovation and the benefits of cost and schedule advantages have prompted WSDOT to use design/build contracts for large construction projects, rather than the more traditional design/bid/build process. The design/build process is used for the construction of the floating portion and east approach of the Evergreen Point Bridge. The contract process for the remaining portions of the project will be determined based on funding and other considerations.

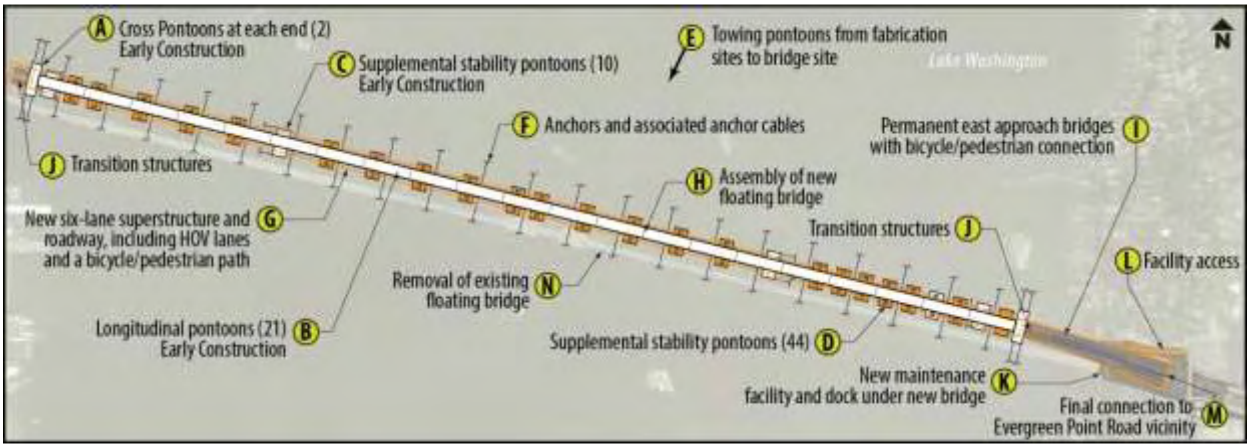


*Figure 5 – SR 520 Floating Bridge Project Elements*

In January 2010, WSDOT awarded a $367.3 million design/build contract to Kiewit-General Joint Venture to design and build a casting basin and to construct enough pontoons to replace the existing four-lane bridge. This work includes a total of 21 large longitudinal pontoons, 2 cross pontoons, and 10 smaller supplemental stability pontoons. These pontoons will be stored in the Aberdeen, Washington, area until they are needed either for the rapid replacement of the facility as a result of a catastrophic failure or for the planned (but not yet permitted at the time of this contract award) SR 520 bridge replacement. The design/build contract for the initial pontoon construction does not include towing pontoons to Lake Washington or construction of the new roadway. The owner and pontoon designer is WSDOT, and design team members for the pontoon casting facility are HNTB, prime consultant; KPFF, casting basin designer; Shannon & Wilson, geotechnical engineer; and Floyd|Snider, environmental and permitting.

The Pontoon Construction Project broke ground in February 2011 at a 22.26-hectare site in Aberdeen, on Grays Harbor, which is located on Washington State's Pacific coast. Since the groundbreaking, WSDOT and contractor Kiewit-General (K-G) have built a casting basin facility featuring a concrete batch plant, on-site water treatment, and a 1.62-hectare casting basin, where pontoon construction is underway.

By the Pontoon Construction Project's completion date in 2015, crews will have constructed 33 pontoons, 21 of which will be the 109.72-m-long, 22.86-m-wide, and nearly 9.2-m-tall longitudinal pontoons. These pontoons are the largest ever built in the state of Washington. Once complete, each batch of pontoons will be floated out of the casting basin, inspected, and towed a distance of nearly 500 km through the Pacific Ocean, Puget Sound, and the Ballard Locks into place on Lake Washington where they will serve as the floating foundation for the new SR 520 floating bridge.

In November 2010, WSDOT awarded a $306.3 million design/build contract to the Eastside Corridor Constructors to design and build the Eastside Transit and HOV Projects. Construction of this project began in April 2011, and this work is to be substantially complete by December 2013. This project is beyond the scope of this paper but interfaces with the east end of the floating bridge project.

In August 2011, WSDOT awarded a $586.5 million design/build contract to Kiewit-General Manson Joint Venture (KGM) for the Floating Bridge and Pontoon Project with the following scope of work.

- Construction of 44 supplemental stability pontoons
- Towing and assembly of 77 pontoons (33 from the Pontoon Construction Project)
- Design and construction of
  - 58 floating bridge anchors
  - floating bridge elevated structure
  - 250-m-long east approach structure
  - 58-m-long transition spans on each end of the bridge
  - 1,115-square-m maintenance facility and dock
- Removal of existing bridge

The team members for the Floating Bridge and Pontoon Project are owner and pontoon designer, WSDOT. Design team and their responsibilities are KPFF and BergerABAM, co-prime designers and overall design project management; KPFF, floating bridge superstructure, anchors, and stormwater; BergerABAM, approach structures, transition spans, roadway civil, and maintenance facility; IBT, low-rise superstructure; Wood Harbinger, floating bridge mechanical/electrical; IBI Group, intelligent traffic systems; Helix, maintenance facility architect; Hultz BHU, maintenance facility mechanical/electrical; The Berger Partnership, landscaping; Hart Crowser, geotechnical for anchors and Pier 36; GeoEngineers Inc., geotechnical for east approach and maintenance facility; HLB, specialty lighting; and Floyd|Snider, environmental and permitting.

Construction, towing, and assembly of floating bridge components are planned at multiple locations in Washington State. They are (1) Lake Washington, (2) city of Kenmore at the north end of the lake for construction of the low-rise precast roadway elements and pontoon anchors, (3) city of Tacoma at Concrete Technology Corporation's (CTC) facilities for construction of the supplemental stability pontoons and prestressed girders for the roadway, and (4) city of Aberdeen at Grays Harbor on the Pacific coast of Washington for the towing of the pontoons cast as part of the Pontoon Construction Design/Build Contract.

## DESIGN CRITERIA AND DESCRIPTION OF MAJOR ELEMENTS

Without a specific code for floating bridges, WSDOT prepared project-specific design criteria for the pontoons. Pontoons are reinforced concrete construction with a design 28-day concrete compressive strength of f'c = 51.7 MPa. The pontoons are post-tensioned in the longitudinal direction using multi-strand tendons. Design was in accordance with the American Concrete Institute Codes (ACI) based on ultimate strength and limited crack widths in accordance with ACI 224 *Control of Cracking in Concrete Structures*. The assembled pontoon joining will be by post-tensioning using American Society of Testing Materials (ASTM) A354 *Standard Specification for Quenched and Tempered Alloy Steel Bolts*, 76-mm- and 89-mm-diameter 965 MPa bolts.

The floating bridge will be designed to withstand a 100-year return period storm of 143 kph (20 second average) and resist a wind-driven wave of 1.92-m significant wave height.

The pontoon compartmentalization and damage analysis will consider loss of any two adjacent anchor cables and multiple cell flooding.

The floating bridge will be held in place by a total of fifty-eight 79-mm-diameter ASTM A586 *Standard Specification for Zinc-Coated Parallel and Helical Steel Wire Structural Strand* anchor cables with a breaking strength of 5,035 kN. There are three types of anchor cables attachment to the lake bottom; fluke, gravity, and drilled shafts. Anchor design conditions under service load are for a maximum lateral deformation of 0.15 m under a 2,491 kN load, and a maximum lateral deflection of 0.30 m under a 3,781 kN load. Extreme anchor design condition is 5,071 kN per anchor.

The drilled shaft anchors (five) are used in shallow water where minimum draft for vessel navigation is a requirement and consist of 3.05-m-diameter drilled shafts varying in depth of 21 to 26 m below lake bottom.

The Lake Washington lake bed soils presented a unique challenge to bridge construction and are the primary reason for development of the floating bridges on Lake Washington. Near-shore soils are competent but drop in depth precipitously and are overlaid by very fine low strength sediments. These lake bottom sediments represent essentially the fine ash materials from eons of eruptions of the Washington volcanoes in the Cascade Mountains, which have landed on the lake and settled to the bottom. This material, known as Diatomaceous silts, exists at the lake bottom to depths of approximately 18 m and sits atop approximately 45 m of equally poor very soft glacial outwash. The combined depths of these very poor soils make use of traditional bridge footings impractical and led to the development of floating bridges on Lake Washington.

Gravity anchors (eight) consist of 12.2-m-long by 12.2-m-wide by 7-m-high concrete boxes filled with ballast rock. These anchors are used in the more competent soils along the slopes of

the lake, and the anchor capacity is achieved through a combination of buoyant weight and base frictional resistance.
Fluke anchors (45) consist of a large reinforced concrete plate, which is jetted into the Diatomaceous silts. These anchors are used in the poor lake bottom soils that lie under the majority of the bridges length. Capacity is achieved by passive soil resistance; and to increase anchor capacity, approximately 1,135 tons of ballast rock is placed on top of the anchor in the soil wedge zone.

The elevated structure roadway has been designed to the American Association of State Highways and Transportation Officials (AASHTO) design criteria for Highway Design and also includes the effects from the floating pontoon behavior. The design will consider options for future high-capacity transportation and light rail. The low-rise roadway section will consist of precast concrete segmental ribbed slabs with segments approximately 17.22 m long by 4.57 m wide. Construction will be by transversely post-tensioned match-cast segments with longitudinal post-tensioning and additional bar transverse post-tensioning between segments taking place after the segments are erected on Lake Washington. Support of the elevated roadway will be on columns spaced approximately 9.14 m on center with expansion joints located at each pontoon joint. To minimize work on the lake, precasting as much as possible is done at the Kenmore facility.

The east and west high-rise sections will be conventional precast, prestressed I-girders with cast-in-place deck supported on column bents at approximately 27 m on center with expansion joints spaced 55 to 110 m apart. The I-girders will be fabricated at CTC's plant in Tacoma and delivered to the site via truck. The cast-in-place cross beam and slab construction will occur on Lake Washington. The east approach will accommodate a navigation opening of approximately 69 m width by 21 m height. This navigation opening is more substantial than that which exists today and permits construction of the new bridge without the need for a drawspan similar to that which exists on the current structure. The fixed shoreside approach structures will be designed in accordance with the "Essential Bridge" and project-specific seismic design criteria.

The transition spans joining the floating structure with the fixed structure have been designed to accommodate wind, wave, and seismic movements.

The maintenance facility supporting daily bridge maintenance activities consists of a 1,115-square-m facility with covered parking, shops, storage, crew offices, showers, eating spaces, and an emergency generator. It will house the communications and bridge utilities control center, power, intelligent traffic systems, closed circuit TV, bridge monitoring systems, and will include a dock for maintenance boat moorage.

## SR 520 FLOATING BRIDGE AND PONTOON CONSTRUCTION STATUS SUMMARY

As of March 2013, the SR 520 Floating Bridge and Landings Project is well underway and one of the major components of the work on the site has been providing site access to both the floating bridge and the east approach structures. The east shore of the bridge is a 30-m-tall bluff that limits access to the lake. To provide access, a series of walls and roadways were constructed throughout 2012. These walls included three soldier pile tie back walls up to 15 m high, a soil nail wall, and many gravity block walls. Together these walls allowed construction of a 300-m-long road with grades as steep as 15 percent. Along with constructing the road, stormwater control has been a major effort throughout 2012. KGM has completed an on-site stormwater control pond, as well as installed a water treatment system, to ensure the quality of Lake

Washington is not jeopardized. All stormwater from the site is directed to the treatment area through pipes and ditches.

Progress on the pontoon casting and floating bridge construction has been steady with 8 of the 23 large pontoons (21 longitudinal and 2 cross pontoons) and 16 of the 54 supplemental pontoons cast. Most of the transverse pontoon anchors have been successfully installed and tested. The eight longitudinal anchors will be tested using the new bridge once the corresponding pontoons are in place.

Two pontoon groups of five pontoons each have been joined and are expected to be in place by the end of the summer 2013, and KGM is currently installing the eastern most superstructure on the pontoon near the shore. Additional superstructure will be constructed as pontoons are joined. Precast panel construction on the low-rise portion is expected to start in the summer of 2013.

Approaches are being constructed from both the east and the west sides. Within the Floating Bridge and Landings project contract is the design and construction of the first pier for the west approach, referred to as Pier 36. At this time, KGM has constructed the four drilled shafts for Pier 36 and has begun constructing the pier caps. The first contract to connect the new bridge to the existing west approach is expected to be released by the owner in the spring of 2013. When this contract is complete, KGM will perform the traffic switch and open the new floating bridge.

For the east approach, KGM is constructing two cast-in-place segmental bridges starting with the Pier 2 south structure, moving to Pier 2 north structure, then to Pier 1 south structure, and finalizing with the Pier 1 north structure.

The upland cofferdam for Pier 2 was installed in the spring of 2012, following with the concrete spread footing. Currently both the columns and transition piers for the two different structures are complete and the travelling formwork system for the segmental casting is being assembled on site. The abutment, Pier 3, is also constructed and the end span segments connecting to the Pier 2 segments are being constructed on falsework.

The in-water cofferdam for Pier 1 has been installed, and the spread footing for the two structures is constructed. Work on the columns and transition piers have begun and are expected to be constructed over the spring and summer of 2013 and ready for the travelling formwork system to be brought over from Pier 2 in 2014.

The upland work consists of the roadway tie-in to the new corridor to the east of the bridge, a new 1,115-square-m maintenance facility, as well as general landscaping, site walls, and park-like finishes.

The maintenance facility is not on the critical path and KGM will begin construction once the east approach bridge above the site is completed. This work is expected to take a year and, once completed, will allow the installation of the complicated Bridge Control Systems with the commissioning and testing program.

Construction of the large pontoons in Aberdeen has incurred some cracking in the ends of the pontoons. These design/construction issues have been resolved by WSDOT. Remedial actions have been successfully implemented, but there will likely be a short delay to the opening of the new bridge to traffic as late as 2014 to mid-2015.

# SEEPAGE WATER CONTROL STANDARD AND METHOD OF DRILLING AND BLASTING SUBSEA TUNNEL

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**ABSTRACT**

*For subsea tunnels excavated with drill and blast method water ingress has tremendous impact on construction, structural design and drainage during the operation period. It is too costly to completely block seepage water. Therefore the reasonable standard of acceptable water ingress must be established on the basis of geological groundwater seepage investigation, and the possible technical measures must be adopted to ensure tunnel construction safety and economical operation. Based on international tunneling experience and practice of the Qingdao Jiaozhou Bay subsea tunnel it is concluded that the water ingress control standard is 0.2m$^3$/m.d for drill and blast subsea tunnels with a cross section area of 60-150m$^2$. Exploration holes are used to ascertain the ground water condition ahead of tunnel work face. The finding of the exploration holes is then used in designing the grouting scheme. Wet sprayed C35 infiltration-resistant shotcrete is used as the primary support. The small amount of seepage water is discharged via the channel between primary support and secondary lining.*

**Key words:** subsea tunnel, drill and blast, water control standard

## 0 INTRODUCTION

In the current era, the world and China's economy is developing rapidly. To meet the requirements of human transportation, more and more subsea space has been utilized. China is entering a period where long tunnels, underground space and subsea tunneling are expanding fast. The construction of Xiang'an subsea tunnel by drill and blast method in Xiamen Fujian, Jiaozhou subsea tunnel, also by drill and blast method in Qingdao and Shiziyang tunnel by shield method in Guangdong set a precedent of subsea tunnel construction in China. There are also a number of subsea tunnels under planning.

The level of tunneling and underground engineering in China is very high and the total tunnel length ranks first in the world. Furthermore, the technology of mountain tunnel engineering becomes mature. But compared with the land tunnels, the subsea tunnels are more complex, and in the construction of some particular subsea tunnels big challenges have emerged, such as instability and excessive seawater seepage caused by faults and fracture zones. In China the design concepts, norms, standards of subsea tunnels, etc are blank or in the initial research phase, and groundwater ingress is the key factor affecting the safety and the economy of subsea tunnels. In order to ensure a tunnel to be constructed economically and reliably, water inflow standards must be formulated. Feasible technical measures on the basis of surveys of groundwater seepage must be taken to ensure the safety of tunnel construction and the economical operation.

For a long time, the practice of underground engineering in China is to both block and drain the water, but mainly to drain. But the long-term massive drainage destructs the original balance of groundwater, resulting in drawdown of groundwater, which leads to subsidence of ground surface and causes serious environmental problems. Building underground structures with high groundwater level, we should take the principle of "mainly blocking, but accepting limited ingress" according to the specific environment around the tunnel. The allowable water ingress is set on the basis of the particular conditions around the underground structure, as the Oslofjord subsea tunnel in Norway. The allowable water ingress is determined according to the capacity of drainage equipment and economic efficiency. While for tunnels located in residential districts and leisure areas on land, environmental authorities made allowable ingress as $0.288m^3/ d{\cdot}m$. The Yuanliangshan tunnel of the Yuhuai railway is in a high water pressure areas and the allowable ingress is designed as $5m^3/d{\cdot}m$. The actual water ingress at the Geleshan tunnel is $0.95m^3/m.d$. The water ingress at the subsea section of the Seikan tunnel is $0.2736\ m^3/d{\cdot}m$. The maximum water inflow standard after grouting at the Jiaozhou Bay subsea tunnel is set as $0.4m^3/d{\cdot}m$ and $0.2m^3/d{\cdot}m$ for the main tunnel and service tunnel, respectively.

## 1 THE PRESENT SITUATION OF GROUNDWATER CONTROL MEASURES

When building underground structures using the drill and blast method the groundwater control measures have big impacts on the construction, operation safety, investments and operation costs. The influence of confined groundwater is big, and if not disposed properly it will cause water bursting, mud inrush, etc, which will affect the safety and investment of the tunnel construction, and the safety of the workers, as well as the operation cost and safety of the tunnel. To ensure the safety of construction and operation and to cut the costs of construction and operation, every country in the world is researching the groundwater control technology. The key points focus on these problems: advanced geological prediction technique, advanced water probing technique, pre-grouting reinforcement technique, new-style drainage systems and materials, and techniques related to prevent water inrush and mud inrush etc.

### 1.1 ADVANCED GEOLOGICAL PREDICTION

Advanced tunnel geological prediction techniques were developed in the nineteen forties or nineteen fifties. Furthermore, Japan, France, England, former Soviet Union, Germany, USA, etc. have listed it as work that must be done when building a tunnel. In China, the research of advanced tunnel geological prediction started at the end of the nineteen fifties, but the technique was not used in tunnel construction until 1970s.

After entering the 21st century, advanced tunnel geological prediction technique gets rapidly developing. The Yuanliangshan tunnel of Yuhuai railway is a representative, in which reflection of elastic wave (the TSP method), reflection of electromagnetic wave (geological radar method), infrared detection and many other comprehensive geological prediction methods of geophysical method combined with the advanced drilling exploration are used in the tunnel construction. The selection of the geological prediction method is based on the engineering geological problems encountered such as karst, high pressure mud and water gush-in, katamorphic zone and etc. These established the present basic ideas and methods of advanced tunnel geological prediction technique.

The advanced geological prediction techniques went through several stages of development: the geology method, the parallel pilot (tunnel) method, the horizontal protruded drill hole, the advance borehole acoustic log and cross hole sonic transmission and the comprehensive geological prediction. While advanced tunnel geological prediction technique is vigorously carried out, the advanced geological forecast technology is developing fast.

### 1.2 ADVANCED WATER-PROBING TECHNIQUE

Probe drilling is used to obtain geological information ahead of the tunnel work face. The method can be used for geological prediction under various conditions, however, it must be used in areas of complex geological conditions, such as fault and fracture zones containing rich water, water-bearing karst areas and so on.

Percussion drilling is normally used where no cores are obtained. However, the information on lithology, rock strength, rock mass completeness, karst caves, underground rivers, development situation of groundwater, etc can be roughly gained by the sound, drilling speed and its changes, rock powder, sticking, drill pipe vibration, flushing fluid color and the variation in discharge, etc. Water inflow from each probe hole can be accurately measured.

In complicated geological sections rotary core drilling may be used. It is only used in special strata for accurate determination of ground properties in laboratory tests.

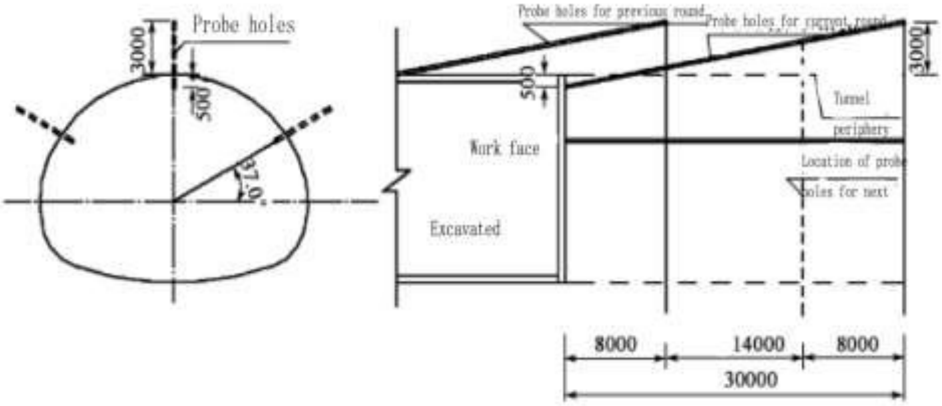


*Fig. 1 Sketch for advance probe drilling (unit: mm)*

### 1.3 PRE- GROUTING REINFORCEMENT TECHNIQUE

Pre-grouting is used to prevent the gushing water or water seepage into the tunnel. Curtain grouting is carried out in faults or fracture zones to prevent water inflow. The main injection materials are ordinary cement single-fl, superfine cement single-fl, specially sulphoaluminate cement single-fl and so on.

In the fracture sections, grouting rings around the tunnel excavation are formed by pre-grouting to seal off the water cracks and the gushing water space in the bedrock. According to the infiltration quantity after pre-grouting, the grouting holes in the ring system of tunnel initial support are adjusted to block the groundwater runoff channels further and reduce the quantity of groundwater infiltration. Supplementary grouting treatment to primary support leakage is conducted before installing waterproof boards. That measure should be valued especially during the construction period. Sketches of pre-grouting are shown in Fig. 2.

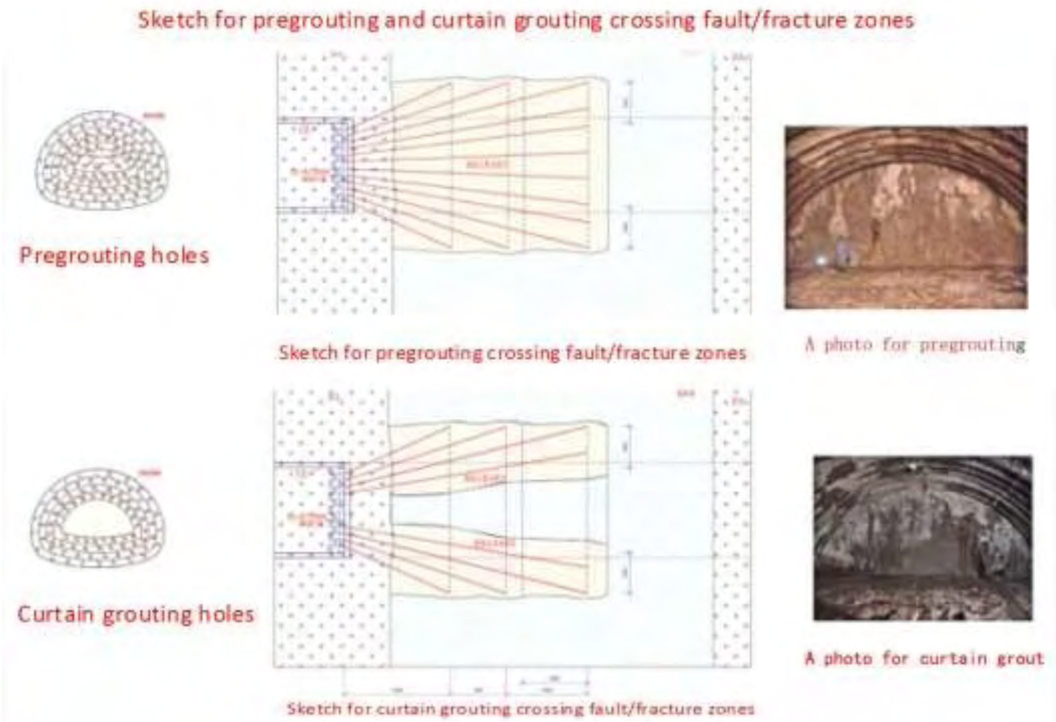


*Fig. 2 Advanced pre-grouting*

### 1.4 NEW –STYLE DRAINAGE SYSTEMS, MATERIALS AND SO ON

Composite lining for tunnel waterproofing consists of pre-grouting of surrounding rocks, radial supplementary grouting, improving the infiltration resistance level of sprayed concrete, the initial support and laying waterproof boards between primary support and secondary lining, waterproofing of the construction joints and deformation joints, self-waterproofing concrete of secondary lining, etc. Drainage measures include circumferential drainage blind pipes (distance of 5~10 meters), longitudinal drainage blind pipes, central drainage ditches, drainage opening and so on. Frequently-used tunnel waterproof and water drainage system is shown in Fig. 3.

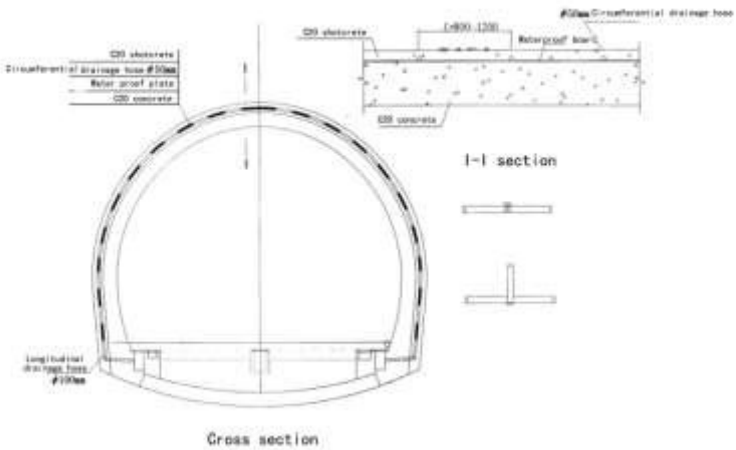


*Fig. 3 Typical tunnel waterproof and drainage system*

## 2 TUNNEL WATER SEEPAGE THEORY

Rock mass is a multiphase material composed of solid (rock), liquid (water and so on), and gas (air and so on). Seepage occurs along these winding passages which are in different shapes and sizes. It is difficult to study the movements of groundwater in individual holes or cracks. Therefore, people don't study individual water particle directly. Instead, they study the overall properties. The rock mass is taken as a continuous porous media, and then the classic continuum percolation theory is used in analyses. In fact, groundwater flow only exists in void space of rock mass. For the sake of the convenience, we use a kind of imaginary flow to replace the real groundwater flow. The properties of the imaginary flow (such as density, viscosity etc) are the same as the real groundwater, which is full of the spaces including not only aquifer void space, but also the space occupied by aquifer rock particles. In addition, when the imaginary flow is moving, the following hypotheses should be erected:

(1) The flow through any section should be the same as the real flow through the same section,
(2) Its pressure or waterhead in a section should be equal to the real one, and
(3) The resistance in any rock and soil mass bulk should be equal to the real one.

The imaginary flow is called seepage flow, short for seepage. The visuospatial area occupied by the imaginary flow is called seepage zone or seepage field. It has two basic characteristic parameters, namely velocity of flow and waterhead, and the former is vector, and the later is a scalar.

Fluid flow acts on rock mass as fluid pressure, and influences the distribution of the rock stress field. The changes in the stress field may change the joint opening or create new cracks, and then impacts the porous property. At the same time there exist phenomena of physicochemical changes, temperature variation, etc. Rock mass is a heterogeneous compound of multi-field coupling.

Since tunnel design needs to consider water pressure in the calculation of load on the structure, the water pressure is taken as direct boundary force acting normally on the surface of the structures. But from the view of seepage, water flow can form seepage field passing through these media. In the condition of given boundary condition, seepage field formed by water in porous media— $H_{(x,y,z)}$ is:

$$H_{(x,y,z)} = Z + \frac{p}{\gamma_w} \qquad \text{(Formula 1)}$$

In the formula above $P$ is pore water pressure; $g_w$ is the unit weight of water; the positive forward direction of Z is the opposite of gravity acceleration.

Because of the gradient of pore water pressure, the pressure components of the seepage body force, which is formed in the process of seepage along the direction of x, y, z can be indicated by the following formula:

$$\begin{cases} p_x = -g_w \dfrac{\partial H}{\partial x} = -\dfrac{p}{x} \\ p_y = -g_w \dfrac{\partial H}{\partial y} = -\dfrac{p}{y} \\ p_z = -g_w \dfrac{\partial H}{\partial z} + g_w = -\dfrac{p}{z} \end{cases} \qquad \text{(Formula 2)}$$

The seepage body force can be divided into two parts: seepage force S and buoyant force f, which is proportional to hydraulic gradient. They can be shown as:

$$\begin{cases} S_x = -g_w \dfrac{\partial H}{\partial x} \\ f_x = 0 \end{cases} \quad \begin{cases} S_y = -g_w \dfrac{\partial H}{\partial y} \\ f_y = 0 \end{cases} \quad \begin{cases} S_z = -g_w \dfrac{\partial H}{\partial z} \\ f_z = g_w \end{cases} \qquad \text{(Formula 3)}$$

As to seepage body force, the buoyant force caused by hydrostatic pressure won't destroy rock mass directly. But it can reduce effective weight of rock mass, so cuts down rock mass's ability of resistance to damage. Thus, it is a kind of negative destructive force. Seepage force is a kind of positive destructive force. It changes effective stress acting on rock framework directly, influencing the stress state and stability of rock mass and supporting structures.

In fact, water pressure is a body force acting on rock mass and substructure under the phreatic line in the process of seepage. Only when there is impervious surface area on computational domain boundaries or in it, do area loads exist along the normal of the impervious surface area. To substructure, body force is the average form of water ballast, and boundary force is in special form.

## 3 INTERNATIONAL STANDARDS FOR SEEPAGE WATER CONTROL OF SUBSEA TUNNEL

Groundwater control of tunnel project is a series of methods and measures to control the loss of groundwater due to the construction of the tunnel, and to ensure that he surrounding environment is not affected by the hazards of water leakage. It is an important element in the tunnel construction.

The allowable amount of drainage of a tunnel is governed by practical limitations related to the excavation process and pumping capacity.

In the tunnel design, following details are considered: expected maximum water pressure, maximum lining surface pressure during the construction stage and completion stage, possible deformation and the highest and lowest temperatures of surrounding rock and structure, and corrosion on the waterproof materials caused by underground water.

Treatment of underground structure for groundwater control can be divided into three classes: waterproof, emission control and drainage.

The waterproof type is mainly adopting various measures to block the water outside the structural lining. The drainage type is to drain off water through the drainage structure. The emission control type adapts the principle of taking limits emissions, when building structures at the regions of high-water level and do not allow excessive emission of groundwater. Measures are taken to let a large amount of groundwater trapped outside the tunnel. The waterproof type must take into account the role of pressure, and the drainage type must ensure the drainage system to be unimpeded.

In recent years, as environmental awareness increases, more and more attention is paid to the problem of environmental protection of groundwater. Many countries began to develop the so-called "acceptable" tunnel groundwater discharge standards. Kveldsvik et al. (2001), describe their approach to a maximum allowable inflow to the tunnel expressed by the percentage of the run - off in a catchment area. For their project, they arrived at the following conclusions:

(1) Inflow < 10 % of the run - off yields no, or small consequences.
(2) Inflow in the range of 10-20 % of the run -off yields medium consequences.
(3) Inflow > 20 % of the run-off yields large consequences.

The run-off from a given area (Q) is defined as the precipitation (P) minus evapotranspiration (E). The project described by Kveldsvik aims at a defined minor negative influence on the surrounding environment. This implies that the strongest class prevails, imposing a maximum allowable inflow of less than 10 % of the run -off. The tunnel will be designed with a number of various sections to meet the local environmental conditions, thus the maximum allowable inflow will vary from 5l/min/100m to 40 l/min/100m, being considered as on the "safe side" as regards to possible damage.

A commonly used figure in Norwegian subsea tunnels is a maximum inflow to the tunnel of 30 l per minute per 100 meters of tunnel (1/min/m)（0.432 $m^3/d\cdot m$）.The water inflow of the cross harbour section of Japan's Seikan Tunnel is Approximately 45 l /min/100m.( 0.648 $m^3/d\cdot m$）. For the Jiaozhou Bay Qingdao Subsea Tunnel the water inflow is 0.4 $m^3/d\cdot m$ for the main tunnels and 0.2 $m^3/d\cdot m$ for the service tunnel, respectively.

## 4 ACTUAL WATER INFLOW OF THE SUBSEA TUNNEL IN NORWAY

In the last 30 years, Norway has built more than 40 subsea tunnels, the total length is longer than 240km, among which 24 are highway tunnels, and the others mainly serve to the offshore oil industry, including pipe holes, cable holes, and so on.

Most subsea tunnels of other Nordic countries adopted the technology of Norway subsea tunnel. Subsea tunnel in Norway are mainly built in hard rock of Precambrian period, and the most typical is granitic gneiss. The drainage standard of the method of drilling and blasting is 30 litres per minute per 100 meters (0.432 $m^3/d\cdot m$).

For the Oslofjord tunnel Q=0.288 $m^3/d\cdot m$. The water inflow of most tunnels in operation is greater than 30 L/min/100m, and the largest water displacement of several tunnels being built is 50-500L/min/100m. The measured value of subsea tunnel in Norway is 20-460 L/min. Part of the tunnel seepage volume statistics of Norway is given below:

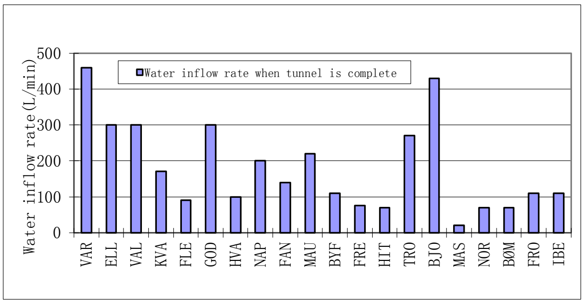


*Figure 4 Statistics for water leakage of some tunnels at completed*

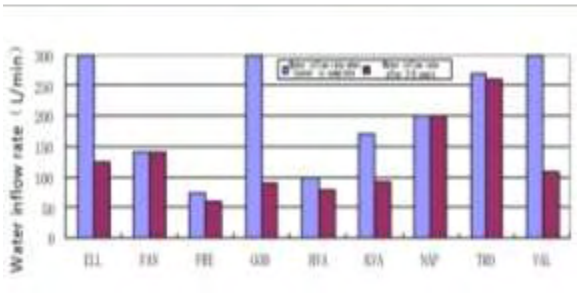

*Figure 5 Statistics for water leakage of some tunnels during the operation*

# 5 CALCULATION METHOD OF GROUNDWATER PRESSURE AND DISPLACEMENT

## 5.1 CALCULATION OF WATER PRESSURE

For the subsea Tunnel, in addition to the pressure of surrounding rock, the supporting structure withstands high water pressure. The pressure of surrounding rock acting on supporting structures can be reduced by strata arch action. But hydrostatic pressure won't be influenced by this.

Design of water pressure value is the key factor to the design of the subsea tunnel lining. Design size of external water pressure is not only related to the head, but also related to groundwater handling (if a certain amount of water inflow is allowed).

The groundwater pressure will not act on the second lining due to the waterproofing layer between the initial lining and the second lining. In this way the second lining becomes an independent structure.

Because the secondary lining is thin, seepage body force within the lining can be simplified to the water pressure on the outer edge of lining.
For a tunnel with an appropriate drainage system and surrounding rock grouted, a simplified calculation model is used (Fig. 6) for the analysis.

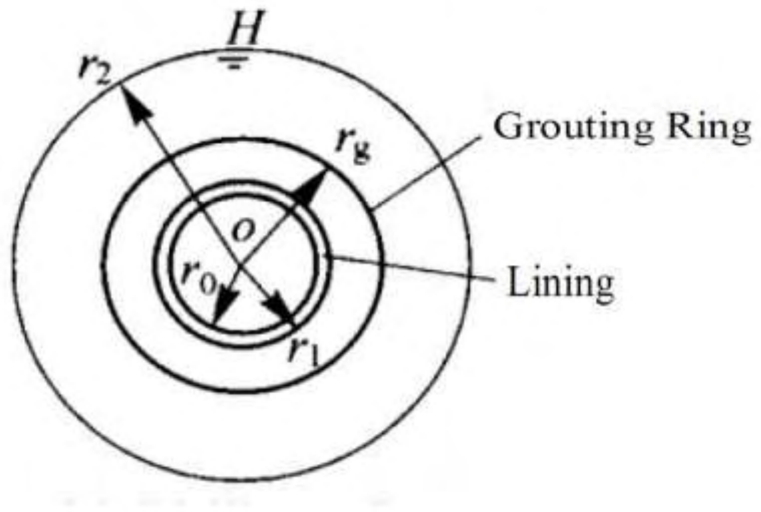


*Fig.6 Theoretical analysis model diagram*

We consider a tunnel with circular cross section within a soil rock with homogeneous and isotropic permeability. With an appropriate drainage system and surrounding rock grouted, the circumference of the tunnel is kept at a constant energy head H.

According to Darcy's Law and continuity equation of water flow: ($\varphi$ -Velocity potential, R—the distance of calculated points to the axis)

$$\frac{d}{dr}\left(r\frac{d\varphi}{dr}\right)=0 \qquad \text{(Formula 4)}$$

Because the lining thickness is relatively small to the groundwater head H, seepage force on the lining can be reduced to external force on the outer of the lining which is the pore water pressure here. According to Darcy's Law and continuity equation of water flow and boundary condition, we can get the volume Q of water and seepage force P

$$Q = \frac{2\pi H k_r}{\ln\frac{H}{r_g} + \frac{k_r}{k_g}\ln\frac{r_g}{r_l} + \frac{k_r}{k_l}\ln\frac{r_l}{r_0}} \qquad \text{(Formula 5)}$$

$$P = \frac{\gamma H \ln\frac{r_l}{r_0}}{\frac{k_l}{k_r}\ln\frac{H}{r_g} + \frac{k_l}{k_g}\ln\frac{r_g}{r_l} + \ln\frac{r_l}{r_0}} \qquad \text{(Formula 6)}$$

$k_l$——permeability coefficient of lining ;
$k_g$—— permeability coefficient of grouting
$kr$—— permeability coefficient of surrounding rock
$r_0$ ——inner radius of lining;
$r_1$——outer radius of lining;
$r_g$ ——radius of grouting ring

Known from the above two equations, use totally enclosed lining, $k_l$ =0, with Q=0, $P = \gamma H$ water pressure without reduction. When $k_l$ tends to infinity, that is, when the lining is permeable, P=0, lining does not withstand the water pressure. When considering the drainage performance of the lining, pressure can be reduced, and the use of grouting in rock let $r_g$ increase or decrease, Q in the equation (5) and P in the equation (6) both decrease.

This means that only in the context of considering lined drainage, surrounding rock grouting can reduce the quality of groundwater discharge and reduce water pressure on lining. In this way, through the formation of grouting and exhaust system behind the lining, groundwater control of emissions was achieved.

A circular subsea tunnel is shown in Figure 7, where $r_a$ is the inner radius of the tunnel, $h_a$ is the bear medial energy head, $r_2$ is the outer radius of the secondary lining, $h_2$ is the outer energy head of the second lining, r1 and h1 is the radius and the outer energy head of the primary support.

Following notations are used: grouting reinforcement ring outer radius is $r_g$, grouting reinforcement ring outer energy head for $H_1$, sufficient distance outer radius for $r_0$, sufficient distance outer energy head for $H_0$. The surrounding ground has the isotropic permeability, permeability coefficient of concrete lining is $K_l$, permeability coefficients of surrounding rock is $K_S$, permeability coefficients of grouting reinforcement ring is $K_S$. Let the ratio of permeability coefficient of surrounding rock grouting reinforcement ring, primary support, and secondary lining respectively be ng, $n_l$ and $n_2$.When groundwater behind secondary lining is all discharged by drainage systems, this leads to $n_2$=0. Fig.8 shows the relationship between the changes of water inflow into tunnel and permeability of initial support:

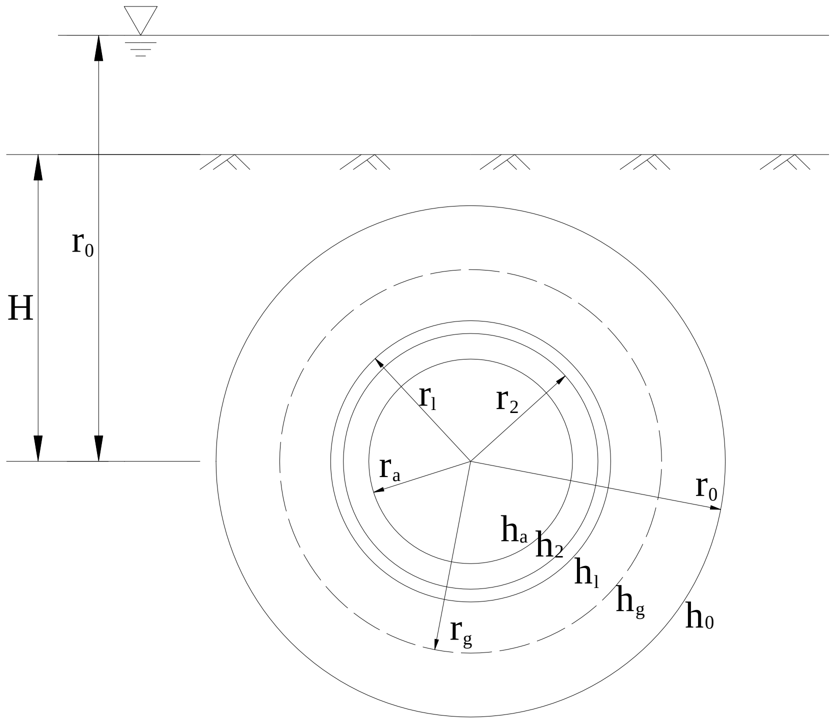


*Fig.7 Calculation model of groundwater seepage field of subsea tunnel*

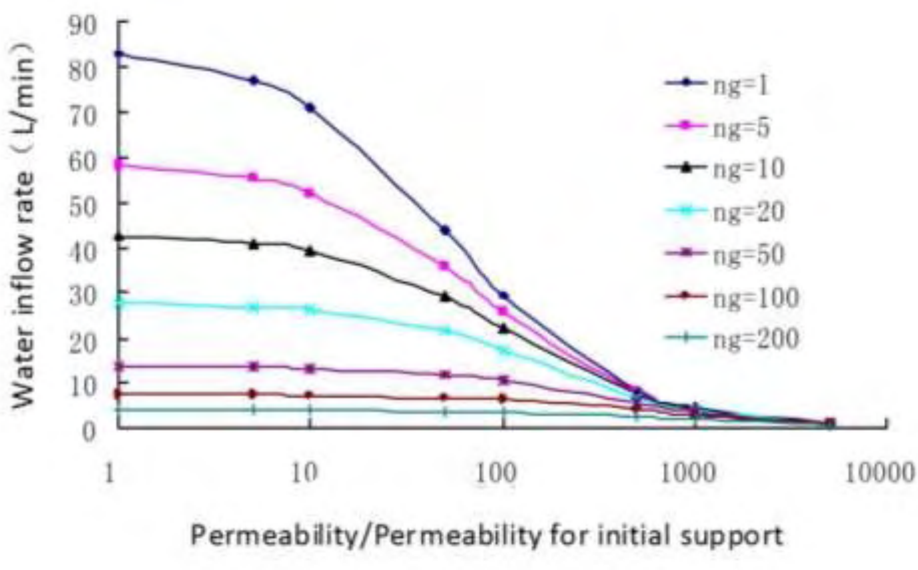


*Fig.8 Curve of tunnel discharge and lining permeability*

(1）When the permeability of grouting ring is constant, the quality of inflow of subsea tunnel reduces as the permeability of primary support decreases, and the stronger the permeability of grouting circle, the more obvious of reducing the trend of inflow.

(2）To unsupported hole without grouting, this is n1=0，ng=1, subsea tunnel seepage is 82.804 $m^3$/m·d. For the tunnel with initial support and without grouting, we can set n1=500, then subsea tunnel seepage is 8.2353 $m^3$/ m•d which is larger than Japan's Seikan Tunnel (0.2736 $m^3$/m·d) and larger than Norway subsea tunnel (0.432 $m^3$/ m•d). However, the permeability coefficient of the initial support, which fully meets its quality requirements of seepage control, is only $1.0\times10^{-8}$m/s.

To achieve the above standard for water control, the permeability coefficient of the initial support is reduced to 1/5000 of the permeability coefficient of the surrounding rock. Therefore, it is not feasible only depend on the waterproofing of initial support when the subsea tunnel is in fault fracture zone.

## 6 ENGINEERING EXAMPLE OF JIAOZHOU BAY TUNNEL IN QINGDAO

There is a fault F3 in the right line of Jiaozhou bay tunnel, the rock formation around which is slightly weathering rhyolite containing fault fracture zones. The geological profile is shown in Fig. 9. The highest sea level in the history is about 44.2m above the surface of the bedrock and the crown of the tunnel is some 27m below the bedrock surface. There exist boreholes near this fault were used to carry out rock hydrogeological survey. The permeability coefficients of rock masses in different depths are shown in Table 1. In comparison, the permeability coefficient of the surrounding rock at the borehole has a smaller discrete range.

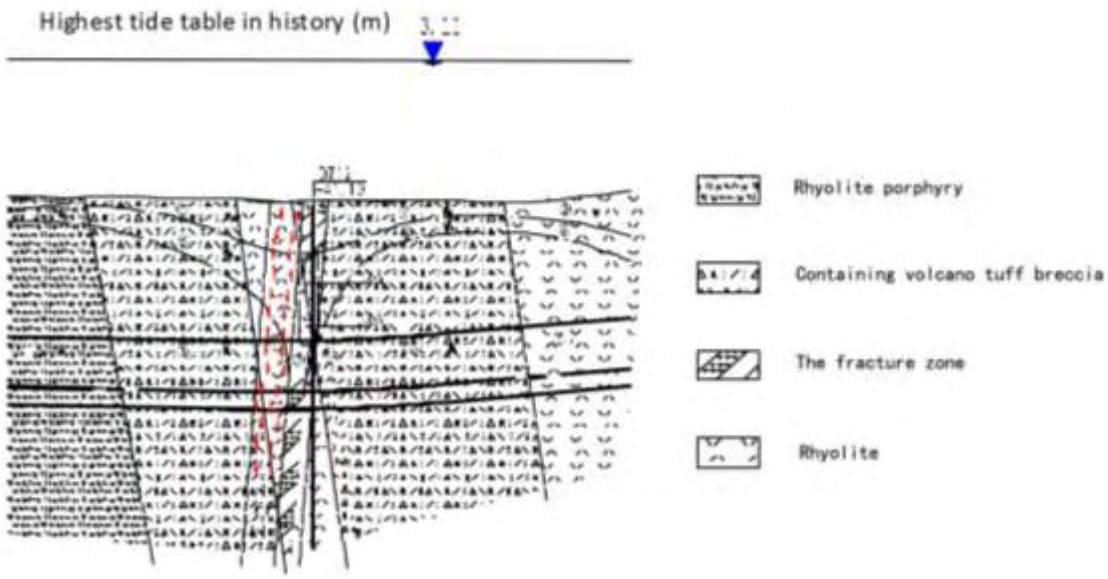


*Fig.9 Fault geological profile*

*Table 1 Permeability coefficient of rock mass at different drilling depth t*

| test type | experimental depth （m） | coefficient of permeability（m/s） |
|---|---|---|
| pressurized-water | 7.10～12.50 | $8.10\times10^{-8}$ |
| | 13.50～18.90 | $5.79\times10^{-8}$ |
| | 22.30～27.70 | $3.47\times10^{-8}$ |

In accordance with the specific geological conditions of the fault, select FLAC$^{3D}$ finite difference computation program for concrete calculation. Analysing seepage, consider rock mass as continuum media and the flow of the fluid in porous media obeys the law of Darcy. When computing, it allows the calculation of fluid models to be independent from the usual calculation of solid mechanics by FLAC$^{3D}$ and just consider seepage; also can calculated with solid model for describing the coupling feature of fluid and solidity. Taking both computational accuracy and the solution time into consideration, the dimension of the model is

400×150×1(length×height×width, unit: m), the surface of the bedrock is 30m away from the surrounding rock of the vault. If there is no especial illustration in the analog in the following, the sea level is 45m away from the bedrock, and the permeability coefficient is rounded to $1.0\times10^{-7}$m/s.

(1) The comparison of the coupling analysis for seepage field and stress field only considering the relationship between seepage field and different constitutive models:

In the comparison, according to the unlined tunnel excavated, consider three kinds of calculation conditions to analyse: ①Only consider the seepage field not stress field; ②Adopt the elastic model as the constitutive relation of the surrounding rock, considering the interaction between seepage field and stress field of the surrounding rock; ③Adopt the plastic model (the Mohr-Coulomb model) as the constitutive relation of the surrounding rock, considering the interaction between seepage field and stress field of the surrounding rock.
The pore water pressures are almost the same in these three different cases. The quantity of gushing water increases considering the interaction of stress field and seepage field. But compared with the condition of only considering seepage field, the change of the quantity of gushing water is smaller than 2.5%. This can also illustrate that seepage field is different with stress field. The pressure of the surrounding rock will reduce as the arching effect of tunnel excavation, while the seepage force will remain unchanged. The changes of the seepage force and the quantity of gushing water caused by stratum deformation are little. But if calculate characteristics in stress field state, such as the stratum deformation force, the lining structure stress, etc, must choose the appropriate constitutive model to analyse and consider the interaction of the seepage field and stress field. In the following calculation, only take the seepage field into consideration, because analyse the seepage field only. Analyse the pore water pressure and the quantity of gushing water.

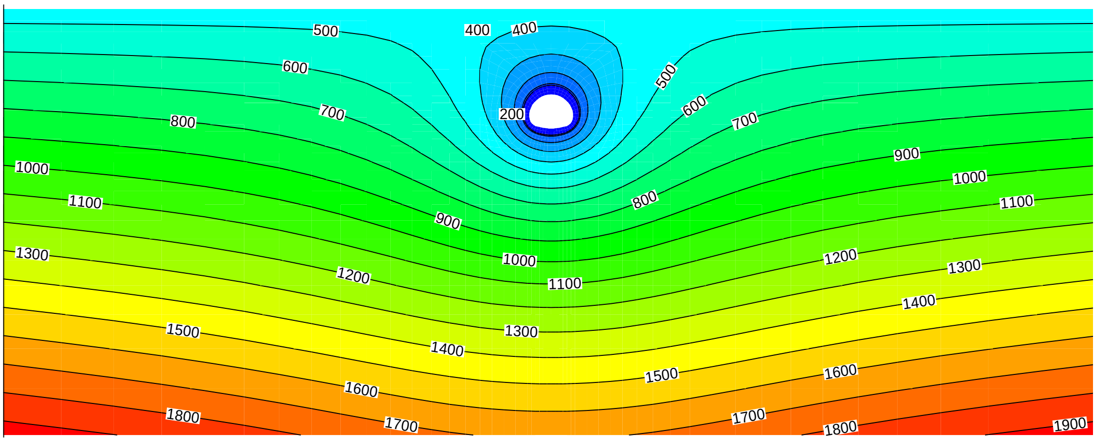


*Fig.10 Distribution of pore water pressure (units: KPa)*

*Table 2 Water inflow under different operating conditions*

| condition | water inflow ($m^3$/（m·d）) |
|---|---|
| ① | 1.928 |
| ② | 1.970 |
| ③ | 1.976 |

In order to satisfy the calculation of Park method, the multi-center circular tunnel can be converted into circular tunnel that can be calculated by equivalent method. The distance (h) from the earth’s surface to the center of the circular tunnel can be indicated by the distance from the earth’s surface to the centroid of the multi-center circular tunnel. The quantity of gushing water calculated by Park method through the method of equivalent area and equivalent perimeter is shown in Table 3.

*Table 3 Water equivalent calculation*

| condition | water inflow ($m^3$/（m·d）) |
|---|---|
| equivalent area | 1.8813 |
| Equivalent perimeter | 1.9003 |

The pore water pressure distributions from the vault to the earth’s surface through the method of equivalent perimeter using Huangfuming method and the pore water pressure distributions under the condition of ① are shown in Fig.11.

Through the comparison of numerical solution and analytic solution, it can be seen that, in the appropriate equivalent conditions, numerical solutions and analytical solutions are in good agreement. The two prove each other, attesting their applicability.

(2) Effects of different permeability coefficient of surrounding rock

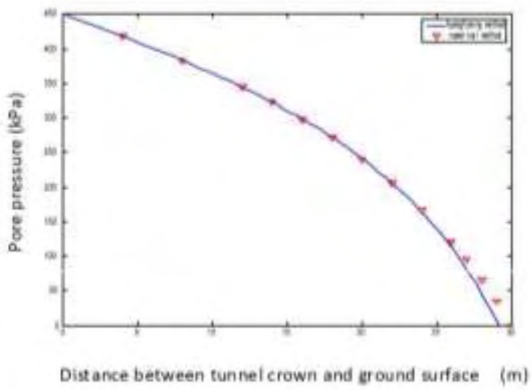


*Fig.11 Pore water pressure on top of the tunnel by different calculation methods*

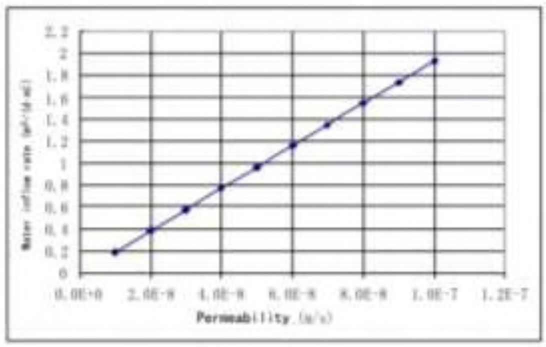


*Fig.12 Relationship of permeability coefficient and water flow*

In analysis, consider seepage characteristics of unlined tunnels under different permeability coefficients of surrounding rock. The pore water pressure distribution is in accordance with the one in Fig.11. This illustrates that the same as circular tunnels in homogeneous isotropic media, the changes of the permeability coefficient don’t influence the distribution of the outdoor rock pore water pressure of the multi-center circular tunnels. The quantity of gushing water along with the change of the permeability coefficient of surrounding rock is shown in Fig.12. It can be seen that the quantity of gushing water is proportional to the permeability coefficient.

(3) Effects of different water depths

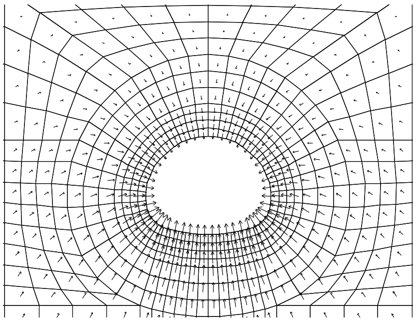

*Figure 13 Flow direction vector diagram*

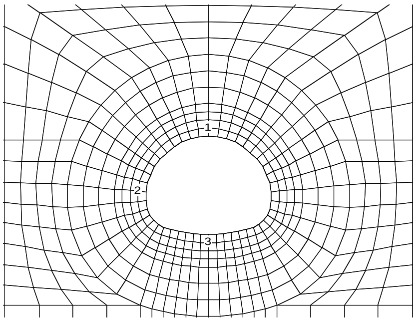


*Figure 14 Monitoring points of pore water pressure*

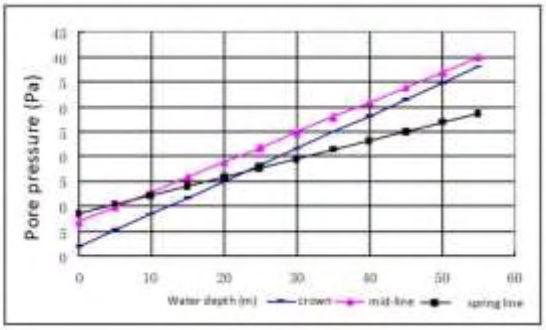


*Figure 15 change of pore-water pressure with depth*

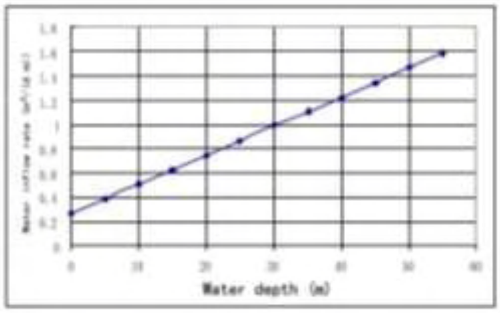


Fig. 16 changes of water burst yield with depth

Vector diagram of water seepage calculated is shown in Fig.13. It can be seen that the flow direction basically is perpendicular to the tunnel periphery. So it can be considered that the direction of the hydraulic gradient is perpendicular to the tunnel contour in the hole week. The pore water pressures (p) at the points 1, 2 and 3—1m away from the vault, haunch and arch bottom, can be calculated. Constant hydraulic gradient are uniformly distributed in the range of 1m. So to the points 1, 2, and 3, the seepage force per unit volume of soil is $j_1 = p_1 + 1$ , $j_2 = p_2$ , $j_3 = p_3 - 1$ respectively. It can be known from Fig. 15 that, pore water pressures of the 3 points will increase with depth. Increasing speed can be seen from the slopes of the three curves: $v_1 > v_2 > v_3$ . The quantity of gushing water with depth is shown in Fig. 16. It can be seen that with the increasing of the depth, the quantity of gushing water increases linearly.

## 7 CONCLUSION

When using drill and blast method for construction of underground works, treatment and prevention measures for groundwater have a major impact on the construction and operation safety, cost of underground project. The groundwater will have greater influence on the subsea tunnel. If not properly handled, it will cause a sudden inrush of water, mud and other safety incidents, such severely affecting safety and investment of tunnel construction, safety of operation of the tunnel and life safety of the construction personnel.

Due to the variability of the formation and the diversity of the practical projects, the accuracy of the current advanced geological prediction of tunnel is relatively low. We can't have an effectively forecasting and prediction of geological conditions and groundwater in front of the tunnel work face. Because of advanced pre-grouting technology has a poor effect on water blocking results and low construction efficiency, construction time and cost are long and high, respectively. At present, China's waterproof and drainage system is mainly not maintained, and small gravels in groundwater block the drainage system easily which lead to shorten life cycle of drainage system. With the practice of Jiaozhou Bay tunnel in Qingdao, we can conclude that for subsea tunnels of cross section area of 60-150 $m^2$, the allowable water inflow standard is 0.2 $m^3/m{\cdot}d$. We decide grouting program by water discharge which were predicted by using advance boreholes in the construction. We can seal rock fracture, change seepage orientation of surrounding rock and control leaking water amount in tunnel. Between initial support and secondary lining the waterproofing board is installed and the drainage cost during the period of operation is significantly reduced.

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# THE ACADEMY OF TUNNELING - AN EXPERT PROGRAM FOR IMPROVING FUTURE TUNNELLING

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## ABSTRACT

*As one of the major tunnelling countries in the world, the Norwegian tunnel community is continuously working to improve methods, management and hence the tunnel product itself. The Norwegian Public Road Administration and the Norwegian Public Rail Administration as the main owners of infra structure tunnels are responsible for approx. 1800 road-and rail tunnels. Over the recent decades, these owners have experienced to a greater extent that long term ownership implies a broad understanding of the tunnel structure in order to meet modern requirements for operation and maintenance. A growing concern for operation cost for modern tunnels, a better understanding of life cycle investments and technical challenge related to rock support and water and frost insulation has led to several processes to improve methods, technical matters and management. One such tool, introduced by the Norwegian Road Authorities and now offered on a regular basis in cooperation with the Rail Authorities is a training program called "The Academy of Tunnelling."*

*This is an expert program both for tunnel managers and tunnelling engineers all specialized in different fields of tunnelling. In order to improve the tunnel as a product, the idea is based on better interdisciplinary understanding between the many specialized fields of tunnelling and the need to work in closer relation with all experts and exchange experience on a regular basis and also recognize that there is a need for continuous education and learning. The program enroles approx. 30 students a year. The students are carefully chosen on behalf of their competance and background, their position, specialized fields as geology or electro, their geographical responsibilities, their sex and age. The aim is to put together a team of tunnel experts that may be able to cover the whole span of challenges within the field of tunnelling.*

## BACKGROUND

A modern tunnel is a complex construction which involves a large span of knowledge and technologies (fig.1). In order to meet international standards and modern demands, knowledge and experience must be shared and recognized by the tunnel planners, constructors and maintenance staff. We must meet future demands for more cost-effective solutions regarding maintenance and operations. As an example, in the city of Oslo, tunnels amount to 3% of the total road length, but they need 30 % of the maintenance budget. We admit that each field has traditions for thinking and working too specialized and too narrow. Each field is continuously getting more complex, and there is a limit to how much one expert can cover. "There is an increased need for professionals who have an overall view" Petter Eiken , CEO , Skanska.

*Figure 10 Modern tunnelling*

## THE ACADEMY OF TUNNELING

On such a background it is believed that improvements for the business are not necessarily related to improving knowledge or technical competence, but rather to improve the way experts cooperate and work together to solve their challenges. The Academy of tunnelling acknowledges this matter and emphasise multidisciplinary or interdisciplinary understanding and involvement of tunnel experts from early planning, through construction to operation, maintenance and reinvestment (fig.2). Regarding the latter, it must be emphazised that maintenance and repair is a field of forgetfulness. There is a growing concern that this knowledge and experience must be taken into account much earlier, which means both in the planning and construction phase. Knowledge and experience related to maintenance and repair must be taken into account by the planners and constructors in order to optimize LCC.

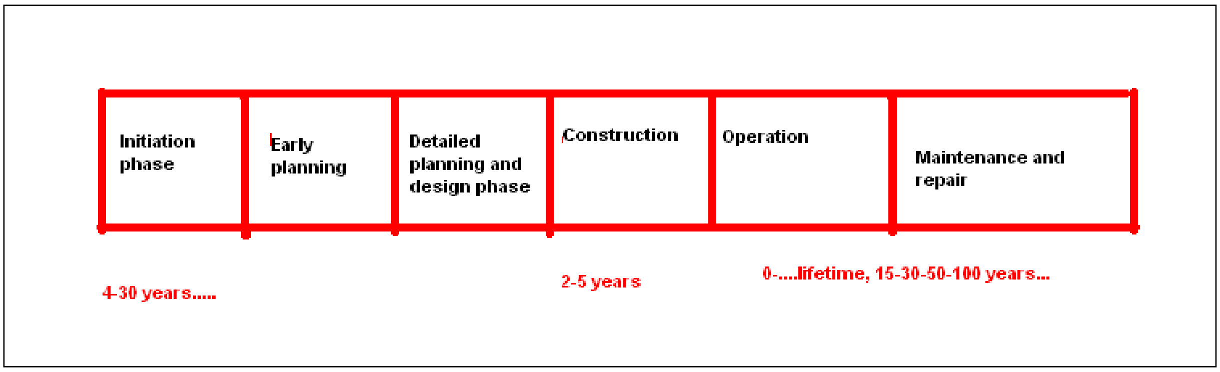


*Figure 11 Different stages in a project life*

The program is now offered on a regular basis after a successfully completed 2 year pilot program, unique of its kind. Maintenance and operation are given high priority, but also planning and construction. Important subjects, like geology and electro technical challenges, are emphasized. Key elements in this training program are to focus on competence transfer from one part of the field to another in order to improve the product. The improvement in result is assumed to be related to improvement in work methods rather than improved technology. By involvement of different knowledges, we believe technology also will gain from such work.

*Figure 12 Tunnelling experts*

So far 120 tunnelling experts have been through the program (fig.3). The lectures are held by employees at the Norwegian Road and Rail Authorities, consultants and contractors, all experts within their field.

**THE EXTENT OF THE PROGRAM AND VOCATIONAL TRAINING**

The expert program consists of five meetings throughout the year with a defined skill theme at every meeting:

- Introduction and overall understanding
- Maintenance and operation
- Planning
- Construction / reinvestment
- Safety, materials, specialized fields

In every meeting technical visits constitute a part of the teaching agenda. This include visiting ongoing tunnel projects, tunnel operators, tunnel control centers and major coming projects which are in the planning stage. The students are prepared to ask questions to the technical guides on every site. Site visits are organized in small groups depending on number of tour guides. The main purpose is to provide a greater closeness between student and technical guide for a more person to person dialogue. In this way, we also believe technical visits have provided meaningful variations in the teaching program.

In addition to lectures and an exam, there are project reports to be prepared between the classes. The intensions of the project reports are multipurpose and shall be done in groups of four to six in order to cooperate and build network. The reports shall provide the students with a deeper understanding of the different subjects, practical training in multidisciplinary work, find modern methods to work effectively together with colleagues all over the country, and give an opportunity to improve presentations skills. There are three project report tasks given during the program, and they have all to be approved in order to take final exam. The program terminates with a final exam prepared by the Norwegian University of Science and Technology. An improved exam can be a part of an experienced based master degree, reviewed by the master program at the Norwegian Technical University.

*Figure 13 Exchange of experiences in a practical way*

The program is now offered on a yearly basis in cooperation with the Norwegian Rail Authorities and is recently also opened for private companies.

## CONCLUSIONS

A holistic approach, interdisciplinary communication and networking have always been the main focus for the Academy of Tunnelling, it is the mission and ideology of the Academy. During the project it has become evident that those core values are very timely. There are more and more talk about interaction and venues for the exchange of experience and knowledge sharing as useful tools. Traditionally, the industry has focused on contractual tools and new models, software tools and technology for increasing the value. Recently however, the interpersonal factors, interaction and learning have come into focus (fig 4). And that is the core values of the Academy of Tunnelling.

# MODERN ROAD TUNNELS
# THE FUTURE OF LONG AND DEEP TUNNELLING - IS THERE ANY LIMIT?

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## ABSTRACT

*The NPRA, Norwegian Public Road Administration, is the owner of in total nearly one thousand tunnels. During the last 30 years there has been a huge increase in tunnels build caused by a demand for having roads to remote cities and villages which in previous times had to relate on ship or sea transportation. Also an increasing demand for safer transport due to rock and snow avalanches and increasingly demands from local authorities solving traffic challenges in our cities, have been important reasons for such a heavy tunnelling activity.*

*In Norway tunnelling has challenges differing from most other tunnelling countries. The most important differences are:*

*-tunnels to remote areas means low traffic*
*-mountainous areas and high altitude means frost*
*-subsea tunnelling means water leakages*

*NPRA has recently finished a 4-year R&D project for developing a holistic strategy for tunnels, from planning, designing, constructing, operation, maintenance and at last upgrading of existing tunnels. A national tunnel school has been started with main focus on a broader understanding of competence connected to different tunnel technical knowledge. A new technical system for documentation of geology and rock support has been developed. An overall system for administration of all the documentation from the constructional phase providing the maintenance phase has been developed. The project has developed strategies connected to safety issues, risk analysis, fire safety, life cycle costs and service lifetime for tunnels in a holistic perspective.*
*One of those strategies implies the challenge of long and deep tunnels – where are the limits?*

## THE CURRENT SITUATION

Norway builds and plans more tunnels than many other countries in the world, and many of our tunnels are also longer than those in most other countries. This means that our knowledge and experience are stretched beyond what could be called generally accepted professional practice. What was considered daring, perhaps impossible, in Norway a few years ago, is being planned and built today. The Directorate of Public Roads is constantly becoming involved in projects where evaluations must be based on sound professional discretion. In other words, frontiers are moved.

In a table for safety equipment/design in Manual 021 Road Tunnels a limit of 10 km is set for tunnel length. Where longer tunnels are planned, special assessments must be made. This limit is somewhat randomly chosen in that there is no documentation to show that it represents a limit with respect to danger or technical issues. The view in the 1990s was that tunnels ought not to be longer than this. Since then, two tunnels have been opened that are longer, one of which is significantly longer than 10 km. The purpose of this limit at the time was that all such tunnel - projects should be specially evaluated, without being precise about the determination of what such evaluation should consist of or how it should be carried out.

The same manual states that the principle for evacuation is based on self-evacuation; in other words, that road users should get out of the tunnel either on foot or by means of their own vehicles.

In recent years, there have been requirements for tunnel safety that have been made more stringent, both in Europe and here in Norway. In the EU, a separate safety directive has been introduced for road tunnels on the Trans-European Road Network (TEN-T). It has also been implemented in Norway by means of separate regulations under the Public Roads Act. This provides clear guidelines/requirements for gradients, design and equipment, but says little or nothing about length and depth.

## GRADIENTS IN ROAD TUNNELS

In the "Tunnel Safety Regulations", the gradient in road tunnels is limited to 5% for most tunnels. When building subsea road tunnels, a supplementary provision is used that says the gradient may be increased where it is geographically impossible to use a lower gradient. In Norway, the possibility of steeper tunnels has recently been restricted by setting a maximum level at 7%. Previously, subsea tunnels have been built with gradients of up to 12%.

The choice of gradient is important in that it often determines the length of the tunnel. A steep gradient results in increased fuel consumption and more pollution, especially from heavy vehicles. Steeper gradients also result in greater evacuation problems, especially for children and the elderly. The advantage is that the tunnels are shorter. Based on the evaluations that have been made, especially of subsea road tunnels, it would seem correct, all in all, to limit the gradient to 7% for subsea road tunnels and 5% for other tunnels.

## VENTILATION – FIRE VENTILATION

In order to be able to control the spread of smoke from a fire in a tunnel with unidirectional traffic, longitudinal fire ventilation of min. 3 m/s is required, and up to 10 m/s is required for "large" fires. There are also several other factors that affect ventilation requirements, for example the gradient and the climate. Adequate fire ventilation will then prevent the spread of smoke to the upstream side of the fire, which is considered to be the safe side in such a tunnel. Downstream from the fire, traffic should in principle be able to drive unimpeded out of the tunnel and to safety. If there is a traffic stoppage downstream of the fire, however, road users there will be in serious danger.

In bidirectional tunnels, smoke from a fire will always have to be forcibly driven by adequate fire ventilation.  There will always be some people on the downstream side of the fire and in the direction of the ventilation. This is a great challenge in these tunnels and a thorough knowledge of the traffic situation in the tunnel during an incident is required before forcibly controlling the fire ventilation direction. In principle, the fire ventilation direction is selected to enable rescue parties to enter the tunnel with the ventilation direction at their back.

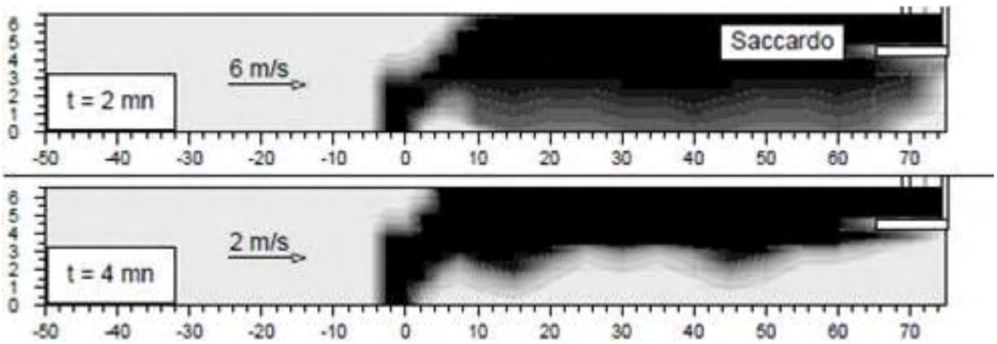


*Figure 2 Spread of smoke in tunnels with ventilation of 2 and 6 m/s, over time.*

The above diagram shows that with a ventilation speed of 6 m/s, 70 m of the tunnel space downstream of a fire is filled with smoke within 2 minutes.

Both control of the fire ventilation and control of smoke spreading will meet increasing challenges both in the longitudinal direction and especially in combination with long, steep gradients in tunnels where ventilation is entirely longitudinal, , as it is in Norwegian tunnels. The alternative will be semi/fully-transverse ventilation where air is directed towards one or more shafts for venting polluted air. For effective smoke control, the air is extracted to ventilation ducts above the false ceiling of the tunnel where both used air from the tunnel and fresh air to the tunnel are controlled.

This is a usual method in Central European tunnels. However, it requires completely different cross-sectional areas and profiles than are usual in our tunnels.

## TRAFFIC SAFETY VS UPHILL/DOWNHILL GRADIENTS

In the work of the NPRA's programme "Modern road tunnels", the question has been raised of what uphill/downhill gradients can be permitted for subsea tunnels with traffic safety in mind.  A study is also currently in progress on remedial measures that would allow large vehicles to be accepted in the Oslofjord Tunnel, a tunnel with high traffic density.

At least four factors must be assessed in order to be able to determine whether the total impact of driving in subsea road tunnels lies within an acceptable safety level to be able to permit unrestricted traffic.

These factors are:
- gradient of the uphill or downhill slope
- the driving distance with an uphill or downhill slope
- the vehicle's braking capacity and
- the weight of the vehicles

Uphill or downhill gradient

The EU allows uphill and downhill gradients of 5% in the TEN-T. The same applies for national road tunnels above sea level in Norway. Due to Norway's particular geography with very deep fjords, the EU has accepted that Norway may have other rules for uphill and downhill gradients in subsea tunnels.

The road length for uphill and downhill gradients

Equal in importance to the gradient of a subsea road tunnel is the length of road with an uphill or downhill gradient. For example, there would be no problem with a downhill gradient of 7% if the height difference to be covered is limited to e.g. 50 metres. This entails a length of road with the gradient limited to about 700 metres. On the other hand, a downhill gradient of 7% with a height difference of 600 metres would mean a downhill stretch of about 8 600 metres, which would not be acceptable.

The reason that this would not be acceptable from a road safety point of view is the risk that braking systems would not tolerate the strain over such a long distance. The danger of heat generation and fire would be considerable.

Vehicle weight

In Norway, vehicles of up to 50 tonnes weight for vehicle and freight combined are generally permitted. Within the EU, the corresponding authorised weight is up to 40 tonnes. In other words, Norway permits a 25% higher maximum weight than similar vehicles that are used in the EU.

In addition, both the EU and Norway have supplementary provisions. The maximum authorised weight of a vehicle in the EU is 44 tonnes with the use of 44-foot containers, while in Norway it is 60 tonnes for modular vehicle combinations.

Vehicle braking systems

There is a direct relationship between a vehicle weight and the strain on the braking system. The heavier the vehicle, the greater the strain the brakes must withstand.

To improve braking systems on heavy vehicles, it is normal in Norway to equip the vehicle with so-called retarders. This is more usual in Norway than in the EU. But both Norwegian vehicles and foreign vehicles drive on the same road network and are therefore exposed to the same strains.

The risk associated with Norwegian use of steep tunnels in Norway

Norway has the most subsea tunnels of any country in the world, approximately 33 in all. Because of the deep fjords and in order to reduce tunnel lengths, tunnels in Norway are built with steep uphill and downhill gradients.

In addition to this, Norway has approved a far greater weight for heavy vehicles than has been accepted in the EU. With a greater proportion of vehicles with retarders than elsewhere in Europe, Norway has a certain advantage. But the advantage is of no benefit to those vehicles that do not have retarders, or the vehicles that are exposed to danger from vehicles that do not have retarders.

Recommendation

There is a need to document what combinations of uphill and downhill gradient, stretches with an uphill or downhill gradient, vehicle weight and braking system provide solutions that are acceptable in terms of traffic safety.

Until adequate documentation is available, subsea tunnels with gradients greater than 5% ought not to be built.

**PERCEIVED RISK WHEN DRIVING IN LONG AND STEEP TUNNELS**

Road tunnels present special traffic safety challenges under normal circumstances and special challenges under critical conditions. Road tunnels differ from open air roads in several ways: they have no roadside activity, have good road conditions during winter, have the same light conditions year round except when the tunnel entrance is in bright sunlight; a certain proportion of road users experience discomfort, it is difficult to judge uphill and downhill gradients, exhaust emissions build up and there are difficult conditions when accidents occur.

Road tunnels are usually at least as safe as or safer than corresponding open air road stretches without intersections, exits, and pedestrian and bicycle traffic. Nonetheless, road tunnels deserve special attention with respect to traffic safety, both because they present special safety challenges under normal conditions and not least because they present special challenges in critical situations.
Road tunnels offer special safety challenges under normal conditions, because they have a relatively high accident risk in the entrance zones, because in contrast to open air roads they lack frames of reference, because a significant share of the population experiences fear and discomfort associated with travelling in them and because they can give road users an experience of monotony and boredom which can lead to reduced attention.

The special challenges that road tunnels present in critical situations are related to traffic stops and build-up, the fact that about half of road users do not know what they should do in critical situations such as fire, and that many remain passive in situations that demand evacuation. In general it appears that Norwegian road users would benefit from training and/or information that explain how they should act in critical situations in road tunnels.

As of today, there are no well founded limits for length and depth or explicit criteria for such limits for road tunnels. The question of how long and deep road tunnels can be is first and foremost a question of what user concerns must be taken into consideration. Four length- and depth-limiting factors may be significant: 1) fear and discomfort, 2) monotony, reduced attention and accident risk, 3) gradient in deep tunnels that creates speed differences between light and heavy vehicles and 4) evacuation times for self-evacuation.

More research is needed on the following questions related to the first two categories of length- and depth-limiting factors: To what extent and to what degree do fear and discomfort increase with the length and depth of the road tunnel? How long or deep does the road tunnel need to be before the share of the population that feels discomfort to choose not to use the tunnel becomes too high? Do fear/discomfort with respect to the road tunnel's length decrease over time in long tunnels or do they remain the same? Are there measures that can reduce fear and discomfort? Do road users who would rather choose a detour than drive through road tunnels choose not to use tunnels at all, irrespective of length or depth, or do these road users only choose not to use tunnels of a certain length and depth? How does the risk of falling asleep increase with tunnel length, and what measures can be implemented to reduce the risk of falling asleep? How long must a tunnel be before monotony becomes dangerous in traffic due to the reduction in attention level?

With regard to how evacuation time limits length and depth, it is assumed that the evacuation situation in cases of fire can be much more critical in single-tube road tunnels than long, dual-tube road tunnels. In case of fire in dual-tube road tunnels, road users can evacuate to the adjacent tunnel tube via emergency exits, thus avoiding smoke. Dual tubes are required when the road tunnel is longer than 10 km and AADT > 12 000. This means that single-tube tunnels can be up to 10 km long. Given that the design walking speed may often be slower than the spread of smoke, we ought to ask whether the potential for a disaster may be greater in long single-tube road tunnels less than 10 km long with bidirectional traffic than in longer, dual-tube road tunnels with unidirectional traffic and many clearly marked emergency exits. In dual-tube road tunnels with many clearly marked emergency exits, those involved will have considerably better chances of avoiding build-up of heat and smoke, and of surviving. Given that those involved avoid the smoke, the question of how evacuation time may have a stronger length and depth-limiting effect may become more a question of how far one should require that involved evacuate themselves on foot in emergency situations. Setting a maximum evacuation time is therefore largely an ethical question.

## FIRE SAFETY ASSESSMENT

Fire safety in Norwegian tunnels is not designed to tolerate an extreme fire, whatever the definition of an extreme fire might be, 200 MW–300 MW? The focus of safety efforts has been directed towards personal safety and less towards structural safety. The structure itself should remain intact so that evacuation can be carried out safely. Requirements for the structure are such that it should not contribute to the further development of a fire that occurs. Ever more robust fire safety methods and equipment are being developed, and there is an ever-increasing pressure to install both more equipment and more technically advanced equipment. All use of safety equipment and the procedures for when and how the equipment is to be used, must be based on it functioning as expected when the need arises. In practice, this means that during the first 15-30 minutes after an incident/fire starts, everything should take place correctly and in accordance with fixed procedures.

Is it actually realistic to believe that those who find themselves involved in such an incident will react and conduct themselves as we presume they will during this first and most critical phase of an incident? The answer is no – this is not how things are in reality, as has been well documented by various incidents and training. Experience shows that many road users hesitated to start an evacuation. Someone must take the initiative. Fires, for example, do not always look dangerous during the initial phase and the question is how to address this? There are two scenarios for "managing" the incident further:

- self-rescue
- active rescue

There are no other alternatives. Self-rescue will have to apply completely for one-tube tunnels and a combination of self-rescue and active rescue is realistic for two-tube tunnels. But during the first and most important part of the incident, self-rescue must apply in two-tube tunnels. No one comes to the rescue during this phase. No matter what tunnels this relates to and no matter the amount and type of technically advanced equipment, the most important thing will always be that those involved in such an incident manage to perceive what is about to happen; to manage to perceive that something dangerous is about to occur and that this requires action. From that moment, there will be a clear relationship between the fact that an incident has been perceived and the pre-conditions that are built into a safety concept of what one should do when the incident occurs. This applies both for self-rescue and if a rescue team is expected to arrive. And in such a scenario, the challenges will increase with both increasing length and greater depth of the tunnels. Increasing depth will be particularly demanding in a self-rescue scenario. More technically advanced equipment is of little help when the distances are long and the gradients steep. And the many challenges of the physically disabled in such an incident haven't even been talk about. Universal design is a challenge in itself for self-rescue.

The figure below illustrates the various phases that a tunnel owner must relate to in an incident scenario and it also says something about the time perspective, both individually and as connected in phases. For those caught up in the incident, it is "only" important to understand what situation is about to occur or has occurred and "relate" to evacuation.

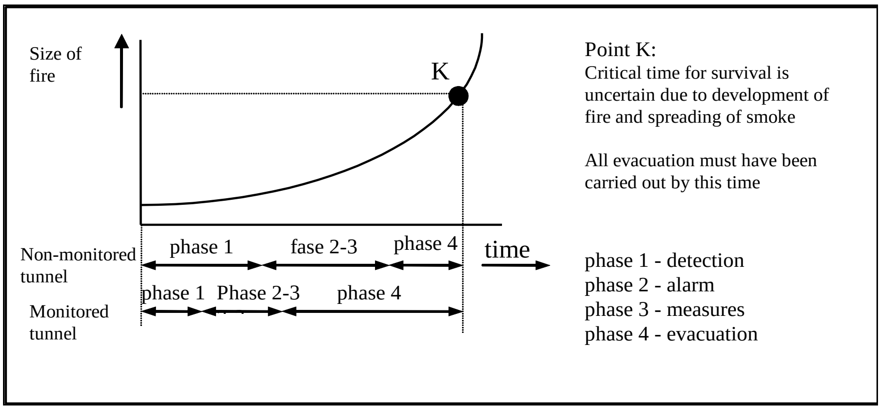


*Figure 4 Phases in the incident scenario*

## TECHNICAL CHALLENGES REPRESENTED BY HEAVY VEHICLES

The technical aspects of vehicles require more attention, especially heat generation in brakes during downhill driving and increased exhaust emissions during uphill driving combined with possible engine overheating. It is important that the air in long tunnels is of a quality that enables vehicle engines to function close to optimally.

On the one hand, vehicles that drive in tunnels with steep uphill/downhill gradients and are loaded to the maximum authorised weight are a safety challenge. On the other hand such considerations must be included as a design assumption based on technical aspects of vehicles that have not taken a prominent place in the risk analyses performed for these types of tunnels.

The regulations concerning minimum safety requirements for certain road tunnels, states the following with respect to tunnel geometry, “more than 5% gradient in the longitudinal direction shall not be permitted in new tunnels unless no other solution is geographically possible” and “In tunnels with gradients of more than 3%, extra and/or stronger measures shall be implemented to improve safety on the basis of a risk analysis”.

Vehicles for use on or off public roads must satisfy a number of technical requirements, including extensive requirements with respect to the vehicle's braking system and performance on long downhill slopes. Commission Directive 98/12/EC regulates these requirements. The technical requirements for vehicles are nearly the same throughout Europe, and the requirements must be included in an assessment of what downhill/uphill gradient a public road may have so that vehicles and infrastructure can be used in a safe and environmentally-friendly way.

Trailers with authorised weights between 750 kg and 3 500 kg

Commission Directive 98/12/EC states that all trailers with an authorised weight of between 750 kg and 3 500 kg shall be equipped with a service braking system that is a continuous, semi-continuous or inertia (overrun) brake type. The inertia braking system is the type most widely used in almost all campers and trailers in the weight class mentioned. The directive gives specific requirements for when the inertia braking system should begin to work (reaction time). Simplified calculations show that the service brakes will begin to brake the trailer when the vehicle and the trailer drive down a gradient of between 3 and 5%. It is assumed that the trailer is loaded to the authorised weight and that the inertia braking system functions in accordance with the directive. The driver of the vehicle cannot prevent the service brakes on the trailer from beginning to brake at a gradient greater than 5%.

Motor vehicles approved in combination with trailers

Commission Directive 98/12/EC also specifies requirements for testing vehicle performance on long downhill gradients. The laden vehicle shall be tested in such a way that the power applied corresponds to that recorded in the same period for a vehicle with load that with an average speed of 30 km/h drives a 7% downhill gradient for a distance of 6 km. During testing, service, emergency and parking brake systems shall not be used. The gear chosen shall be such that the speed of the engine does not exceed the maximum value prescribed by the manufacturer. An integrated retarder may be used, providing that it is properly adjusted so that the service brake system is not affected. (If the vehicle's maximum mass exceeds 26 000 kg, the test mass shall be limited to 26 000 kg.)

Simplified calculations show that vehicle combinations loaded to the maximum authorised axle load of 44 000 kg (in accordance with Directive 96/53/EC, most recently amended by Directive 2002/7/EC), have a speed of 80 km/h in a downhill gradient of 5%. The vehicle combination will manage to maintain an almost constant speed per 1 000 metres driven, with a deviation in speed of less than 3%. This is achieved only by utilising the gears, engine brake and possibly retarder. If a corresponding vehicle combination is used at the same speed, but is now loaded to the maximum authorised weight in accordance with national weights, i.e. 50 000 kg, and drives down a gradient of 7%, the speed will increase by approx. 40% per 1 000 metres driven. To maintain a constant speed down this 7% slope, braking must be carried out. . The braking can be carried out using available retarder or service brakes. Currently there are no requirements that vehicles have retarders, but most Norwegian lorries have them mounted as extra equipment. Since 2005, the Norwegian Public Roads Administration has carried out technical inspections of heavy vehicles. Among other things, these inspections examine the condition of the brakes. The results from 2009 show that 22% of foreign-registered heavy vehicles and 15% of Norwegian-registered heavy vehicles have faulty brakes. Faulty brakes will affect heat generation in that other axles must do a greater amount of the braking work to achieve the same deceleration when the service brake is applied.

Increased fuel consumption in heavy vehicles

Vehicle combinations loaded to the authorised weight on uphill gradients require both the right quality and a large amount of air to obtain adequate power and cooling. The large power drain from the lorry's engine will make heavy demands on the vehicle's cooling system and the exhaust system's ability to purify the exhaust emissions. This is because the vehicles will have relatively little speed uphill and therefore little air flow around the engine and exhaust system for cooling purposes. Fuel consumption will also increase significantly, and in the study *Assessment and Reliability of Transport Emission Models and Inventory Systems* (ARTEMIS), tests have been carried out that show that if a heavy vehicle of only 20 000 kg drives up a 6% gradient (50 km/h), the consumption will increase by approx. 420-445% compared to driving on a horizontal road (80 km/h). The waste gasses NOx, Pm and HC will increase somewhat but not by the same percentage as consumption. There is a common view that these data must be included as important parameters in the assessments of the steady state situation of tunnel ventilation systems.

Recommendation
Based on these technical assessments, it is proposed that tunnels should be built with a maximum gradient of 5%. This is also consistent with Council Directive 2004/54/EC which concerns minimum safety requirements for tunnels in the Trans-European Road Network (the Tunnel Directive).

**CONSTRUCTION CHALLANGES**

Current knowledge within Norwegian tunnel construction places almost no limits on how long a tunnel can be. Today's longest tunnel is 24.5 km long. Technically, the construction process is without problems, and the rock reinforcement process is the same irrespective of length. During the actual excavation phase, ventilation conditions to ensure a satisfactory working environment will be a prerequisite. However, this will not strongly influence how long the tunnel can be. The consequences for any operation to evacuate construction personnel will vary somewhat depending on the distance to the open air. During this phase there is only one exit in case of evacuation. There might be a limit somewhere, but it can hardly be considered a governing factor either.
On the question of how deep a tunnel can be, however, there will quickly be more technical engineering challenges where there is at least greater uncertainty associated with setting limits. Today, there are plans for subsea tunnels of -650 m at the investigation stage. And there will certainly be more, and at deeper levels. The technical challenges purely with regard to construction will primarily be related to water leakage and challenges related to stopping such leakages with injection techniques. It will be solvable, but is challenging and, not least, costly.

**GROUND SURVEYS AT GREAT DEPHTS**

Preliminary geological investigations should provide the basis for establishing feasibility, reinforcement needs and costs related to tunnel projects. The investigations shall be carried out in accordance with the requirements described in applicable regulations. It is crucial to have the best possible map basis both on land and at sea. This is especially important when planning subsea tunnels at great sea depths.

Geological investigations should include detailed geological and engineering geological mapping. The mapping is to be supplemented with geotechnical, hydro-geological and geophysical investigations.

The rock mass should be classified according to the Q-system (rock mass classification system) with the help of core drilling, field mapping and geophysical investigations, so that a prognosis of the reinforcement necessary for stability can be made.

The question related to the ground investigations will be to find the correct relationship between the refraction seismic velocity in the rock mass and the rock mass quality because of the difficulty in taking realistic samples at great depths of water. It is difficult to achieve adequate core drilling at the locations concerned. It is possible to use a drilling ship, but the costs are high so there must be absolute certainty about drilling at the correct location.

It appears that in due course refraction seismic tomography might be a method that at least provides information more about zone width and alignment with respect to the tunnel alignment. However, this method is still at the experimental stage.

## SINGLE-TUBE TUNNELS WITH BIDIRECTIONAL TRAFFIC

It is for these tunnels that a length limit may be most relevant. It is also here that the consequences could be great, but normally the traffic will be relatively light. It is also important to be aware that while the probability that incidents may occur primarily depends on the number of vehicles, the consequences will be dependent on how many people are in the tunnel at any given time. The basis for assessments ought therefore to be the number of people who may be in the tunnel simultaneously.

The problem is that in a worst case scenario, these people must escape the whole length of the tunnel or parts of it. Walking steadily without a rise, a normal person will be able to manage a speed of 5-6 km an hour. Given a rise of 7%, the same person will be able to walk half of this distance in an hour. These are "normal figures" and will vary considerably depending on the age and physical condition of the road users to be evacuated.

Even in a small fire (passenger car), the smoke will spread as fast as the walking speed of a "normal person" on a flat road and with a 7% rise, the smoke will spread twice as fast as the walking speed. With a 5% rise, which is the maximum gradient according to current guidelines, the spread of smoke from a passenger car will be faster than "normal" walking speed.

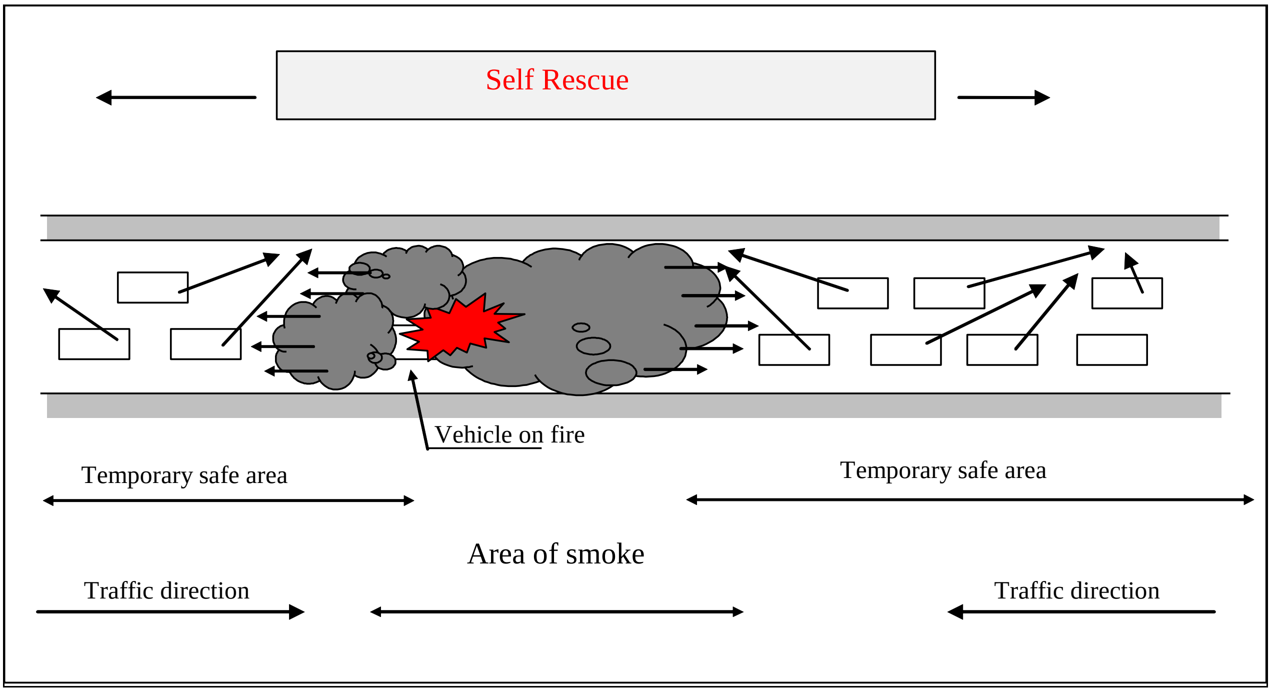


*Figure 5 Emergency egress in single-tube tunnels*

In other words, a "normal person" has a walking speed that is equal to the speed of the spread of smoke. If time is included to "understand" the situation, assess what should be done and possibly which way to go etc., smoke will rapidly catch up and surround the person. At the same time, the person must know that he or she is in a very critical situation a long way underground and with no knowledge of the extent or the quality of the rescue effort that might arrive. The sense of distance out to the open air (and rescue) will vary considerably, but everyone will perceive that the longer the distance the greater the problem. How far is then far enough will perhaps be decided by how far a person is obliged to walk before external rescue services are expected to arrive. Both the time perspective of the detection phase and the alarm phase will play a part here, as will the distance of the external rescue service from the incident site. In Norway, this time can be very long.

Today's regulations require a dual tube when a tunnel is over 10 km long and the AADT > 1 2000, and no distinction is made between conventional tunnels and subsea tunnels. A risk analysis of local circumstances and conditions can change this. Based on the self-evacuation principle and bearing in mind the usual walking speed of a "normal person" and not least the speed at which the smoke spreads in subsea tunnels with gradients up to 7%, the following questions ought to be raised:

## TWO-TUBE TUNNELS WITH UNIDIRECTIONAL TRAFFIC

These tunnels are usually assumed to have high safety for each individual road user and special safety arrangements for evacuation purposes are not usually required. However, there may be large numbers of road users in the tunnels simultaneously and panic can easily occur. This is a great challenge in itself, and an evacuation must take place through cross-connections to the other tube. The distance between such cross-connections, pursuant to current regulations, is 500 m or 250 m depending on the tunnel class. An assessment must also be made of whether to enable rescue personnel to attack the fire through this passage. Such an assessment was done for the Bjørvika/Ekeberg tunnels. It is recognised that the more stationary traffic that can occur in a tunnel, the greater the challenge for an effective evacuation

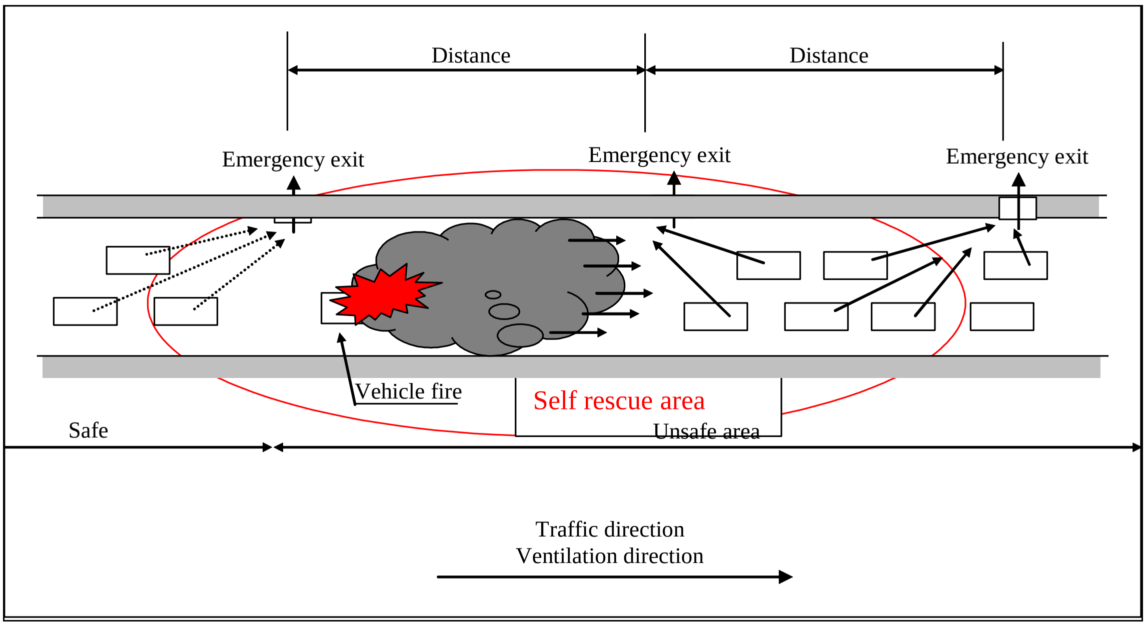


*Figure 6 Emergency egress in two-tube tunnels*

In a 30 km long, level tunnel, the "worst case" will be evacuation 15 km from the tunnel entrance. In a self-evacuation scenario (which in any case will be the governing scenario during the start phase of an incident) this means that a "normal person" must find out what has happened, assess the situation so that he or she does not end up on the wrong side of the fire, find out which way to go and find the nearest emergency exit (which may be 200 m from where he or she is standing). It could well be that the time it takes to find the emergency exit to the other tunnel could be about 15 min. In the worst possible case, it could take two hours to get out of the tunnel if self-evacuation provided necessary. In reality, external rescue services will meet up somewhere along the way from the emergency exit at the incident location and 15 km out. But the reality will also be that those who are involved in an incident of this type will not stay somewhere along this stretch and wait for an external rescue service that they know nothing about; either when it will come or even if it will come. They will attempt to get out of the tunnel on their own.

There is also a not insignificant uncertainty linked to the assumption that the smoke will not spread to the second tunnel tube. In a self-evacuation situation (which realistically will exist until people access the other tube), the people concerned will naturally enough have insufficient or no knowledge about either ventilation or airflow conditions in tunnels.

A tunnel that is 30 km long and level is perceived as long, and for most people it feels even longer to actually drive through. Self-evacuation from such a long tunnel, this entails in the worst case having to walk for one or two hours, assuming that no external rescue service comes. And all the while under pressure to get away from the incident and under stress because one does not know what is happening. Added to this, we are talking about "normal people"; none of the others are considered (handicapped, old etc.). Therefore there is every reason to assume that this sort of time perspective for self-rescue has been stretched to the absolute maximum and that the objective owner responsibility for road user safety in a closed tunnel space has also been stretched far enough.

In tunnels with gradients, conditions will be even worse for those evacuating. Walking speed is almost halved with a 5% uphill gradient compared with a level road. And the speed at which smoke spreads increases greatly due to the chimney effect caused by the gradient, and of course depends on the size of the fire.

A subsea dual-tube tunnel that is 15 km long implies in the worst case an evacuation stretch of 7.5 km, with a 5-7% uphill gradient. A "normal person" would take one to two hours to get out of such a tunnel unaided. Even if external rescue services arrive, such a long subsea tunnel represents a significant challenge with respect to safety. Perhaps the perceived safety (or rather lack thereof) will be the true limit for how deep and long one goes. Ought we not we ask ourselves the following question:

## CONCLUSION: WHEN IS LONG LONG ENOUGH AND DEEP DEEP ENOUGH?

How long or how deep a tunnel can be must be determined by the safety challenges for road users. Driving heavy vehicles and evacuation in the case of an incident are examples of such challenges. An injury situation that requires rescue services becomes more difficult the more the limits for length and particularly depth are being stretched.

The fire in Oslofjord Tunnel during the summer of 2011 started with heat generation in the braking system of a heavy vehicle. The tunnel is 7.3 km long and has an uphill/downhill gradient of 7% over a stretch of 3.5 km. Due to the tunnel's geometrical design, which admittedly presents great challenges for heavy vehicles, the tunnel was closed until further notice for heavy vehicles with a weight > 7.5 tonnes. Currently, a study is being carried out regarding remedial measures that would allow the tunnel to be opened once again for heavy vehicles. This work will provide a valuable contribution to determining the limits that must be set for uphill/downhill gradients and appurtenant lengths of road tunnels. The tunnel was opened for heavy vehicles in February 2012.

To ensure that the total stress of driving in subsea road tunnels lies at an acceptable safety level with unrestricted traffic, the limits for the following factors must be assessed:

- downhill and uphill gradient
- driving distance with this downhill and uphill gradient

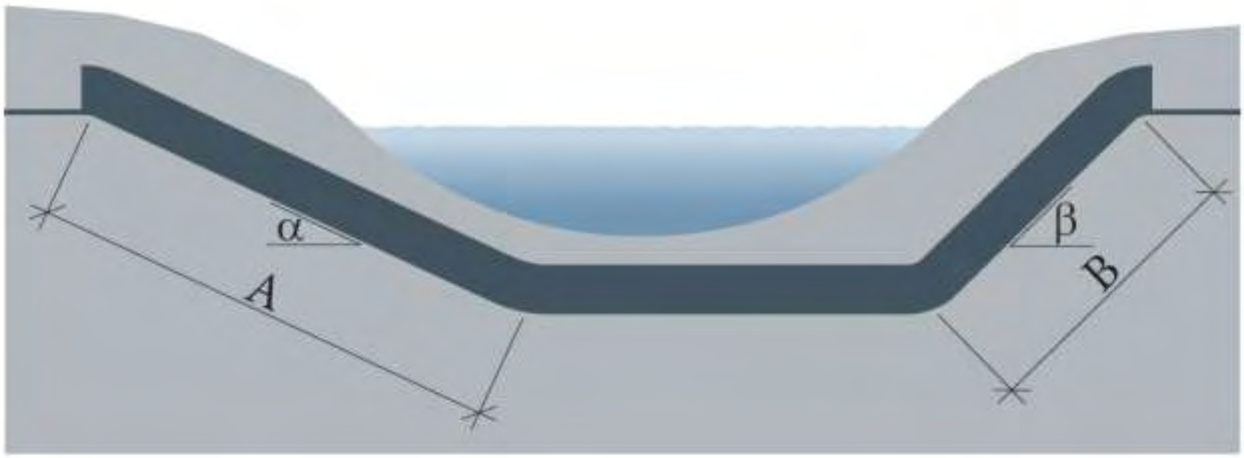


*Figure 7 Subsea tunnel: Tunnel tube*
*α and β : downhill or uphill gradient*
*A and B : driving distance with downhill or uphill gradient*

The combinations of uphill/downhill gradients and driving distances that provide acceptable safety solutions need to be documented.

Based on technical challenges affecting both large and small vehicles that drive on steep downhill and uphill slopes, it seems clear that tunnels should not be built with gradients of more than 5% unless the uphill/downhill distance is limited.

# INNOVATIVE DESIGN FOR SAIVAN DOUBLE-DECK BRIDGE WITH TWIN PC GIRDER IN MACAU

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## ABSTRACT

*The Saivan Bridge in Macau cross the bay is the first cable-stayed bridge with double-deck and twin PC girder in the world. This paper introduces how the design engineer conceived the bridge and realized harmonious combination between traffic capacity and architect arts in civil engineering design and achieve the objective that a complex project was simplified by innovative design by introducing implementation of the Saivan Bridge, of which multi traffic lane and heavy load are required under restraint marine construction condition. The main span layout of the cable-stayed bridge is 110m+180m+110m, The main girder are made up of twin prestressed concrete single box single cell structures carrying three carriageways and enclosing a typhoon refugee carriageway and a light-rail line each. The box girder is 6.1m deep, 14.5m wide. The main concrete pylon is an "M" shaped monolithic structure supporting the twin girders by four cable planes. The approach bridge was designed with span 60m of PC Box girder same size with the main span for connected the double deck traffic. So it is a example for bay bridge with all concrete structure for all-weather to opening.*

## 1. BACKGROUND

In order to ease the traffic congestion of the two existing bridges between Macao peninsula and Taipa island due to the increasing traffic volume, the government of Macao Special Administrative Region decided to invest for the third bridge. Internationally public bidding invitation was made in April 2002 for the project. Owing to the special geographic location and the potential technical challenge, there are 11 international joint ventures attracted to compete for the biding. After fierce competition and comprehensive selection, the author's (chief designer) proposal won out of the 23 proposals and became the final implemented proposal, viz., CTMB (China Railway Major Bridge Reconnaissance & Design Institute, China Railway Construction (Macau) Corporation, China Railway Major Bridge Engineering Co., Ltd) Joint Venture was granted the bid to implement the design-built contract of the project. What follows in the passage emphasizes to introduce the origin of the innovation design and its technical characteristics.

## 2. TRAFFIC CAPACITY REQUIREMENTS

The bridge is required to be designed to meet the traffic demand of 6 carriageways and dual light railways, at the same time two carriageways shall be taken into consideration for the cars to refuge from typhoon under emergent conditions, which would supply the other two bridges' want of such a function. The clearance requirement of the main navigation span of the bridge is

150m (net width)×28m (net height), which is required for the navigation of the 4000DWT seagoing ships.

## 3. CONCEPT DESIGN

It is an integrated challenge for the bridge engineers to conceive a cost-effective proposal with reasonable force condition, applicable functions, both technically and in creativity aspects, to meet the complex requirements including traffic function. The owner didn't put forward any additional condition about the bridge type and cost to leave room for creativity of the designers. The project design covers 3km including the 1825m main and approach bridges, 246m box culvert on both sides, 791m ramps and the attached fire fighting and lighting design.

### 3.1 Technical requirements

The following points are concluded to describe traffic function of the bridge:
a) multi-carriageways demand;
b) heavy load;
c) Strict requirements for rail traffic;
d) extra requirement for typhoon refuge;
e) large diameter water supply pipes;
f) constraint of construction condition.

### 3.2 General Concept and Technical Proposals

In order to solve the problems on above mentioned, the author proposed the corresponding measures in design:
a) A double-deck all prestressed concrete box girder carrying motorcar carriageways on the upper (top of the box) and light rail and refugee carriageways on the lower (inside the box) is proposed to meet the multi-carriageway and typhoon refuge requirements. It is shown in Fig.1.
b) A separated twin girder structure other than a huge box is conceived to make the force flow simple and clear, as a result, the girder slab and segment is reduced in thickness and weight,which help the construction be easier and the quality of the completed bridge can be ensured.
c) The space between the two main girders was taken good use by arranging the larger diameter water and cable pipes. Then the function sections of the bridges are separated in case any cause of interference from each other. The sectional details are shown in Fig. 2.

*Fig.1 Twin Girder of the Saivan Bridge*

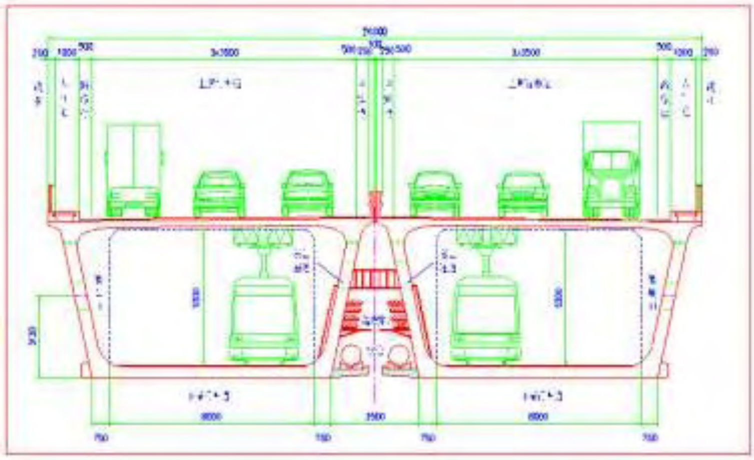

Fig.2 Twin Girder with PC Box

### 3.3 Cable stayed bridge

A cable-stayed bridge with a main span of 180m was proposed in the conception design to meet the navigation requirements. As a support to the twin bridge girders, the cables stays have to be arranged in 4 planes, which is more liable to visual dazzling. To avoid this disadvantage, the stays are sparsely laid out in a harp shape. This arrangement also makes good use of high rigidity of the deep bridge deck. The side span is consequently 110m long and can provide a spare navigation channels for vessels of small size. General layout of cable-stayed bridge is 110m+180m+110m as shown in Fig 3.

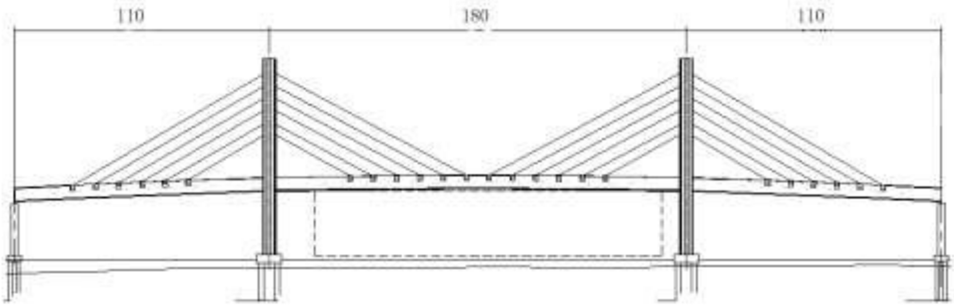


*Fig. 3 General Layout of the Saivan Bridge*

The pylon, as shown in Fig.4, is composed of 3 columns that are transversally connected as a whole by two struts at top and middle. The central column stands in the separation zone of the two traffic ways and provides anchorage for the two middle planes of stays. The two side columns stand outside the girders only carrying the outer planes of stays to both sides.

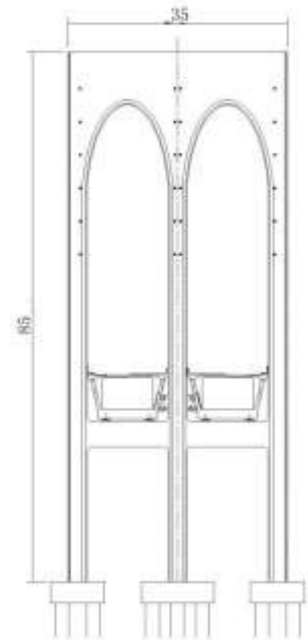


*Fig. 4 Main Pylon of the Saivan Bridge*

### 3.4 Approaching Viaduct and Box Culvert

Girder of approaching viaduct and box culvert has the same profile and dimension as that of the cable stayed bridge. It is necessitated by the requirement to continue the double deck traffic of main bridge. And also the constant cross section of bridge deck provides great convenience for fabrication and erection. Span length of approaching viaduct is decided by depth of bridge girder henceforth, 60m continuous crossing system is adopted upon analysis. As for the box culvert, span length varies according to spacing of bearing piles for the driven piles, which depends on site geological condition.

### 3.5 Foundation

According to the force subjected to the superstructure, the pylon has to be based on 3 separated waterline pile caps, of which the central one has 7@2.2m diameter bored piles while each of the two side footings has 4@2.2m diameter bored piles. The other spans are supported by separated offshore piers founded on 4@1.5m diameter bored pile. Taking account of the heavy dead load and stringent requirements of rail traffic, and to control the differential settlement of foundation in tolerance, the piles are all designed as column type, and pile tips are required to have enough Slightly Decomposed Granite socket during construction.

### 3.6 Construction planning

As well known, Macau is a much crowed town and its revenue mainly comes from tourism, suitable site for construction is not available and shortfall of manpower as well as material supply restrict major civil construction. The bridge girder is prefabricated in segments and erected by balanced cantilevering method. The prefabrication yard is located at western nearby Hengqin island, which belongs to Mainland China and is under developing and has easy access to abundant manpower resource and construction material of Mainland. To make use of this advantage to a further extend, most of major construction activities such as prefabricating steel structures and concrete batching are also undertaken on the island. Construction planning in this way saves the cost, facilitates oversea construction and also reduces adverse impact on environment. Typical balanced cantilevering erection of segments is show at Fig 5

*Fig. 5 Cantilever Erection of the Girder*

### 3.7 Bill of quantities of main bridge

The quantities of the cable-stayed bridge (110+180+110m):

- concrete: 28600 m3
- conventional reinforcement: 3789 t
- prestressing steel: 424 t
- prestressing steel strand: 609 t
- high strength steel wire: 323 t
- structural steel: 2119 t
- temporary construction steel: 5000 t

The quantities of the approach bridge (22*60m):

- concrete: 71300 m3
- conventional reinforcement: 7260 t
- prestressing steel: 950 t
- prestressing steel strand: 1960 t
- temporary structural steel: 13200 t

## 4. COMBINATION OF BRIDGE ENGINEERING STRUCTURE AND ARCHITECTURAL ART

According, under the premise of providing proper traffic function, the structure force and construction planning is firstly taken good consideration in the design proposal of the bridge, which is the structure engineer’s duty, moreover, attention is paid to architectural art design, which embody human landscape to the concrete civil structure, by which to achieve the harmony between traffic function and architectural art and capture the imagination of the public as well.

In consideration of the structure force characteristic of twin girders the following measures are adopted to give attention to the construction effect:

a) **Cable:** The load bearing cables are arranged parallel in case any cause of disorder impression from any angle of view.

b) **Main girder:** The anchorage points to both sides of the main girder are designed exposed to meet the force requirements, which is favorable for the maintenance and inspection and enhancing lasting appeal for the thick girder structure.

c) **Main pylon:** Harp shape cable planes are adopted to enhance the height and the forceful impression of the main pylon; the main pylon is designed with hollow columns to ensure its bridge longitudinal width according to the need of visual proportion and give it an impression of

stability.
d) **General layout:** The transversally connected pylon columns is designed as double archway, which takes on Chinese architectural tradition, and also benefit from and work in concert with the arch porch frequently used in the numerous occidental churches in Macau, meanwhile, it implies the first letter of Macau, with a deep meaning in it.
These design implementations won praises for the designers when the image of the winning scheme was shown to the public on the bid open day.
Besides, architects are especially commissioned to design the details, such as the washboard, skirt plate, airway, pier shape and edges of main pylons. Meanwhile, in order to achieve an overall architectural art view effect, optimization design has been done to the daytime painting color of the bridge and the night illumination.
The impression image of the bridge is shown in Fig. 6

*Fig. 6 Construction picture of the Saivan Bridge*

## 5. DESIGN INNOVATION MAKES A COMPLICATED STRUCTURE SIMPLE

It is a prominent feature of this design that innovation makes a complicated structure simple. The designer conceived an open multi-direction prestressed concrete box girder other than a conventional steel or composite steel-concrete structure and found a solution for complicated structural conditions including requirements from carriageways and loads.
This designer divides a generally conceived huge single box girder into a symmetrical twin girder by skillful conception, which is independent of each other and connected in fact. The prestressing layout and detail structural arrangement make the force meet both structural and function. This solution is also down to simply constructibility and economics and special artistic effect are achieved by architectural design. It is a pioneer project among those structures of the same type internationally.

## 6. CONCLUSIONS

The project time limit is only 2 years, because it is anticipated by the Client to be inaugurated on the five-years' anniversary of the return of the Macau Special Administration Region of December 2004. Fig.7.The working drawing design started at the bid open day, urgent construction period and unconventional design process allow no accident or repeating, it is a severe test for the design and construction. The design has been implemented on time successfully and no accident is found. The load test indicates that the total structure has excellent performance and all the indexes meet requirements of design criteria and function. The light rail projects are still under designing because the systems were delay.

*Fig.7 The General View of Saivan Bridge*

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# WORLD’S FIRST BATTERY-DRIVEN CAR FERRY

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## ABSTRACT

*An environmentally friendly ferry operation is no longer a future dream. Today there are several energy carriers available that help the shipping industry to reduce or eliminate local and global emissions.*

*A few of the most environmental friendly options are:*

- *Bio diesel*
- *Bio gas*
- *Hydrogen*
- *Electrical energy*

*In 2012, Norled won the contract to build and operate the world’s first battery driven ferry on the Lavik-Oppedal route in southwest Norway.*

*The focus of this paper is the choice of electrical energy as energy carrier for ferry transports in Norway. In this area, electrical energy is the carrier with highest availability, while both complexity and risks are low.*


## 1. INTRODUCTION

Domestic ferry transports in Norway emit approximately 400 000 tonnes of $CO_2$ per year. This represents approx. one percent of Norway’s total $CO_2$ emissions [1].

In recent years, many initiatives have been taken to compensate for increasing emissions. One popular incentive is the choice of LNG as energy carrier. LNG with its local benefits should fit well into the sensitive areas of Norway, but the uncertainty of global impact forces the shipping industry to look one step further. Also taking into account the lack of infrastructure for distribution, the reasons to seek alternative energy carriers are clear.

In Norway, the process toward emission free transportations is accelerating as the Norwegian Public Roads Administration has started to encourage environmental friendly concepts in contract bids. One example is the tender for the Sognefjord, where Norled in 2012 won a bid when offering an emission free battery driven ferry for the crossing Lavik-Oppedal.

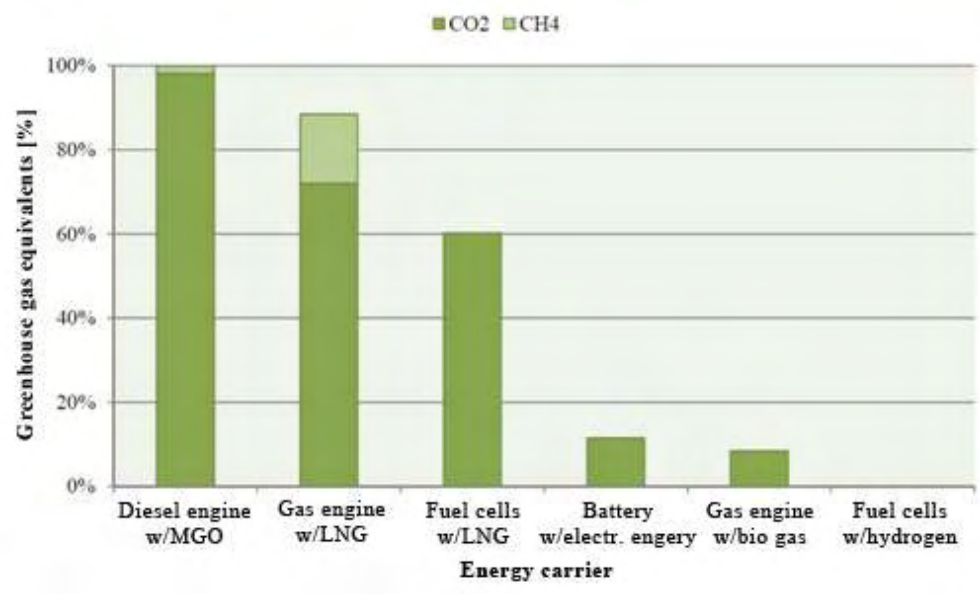


*Fig.1 Emissions for different energy carriers (Source: DNV).*

Battery technology has developed quickly during recent years, reducing both weight and cost. The success has its roots within the automotive industry, which have made the technology available for use in other areas, like in the shipping industry.

The large number of fjords and islands in Norway make several crossings suitable for battery operation. A study made by Fjellstrand Shipyard shows approx. 40 potential crossings that, in the future, could be operated by battery driven ferries.

Through collaboration with some of the industry's main players, the world's first battery driven car ferry is now under construction at Fjellstrand Shipyard and will start operating 1st of January 2015.


*Fig. 2 The world's first car ferry without any direct $CO_2$ emission.*

## 2. REQUIREMENTS

Several factors need to comply in order to provide optimal conditions for battery operation. Some of the most important are:

- **Crossing time/Energy consumption**
  Using today's battery and recharging technology, approximately all crossings of up to 30 minutes in duration or 200 kWh could be served by electrically powered vessels.

- **Size of vessel**
  With a max energy consumption of 200 kWh per crossing, the possible size of the vessel is limited to approximately 120 PCU.

- **Moderate speed**
  Energy efficiency will vary with speed. A slower crossing will give more advantageous values but also reduced charging time. The speed-power profile for the Lavik-Oppedal case gives an optimal speed of 9,5-10,5 knots.

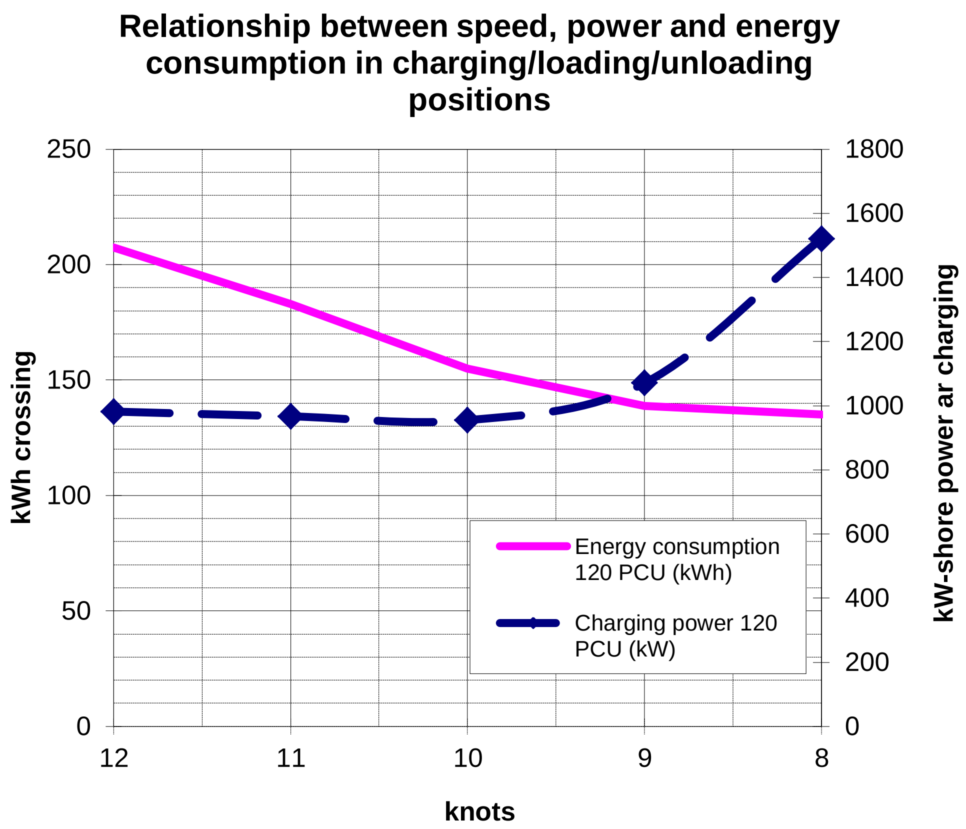


*Fig.3 The figure shows how the required power from shore power escalates when available charging time is reduced. Energy and power includes complete tour including consumption docked during charging. 10% sea margin included [2].*

- **Availability of electrical energy**
  With today's technology, the ship's batteries have to be recharged between crossings while it is loading and unloading passengers and cars. One key requirement is a sufficiently developed infrastructure for distribution of electric power at the ferry quays.

- **Adjusted timetable**
  Normal time at quay of around 5 minutes would significantly complicate battery operations. Another requirement is a timetable that allows both slower cruising speed and extended time at quay for charging.

- **Energy-optimized hull and systems on-board**
  The vessel should be optimized in terms of weight and hull shape to reduce emissions and energy consumption. In this case, a low weight in combination with a catamaran hull is reducing resistance with 30 percent compared to a traditional steel ferry [2].

  Other ways to promote energy efficiency are by an optimized propeller solution, including feathering possibilities, as well as several other factors presented in chapter 4.

## 3. DESCRIPTION OF THE CROSSING

In 2012 Norled won the competition concerning operation of the route E39 Lavik-Oppedal in the period 1.12015 to 31.12.2024. The ferry line crosses the famous Sognefjord, a popular tourist destination in the area.

*Fig.4 The Lavik-Oppedal crossing, Sognefjorden.*

Combining the south and north of Norway, the Lavik-Oppedal route is frequently travelled with almost one million cars crossing each year [3].

The new ferry will operate the actual route with 34 crossings a day, 365 days a year. The crossing covers a distance of 5,6 kilometres thus giving an operation profile of 20 minutes of sailing and 10 minutes at quay.

## 4. DESCRIPTION OF THE FERRY CONCEPT

### 4.1 General

The world's first large all-electric car ferry is a 100% battery driven catamaran in aluminium. The 80-meter long vessel will be able to carry 120 cars and 360 passengers across the Sognefjord between the villages of Lavik and Oppedal. Its key characteristics are:

| | |
|---|---|
| Length | 80 m |
| Beam | 20.8 m |
| Passenger capacity | 360 |
| Cars | 120 PCU |

Propulsion system:

2x450kW Azimuth Thrusters
2x450kW Electric motors

The ferry is designed with energy efficiency as highest priority, effecting operation, hull shape, material and systems on board:

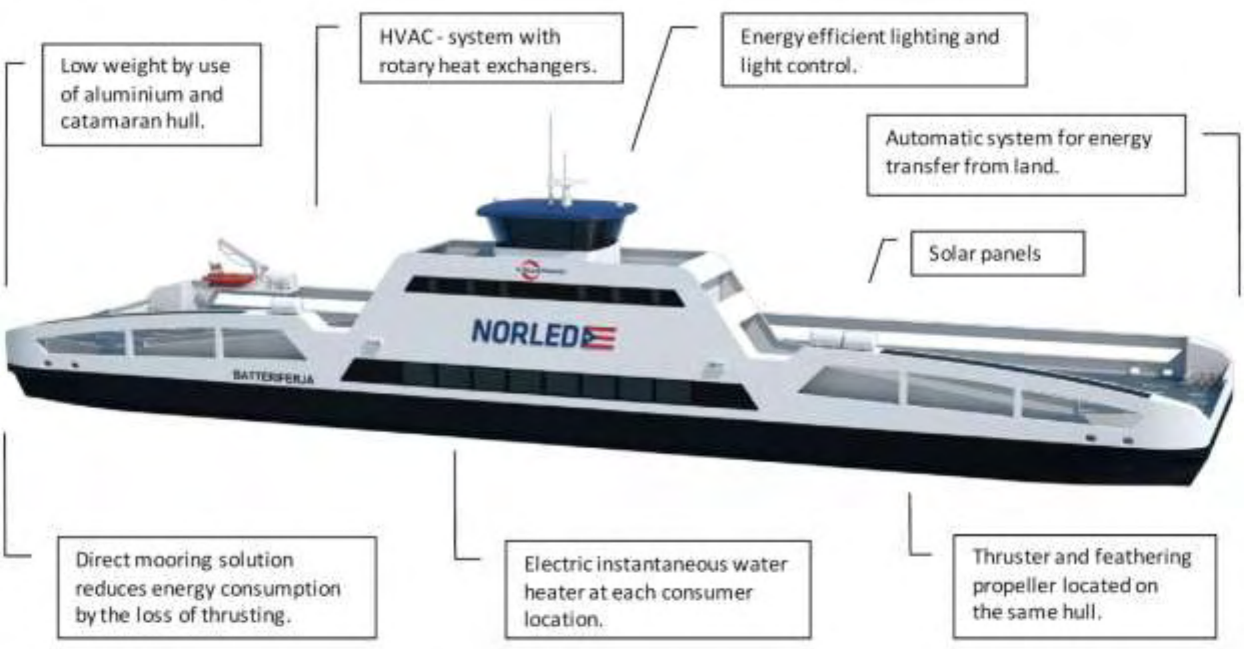


*Fig.5 Energy saving parameters*

### 4.2 Hull

The ferry is of catamaran type with both hull and superstructure of seawater resistant aluminum. Aluminum is used in large and high-speed crafts, and also has very good characteristics for vessels at lower speeds.

Both hulls are of slender type with minimal resistance, without the vessel having to be ballasted to meet stability requirements. Low weight and slender hulls mean less resistance in constant speed and also energy saving in maneuvering operations.

The hull shape and material result in energy savings of approximately 4,5 % [2].

### 4.3 Propulsion system

With speed and hull lines as given parameters, the propulsion system is optimised with variable speed, controllable pitch and feathering possibilities. During transit, the ferry runs on the aft propeller only, with the other in feathered position. The system results in energy savings of around 5 % [2].

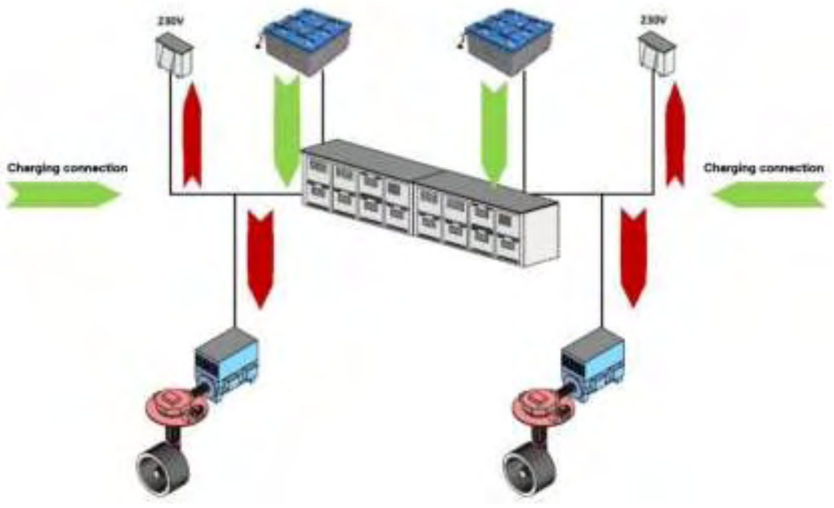


*Fig. 6 Energy flow for propulsion system 'Blue Drive Plus C' (Source: Siemens).*

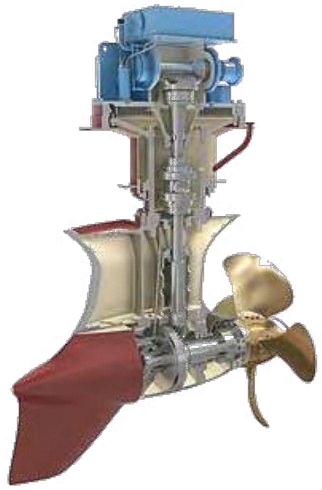

*Fig. 7 Sectional drawing of a Rolls Royce Azipull with angle gear and feathering propeller (Source: Rolls Royce).*

### 4.4 Operation

The ferry will run 34 times each day, with a crossing time of 20 minutes. Time at port is 10 minutes, time which will be used to fully charge the 1MWh lithium-polymer battery pack on board.

This amount of electric power delivered in such a short time is far beyond the capacity of the electrical grid serving the villages of Lavik and Oppedal. The solution is to install battery buffers at both ports. These batteries can be continuously charged from the grid with 250 kW, then rapidly provide a quick dump to the ferry's batteries.

During night, the ferry has a period of seven hours for top-charging of the batteries.

For one crossing under normal conditions operating at a speed of 10 knots, battery power of only 150-200 kWh is needed, thus providing a wide safety margin.

*Fig.8 Illustration showing principle with a battery bank continuously charged from the public grid.*

Compared to a standard diesel ferry serving the same route, the electric ferry will save about one million litres of fuel annually, as well as preventing 570 tonnes of carbon dioxide from entering the atmosphere [4].

### 4.5 Batteries and safety

The redundant battery pack on-board the vessel is of Li-ion type, with good characteristics for rapid charge/discharge rates. It will claim 10 $m^3$ of space, have a weight of 10 tons and is able to store 1MWh of energy. Under the circumstances specified for Lavik-Oppedal, the battery lifetime is calculated to approximately 10 years.

To ensure safe operation, the batteries are of dry type, with current- and temperature surveillance in each control cell.

**5. CONCLUSIONS**

The electric car ferry now being produced at Fjellstrand Shipyard does not discharge to the environment, neither greenhouse gases, $CO_2$, methane or nitrogen oxides. Besides clear environmental benefits, the ferry will run with both lower operational and maintenance costs.

Passenger and crew comfort will increase due to cleaner environment and minimal noise and the small disadvantage of not having a fully equipped kiosk onboard will hopefully be accepted in time.
In time, we will certainly also see more than one electric ferry crossing fjords in Norway. Norled will continue the development of electric ferries and encourage the rest of the shipping industry to follow. To facilitate and speed up the process, national authorities play a great role. That the NPRA continue to ask for environmental friendly alternatives is essential for encouraging such incentives.

In time, we will also hopefully see a removal of the electricity tax for maritime transports.

Although domestic ferry operations in Norway do not represent a large proportion of the total emissions, it is important to cut where possible. To improve the characteristics of one specific car ferry is certainly a great step for the ship-owner. But even more important is to spread the knowledge and technology to the rest of the shipping industry, contributing to large reductions of pollution, worldwide.


*Fig. 9 Smart, clean and silent. Trafficking Lavik-Oppedal from 1st of January 2015.*

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# AUTOMATIC MOORING - A STEP TOWARD AUTOMATED TERMINALS

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## ABSTRACT

*A conceived vacuum-based automated mooring technology that safely moors ferries and even the largest 450,000dwt bulk vessels and 18,000TEU container vessels in a matter of seconds eliminates the need for conventional mooring lines. Remote controlled vacuum pads, recessed in, or mounted on the quayside and attached to hydraulic actuated arms, extend, attach and moor ships in a few seconds.*

*The technology is proven with over 54,000 successful moorings and the operator´s benefits include:*

- *Vessels can be all secure in 30 seconds and released in 15 seconds saving up to 1.5 hours per vessel compared when to conventional mooring conventionally taking 20-90 minutes involving mooring gangs and ships crew.*

- *Personnel safety is improved as only one operator is required to remote control the system with no "man-handling" of heavy ropes in most cases.*

- *Some automated systems hold the vessel at a pre-set distance from fender line resulting in reduced wear on fenders.*

- *Some systems only attaches to the parallel ship body making berth overhang possible, therefore the need for berth extensions or mooring dolphins is unnecessary.*

- *Operational efficiency is increased and breakwaters or breakwater extensions may be unnecessary wher the system is able to reduce vessel motion due to waves.*

- *In some cases the mooring units can also be used to shift vessel position without the need for ships propulsion or tug assistance.*

- *Due to the speed of operation air quality in the port is improved as the engines can be shut down sooner.*

- *Due to speed of operation vessels can reduce their speed and still meet the required schedules.*

- *The system incorporates continuous load monitoring and sophisticated alarm functions relayed in real-time to operations personnel onshore, on-board and/or in port control office. Alerts can be sent to pagers, mobile phones and other devices.*

**INTRODUCTION**

Cavotec has been developing Automated Mooring systems for more than 14 years and is now assisting ports and shipping operators as they enter the new era of automation. MoorMaster™ automated mooring products are designed to completely automate vessel mooring using proven vacuum technology, with over 54,000 moorings performed to date (see Figure 36 & Figure 37). Berths utilising MoorMaster™ moor vessels that include 140 m Ro/Ro, Ro/Pax, bulk Lakers, cape size bulkers and 362 m container vessels.


*Figure 35: Typical Ferry Installation*

Based on the accumulated knowledge of key MoorMaster™ installations, Cavotec believes ferry terminals will benefit greatly from automated mooring technology through savings on turnaround time, reduction in fuel consumed at the berth, fuel savings from “slow steaming”, reduction of infrastructure costs and increase of safety both onshore and on-board through the elimination of ropes. Furthermore, the technology offers users the opportunity to reduce labour requirements on the terminal and vessel for mooring / let go operations.

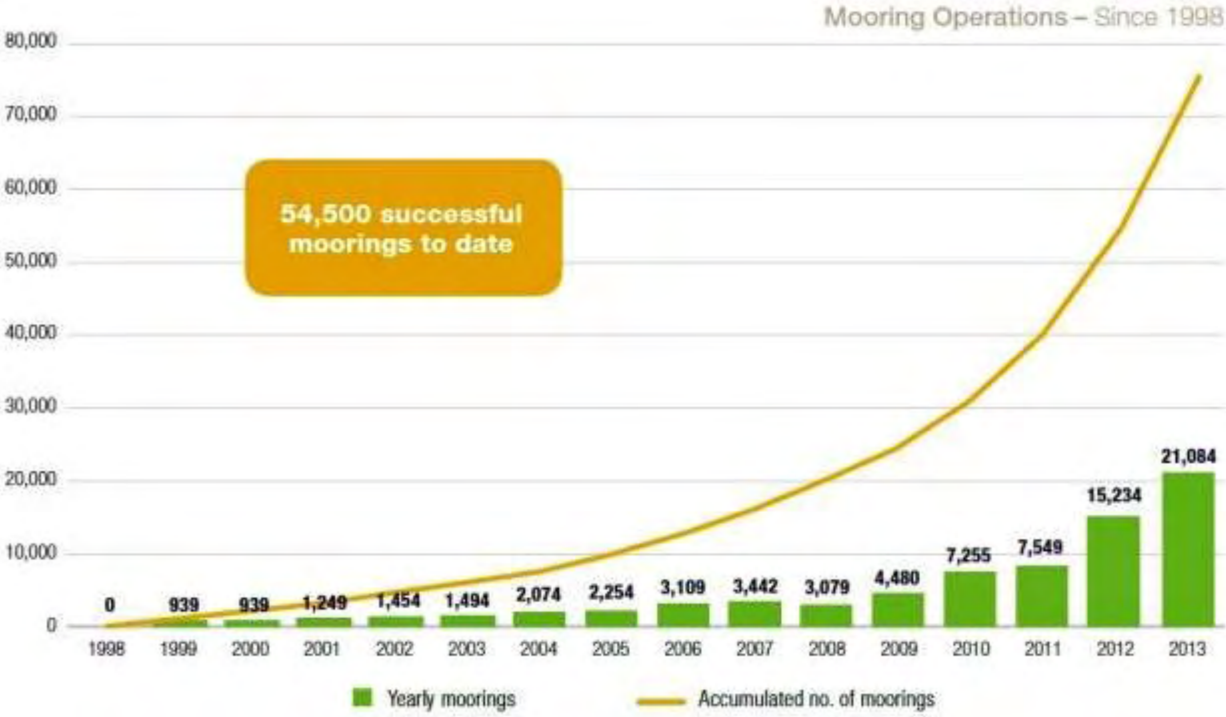


*Figure 36: MoorMaster™Automated Mooring Operations*

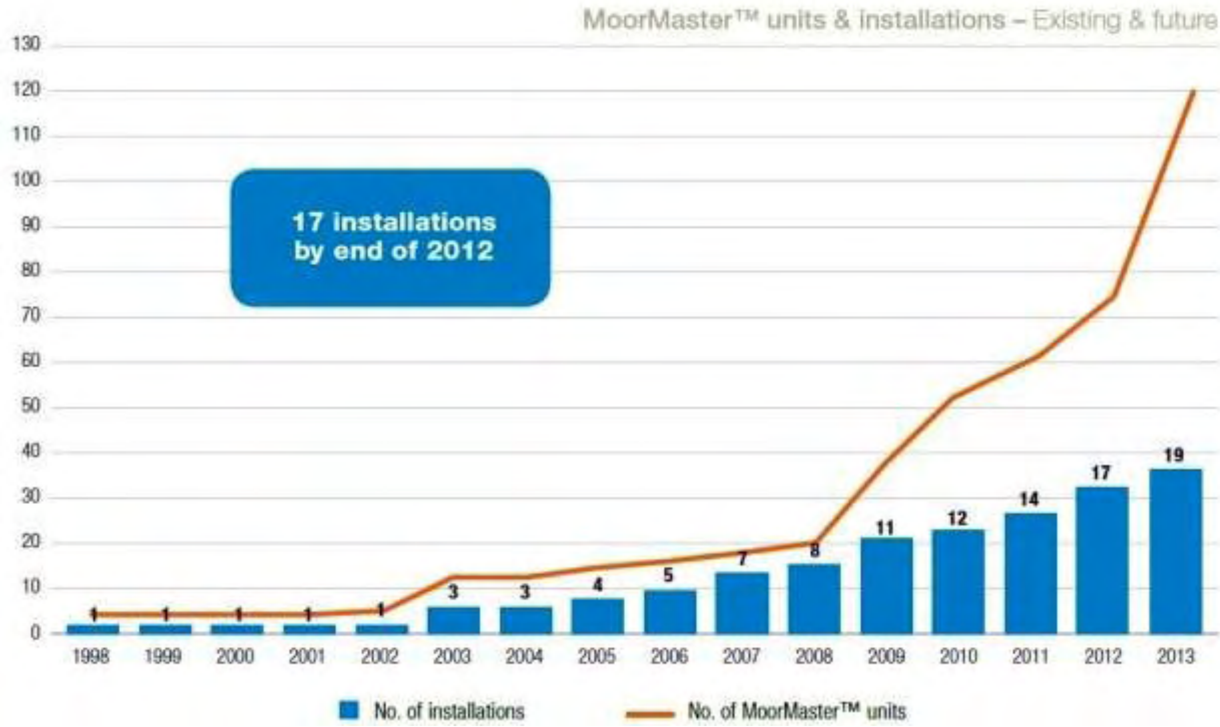


*Figure 37: MoorMaster™ Automated Mooring Installations*

**WHY AUTOMATE MOORING?**

Firstly it is important to distinguish fully automated mooring systems from systems that use automated mechanisms to deploy and simulate the behaviour of conventional moorings. While these systems will offer some of the advantages of a fully automated system they cannot offer them all. This is mostly because many of the short comings of conventional mooring are the ropes themselves.

It is currently not uncommon for ferries to use equipment on the vehicle bridge to restrain vessel surge. These systems can be used in conjunction with automated mooring to reduce the cost of the automated mooring system. However, the full benefit of the automated mooring system is not likely to be realised due to the speed of the bridge system and the required sequence of operation.

**1) Speed**

Conventional moorings employing ropes are time consuming and dangerous to deploy. A number of dock side staff and ships crews are required to deploy and release them. This costs a ferry operator time and money. Automating moorings on a ferry operation allows the vessel to be moored from the vessel bridge by the master or other ships officer in a matter of seconds using a remote control, Figure 38.


*Figure 38: Typical Remote Control*

**2) Fuel and Staff Savings**

Automated mooring allows the vessels to employ smaller on-board crews and negates the needs for shore mooring crew totally.

Mooring is completed in a matter of seconds allowing the ship to shut down engines sooner. This reduces fuel consumption and increases air quality in the port. The time saving can be additionally leveraged by the savvy ferry operator by reducing sailing speeds while maintaining the same loading / unloading time in port resulting in further fuel savings. Additionally these savings are increased in more severe conditions if the fully automated mooring system employed is able to reduce vessel motions in severe but not operationally limiting conditions.

Further savings in this area are achieved if the vessel is required to be relocated along the berth after mooring to ensure proper alignment of tall dock side services. The automated system is able to warp a vessel along the berth at a touch of a button.

This benefit is further increased in the case of ferries that do not use moorings at all but rely purely on vessel thrust to remain alongside. In these cases automated mooring will not increase the speed of mooring but will dramatically reduce the fuel consumed while alongside. Furthermore, the damage to or maintenance to the seabed required as a result of the constant disturbance caused by the propulsion can be very costly. This fuel saving could be achieved with rope moorings however the time savings would not be achieved.

### 3) Staff Safety

Conventional moorings are also dangerous once deployed and often the cause of dock side accidents. This is a result of breakages due to overloading resulting from either poor rope condition or severe environmental conditions. This danger is compounded in ports where "storm moorings" are deployed once conditions have worsened to the point that the present moorings are at risk. Sending personnel into the danger zone is extremely dangerous and completely unnecessary with automated mooring.

As above vessel warping is not only time and fuel consuming it is very dangerous if conventional moorings and winches are used to achieve this movement. Using an automated mooring system for this function completely eliminates all risk to personnel, vessel and other equipment.

### 4) Vessel Security

The condition and status of a well designed and developed automated mooring system is known at all times unlike ropes that could be in poor condition. This could be done via the control and monitoring system which can be bridge and/or terminal mounted, see Figure 39, and can also be integrated into the ships alarm system.


*Figure 39: Typical Ferry Bridge Installed Equipment*

Additionally these monitoring systems can be integrated into the ferry operations computer network (if required) and also be equipped to send status messages via SMS/TXT or email.

### 5) Infrastructure Savings

Infrastructure savings arise in some cases as certain systems only attach to the parallel body of the vessel. This allows a ferry operator to build a shorter berth and to moor a vessel of a certain size. An example of this approach is shown in Figure 40.

*Figure 40: New Berth with Vessel Over-Hang*

Alternatively if an operator increases the size of the ferries on their route(s) they can avoid extending the pier or building mooring dolphins as was the case shown in Figure 41 where a berth extension of approximately 50m was avoided.


*Figure 41: Existing Berth - Larger Vessel*

Furthermore, due to the manner in which some systems transmit the mooring loads into the berth structure the high point loads associated with bollards no longer occur. This can potentially result in the design life of existing structures being extended.
Infrastructure can be further reduced due to the vessel motion reduction. This ability allows for more severe conditions at the berth. Therefore a breakwater could be avoided, reduced at the design stage or not extended as the case may be.

### 6) Reductions in Maintenance

A thorough automated mooring system will hold the vessel off the fenders once the vessel is berthed. This results in less wear and tear on the fenders extending their life and saving on replacement and repair costs.

Additionally this can result in less wear and tear on the vessel hull resulting in cost savings when the vessel is sent for refit. For example one MoorMaster™ system owner has reported savings in this area of up to €150,000 per year.

## CONCLUSION - FULLY AUTOMATED TERMINALS?

Fully automating a ferry terminal requires more than simply automating the moorings. Vessel bunkering/refuelling, water bunkering, land power deployment where cold ironing occurs and automation of the vessel link spans is also required. However, for these other operations to be automated first the vessel must be securely moored in a repeatable position and be as stable and secure as possible. A properly specified and designed automated mooring system will provide this base platform for the automation of these other vital ferry operations along with providing the additional benefits of:

- Faster vessel-turnaround enables larger number of ship calls.
- Cargo and crew transfer can start earlier.
- Increased cargo throughput.
- Mooring gangs not required.
- Improved utilisation of terminal length if berthing distances reduced.
- Vessels longer than berths can be moored with overhang, enabling substantial savings on quay extensions or dolphin investments.
- Restricted waterways not disturbed.
- In some cases, and with a well desined system the need for breakwater construction may be eliminated.
- Personnel safety improved.
- Potential reduction in insurance premiums.
- Mooring load status constantly monitored and event logs can be reviewed.
- Less wear and tear on fenders.
- Cargo operation less dependent on weather conditions.
- Reduced use of the vessel’s propulsion system diminishes fuel consumption and emissions.
- Faster connection to shore power, where available.
- Potential slower cruise speeds for vessels.
- Ship’s crew can use their time for more productive jobs and keep uninterrupted rest hours.
- Less wear and tear on ropes, winches, ship’s hulls and plating.
- Automatic ship repositioning / “warping” facility results in fuel saving for vessels.

# TUNNEL LIGHTING

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**ABSTRACT**

*Tunnel lighting technology is developing rapidly. New light sources and better lighting control systems make new possibilities for energy savings and reduced maintenance costs. LED luminaires have for a few years been used for interior zone illumination, and during the last two years the first tunnels are equipped with full LED lighting, even in the threshold and transition zones. As prices go down and quality goes up, new technology will be more applicable and profitable in new tunnels and in upgrading of existing tunnels.*

*However, some uncertainty factors regarding LED and other new technology need to be studied. It is important to gather experience and learn from the first LED installations, to avoid bad choices in future investments.*

*From January 2013 Brekk tunnel in the middle of Norway is used for testing several factors influencing the effects and costs of tunnel lighting. Other Norwegian tunnels are also used to study benefits and costs of cleaning luminaires and tunnel walls and white painting of tunnel walls.*

*State of the art in tunnel lighting and results from Norwegian studies are briefly presented in this paper.*

## INTRODUCTION

Tunnel lighting technology is developing fast. LED was introduced as light source for tunnel lighting a few years ago, and today dimmable LED luminaires in a lighting control system is a real alternative to traditional tunnel lighting. Such a system provides opportunities for optimisation of lighting level, energy savings and reduction of maintenance costs.

LED has some advantages compared to traditional light sources for tunnel lighting. It is slightly more efficient, has probably longer life, is more useful for dimming, and the quality of the light seems to be equal or better. Each year the efficiency and quality has increased while the price is reduced. However, there are considerable uncertainties related to the use of LED. We do not know how much we can trust the information we get from the manufacturers and suppliers. Experience and knowledge is limited, especially about life and light loss over time. Standards, requirements, product specifications, and methods for calculations and measurements are mostly made for other light sources and are perhaps not very useful for LED. Studies must be carried out and experiences must be gained before we can be sure to make the right choices and get good LED installations.

During the last couple of years LED is installed as interior zone lighting in a few Norwegian road tunnels, complemented with traditional high pressure sodium luminaires or metal halide luminaires in the threshold zone and transition zone. In March 2013 we got the first tunnel, Brekk tunnel, fully equipped with LED luminaires, using high power LED luminaires for the threshold zone and transition zone. Brekk tunnel is a one tube tunnel on E39 in the middle of Norway, with average daily traffic volume of 9000 vehicles. It was built in 2005 with a traditional tunnel lighting installation. The same private company that built the tunnel got a 25 years contract for operating and maintaining the tunnel, and after 8 years of maintenance the contractor decided to change the whole tunnel lighting to LED. Thus, the NPRA got an opportunity, in cooperation with the contractor, to study LED technology, lighting level and quality, energy consumption, driving comfort and safety.

Brekk tunnel and several other Norwegian tunnels are used to study LED technology and advanced lighting control in a two years research and development project that started in March 2013 and will be finished in 2015. The project also includes other study objectives related to energy savings, traffic safety and the forthcoming revision of the Norwegian tunnel lighting standard.

The first results from the project are presented in this paper.

## STUDY OBJECTIVES

### LED technology

LED technology will be studied in several Norwegian tunnels in 2013and 2014.

*Table 1: Norwegian road tunnels with LED lighting, to be studied in 2013 - 2014*

| Name of tunnel | Length of tunnel | LED lighting | Year of LED installation | Product |
|---|---|---|---|---|
| Ljabrudiagonalen | 110 m | Basic lighting | 2004 | Dellux |
| Vabakken | 650 m | Basic lighting | 2010 | AEC |
| Prestura | 1370 m | Basic lighting | 2011 | Thorn |
| Jønjilju | 764 m | Basic lighting | 2011 | Thorn |
| Brekk | 1300 m | All lighting | 2013 | AEC |
| Harang | 790 m | All lighting | 2013/2014 | ? |

Quality, benefits and challenges will be studied through measurements of light and electricity parameters. Two of the tunnels, Brekk and Harang, will be fully equipped with LED lighting. The other tunnels have LED luminaires for basic lighting, complimented with high pressure sodium luminaires for threshold and transition zones.

### Lighting control

Lighting control systems will be studied in Norwegian tunnels during 2013 and 2014. While two of the tunnels, Brekk and Harang, have LED as light source even in the threshold zones, a third tunnel, Strindheim tunnel, has traditional light sources in all zones. Lighting levels in the threshold and transition zones will be controlled according to daylight level outside the tunnel, Different methods for assessing the right lighting level will be evaluated, and quality, application and possible benefits and challenges of different control technologies will be evaluated.

Lighting control according to traffic density will be tested in some low traffic tunnels, and the potential for energy savings will be evaluated. The potential is believed to be large.

**Lighting level and quality requirements**

The third objective is to test the Norwegian requirements for tunnel lighting levels and quality in some real tunnels. Brekk tunnel is used for this purpose. A test group of people at different ages will drive through the tunnel during different daylight conditions and experience comfort and feeling of safety at different lighting levels inside the tunnel. The test persons will classify the lighting levels in the different zones of the tunnel as either too low, suitable or too high. The results of the study may be helpful for revision of the tunnel lighting standard. Beside the lighting level, lighting uniformity, wall luminance, colour of the light, and glare from the luminaires are important issues.

**Effect of white tunnel walls**

*Figure 1: Common Norwegian tunnel entrance.*

*Figure 2: The first test of white light on ceiling and walls in 2008*

A forth objective is to study the effect of light tunnel walls on driving comfort, traffic safety or possible energy savings. Studies carried out in 2008 with white light on light tunnel walls showed potential for energy savings and improved comfort and safety. New studies will be carried out in 2013 and 2014 in other tunnels. An on-going project in Askimporten tunnel testing different kind of surface treatments of concrete wall elements will be extended with measurements of wall luminance before and after washing the walls. Other tunnels in Western Norway are painted with white coating once a year or every second year. Painting is performed on different kind of wall materials, even on wet rock walls. Measurements will be carried out before and after painting, to calculate the effect on wall luminance, road surface luminance and possible energy savings. The effect on comfort and feeling of safety will be assessed by a test group of persons. Distance for detection of obstacles will be measured.

**Effect of cleaning**

The effect of cleaning the luminaires and tunnel walls on lighting levels, traffic safety and/or energy savings will be evaluated. Measurements of horizontal and vertical illuminance, road surface luminance, and wall luminance will be carried out before and after washing the luminaires and before and after washing the tunnel walls. Measurements will be carried out in tunnels in Bergen, Askimporten and other tunnels in 2013 and 2014.


*Figure 3: Arnanipa tunnel before washing, after washing of the whole tunnel, and after painting of walls*

**Effect of guide lights**
The traffic safety effect of wall mounted guide lights will be evaluated in 2014.

## METHODS

Illuminance inside the tunnel is measured with a handheld calibrated Haegner lux meter. Luminance of road surface and tunnel walls is measured with a CCD based luminance camera from Techno Team. It is a modified Cannon EOS 550D camera, mounted on a tripod. Pictures are analysed in a computer program from LMK. Measurements are carried out according to procedures described in EN-13201-4, 2003.

In Brekk tunnel continuous measurements of illuminance, colour rendering index, and colour temperature are also carried out with a Specbos 1201 mobile spectroradiometer mounted on a trailer behind a car.

At Brekk tunnel, a luminance camera is mounted 100 m outside each tunnel entrance to detect the drivers' adaptation luminance before entering the tunnel. Adaptation luminance is the main input to the control system to dimension the lighting level in the threshold and transition zone inside the tunnel. The luminance level in the threshold zone is set as some per cent of the adaptation luminance outside the tunnel. Lighting levels will be evaluated according to the test drivers' feeling of comfort and safety and according to visibility of objects.

At Brekk tunnel and some other tunnels, radars will be mounted outside the tunnel entrances to detect oncoming vehicles. During low traffic hours, the tunnel lighting will be turned off or set to a very low level, but increased to a higher level when a vehicle approaches the tunnel.

Electrical parameters are measured with an oscilloscope and a calibrated power quality analyser.

A test group of persons of different age and gender is used for visual assessment of lighting quality, driving comfort and feeling of safety.

Norwegian requirements for tunnel lighting levels and quality are used for comparison.

## RESULTS

### Lighting Quality

Lighting quality is measured in five tunnels having LED interior zone lighting: Ljabru, Vabakken, Prestura, Jønjilju and Brekk tunnel. The Norwegian requirements for luminance level and longitudinal uniformity are met in all tunnels. Required value for overall luminance uniformity is also met in all tunnels, except Vabakken tunnel, where the luminance in the middle of the road is too high compared to the luminance at the edges.

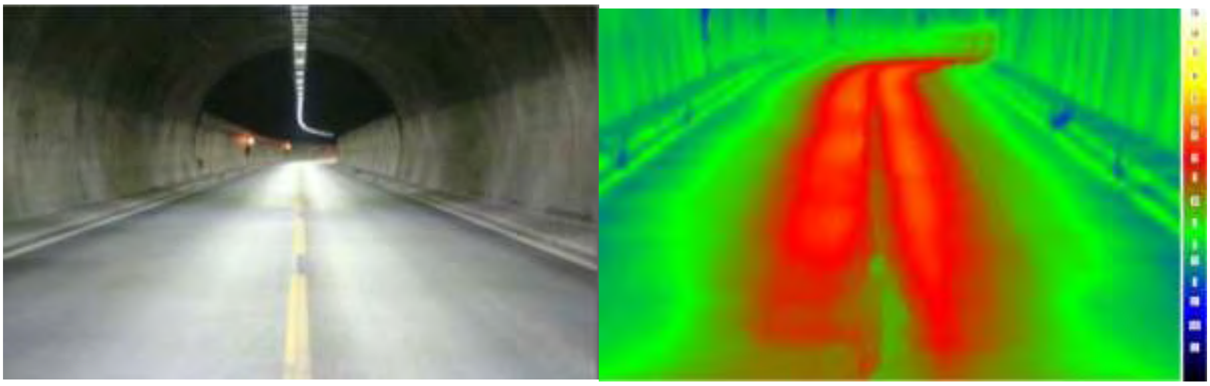

*Figure 4: LED for interior zone lighting in Vabakken tunnel. Lighting level and longitudinal uniformity is good, but too much light in the middle of the road.*

Horizontal and vertical illuminance is measured continuously through Brekk tunnel with a mobile instrument, and the measurements confirm that the LED installation fully meets the Norwegian requirements for lighting level in all the zones.

Glare from the luminaires is an important issue because it affects traffic safety and comfort, although the Norwegian standard does not give any requirements on glare. Calculations of disability glare (TI) for Prestura, Jønjilju and Brekk tunnel show quite good results compared with the recommended limit, TI < 15 %, in some international technical reports.

*Table 2: Calculated disability glare in some LED tunnels*

| Tunnel | Calculated disability glare (TI) from LED luminaires | |
|---|---|---|
| | Entrance zone | Interior zone |
| Brekk | 6.8 % | 8.1 % |
| Prestura | | 2.8 % |

However, the observed glare from LED luminaires seems to be higher than indicated by glare calculations. The methods for glare calculations are probably not quite applicable for LED luminaires, and results from glare calculations for LED luminaires and other luminaires should therefore not be compared.

Glare in tunnel lighting cannot easily be measured, but some kind of simplified glare evaluation is done for LED installations in Brekk and Prestura tunnels and for traditional lighting in Brekk tunnel. The evaluations indicate that both discomfort glare and disability glare is significantly higher from LED luminaires than from traditional luminaires. Vertical illuminance on the eye of the observer is higher, and the luminance of the luminaire is higher when LED is used. Further studies of glare will be carried out.

Several other lighting and electrical parameters will also be measured and evaluated, and LED installations will be compared with traditional lighting installations and checked against Norwegian tunnel lighting requirements. Feeling of comfort and safety will also be assessed

**Maintenance**

The effect of cleaning the luminaires is evaluated for two tunnels, as shown in table 3. Average horizontal illuminance at the road surface is measured in the interior zone before and after washing. Both tunnels have high pressure sodium luminaires. In Arnanipa tunnel the whole tunnel was washed. However, only four summer months had passed since last washing, and the luminaires were not much dirty. In Brekk tunnel only the luminaires were washed, and they were quite dirty after 3 winter months since last washing. According to the maintenance contract the luminaires should be cleaned after 12 months.

*Table 3: Average horizontal illuminance in interior zone before and after washing of luminaires in Arnanipa tunnel and Brekk tunnel*

| Tunnel | Average horizontal illuminance (lux) | | Increase after washing | Time since last washing |
|---|---|---|---|---|
| | Before washing | After washing | | |
| Arnanipa | 31.1 | 32.43 | 4.3 % | 4 summer months |
| Brekk | 70.5 | 81.5 | 15.6 % | 3 winter months |

The measurements in Brekk tunnel show that dirt depreciation of the tunnel lighting is more than 15 % after three winter months. Average daily traffic volume is 9700 vehicles.

Measurements before and after washing will also be done in other tunnels in 2013.

**Light tunnel walls**

Measurements in Arnanipa tunnel show that white painting of rock walls contributes to increased road surface luminance. The increase in this case is 12.3 %, as shown in table 4.

*Table 4: Average horizontal illuminance in interior zone before and after white painting of tunnel walls in Arnanipa tunnel*

| Tunnel | Average horizontal illuminance (lux) | | Increase after painting | Time since last painting |
|---|---|---|---|---|
| | Before painting | After painting | | |
| Arnanipa | 32.43 | 36.42 | 12.3 % | 12 months |

This means that the required road surface luminance can be obtained with 12 % less light flux from the luminaires and about 12 % less consumed power in case of LED lighting. Some light colour from the previous painting one year ago was still visible. If the walls were naturally dark, the effect of painting would have been higher.

The benefit of light coloured walls is not solely the reflected light on the road surface. Light walls also provide better visibility of objects in the tunnel as well as better feeling of comfort. Measurements in Damsgård tunnel show that white painting of concrete walls increases the reflection factor from 0.25 to 0.67.

*Table 5: Reflection factors before and after white painting of walls in Damsgård tunnel*

| | Before painting | After painting | Increase after painting |
|---|---|---|---|
| Reflection factor of walls | 0.25 | 0.67 | 168 % |

Requirements for tunnel wall luminance will be incorporated in a revised Norwegian tunnel standard, and with white coloured walls the required luminance can be obtained with less light flux and less power.

In Askimporten tunnel nine different surface treatment products are tested on nine concrete wall sections. All products provide some kind of impregnation of the concrete, and some of the products also provide white colouring of the surface. Long term effect is tested with respect to maintaining a light grey surface colour and reducing the environmental impacts on the concrete sections. Figure 5 shows the reflectance factors before and after washing the walls, three years after the surface treatment and one year after the previous washing.

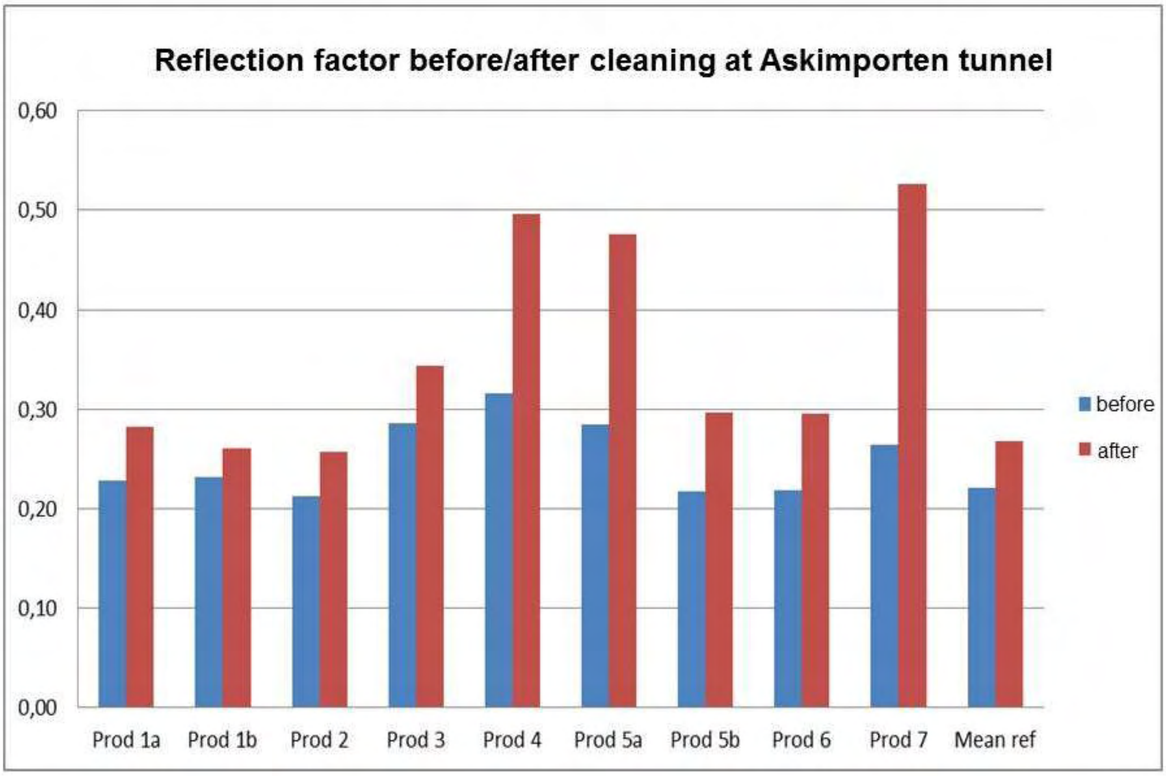


*Figure 5: Reflection factor before (blue) and after (red) washing of wall elements with different kind of surface treatment in Askimporten tunnel*

After three years the treatment is still effective, more or less. Compared with the untreated reference sections, the reflection factor is higher on most of the treated sections, before and after washing.

Before washing, the best performing product (Prod 4) showed a 40 % higher reflection factor than the reference surface.

After washing, three of the products showed nearly twice as high reflection factor as the untreated reference sections. These products are:

Prod 4. Water based epoxy coating with white pigment
Prod 5a: Hydrophobic impregnation with white pigment.
Prod 7: Cement based coating with white pigment

Benefit/cost calculations should be done.

# CONCLUSIONS

Some conclusions can be made on the first results from the two years Norwegian research and development project within tunnel lighting.

LED is today a useful light source for tunnel lighting. Lighting requirements as luminance level and luminance uniformity are easily met with LED luminaires even if the spacing between luminaires is increased compared to traditional tunnel lighting. However, glare from LED luminaires can be a problem, especially when counter beam luminaires are used in the threshold and transition zones. Measurements of vertical illumination at the eye of the observer and luminance of luminaires indicate that calculations of disability glare from LED luminaires give too low values. Further studies about disability glare and discomfort glare from LED luminaires are needed.

Dirt depreciation of tunnel lighting is considerable and often underestimated when determining the maintenance factor. Dirt depreciation alone often accounts for more than 30 % loss of light after a winter season, and a realistic maintenance factor for the lighting installation is about 50 %. More frequent cleaning will allow for a higher maintenance factor, which means fewer luminaires and lower investment costs. With dimmable luminaires the light flux can be reduced after cleaning, and electrical energy is saved each time the luminaires are cleaned. With a not dimmable installation the visual conditions and traffic safety will be improved after cleaning. Benefit/cost evaluations are needed.

White painting of tunnel walls contributes considerably to road surface luminance, and with a dimmable lighting installation energy can be saved by reducing the light flux about 12 % when the painting is new. The main benefit of light walls, however, is that they make a light background for better detection of dark pedestrians, cyclists, mopeds, other vehicles or other objects. Light tunnel walls also contribute to make the tunnel more pleasant. Tests in Askimporten tunnel have shown that surface treatment of concrete walls with suitable products can last for several years and make the walls significantly lighter, before and after washing. Experiences from Arnanipa tunnel shows that white painting can be useful even on rock walls, but rock walls need to be repainted more often than concrete sections. Benefit/cost calculations are needed.

Studies will be carried out with lighting control systems and dimmable LED luminaires. The potential for energy savings is assumed to be large. In low traffic tunnels the potential for energy savings is especially large when tunnel lighting is controlled by oncoming vehicles.

# EXPERIENCE FROM USE OF COMPETITIVE DIALOGUE IN COMPLEX PROJECTS

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**ABSTRACT**

*Crossing of major barriers are usually based on combination of knowledge, experience and innovation. Also when it comes to road construction and strait crossings, whether barriers are related to topographical, geological, technological, climatic or environmental conditions. Visionary leaders and politicians set new goals, - partly outside the known limits! This means big challenges for engineers which are set to meet these goals. Planning, procurement and construction can be limited by existing competence, capacity, policies, laws and regulations.*

*In some certain particularly complex project NPRA has admitted that they have not been able to describe all solutions, conditions or circumstances that can meet the project's needs and objectives. Norwegian law and regulations for public procurement allows for a special procurement procedure, "Competitive Dialogue", for such occasions. The procedure is introduced along the lines of EU's procurement Directive EF/18/2004[5]. Norway chose to introduce the procedure to obtain more options and greater flexibility in procurement process for particularly complex contracts.*

*Competitive Dialogue is a procedure in which any economic operator may request to participate and whereby the contracting authority conducts a dialogue with the candidates admitted to that procedure, with the aim of developing one or more suitable alternatives capable of meeting its requirements, and on the basis of which the candidates chosen are invited to tender.*

*NPRA has so far used Competitive Dialogue on three occasions:*

- *Turnkey contract regarding E6 tunnel through sensitive clay in the district Møllenberg in Trondheim.*
- *Development of the most energy - and environmentally efficient ferry for operation of E39 Lavik - Oppedal in the period 2015 - 2025.*
- *Feasibility studies for crossing the Sognefjord with respectively a floating bridge and a submerged floating tunnel.*

*This paper explains NPRA experiences with the implementation of Competitive Dialogue in connection with these complex projects.*

## 1. INTRODUCTION

NPRA usually prepares detailed technical descriptions as basis for procurement of engineering, services and construction works. Established traditions, experience, manuals and other standardized document support efficient procurements within this traditional form. In connection with some particularly complex projects, innovation tasks, development etc, it may occur that the NPRA's normally available expertise is not sufficient to define the means of satisfying their needs or of assessing what the market can offer in the way of technical solutions or financial and

[5] Directive 2004/18/EC of the European Parliament and of the Council of 31 March 2004

legal solutions. NPRA has on three occasions in the last five years found the possibility to use a new procurement procedure which offers more flexibility with such particularly complex projects.

## 2. THE PROJECTS

### 2.1. E6 Trondheim-Stjørdal, Nidelv Bridge – Grillstad, Western surface work zone

The original development of E6 along the stretch Trondheim – Stjørdal has had a very long period of planning, followed by implementation in several stages. It is the end sections in Trondheim and Stjørdal municipalities, respectively, which now are under construction. These sections will for the time being complete the E6 Trondheim – Stjørdal project.

The end section in Trondheim includes building approx. 4.5 kilometres of new or improved four-lane E6 along the stretch from the Nidelv bridge – Grillstad. This also includes a 2.4 kilometres long tunnel from Møllenberg to Strindheim. During planning of this tunnel it was identified great challenges with respect to execution of “Western surface work zone” – cutting through an old, urban residential area on sensitive clay in the district Møllenberg.

“Western surface work zone” includes engineering and construction of the new E6 from Nidelv bridge to approx. 30 metres within the entry point of the rock tunnel under Møllenberg. Very demanding soil conditions with quick clay and excess pore pressure against the bedrock have been revealed. Work which entails a risk of disturbing sensitive soil masses and/or changes in the pore pressure require comprehensive planning and control while the work is carried out, in order to avoid unacceptable conditions along the way. Through engineering and construction it must be ensured that disturbance of quick clay is avoided, as this can have catastrophic results. The engineering and implementation must also take into account that the construction is taking place in densely built-up areas and show due consideration for noise and vibrations.

Execution of “Western surface work zone” affects many buildings and residents at Møllenberg. Stringent requirements had to be implemented in order to ensure that consequences for buildings and residents should not become greater than necessary.

The contract strategy for “Western surface work zone” was established under following assumptions:

- Design engineering and execution of the work are particularly complex.
- Sensitive ground conditions and surroundings
- Lack of relevant reference projects
- High and uncertain cost estimates
- Great uncertainty as to whether the NPRA preliminary design will meet the project's needs and objectives.
- Doubts as to whether NPRA and their advisors have adequate qualifications to define the technical means which will meet the project's needs and objectives.

It was assumed that the most adequate qualifications were to be found among construction companies with special skills in earthworks and foundations. Under the assumptions above it was in June 2008 decided to utilize Competitive Dialogue for this part of the project.

## 2.2. E39 Lavik – Oppedal, Operation of Ferry Service.

Ministry of Transport and Communications issued in the budget proposal for 2011 guidance to NPRA to provide for technical innovations with respect to transport equipment. The Ministry requested specific that operation of the ferry service Lavik - Oppedal should be announced as a development contract, where the industry should be invited to compete for the delivery of the most energy - and environmentally efficient ferry. The Ministry noted that electrically operated ferry or ferry with biofuels could be relevant in the competition, but gave no other specifications or guidance with respect to needs and objectives for the development.

NPRA decided to use this opportunity to create a "showcase" for innovative ferry design, and wanted to achieve at least 15 - 20% improvement with respect to energy - and environmental efficiency compared to today's mainstream construction. NPRA was not objectively able to specify technical means to obtain such objectives, but wanted the industry to focus on energy efficiency by reducing fuel consumption and environmental efficiency through reduced emissions as a result of selected energy carriers and technical solutions.

NPRA decided in April 2011 to utilize the procurement procedure Competitive Dialogue for the granting of permits and contract for operation of the ferry service. The competition included operation for a period of 10 years, starting 1st January 2015. The ferry service shall be operated by three ferries, of which the “Development Ferry” shall be the main one.

## 2.3. Crossing of the Sognefjord, Feasibility Studies

NPRA has been commissioned to investigate the potential for trade and industry, regional employment and settlement patterns of eliminating all ferries along the western corridor (E39) between Kristiansand and Trondheim. Further, this project will explore the technology required for the remaining fjord crossings.

The fjord crossing study explores technological alternatives for the eight fjord crossings still being operated by ferries. The Sognefjord, which is about 4 km wide, is considered to be a relevant pilot site for developing new concepts for extreme bridges. With its vast depths of up to 1300 m and 200-300 m of bottom deposits above the rock, the Sognefjord is considered the most difficult and challenging fjord to cross. The depth is rather extreme; the other fjords along the route are more typically some 500 – 600 m deep. Different depths and lengths may require different concepts and solutions, and the technological implications and costs related to anchoring systems for floating structures will be of particular interest.

The fjord crossings study is looking into three main alternatives for the Sognefjord:

- a suspension bridge,
- a floating bridge, or
- a submerged floating tunnel.

Combinations of the three are also being considered.

The studies require cutting-edge national and international knowledge, including experience and technologies from marine and offshore installations and operations. Studies also include risk analyses related to shipping operations on the Sognefjord for the different alternatives. NPRA has not been objectively able to specify neither needed qualification nor technical means necessary to create constellations and concepts as substantiating basis for the feasibility studies of alternatives for floating bridge and submerged floating tunnel. It was therefore in August 2011 decided to utilize the procurement procedure Competitive Dialogue based on a prequalification and a functional description to find the right experts and the right concepts as basis for these two feasibility studies.

## 3. THE PROCEDURE

Competitive dialogue is a procedure in which any economic operator may request to participate and whereby the contracting authority conducts a dialogue with the participants admitted to that procedure, with the aim of developing one or more suitable alternatives capable of meeting its requirements, and on the basis of which the participants chosen are invited to tender. The procedure is governed by the Law[6] and Regulations[7] relating to public procurement, and may be relevant when awarding particularly complex projects.

The procedure is governed by Section 4-2, litra c) of the Regulations:

> *procurement procedure where the contracting authorities, in one or more rounds, carries out a dialogue with suppliers regarding alternative solutions before competing tenders are submitted*

The conditions for use of the procedure are governed in Section 14-2 of the Regulations:

> *(1). Contracting authorities can use competitive dialogue when awarding particularly complex contracts.*
>
> *(2). A contract is particularly complex when the contracting authorities are not able to:*
>
> > *(a) .....to objectively and precisely define the technical means satisfying their needs and objectives, or*
> >
> > *(b) to objectively and precisely define the legal and/or financial make-up of a project.*

The procurement procedure entails that each individual participant can suggest the solution that it finds most suitable. The procedure includes a dialogue phase where the Client engages in dialogue with each individual participant about the proposed solution, with an eye to possibly further developing it. This will provide opportunities for creative solutions and optimal implementation.

The dialogue phase will be characterized by which solutions the participants have presented, and how the Client responds to these solutions. It is likely that this phase will be split into partial phases with more rounds of dialogue where some solutions and participants may be abandoned along the way. During the dialogue, all aspects of the contract can be discussed with the admitted participants. The Client must ensure that all participants are treated equally. In particular, the Client must make sure that there is no differential treatment in the form of providing information which could give some participants a better position than others. All information which comes from the Client must be presented simultaneously and in the same manner to all participants in the dialogue. The Client may not reveal to the other participants solutions or other confidential information which one participant has given to the Client, without that participant's consent.

When the Client has identified the solution(s) which fulfil(s) the need, the Client declares the termination of the dialogue. The Client then invites the participants to submit their final tenders based on the solutions which have been presented and possibly modified in the dialogue phase. For this work, the participants will be tendering for a general contract.

[6] LOV 1999-07-16 nr 69: Lov om offentlige anskaffelser.
[7] FOR-2006-04-07-402: Forskrift om offentlig anskaffelser.

Project-specific award criteria will be prepared, and these shall be used to determine which proposed solution will be most suitable for this project. The contract award will take place on the basis of which tender is the most economically advantageous.

## 4. EXPERIENCES AND RECOMMENDATIONS

Given the challenges identified and the experience gained after the implementation of “Competitive Dialogue”, NPRA recommends following conditions to be emphasized during future application of the procurement procedure

Anchoring
Managers, professionals and experts in all relevant organizational units should be involved as early as possible. Measures should be implemented to ensure:

- Knowledge of relevant sections of Laws and Regulations for public procurement.
- Understanding the conditions for the use of Competitive Dialogue.
- Involvement in the choice of procurement procedure and implementation of this.
- Acceptance and understanding of the consequences, when Competitive Dialogue is the chosen procedure.

Application of Competitive Dialogue as procurement procedure should be verified legally before implementation. It further strengthens the preparation and implementation if general and project-specific information relating to the procurement procedure can be supported by professionals with relevant legal expertise and experience

Organizational structure
Implementation of Competitive dialogue is time - and resource-intensive. To ensure a timely and adequate implementation, the procurement process should be organized as a separate project. At least it is appropriate to coordinate preparation of documents and decisions related to the procurement process in a single purpose organizational unit, with a leader who does not have parallel tasks or responsibilities in the affected units.

Marketing
It is a condition for the use of Competitive Dialogue that the Client is not able to clarify all means and matters related to the specific project. Thus it will per se be more than the customary uncertainty associated with project implementation. NPRA recommend a targeted information campaign to ensure that the market is given reason to interpret this uncertainty as an attraction rather than a threat.  This requires that the Client clarifies uncertainties as identified, in nature and scope. It should be further clarified for the market to what extent the participant’s expertise can be used for competition.

Basic responsibilities
The participants should bear complete responsibility for the contract work based on their final tenders and their individual solutions presented and specified during the dialogue. It is although conceivable that the Client prepares a contract basis based on elements from several solution proposals, and that the Client shall bear the engineering responsibility.
This requires, however, that participants have previously given their consent. One common solution will probably undermine the participant’s ability to utilize their own expertise to competitive advantage, and it is doubtful whether this would be appropriate. Responsibilities

related to the choice and convenience of the solution should be clarified before the competition starts.

Announcement

It should be stated in the announcement if there are conditions related to the contract that is already established and therefore will not be open for dialogue. Similarly, avoid describing conditions in the announcement that can lead to unintended restrictions with respect to proposal of solutions and the later dialogue phase.

Requirements - contract work

To gain a successful implementation of Competitive Dialogue it is necessary that the Client prepares clear and unambiguous descriptions of his needs and objectives – Functional Descriptions. These descriptions should clearly reflect the issues where the Client is not able to define terms and conditions to fulfil needs and objectives, and thus support the conditions for use of the Competitive Dialogue. It is also important in this context to avoid specifications that can lead to unintended restrictions in the dialogue phase.

It is recommended that scope of work for the final tender to the smallest extent possible include deliveries and items beyond what is covered by the dialogue phase.

Award criteria

Where the Client choose to award a contract to the most economically advantageous tender,
he shall assess the tenders in order to determine which one offers the best value for money. In order to do this, the Client shall determine the economic and quality criteria which, taken as a whole, must make it possible to determine the most economically advantageous tender for the Client. The determination of these criteria depends on the object of the contract since they must allow the level of performance offered by each tender to be assessed in the light of the object of the contract, as defined in the functional description and the value for money of each tender to be measured. The award criteria shall be described in the announcement or latest in the invitation to the dialogue phase, not yet knowing the spread of solutions. The criteria shall be the basis for all the reviews, priorities and selections with regard to the participant’s solutions and services made by the Client during all phases of the procurement. It is therefore very important to do a thorough analysis of needs and objectives before description of the award criteria, to ensure that all relevant factors are taken into account with reasonable weight and to avoid unintended effects in subsequent evaluations.

Requirements – proposals for solutions

It is recommended for future implementation of Competitive Dialogue to start the dialogue phase on the basis of the participant’s documented proposals for solutions. Requirements should be described primarily to ensure that the participant’s proposal addresses the project specific issues underlying the choice of procurement procedure. It is appropriate to avoid that the requirements entail demanding work that participants can perform just as well later in the procurement, or which may be performed after contract award.

Participants - remuneration

Participants’ estimates with respect to use of resources in the procurement process may be a contributing factor when selecting where to take part among several opportunities in the market. With reference to the Regulations it allowed to enter into a remuneration agreement with the participants in the dialogue phase. It is important that the Client makes a thorough analysis of to what extent and how this instrument may be used for future implementation of Competitive

Dialogue. Analysis should be based on the significance of means may involve long-term achievement of the Client.

Process management

It is a challenge to separate process management from project development, and to put emphasis on the former. The main Client task during the dialogue phase is to manage the process, where the respective participants' project developments are parallel elements that are part of the total process. Important instruments in this management should be:

- Defined goals.
- Defined schedule.
- Detailed plans for dialogue meetings
- Written feedback.
- High degree of documentation.
- Effective distribution of information.
- Administrative procedures.
- Systematic and documented risk assessment.

Confidentiality and trust

Representatives of the Client should be selected with great emphasis on the ability and willingness to act with great integrity and credibility in internal and external communication.

## 5. RESULTS AND ADDES VALUE

NPRA believes that the implementation of Competitive Dialogue has led to results and added value beyond what one could have expected with the use of traditional procurement procedures

### 5.1. General

Project organization - development

NPRA project personnel are extremely well prepared to carry out the demanding contract works, as a result of going through the dialogue phase with all participants. Procurement processes have beyond that caused enhanced involvement of central and regional management in the NPRA. This has contributed to good decision making and that issues related to the procurement process, contract terms and contract award are well anchored on all levels within NPRA.

Market contact

Implementation of Competitive Dialogue has contributed to the establishment of good relations with experts, professionals, advisors and other suppliers, otherwise less available for NPRA.

Representatives of NPRA have received new and useful experience regarding supplier's responsibilities and actions in the procurement process, and there are reasons to believe that the participants have improved their understanding of public procurement and NPRA tasks.
Increased knowledge and understanding of the other part's roles and responsibilities will in general be positive contributors to the establishment of future relations and cooperation.

Relations between the contracting parties

Implementation of Competitive Dialogue has led to cooperation between NPRA and project participants' project organizations for a long time before contract award, and given unique opportunities to establish early relationships that have strengthened the cooperation and the project development in the further implementation of the contract.

Contract works – risk management
NPRA was particularly motivated to describe projects' needs and objectives prior to the procurements, due to implementation of Competitive Dialogue. These were done in the form of functional descriptions. Engineers fall easily to the temptation to describe a solution, rather than to provide premises for the solution. The needs to describe the works in terms of premises have initiated analysis and uncertainty assessments that may have reduced the likelihood of adverse events to a greater extent than if it was chosen to describe solutions.

The consequences of the projects' needs and objectives have been systematically identified and assessed, through these analysis and risk assessments performed incrementally and in parallel with several proposals in dialogue with all participants. Corrective actions are taken as necessary. It is doubtful whether use of other procurement procedures could have resulted in similar thoroughly development before contract award.

Responsibilities
Responsibilities related to the implementation of the Contract have been thoroughly investigated in dialogue with the participants before contract award. This has led to greater awareness of the responsibilities than normally achieved by the use of other procurement processes.

Development of solution – feasibility
All project specific risk factors that led to the choice of procurement procedure have been thoroughly analysed. Significant risk factors is eliminated or reduced through Competitive Dialogue, and the respective solutions have been systematically developed with the aim to define solutions that surely will meet the project's needs and objectives. This has resulted in increased feasibility, caused by following factors:

- Development of the projects' critical elements
- The establishment and development of risk management model
- Development of NPRA organization
- Development of the contractors' organization

### 5.2 Projects

E6 Trondheim-Stjørdal, Nidelv Bridge – Grillstad, Western surface work zone
NPRA received three requests for participation in the Competitive Dialogue:

- NCC Construction, with the assistance of geotechnical specialists at Sweco. Earthworks and foundations performed by the subcontractors Kynningsrud Fundamentering (a subsidiary of Veidekke) and Hercules Grundläggning (subsidiary of NCC).
- Bilfinger Berger Ingenieurbau GmbH, with the assistance of geotechnical specialists at NGI.
- Joint Venture AF Gruppen / Züblin, with the assistance of geotechnical specialists at Multiconsult.

NPRA signed a contract with NCC Construction 26th October 2009. Design of the contract work is completed, and all critical work in the sensitive zone is completed with good results. Backfilling and reassembly of temporarily relocated buildings above the tunnel have started. It is expected that the tunnel opens for public traffic in October 2013.

E39 Lavik – Oppedal, Operation of Ferry Service.
NPRA received four requests for participation in the Competitive Dialogue:

- Torghatten Trafikkselskap, with assistance from Nordnorsk Skipskonsult.
- Tide Sjø (later Nordled), with assistance from respectively Fjellstrand, LMG Marin and Westcon Power & Automation and AGR.
- Fjord1 Fylkesbaatane (later Fjord1), with assistance from Multi Maritime
- Boreal Transport Nord, with assistance from STX Europe

NPRA signed a contract with Nordled on 9th November 2012, based on their battery ferry with catamaran hull of aluminum. Design and engineering are in progress, and it is expected that Development Ferry will be ready for operation as planned on 1st January 2015.

To describe the diversity, it may be added that NPRA received solutions with the following energy carriers:

- Batteries
- Super Capacitors
- Hybrid - batteries and gas
- Hybrid - biogas- and LNG-generator
- Hybrid - biodiesel- and dieselgenerator
- Hybrid - biogas/fuel cell/battery
- Hydrogen Fuel Cell

These energy carriers were in different solutions also supplemented with ORC-heat recovery systems, Flettner-rotors, Solar panels etc.

Crossing of the Sognefjord, Feasibility Studies
NPRA received six requests for participation in the Competitive Dialogue:

- Croup consisting of Aas-Jakobsen, Johs. Holt, Cowi, NGI and Skanska Norge
- LMG Marin with subcontractors Marintek, Smidt & Ingebrigtsen, Teknisk Data and Fondasol
- Multiconsult
- Norconsult, with subcontractor Haug og Blom-Bakke
- Group consisting of Reinertsen og Dr.techn Olav Olsen
- Sweco, in cooperation with Instanes Polar, Hans-Petter Brathaug og Sivilingeniør Andres Bleie.

In July 2012 NPRA signed contracts with respectively Aas-Jakobsen Group regarding feasibility study of a floating bridge, and Reinertsen/Olav Olsen regarding feasibility study of a submerged floating tunnel. The feasibility studies have been completed with promising results. The studies are presented in more detail in separate papers.

## 5.3 Conclusion

NPRA believes that the overall objectives are met in all three projects:

- The most adequate qualifications have been made available for the project development.
- There have been good development processes that give reasons to expect that the selected solutions will satisfy the project's needs and objectives.
- NPRA has entered into contracts with suppliers who through Competitive Dialogue have proved competence, capacity and cost efficiency.

- Established a practice for implementation of Competitive Dialogue as a possible procurement procedure in NPRA.

# CROSSING THE DEEP AND WIDE FJORDS ON THE WESTERN COAST OF NORWAY WITH FIXED CONNECTIONS

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## ABSTRACT

*The Coastal Highway Route E39, running from Kristiansand to Trondheim, crosses seven of the most extreme fjords on the western coast of Norway. Currently, these fjords are crossed by ferries, but the Norwegian Public Roads Administration plans to replace the ferry connections with fixed links. These fjords vary from 2 to 8 km in width and 300 to 1300 m in depth, many of them exposed to harsh weather conditions. In addition, the bridge constructions need to allow free passage for cruise liners visiting the fjords. Consequently, fixed connections across these fjords are beyond any bridge structures built as of today.*
*As a part of the Coastal Highway Route E39 Project, a component project, the Fjord Crossings Project was established to develop the technology required for crossing these extreme fjords. The Fjord Crossings Project has investigated three principally different ways of bridging the fjords; super-long suspension bridges, floating bridges and submerged floating tunnels. Within all of these different bridging concepts, there is a great variety of alternative solutions. Applying offshore technology in combination with traditional bridging technology has shown to be a key factor in developing new technology for fjord crossings. Existing technology from floating platforms, in combination with fixed bridges like suspension bridges, represents one alternative way of designing floating bridges.*
*The paper gives a brief overview of the challenges met, and the findings so far.*

## THE EXTREME NORWEGIAN FJORDS

The seven fjords crossings along the Coastal Highway Route E39 (see fig. 1) are unique, compared to straits, sounds or inlets in other parts of the world. Many of the crossing sites are exposed to wind and waves from the North Sea. The fjords are visited by a number of cruise ships every year, and a fixed connection should not restrict future ships traffic. The largest of these ships requires a free height of more than 70 m.

The fjords vary in width from about 2 km to about 8 km. This is not extreme values for straits or sounds, looking at width only. Wider straits can be found, and have been crossed with fixed, links both in Scandinavia and the rest of the world. The fjords are, however, very deep. The deepest one, the Sognefjord, is 1300 m at the deepest. Normally, depth is varying from about 300 m to about 600 m.


*Figure 1: The fjord crossings along Coastal Highway Route E39*

## LIFE CYCLES COSTS AND ENERGY CONSIDERATIONS

So far, energy consumption and life cycles costs (LCC) have had little attention when considering different fjord crossing methods. In the future, however, these factors are expected to be of great significance when alternative fjord crossing methods are considered. The cost and energy needed to operate the long and deep subsea rock tunnels is also of growing concern. Preliminary calculations [1] indicates that operation and maintenance of a high standard subsea rock tunnel will be in the order of 1600 NOK/year/m tunnel length. LCC for subsea rock tunnels are calculated to be in the order of 30-50% of construction costs during a period of 100 years. Preliminary studies from 2009 indicates that operating a deep subsea rock tunnel might need as much as 300 kWh/year/m tunnel length [2], and that emission in terms of CO2 and NOx is significantly higher for a tunnel crossing than a bridge crossing [3] (see fig. 2).

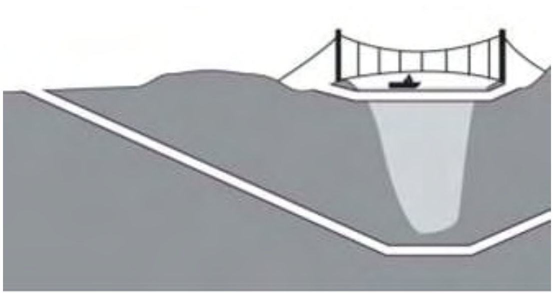

*Figure 2: Alternative fjord crossing methods (rock tunnel, suspension bridge, car ferry, submerged floating tunnel)*

Hence, as the remaining fjords are very deep, the Fjord Crossings Project has focused on bridge structures only, and not subsea rock tunnels.

## RECENT DEVELOPMENT OF FJORD CROSSINGS IN NORWAY AND ABROAD

When it comes to the longest free spans, suspension bridges are superior to cable stayed bridges. The Akashio Kaykio suspension bridge in Japan has a main span of 1991 m, which is the longest in the world. A bridge between Sicily and Italy will have a main span of about 3300 m, according to the plan [4]. In South Korea, the “super-long span bridge” project is looking into ways of building suspension bridges with main span of about 2800 m. One important element in this research is the development of stronger steel in the cables [5].

The Hardangerfjord suspension bridge in Norway, with a free span or 1310 m, might be close to the limits of how long and slender the free span could be before dynamic instability causes a serious problem.

Floating bridges have been in operation for a long time. The longest in the world has a floating section of more than 2000 m. They are normally designed with a continuous series of pontoons, and side anchored to the seabed. There are two floating bridges in Norway, both with separate pontoons. The Nordhordlandsbrua has a 1246 m long floating section, in combination with a cable stayed bridge above the ships channel. This floating bridge is the longest in the world with end anchoring only (see fig.3).

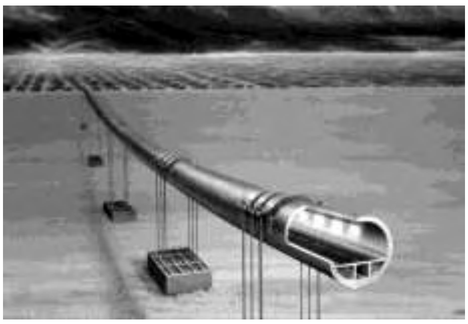

*Figure 3: The Nordhordland floating bridge, and the proposed Høgsfjord SFT*

So far there has not been built any submerged floating tunnel (SFT) anywhere. Nevertheless, this crossing method has been considered for strait crossings for many years. SFT solutions have been proposed in different countries, especially Italy, Japan, China and Norway [6]. During the 1980-s and 90-s, a 1400 m long submerged floating tunnel (SFT) was planned for a fixed connection across the Høgsfjord in Norway. Four different solutions were considered, one anchored to the seabed at 155 m depth, and three solutions with pontoons at the sea surface. The SFT was approved as solution for the crossing, but was never built due to political reasons [7] (see fig. 3).

## THE CHALLENGES

All of the remaining fjord crossings along the Coastal Highway Route E39 are beyond any bridge structure built as of today. There is no bridging project in the world today that can meet the challenges with regard to width, depth, environmental conditions or ships traffic, that are to be met when considering the Norwegian fjord crossings (see fig. 4). Consequently, limits of existing bridging technology had to be broken, and new technology has to be developed, in order to establish fixed connections across these extreme fjords.

*Figure 4: The wide and deep Sognefjord*

Normally, a fjord is seen as a barrier. However, the possibility of using buoyancy of floating elements to carry loads from bridges, turns the fjord into a possibility, not only a barrier. Within the offshore industry, floating platforms, anchored to the seabed, have been operating under very rough environmental conditions in the North Sea, and all over the world.

One major question has been how knowledge and experience from offshore structures and anchoring techniques could be applied to bridging structures across the deep fjords [8].

All the fjord crossings are unique, consequently, different crossing methods had to be developed. Hence, it was decided that suspension bridges, floating bridges, submerged floating tunnels, and possible combinations of these, should be developed further.

## SAFETY CONSIDERATIONS

When considering new alternative fjord crossing methods, safety is a fundamental aspect. New concepts, exceeding the limits of existing bridging structures, bring up new sets of questions to be answered. Three major aspects should be mentioned:

1. Safety of the people, car drivers and passengers, and ship crew and passengers.
2. Safety of the bridge structure and ships.
3. Safety of the environment.

Ship collision might in worst case be catastrophic to any floating structure. If the risk of ship collision is considered higher than the acceptable level, the designs are made for absorbing collision energies without the risk for fatal accidents for road traffic as well as for passengers or crew on-board ships. To avoid critical damage on a floating bridge or a submerged floating tunnel, vital elements must be redundant.

## THE FEASIBILITY OF CROSSING THE SOGNEFJORD

To develop the technology needed, the Sognefjord, was chosen as case in a feasibility study. The aim of this study was to prove that extreme fjord crossings like this can be built. At the crossing site, the Sognefjord is 3700 m wide and 1300 m deep. A number of cruise ships are visiting the fjord every year. Hence, the navigable ship channel has to meet the requirement of 70 m free height, a free depth of 20 m, and a free width of 400 m.

The feasibility study started with a number of alternative solutions, and was completed with three main bridging concepts: suspension bridge in one span, floating bridge, and submerged floating tunnel.

A group of bridge engineers from the Norwegian Public Roads Administration has studied the suspension bridge solution. Their conclusion is that it is possible to cross the fjord in one span. To solve the problems with aerodynamic instability, the box girder has to be divided into two parallel girders, connected with a framework [9] (see fig. 5).

*Figure 5: Suspension bridge across the Sognefjord*

As the NPRA has limited experience with regard to design of floating structures, engineering companies were invited to develop one floating bridge concept, one submerged floating tunnel concept, or a combination of these. Competitive dialogue was used, in order to develop the solutions to be studied in a dialogue between the NPRA and the bidders. The experience from the competitive dialogue is very positive. During the dialogue the client and the bidders established a good relationship, which resulted in a set of different possible crossing alternatives.

The Aas-Jakobsen group was chosen to study a floating bridge alternative. Their alternative solution was a floating bridge which consisted of a suspension bridge in three spans, with two pylons set on floating pontoons, anchored to the seabed in the middle of the fjord [10] (see fig. 6).

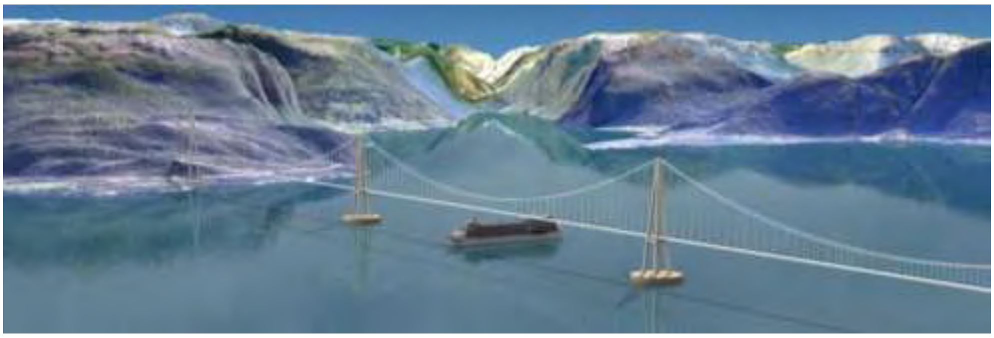

*Figure 6: Floating bridge across the Sognefjord*

The Reinertsen/Olav Olsen group was chosen to study a submerged floating tunnel alternative. Their alternative solution was two tubes, connected with a framework, formed like an arch. The tubes are kept in position by connections to pontoons at the surface, and fixed connections to the shore on both sides of the fjord [11] (see fig. 7).

The feasibility study shows that it is possible to cross the Sognefjord with a fixed connection. Furthermore, that there are several crossing methods to choose between: a suspension bridge in one span, a floating bridge, or a submerged floating tunnel.

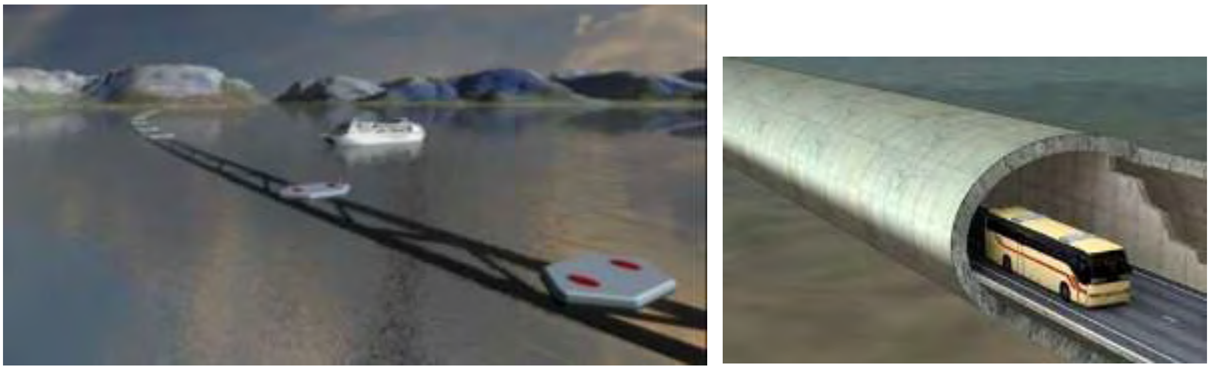

*Figure 7: Submerged floating tunnel across the Sognefjord*

A risk assessment study, made by Rambøll, shows that both the floating bridge and the submerged floating tunnel can meet safety requirements for the structure, and also regarding the ship traffic [12],[13].

The feasibility study has shown that there are numerous alternative solutions of how to design a floating bridge and submerged floating tunnel. This is a very important result of the study, as it means that at set of alternative solutions will be available, not only for a fixed connection across the Sognefjord, but for any of the fjords along the Coastal Highway Route E39. All the fjord crossing sites are different in one way or another, consequently it is important to have a set of solutions to choose between [14].

## CROSSING THE BJØRNAFJORD

The Bjørnafjord is 5 km wide at the narrowest, and thus even wider than the Sognefjord. The depth is about 550 m. Consequently, the solutions from the Sognefjord feasibility study cannot be copied directly. Crossing the Bjørnafjord, it was reasonable to look for alternative solutions, anchored to the seabed in one way or another.

In the deepest part of the fjord the seabed consists of soft sediments, thus anchoring technology from floating platforms in the North Sea might be used in this part of the fjord. In the shallower parts the ground is solid rock and no soft sediments. At the north side of the fjord there is a rock plateau, with depths varying from 30-150 m.

With a main foundation at 120-150 m depth, the main span of a suspension bridge will be about 3300 m, which is less the span across the Sognefjord. It is, however, a great challenge to put one of the pylons on a foundation at 140 m depth.

Due to the width of the fjord, various floating bridge alternatives have been studied. One is a floating bridge, based on a suspension bridge with towers on one or two floating pontoons in the middle of the fjord.

*Figure 8: Floating bridge (TLP) across the Bjørnafjorden*

In this case the main spans of the suspension bridge will be shortened to about 1500 m or less. On the other hand, it will be a multi-span bridge with pylons on floating pontoons. Due to the depth and the ground conditions this alternative solution is based on tension leg platform (TLP) technology. The proposed pontoons are small TLP-platforms with vertical tension anchoring to the seabed (see fig. 8).

*Figure 9: Side-anchored and end-anchored floating bridge across the Bjørnafjorden*

Two more traditional floating bridge alternatives are a side-anchored alternative, and an end-anchored alternative. For both of these alternatives, the ship channel is placed in the northern part of the fjord where the floating bridges are combined with a traditional cantilever bridge or cable-stayed bridge. For both of the alternatives, the floating bridge part will be 4000 m or longer. Separate pontoons will be used. For the side-anchored bridge there might be a number of alternative designs, like distance between the pontoons, anchoring system (two-sided anchoring, or one-sided anchoring), number of anchors, and so on. For the floating bridge alternative with end anchoring only, the carriageway / box girder is split in two parallel boxes connected with a framework, the horizontal alignment following a circular arch (see fig. 9).

It is also possible to cross the Bjørnafjord with a submerged floating tunnel. Due to the length, a single-tube, side-anchored solution is proposed. In the northern part of the fjord, where the depths to the rock plateau are in the range of 80-30 m, the proposed SFT is resting on pillars to the rock.

Cost estimates indicate great variations between the different alternative solutions. The side-anchored floating bridge is calculated to approximately 9 billion NOK, while the single-span suspension bridge is calculated to approximately 20 billion NOK.

The result from the Bjørnafjord study has been very important in the ongoing discussion about the location of the ferry-free Coastal Highway Route E39 in the region between Bergen and Stavanger. Until recently it was assumed that a crossing of the Bjørnafjord with a fixed highway connection would not be possible in the foreseeable future. Hence, a ferry-free E39 was proposed to be located east to the fjord. Now, however, as it is possible to cross the fjord with a fixed link, a straight and much shorter location of the ferry-free E39 turns out to be the most favorable location [15].

## CROSSING THE BOKNAFJORD

The Boknafjord, just north to Stavanger, is the most extreme of all the fjords along E39. At the narrowest it is approximately 7,5 km wide, and the deepest part in this area of the fjord is about 550 m deep. In addition the fjord is open to wind and waves from the North Sea. This fact is a real challenge, and it was obviously that not all of the alternative solutions from the studies of other fjords would be feasible. In addition there is heavy ship traffic in the area, including a great number of gas-tankers. Consequently, possible restrictions on ship traffic should be minimized.

The NPRA ended up with a solution, based on experience from offshore industry in the North Sea [16]. Here vertically anchored TLP-platforms are located under extreme weather conditions. The study was limited to a floating bridge, a solution based on a multi-span suspension bridge on a series of pontoons in the fjord. With four TLP-like pontoons, vertically anchored to the seabed in the middle of the fjord, the multi-span suspension bridge has five spans, each span 1400-1450 m long. With a free sailing height under the bridge of 70 m, the pylons will rage 220 m above the sea. The TLP-pontoon will have a depth of about 70 m, thus the total height of the pontoon and the pylons will be close to 300 m. The box girder has a total width of approximately 25 m.

With regard to pontoon and pylons, both steel and concrete solutions were considered. The steel option seemed to be the most favorable, both when cost and production time were considered. The steel pylon and platform might be built in one operation on a yard, brought to the site on a barge, tilted and installed (see fig. 10).

*Figure 10: Floating bridge (TLP) across the Boknafjord (concrete and steel pontoons)*

Total cost of the floating bridge across the Boknafjord was estimated to approximately 30 billion NOK. Operation and maintenance cost over a period of 100 year was estimated to be about 4% of production cost, when discounted to the year of opening.

Mainly due to the cost of the bridge, it is decided to build a 25 km long subsea rock tunnel, going down to approximately 390 m below sea level. This will be the longest subsea rock tunnel for highway traffic in the world.

## FURTHER WORK

All the three main concepts, the super-long suspension bridge, the floating bridge, and the submerged floating tunnel, require further studies to verify technical solutions and develop various alternative designs. All of them are beyond any bridge structure built as of today. Consequently, design standards, analytic tools and technical approval regimes have to be developed. Even though there seems to be great variations in construction cost, it is not recommended to eliminate any of the three main concepts at this time. The many crossing sites are so different with regard to geographic and environmental conditions, requirements regarding ships traffic and environmental requirements, that none of the concepts on a general basis will be supreme to the others.
Further technical research and development are proposed undertaken during the period 2014-2018. It is recommended to be organized in a research and development project, mainly funded by federal grants. This program will include design, materials, instrumentation and monitoring systems, construction, operation and maintenance, and testing regimes [14]. Safety has to be an important factor, especially due to the risk of ship collisions. The project is recommended administered by the NPRA, but universities, engineering companies, contractors and other, will be involved. The main objective is to provide access to technology that will enable construction of the fjord crossings along the Coastal Highway Route E39, or elsewhere. The project also aims at recruiting skilled personnel to NPRA, universities, engineering companies and contractors.

## CONCLUSION

The general conclusion is that all the fjords along the Coastal Highway Route E39 can be crossed with a bridging structure. In most cases super-long suspension bridges, floating bridges and submerged floating tunnels are feasible. Furthermore, the study shows that there are numerous alternative solutions of how to design a floating bridge or a submerged tunnel.
A research and development project is recommended to bring all the three main crossings methods to the level needed to start design and construction of the different projects along the route.

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# BRIDGE CROSSINGS AT SOGNEFJORDEN – ENSURING TECHNICAL FEASIBLE AND SAFE SOLUTIONS

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## ABSTRACT

*The Norwegian Public Roads Administration, Statens Vegvesen (SVV) has been given the job to improve the existing E39 road between Kristiansand in south and Trondheim in north such that the eight existing fjord crossings being a part of E39 – presently ferry crossings – are replaced with fixed crossings either as tunnels or as bridges.*

*Some of the crossings are highly complicated due to the large water depths and the width of the fjords. For this reason, SVV has asked Ramboll to carry out ship collision risk studies for two different bridge designs for the Sognefjorden crossing – a 3.7 km wide and up to 1.3 km deep fjord - in order to determine potential ship collision accidents and corresponding consequences due to ship collisions.*

*The present paper describes the applied methods - using the initial ship collision frequency results – to determine significant risk contribution – specifically the risk related to the ship traffic. Risks are in this context formulated in terms of safety for passenger and personnel onboard the ships. Furthermore, environmental risks related to oil spillage as a consequence of a ship collision with the bridge will be determined.*

*The consequence study takes into account different consequence modeling techniques, e.g. fault tree modeling and Bayesian network modeling in order to give estimates of the consequences of ship collisions related to the considered bridge designs. Consequences are formulated in terms of human safety (fatalities) and in terms of environmental safety (oil spillage, dangerous substances). The calculated risk results will be compared to established risk acceptance criteria to ensure that the suggested design will not pose unacceptable high risks for the ship traffic.*

## INTRODUCTION

The existing E39 road in Norway between Kristiansand in south and Trondheim in north shall be improved such that the eight existing fjord crossings being a part of E39 – presently ferry crossings – are replaced with fixed crossings either as tunnels or as bridges. Some of the crossings are highly complicated due to the large water depths and the width of the fjords.

For this reason, SVV has asked Rambøll to verify that establishing a fixed crossing for these fjords is technical possible and safe for the users. Rambøll has been involved in many large bridge and tunnel projects including e.g. Øresund Bridge and Tunnel (DK/SE), Femern Belt

Immersed Tunnel (DK/DE), Forth Replacement Crossing (UK) preparing design requirements and demonstrating that risk is acceptable for users of the bridge, for third party and for the environment.
For use in these projects Rambøll has established a ship collision risk model, [1], [2] and [3], that determines collision probabilities and consequences based on given information regarding bridge geometry, bathymetry, geography, current and weather and also based on quantification of human errors.

SVV has specifically prepared preliminary designs for two types of crossings at Sognefjorden. The ship collision frequency assessment related to these designs is described in details in ref. [4]. The present paper summarizes these results and describes the risk modeling aiming at ensuring that the risk related to ship collision will be acceptable. The risks are expressed in terms of human risk (fatalities per year) and in terms of economic risk (NOKK per year) from property and environmental damage. The calculated risks are compared to established risk acceptance criteria.

## BASIC INFORMATION

Sognefjorden is Norway's longest, and the world's second longest fjord. It stretches 205 kilometers and is up to 1300 m deep. Part of Sognefjorden and the location of the proposed fixed link are seen below in Figure 1.


*Figure 1. Proposed fixed link crossing of Sognefjorden.*

The large water depth makes bridge solutions far more feasible than tunnel solutions below seabed. However, the bridge alternative forms obviously an obstacle to the ship traffic and in order to estimate whether this obstacle poses an acceptable or unacceptable risk to the users of the bridge, to the marine traffic and to the environment, a risk model has been established.
The ship collision risk model relies on a set of input that has to be available and described. This includes bridge design (shape and location of structural elements), geographical and meteorological information and ship traffic information – number and types of ships and location on routes. Finally, failure modes (human and technical failures) for ships on the routes must be described. This information is given in the following sections.

### Bridge design

Sognefjorden is a very deep fjord – up to 1.3 km deep. This means that usual bridge designs founded on the seabed is not possible. In order to account for this the proposed design of bridges are floating bridge designs using pontoons as supports. Two different designs have been investigated – a floating bridge with piers supported on pontoons and a submerged floating tunnel (SFT) where tunnel tubes are kept in place by fixation to floating pontoons. Examples of the two conceptual designs are shown in the figure below.

*Figure 2. Conceptual bridge designs. Floating bridge (left) and submerged floating tunnel – SFT - (right).*

The floating bridge is supported on two floating piers with a distance between of 1233 m. The SFT is supported by 16 pontoons. The distances between pontoons vary from 175 m to 430 m in the middle span.

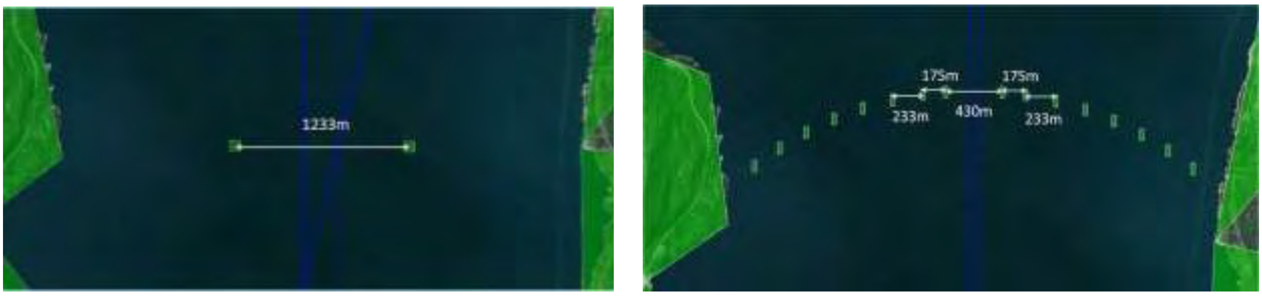


*Figure 3. Pontoon distances. Floating bridge (left) and SFT (right).*

The floating bridge has concrete pontoons circular shaped with a diameter of 75m. The SFT has steel pontoons rectangular shaped (80m x 30m) with length axis parallel to the main sailing route.

**Geographical and meteorological information**

All coastlines and water depths are stored with coordinates such that grounding issues can be included in the analysis. Wind and current influences drifting behavior of ships with failure on propulsion machinery. A wind rose from the wind station Takle, the nearest weather station around 10km west of the location of the fixed link, has been used to estimate current patterns in the vicinity of the bridge crossing.

**Ship traffic**

The ship traffic in Sognefjorden has been described based on analysis of registrations of ship movements in 2010. The ship movement registrations are based on AIS (Automatic Identification System) data. The AIS is an automatic tracking system used for identifying and locating vessels by electronically exchanging data with other nearby ships and AIS Base stations.

In the area where the bridge are planned to cross Sognefjorden there are presently four major sailing routes. 1) The main sailing route for commercial ship traffic is located in the centre of Sognefjorden. 2) High speed passenger crafts (HSC) go from Bergen to Sognefjorden and use a sailing route closer to the northern coast line. 3) Local traffic to Instefjord uses a sailing route close to the southern coast line. 4) The ferry route crossing Sognefjorden as a part of the E39 connection from Kristiansand to Trondheim. In order to use the ship traffic data in the risk model, the registered number of ships on each route is assigned to a GT class (1-9) according to their size and a traffic forecast for 2030 have been carried out. Forecasted values are based on the registered present values and corrected in order to account for national economic development and local initiatives concerning cruise ship operations. In Figure 4 is shown an intensity plot of the registered AIS data and an annual counting of ships on the three main routes in the area together with forecasted annual numbers of passages divided into ship classes on the routes.

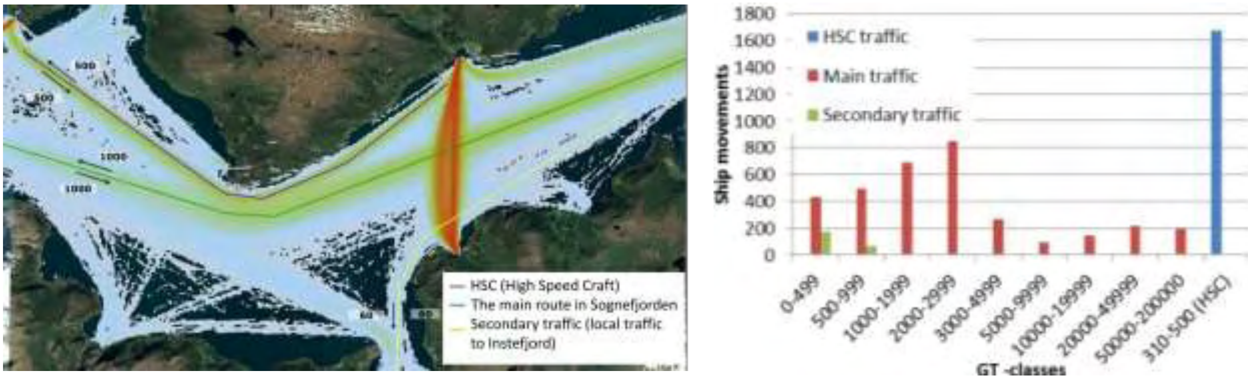


*Figure 4. Intensity plot of ship traffic in Sognefjorden.*

## SHIP COLLISION RISK MODELING

In connection with design and construction of the Fehmarnbelt Link – proposed bridge and tunnel designs for a fixed link between Denmark and Germany – detailed sip traffic studies have been made and an advanced ship collision risk model have been established, Mølsted Rasmussen et al. [1]. This reference reviews existing models, Fujii [5], Macduff [6] and Terndrup-Pedersen [7], and estimated data input, Randrup-Thomsen [8], in renewing the ship collision risk model.

The risk model deals with a set of ship accident scenarios including e.g. groundings, ship-ship collisions and ship-obstacle collisions. The risk model determines collision frequencies and collision consequences. Modeling for both frequencies and consequences are given in the following.

### Frequency modeling

The basic concept in the risk modeling is that the ships may – based on the location on the considered route – be on collision or grounding course, but will normally make proper evasive actions such that an accident does not occur, Terndrup-Pedersen [7]. An accident only occurs in cases, where a failure occurs and an evasive action is not made. Hence, the frequency of an accident relates to the two probability contributions: 1 'The probability of a ship being on collision or grounding course' and 2 'The probability that the navigator(s) does not make evasive actions in due time'. Hence, the risk model is based on a modeling of ships on defined routes and a modeling of ship behavior on these routes.

*Route modeling*
Obviously the routes are changed compared to the current ship pattern due to the presence of the bridge. The total ship traffic volume must therefore be split into a set of defined routes. Possible routes have been discussed with pilots from The Norwegian Coastal Directorate (Kystverket) having large experience in maneuvering in Fjords. Based on their statements and based on experience from AIS registrations from similar bridge crossings, the routes shown below are applied to the model.

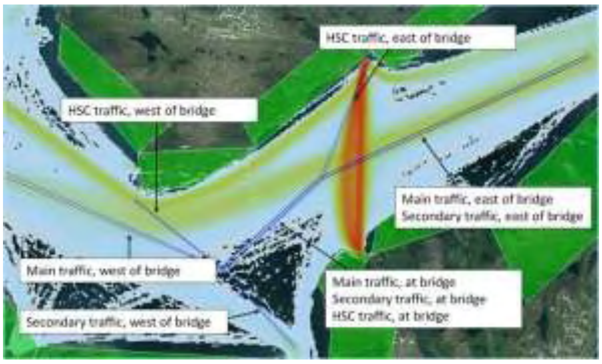


*Figure 5. Intensity plot of current ship traffic in Sognefjorden and estimated sailing routes when bridge is present – floating bridge (left) and SFT (right).*

*Geometrical modeling*
The geometrical modeling of location of ships on the described routes is based on knowledge on how ships are located on the routes today. Far away from the bridge there will be no difference from today whereas close to the bridge the ship location on the routes is influenced by the presence of a bridge. The geometric modeling of the routes will be based on AIS registrations of actual observations on ships on the routes and a suitable probability distribution fitting to the observations. Close to the bridge the geometric modeling are based on observations from similar route types in Øresund (Øresund Bridge) and in Great Belt (Great Belt Bridge) in Denmark.

*Accident scenarios*
Accident scenarios are driven by two different failure modes - human failure and technical failure (loss of propulsion or steering machine failure).

The probability of human failure – the probability that a ship on collision course does not make an evasive action - is estimated based on a large number of studies referred in details in Mølsted Rasmussen et al. [1]. This includes Fujii [5] and [9], MacDuff [6] and Terndrup Pedersen [7]. The studies all find values for human failures in the region of between $0.8 \cdot 10^{-4}$ and $5.0 \cdot 10^{-4}$. The present model uses an annual failure probability of $2 \cdot 10^{-4}$.

Technical failures are related to situations where the navigator loses control of speed and course. Two scenarios dealing with technical failures are included: loss of propulsion (leading to a drifting ship) and steering machine failure (leading to a ship sailing in circles). According to general ship navigator and engineering judgement, the propulsion machinery on a ship is assumed to fail approximately once during a year in service. Based on this the frequency $f_{drift}$ of loss of propulsion machinery has been estimated to $1.5 \cdot 10^{-4}$ failures per hour per ship. The frequency of failure of the steering system $f_{steering}$ has based on a U.S. investigation [10], been estimated to $6.3 \cdot 10^{-5}$ failures per hour per ship. Examples of failure modes are given in Figure 6 below.

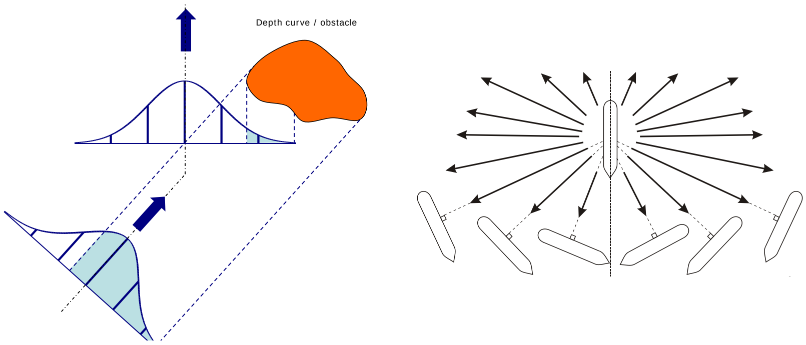


Figure 6 Illustration of failure modes - human failure (left) and failure on propulsion machinery (right)

## Consequence modeling

The consequence modeling is based on a definition of consequence types to be considered. Consequence types are divided into consequences for the bridge and consequences for the ship traffic in case of a ship collision. The types given in Table 1 are used in the risk analysis.

*Table 1. Consequence types*

| Consequence type | | Description |
|---|---|---|
| Fixed link | User fatalities | Fatalities amongst road users - bridge or SFT. |
| | Property damage | Repair or re-construction cost - bridge or SFT. |
| | Disruption | Totally or partly disruption time for the link |
| Ship traffic | Fatalities on ship | Fatalities amongst passengers and crew |
| | Property damage | Cost of repairing ship damages. |
| | Environmental impact | Size spill of bunker or cargo oil in tones. |

*Acceptance criteria – fixed link*

The risk policy for the societal risk for users of the bridge is formulated as 'The risk for road user fatalities shall not exceed the average risk for road user fatalities on Norwegian roads with one lane in each direction and the same traffic intensity and length'. In order to operationalize the risk policy, the average number of fatalities per year on Norwegian roads (bidirectional, one lane in each direction) has been estimated by use of the TUSI-database, ref. [11]. It is found that the average number of fatalities per year on $N_{fat}$ on a bidirectional, one-lane road of a length of 3300 m can be determined to $N_{fat}$ = 0.06 fatalities per year. This average number covers both small accidents with few fatalities and larger accidents with substantially more fatalities.

Risk from the consequence type 'property damage' is measured as expected yearly cost (NOK) of repairing or re-constructing the SFT after ship collision. Risk from the consequence type 'disruption' is measured as the expected number of days per year, where the fixed link is totally or partly disrupted for SFT users as a result of ship collision.

For these consequence types the risk acceptance criterion established is a relative criterion (ALARP: As Low As Reasonable Practicable) which in terms of cost-benefit methods determines whether or not the considered risk is deemed acceptable or whether introduction of risk control options are reasonable. If cost-benefit evaluation shows that introduction of proposed risk control options is not cost-beneficial then, in principle, the risk is considered acceptable.

*Acceptance criteria – ship traffic*

When analyzing the risk from ship collisions to a fixed link across Sognefjorden passengers and crew on ships passing the SFT is considered third party. For the individual risk to third party, a proposed average yearly number of third party fatalities $N_{fat}$ is given in British, Danish and Norwegian guidelines, ref. [12], [13], [14] and [15]. Their values vary slightly around $N_{fat}=1\cdot10^{-6}$

per year. For ship passengers and for ship crew this value is used as an upper bound of the acceptable risk.

Risk from the consequence type 'property damage' is measured as expected yearly cost (NOK) of repairing ship damages after collision. Risk from the consequence type 'environmental impact' is measured as expected volume/mass (tons) of cargo or bunker oil. For these consequence types the ALARP criteria as described above is applied.

Even if the risks related to ship traffic are considered acceptable according to the risk acceptance criteria, it is recommended in the continued design phase to consider involvement of navigators on ships assumed to be passing the SFT (pilots, captains on cruise ships and captains on HSC) in order to discuss further risk control options to be considered.

*Consequences related to the fixed link*
The consequences related to the fixed link, 'user fatalities', 'property damage' and 'disruption', are estimated based on information from the SFT and bridge designers together with information about the road traffic. The consequences are divided in two classes depending on the criticality, namely 'Total loss' and 'Considerable damage'. Obviously, the situation with a total loss is closely connected to the design load and the probability of a total loss is hence defined according to Eurocode specifications, ref. [16] and [17].

Quantification of consequences for total loss and for considerable damage are given separately for the floating bridge and for the SFT in Table 2 below.

*Table 2. Quantified consequence assessment – fixed link*

| | **Total Loss** | **Considerable damage** |
|---|---|---|
| User fatalities (Cost of one fatality ~15.4 MNOK) | It is assumed that all people present on the bridge (SFT/floating bridge) will die in case of a collapse. A collapse will therefore result in 13 fatalities in average (average number of persons on the bridge). | No consequence as the SFT/floating bridge is assumed intact. |
| Property damage | The cost of establishing a new SFT is assumed to be 10 billion and 12.5 billion NOK to establish a floating bridge. | SFT: 45 MNOK if pontoon can be repaired and reused (p=90%) and 145 MNOK if pontoon is lost (p=10%)<br>Floating bridge: 25 MNOK if ship collision is caused by ship in ship class 5 or greater but less than design ship. 60 MNOK if ship is greater than design ship (31.456 tonnes displacement). |
| Disruption (cost of one disruption day ~1.2 MNOK) | It will take around 7 years to establish a new SFT and 5 years to establish a floating bridge. | 14 days for SFT and 2 months for floating bridge. |

The above consequence values from damage are to be updated as design progresses. Fatality costs are taken from Risk Evaluation Criteria, Safedor, ref. [18]. Disruption costs are based on cost considerations for using the bridge and compared to bridges in DK and SE.

*Consequence related to the ship traffic*
The consequences related to ship traffic 'fatalities onboard involved ships', 'property damage', and 'environmental impact' is modeled using a Bayesian network model (BN model). The principle in a BN model is to define probabilities of various system states (speed level, ship size,

damage to ship etc.) in various nodes. The nodes are then related defining conditional probabilities of occurrence of a state in one node given occurrence of a state in a related node. In many cases the conditional probabilities in the Bayesian network are based on engineering judgment and general background information. Therefore, the consequences should be taken as initial consequences. Due to this fact the model is adjusted where it is found necessary, as described below, in order to better reflect the conditions in Sognefjorden.

Hence, the BN model takes input in terms of collision frequencies for different ship classes (size, speed, ship type) and calculates consequences in terms of damage to the ship, environmental damage (volume of spilled oil) and user fatalities. The BN model for estimating consequences in case of a ship collision is shown in Figure 7.

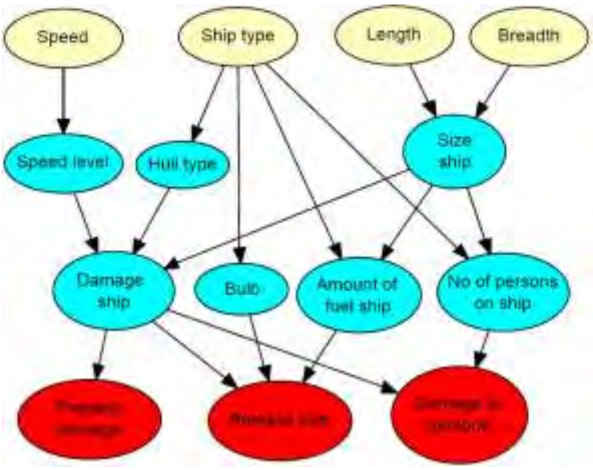


*Figure 7 Bayesian network for estimating consequences in case of a ship collision*

The four yellow nodes "speed", "ship type", "length" and "breadth" are input nodes defining the ship particulars and speed. The blue nodes are intermediate nodes where particulars of the colliding ship are determined and the damage to the ship from the collision is estimated. The three red nodes "property damage", "release size" and "damage to persons" are the result nodes where the probability of occurrence of each of the consequence states are estimated. The Bayesian network is coupled to the ship accident frequency model, ref. [1], such that the consequences depend on which ship accident has occurred. Thus, the consequence model inherits the accident characteristics. The original consequence calculations in the Bayesian network are based on a ship collision with a fixed concrete pier. The results have therefore been modified to take into consideration that the ships will collide with a less stiff and flexible structural element – separately modeled for SFT and floating bridge. Other minor adjustments have also been made to resemble the traffic in Sognefjorden more correct.

Further, the consequences calculated by the Bayesian network are based on input regarding the shape of the structural element, e.g. it is designed to protect the colliding ship as much as possible. Therefore, it is assumed that the pontoons and pylons are designed to inflict minimal damage upon the ship, i.e. no sharp edges and a design that ensures that a ship can glance along the pontoon/pylon without being ripped open by the pontoon.

The high speed craft (HSC) has been considered separately since the high speed will yield an accident with more severe consequences compared to other ship types. In 1999 the high speed craft M/S Sleipner grounded on rocks and capsized and 16 of the 85 passengers did not survive, ref. [19]. It is assumed that an accident this severe will occur in 1% of all HSC accidents. In the remaining collisions the consequences will be considerable lower. Therefore, it is assumed that in 9% of all accidents one fatality is seen and in the remaining 90% of all collisions no fatalities are seen. The cost of an HSC ferry is estimated to be 30 mil. NOK. This is based on the cost of the HSC ferry Tansøy which operated in Sognefjorden in 2010, ref. [20]. It is estimated that in

5% of the accidents a cost equal to total loss of the ship can be assumed. For 75% of the accidents it is considered that only minor repairs are needed.

Estimating the cost of an oil spill is not straight forward, since it will depend on the location, size of oil spill and type of oil spilled. In addition to this comes less tangible factors such as loss of reputation and wild life and also the season and weather can affect the dissipation pattern of the oil spill. In the present analysis the cost related to clearing and clean-up of an oil spill (environmental damage) is estimated based on information given in Safedor, ref. [18], where a coarse model with an economical cost of 12 700 US$ per spilled ton of oil is proposed as an average cost per ton independent of the oil spill size.

## COLLISION RISK

The ship collision risk model has been applied to both bridge solutions – the SFT and the floating bridge. Results in terms of collision frequencies and collision consequences are given in the following.

### Collision frequencies

The overall collision frequency for the SFT is $1.1 \cdot 10^{-1}$ corresponding to a return period for ship collisions of approximately 10 years. This includes all types of collisions including minor glancing of the pontoons and collisions from very small ships. Only a fraction of the total collision frequencies originates from more serious collisions. It is noted that collisions for SFT only covers collisions with pontoons. Collisions with the tunnel itself are estimated not to be possible as it is situated too far below sea level.

The overall collision frequency for the floating bridge is $9.2 \cdot 10^{-3}$ corresponding to a return period for ship collisions of approximately 100 years. Resulting frequencies distributed on single pontoons are shown in Figure 8 divided into head-on (HOB) collisions and sideway collisions. The navigational route for the SFT goes between pontoon 8 and 9.

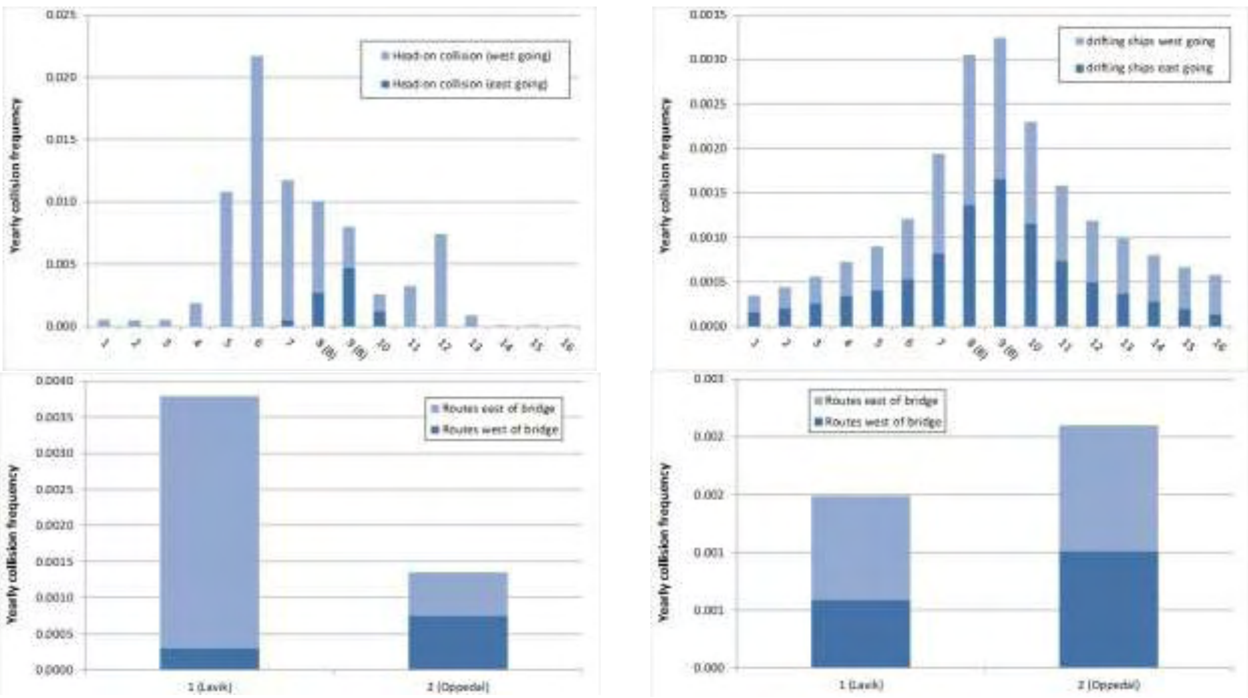


*Figure 8. Collision frequencies – SFT (top) and floating bridge (bottom), HOB (left) and drifting (right)*

**Risk related to the fixed link**

The risk related to the fixed link in terms of user fatalities, disruption period and property damage are given in the following separately for the floating bridge and for the SFT. The risk is determined by combining the frequency and consequence calculations as described in previous sections. The risk is determined for all consequence types and are also capitalized, i.e. expressed in expected annual costs.

*Table 3. Risk assessment fixed link*

| | Fatalities [ per year] | Property [NOK/year] | Disruption [days/year] |
|---|---|---|---|
| Floating bridge | $9.61 \cdot 10^{-5}$ | 128,781 | 0.073 |
| SFT | $4.83 \cdot 10^{-5}$ | 2,062,903 | 1.0 |

From Table 3 it is seen that the risk of fatalities are very limited and only a very small fraction of the total acceptable risk of user fatalities of $N_{fat}$ = 0.06 fatalities per year. The risk of user fatalities is hence considered acceptable low. Similar the risk of disruption and the risk of property damage is considered limited without however having being compared to established acceptance criteria. The risk should hence be treated in accordance with the ALARP principle (As Low As Reasonable Practicable) and shall be minimized if measures to do so are cost-beneficial. There is a number of risk reducing measures that could decrease the risk further. This includes e.g. implementation of some kind of surveillance of the ship traffic (VTS). By using the numbers for transforming fatalities and disruption period into economic consequences, the figures given in Table 4 applies.

*Table 4. Capitalized risk assessment fixed link*

| | Fatalities [NOK/year] | Property [NOK/year] | Disruption [NOK/year] |
|---|---|---|---|
| Floating bridge | 1,479 | 128,781 | 87,992 |
| SFT | 743 | 2,062,903 | 1,249,036 |

Almost the entire risk for the consequence types 'property damage' and 'disruption' derives from collisions leading to repair. This is due to the frequency of repairs being much larger than the frequency of collapse. The number of fatalities is on the other hand very limited as fatalities are only seen for collisions leading to collapse.

In general, there is a higher risk for the SFT than for the floating bridge primarily due to the fact that the repair collision frequencies are two decades higher.

**Risk related to the ship traffic**

The risk related to the fixed link in terms of user fatalities, disruption period and property damage are given in the following separately for the floating bridge and for the SFT. The risk is determined by combining the frequency and consequence calculations as described in previous sections. The risk is determined for all consequence types and are also capitalized, i.e. expressed in expected annual costs.

*Table 5. Risk assessment ship traffic*

| | Fatalities [ per year] | Property [NOK/year] | Environment [tonnes/year] |
|---|---|---|---|
| Floating bridge | $1.72 \cdot 10^{-4}$ | 128,854 | 0.31 |
| SFT | $2.26 \cdot 10^{-4}$ | 465,070 | 1.23 |

Regarding ship traffic fatalities there is a criteria regarding individual risk of $N_{fat}=1 \cdot 10^{-6}$ fatalities per year. This acceptance criterion is related to the most exposed person on board a ship passing the bridge. As the HSC ferry has a large contribution to fatalities and a lot of regular passengers it can be assumed that the most exposed person is a regular passenger onboard a HSC vessel. By assuming that a regular passenger pass the bridge twice 200 days a year and that the number of passengers onboard a HSC vessel is 30 the risk for the most exposed passenger can be estimated to be $4.1 \cdot 10^{-7}$ for the SFT and $2.4 \cdot 10^{-8}$ for the floating bridge. The risk of ship traffic fatalities is hence considered acceptable low.
The risk of environment and the risk of property damage are also considered limited without however having being compared to quantified acceptance criteria. The risk should hence be treated in accordance with the ALARP principle (As Low As Reasonable Practicable) and shall be minimized if measures to do so are cost-beneficial. There is a number of risk reducing measures that could decrease the risk further. This includes e.g. implementation of some kind of surveillance of the ship traffic (VTS). By using the numbers for transforming fatalities and disruption period into economic consequences, the figures given in Table 4 applies.

*Table 6. Capitalized risk assessment ship traffic*

| | Fatalities [NOK/year] | Property [NOK/year] | Environment [NOK/year] |
|---|---|---|---|
| Floating bridge | 2,641 | 128,854 | 22,955 |
| SFT | 3,475 | 465,070 | 90,810 |

Even though the collision frequencies are higher for the SFT than for the floating bridge the risk for the ship traffic is of the same magnitude. This is primarily due to the fact that the consequences of a collision with the SFT being smaller compared to the floating bridge.

## CONCLUSION

It is demonstrated that construction of a fixed link crossing Sognefjorden – either as a floating bridge or as an SFT – does not pose unacceptable risks, neither for the ship traffic nor for the users of the fixed link. There is however the possibility of implementing different measures to decrease the risk further. The most obvious is to extend the existing surveillance of the ship traffic (VTS) in the area to include Sognefjorden. Obviously, as the design of the bridges progresses it is important to keep track of the actual risk level in order to ensure that the risk at any time stays at an acceptable low level.

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# SUB-BOTTOM INVESTIGATIONS FOR A FLOATING STRUCTURE ACROSS BJØRNAFJORDEN ANCHORING CONDITIONS

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**ABSTRACT**

*The Norwegian Public Road Administration hired MULTICONSULT to investigate the seabed conditions for a prospective anchored floating bridge or submerged tunnel across Bjørnafjorden. The investigated corridor at the mouth of the fjord is approx. 5km long and 2.5-3.5km broad with an area of 16km$^2$. Deepest parts of the fjord are 500-560m and make about a third of the investigated area.*

*The introductory investigations have consisted of:*

1. *Bathymetric information in form of existing Multi Beam Echo soundings collected from Norwegian Hydrographic Service.*
2. *Acoustic profiling by high-resolution, surface towed Sparker. Performed by GEOMAP from M/B "Frifant".*
3. *Soil sampling in four stations from the top 3-4m of seabed sediments by Ø100mm gravity corer from research vessel "G.O.Sars".*
4. *Geotechnical laboratory testing of collected clay samples*

*The Sparker survey reveals loosely deposited sediments with typical thicknesses of 20-30m in the deepest and central part of the investigated corridor. Depths to seabed in the selected sampling stations, with spacing approx. 1km, vary between 492 m and 560 m. A comprehensive laboratory programme reveals remarkable homogenous conditions with very soft clay of high plasticity in the full sample length (3-4m) at all the stations.*
*The uncovered sub-bottom conditions suit suction pile type anchors well, provided that the sediment thicknesses are sufficient for installing designed length of piles. The Sparker results give positive indications in that respect.*
*The investigations show in which parts of the fjord installation of suction pile anchors is feasible. Design of suction piles needs information in terms of geotechnical parameters deeper into the sediments, say min. 20m. Relevant investigations in that respect are CPT soundings combined with deeper soil sampling from rig resting at the seafloor, operations that will have to be carried out from a suitable vessel.*

## INTRODUCTION

E39 is the coastal road in Norway between Kristiansand in the south, via Stavanger, Bergen and Molde, to Trondheim in the north. This road has today eight fjord crossings by ferry. The vision is to get the distance between Kristiansand and Trondheim free of ferries.
One of the longest existing ferry distances is across Bjørnafjorden in Hordaland County, 30-35 km south of Bergen. Two alternative corridors of crossing Bjørnafjorden exist. The Norwegian Public Roads Administration has carried out a feasibility study of prospective crossing solutions for the deepest part at the mouth of Bjørnafjorden. Due to width and depth of the fjord relevant

solutions is a floating bridge or submerged floating tunnel, both of which anchored laterally to the seabed.
In that connection Multiconsult AS (MC) has been hired by The Norwegian Public Road Administration (NPRA) to investigate seabed and sub-bottom conditions as basis for a feasibility study of the anchoring conditions in relevant parts of the fjord.
The road authority first contacted MC on the 28$^{th}$ of November 2011 with a specification of the project. Final report was expected to be available at the 1$^{st}$ of March 2012. Preliminary survey results and conclusions as to suitable seabed anchors were submitted to the client at the beginning of March 2012.

## SITE DESCRIPTION

The investigated area is a corridor for prospective fjord crossing by floating bridge or submerged floating tunnel at the mouth of the fjord as shown in a section of the sea chart in fig. 1. The corridor length across the fjord is 5km and the width varies between 2.5 and 3.5km. The area is about 16km$^2$, whereof the deepest part with water depths of 500-560m makes a third. The seabed climbs generally steeper from the fjord floor towards land in south than in north. See deep water chart in figure 1.

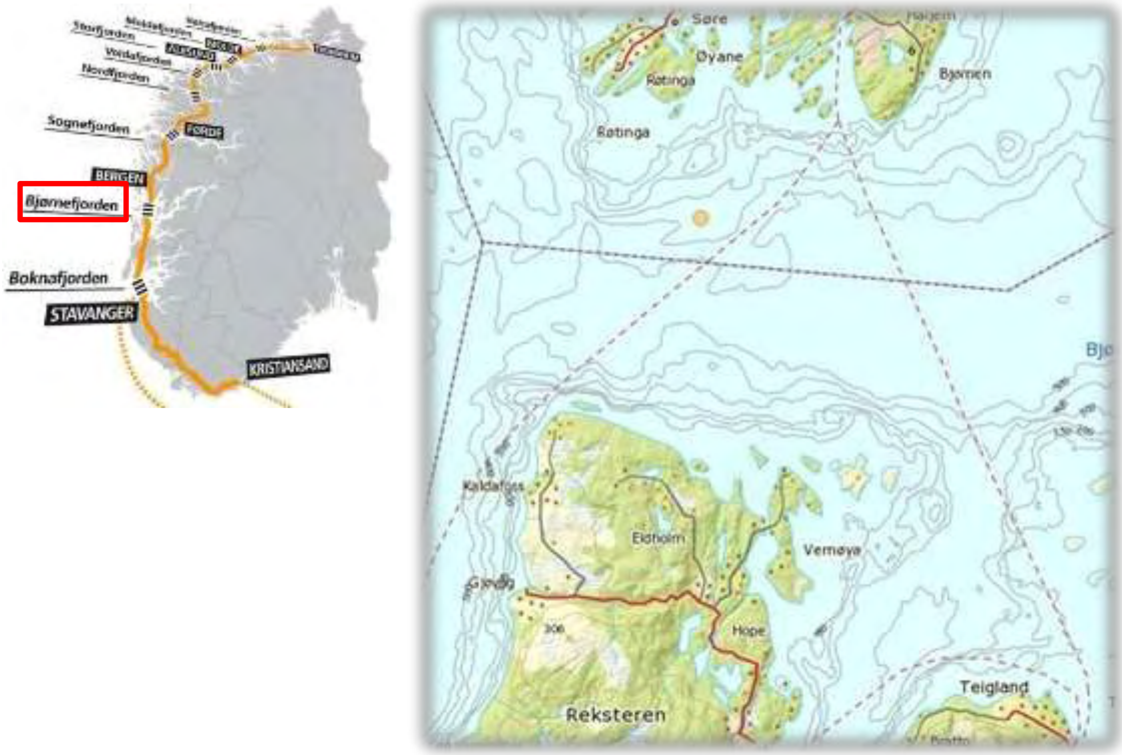


*Figure 1. Sea chart. Mouth of Bjørnafjorden*

## PURPOSE OF INVESTIGATIONS. SCOPE

The purpose of the investigations defined by NPRA was to collect information about the seabed conditions to make a basis for assessing relevant anchoring solutions of known or feasible technology for floating structures. Desired information in that respect is detailed seabed topography and an acoustic image of the seabed indicating the distribution of exposed bedrock and areas of sediments/deposits. In addition, thickness of deposits is of interest in selected

sections of the seabed, especially if consisting of soft clay, as well as information/indication of stratification and sediment types with geotechnical data down to about 15 m beneath seabed.
If possible and suitable within available time the client also wanted soil sampling in selected stations to verify the acoustic profiling findings and collect geotechnical data from the sediments.

## INVESTIGATION PROGRAMME

With basis in the scope defined by NPRA the following investigation programme was advised from MC:
Bathymetric information in form of existing Multi Beam Echo soundings
1. Acoustic profiling by high-resolution, surface towed Sparker
2. Soil sampling from the top 3-4m of soft seabed sediments by Ø100mm gravity corer
3. Geotechnical laboratory testing of collected clay samples

## BATHYMETRIC INFORMATION

From Norwegian Hydrographic Service`s deep water database the NPRA obtained data covering the area of interest delivered as point files with position and depth from Multi Beam Echo soundings carried out in March 1998 and August 1999.
MC has had the data matrix displayed as a terrain model with shadow relief and depth colours to visualize the seabed character as shown in figure 2.

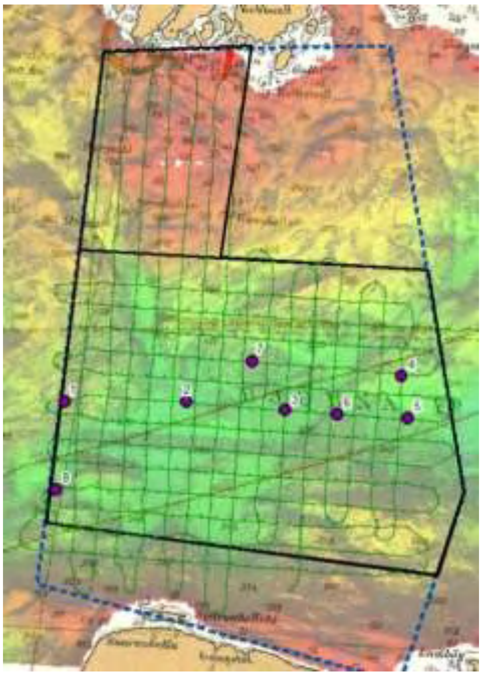

*Figure 2. Acoustic image with survey lines and sampling stations*

## ACOUSTIC PROFILING

Queries were distributed to Norwegian companies performing geophysical surveys at sea. Due to large water depths and desired penetration into sub-bottom sediments it was supposed that the acoustic survey would have to be conducted with sparker equipment which has sufficient energy and penetration properties.

Two companies submitted offer on the queried survey. One company offered the survey performed by inshore equipment with a sea surface towed sparker system while the other company recommended and offered a hull-mounted system from an advanced multipurpose vessel classified for offshore tasks. Naturally, the difference in quoted costs between the two offers was substantial. Since the survey was to be a part of a feasibility study with corresponding demand to accuracy and limited budget it was decided to choose an acoustic survey by ordinary inshore equipment. Geomap AS was hired and the field work was executed in weeks 6/7 2012.

**Survey programme**

GEOMAP hired the local working craft "Frifant" for the survey. The surveyed area is profiled systematically along parallel lines with spacing 200m in directions N-S and E-W as shown in figure 2. In the western part the N-S lines are extended north towards the shore to uncover any deposits in the area of possible rock tunnel for a submerged floating tunnel solution across the deepest part of Bjørnafjorden. Totally profiled survey lines are about 100km.

**Survey equipment and positioning system**

The survey was carried out applying Geo-Spark 200/1000 system 100-1000joule, EPC 2086 plotter and Geo Resources streamer.
For positioning differential GPS, Trimble 12 / Starfix was used.
Horizontal datum is Euref89 zone 32.

**Interpretation and reporting. Results**

An interim interpretation of the sparker data was carried out by Geomap at MC`s office in Bergen the day after completed field works in week 7/12. The immediate purpose was to identify areas were the survey data indicated sections of soft sediments as silt and clay with an eye to the closely following soil sampling operation. The preliminary interpretation concluded with a suggested programme of eight soil sampling stations.

Final interpretation and reporting of the acoustic survey included identification of areas with exposed bedrock, adding to depths of soft sediments and showing sections of moraine or other coarse deposits ("mixed deposits"). The sparker data are interpreted manually and have been converted from "time depths" to depths in meters applying assumed sound velocity 1600m/s in soft sediments, typical for clay/silt, and 2000m/s in assumed moraine/"mixed deposits". Beneath the sections of soft deposits the reflections from bedrock are generally distinct, even for overburden thicknesses of 50-60m. Also strata indicating moraine deposits on bedrock are identified in some areas.

The overall interpretation results are presented as a deposits distribution chart that is shown conventionalized in fig. 3.

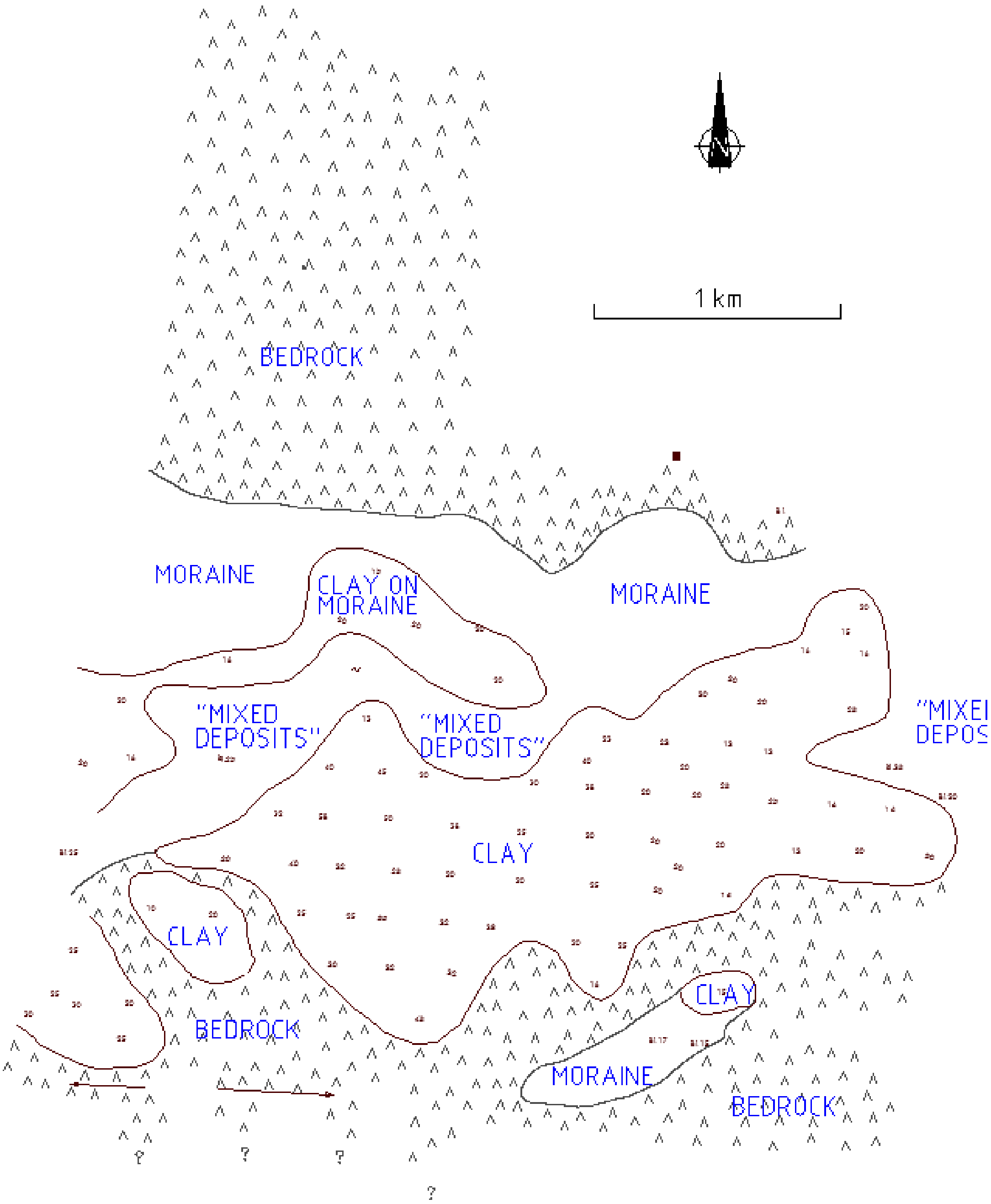


*Figure 3. Customized deposits distribution chart of investigated area*

The sparker survey has detected existences of soft sediments in a greater part of the central and deepest section of the investigated fjord area. More isolated sections of soft sediments are also found. Deposit thicknesses are indicated in numbers on the chart. Typical interpreted thicknesses of soft sediments are 20-30m with a maximum up to 50-60 m. From the shoreline in north and 2km southwards the investigated fjord bottom consists of exposed bedrock or slight occurrences of deposits, which also is the case in the steep bottom slope in south.

## PROJECT ORGANIZATION

Contact persons at NPRA have been Bodil Oust in Region West (Leikanger), who managed the project, and Johannes Veie from the bridge section in Region East (Hamar). Project leader in MC has been Harald Systad, and Arne Stordal has been in charge of soil sampling and testing, including geotechnical reporting.

NPRA by Bodil Oust provided for arrangements with Norwegian Hydrographic Service regarding MBE-data and G.O.Sars from UiB for sub-bottom soil sampling.
Rest of the field operations, including hiring GEOMAP for the acoustic survey, was coordinated by Harald Systad, MC.

## SOIL SAMPLING

In advance of the survey, eight possible sampling locations were defined and given a certain priority from the acoustic mapping. It was decided a schedule of one day for sampling. Samples marked with priority numbers are presented on figure 2.

Sample locations were transformed from Euref-coordinates to degrees of longitude and latitude in a hurry when we discovered that the vessel positioning was based only on this system.
Two representatives from Multiconsult mobilized at 14$^{th}$ of February 2012, and were transported on board G.O.Sars by a landing craft from Halhjem in Os municipality on the north side of Bjørnafjorden.

The gravity corer consisted of an adjustable conical weight of 350kg, see figure 4.

*Figure 4: Gravity corer with adjustable weight*

After wire launching to approximately 10m above sea bottom, the falling velocity was reduced to less than 1m/s during penetration. The low penetration velocity might result in moderate disturbance at the surface of the 100mm diameter sample, but the core is assumed to remain more or less undisturbed. During the one day survey sampling was carried out in four locations with sea depths in the range 492 – 560m as presented on Table 1. Totally 13.5m clay samples of good quality were recovered. The four sample cylinders were 3.0-3.8m long, and were divided into 1m cylinder lengths before sealing and transport to laboratory.

*Table 1: Details from sampling*

| Sample No. | Time | Sea depth | Tube length | Sample length |
|---|---|---|---|---|
| 1 | 11:45 | 527m | 4m | 3.30m |
| 2 | 12:45 | 560m | 4m | 3.80m |
| 3 | 13:35 | 550m | 4m | 3.35m |
| 4 | 14:40 | 492m | 3m | 3.00m |

*Figure 5: PVC-tube mounted with steel edge and sample protection*

## LABORATORY TESTS

A comprehensive laboratory program was outlined by Multiconsult, and investigations were carried out at our laboratories in Bergen and Oslo. Since ø100mm is not a sample standard for geotechnical laboratories, we reconstructed the sample extruder at our laboratory in Bergen.

The laboratory program consisted of the following tests:

- Index properties of 15 cylinders
- 7 grain size distributions
- 4 oedometers with stepwise loading
- 4 oedometers with continuous loading
- 4 active undrained triaxial tests (CAU)
- 2 bender element tests

## CLAY CHARACTERISTICS

Descriptions of clay characteristics are divided into index properties and grading, settlement parameters, shear strength parameters and stiffness parameters.

## Index parameters

Grain size distributions revealed clay content (<0.002 mm) in the range 40%-52% with one exception for a sample of silty clay with 21%. Laboratory testing revealed very plastic, soft clay with high values of moisture content, very low unit weight and moderate apparent organic content in two samples. Testing with hydrochloric acid showed that this latter reaction is rather a confirmation of calcareous content.

*Table 2: Ranges of index properties for the soft clay*

| Sample serie | w (%) | $I_p$ | $O_{Na}$ (%) | $\gamma$ (kN/m$^3$) | $s_{uk}$ (kN/m$^2$) | $S_t$ |
|---|---|---|---|---|---|---|
| 1 | 65-133 | 47-64 | 0.7-1,0 | 13.3-15.2 | 5-26 | 1.3-2.9 |
| 2 | 77-123 | 42-69 | 1.0 | 13.2-14.7 | 4-14 | 2.8-7.3 |
| 3 | 62-129 | 40-70 | - | 14.3-15.7 | 7-38 | 1.5-6.2 |
| 4 | 66-98 | 58-66 | - | 14.7 | 9-24 | 1.8-4.3 |

## Settlement parameters

Consolidation tests in two types of oedometer were chosen to determine creep parameters in addition to consolidation parameters. Stepwise oedometer is best suited to determine creep parameters, whereas continuous oedometer is most efficient and accurate for consolidation parameters.

Parameters are interpreted according to the modulus concept by Janbu /3/, and also the creep parameters are in accordance with Janbu's creep model. The general equation for the constrained modulus, M, according to Janbu is:

$$M = m \cdot p_a \left(\frac{p'}{p_a}\right)^{1-a}$$

m=modulus number
$p_a$=100 kN/m$^2$
a=model exponent
p'=effective stress

Normally, the exponent value of normal consolidated clays is a=0, and the constrained modulus is linear governed by the modulus number. However, Janbu also describes the model where a<0 as a model for "extra sensitive" soil.

Results from consolidation tests are presented in table 3, and the results are remarkable compatible despite of major distance between sample locations and different testing methods. From eight tests we found the modulus number in the range m=8-11 and the exponent variation in the range a= -0.19 to -0.26.
The consolidation coefficient is very low, showing that consolidation process will develop very slowly. Creep numbers are in the lower range of empirical clay data and increases with stress level.

*Table 3: Settlement parameters from oedometer tests*

| Test | Sample | Depth (m) | Modulus number, m | Exponent, a | Consolidation coefficient, $c_v$ ($m^2$/year) | Creep number, $r_s$ |
|---|---|---|---|---|---|---|
| Trinnvis 1 | PR I | 0.50 | 8 | -0.23 | 0.6-3.3 | 26-180 |
| Trinnvis 2 | PR II | 0.40 | 9 | -0.25 | 0.7-5.7 | 29-225 |
| Trinnvis 3 | PR II | 2.50 | 9 | -0.22 | 0.9-6.7 | 29-280 |
| Trinnvis 4 | PR III | 1.30 | 11 | -0.19 | 0.4-5.8 | 14-300 |
| CRS 1 | PR II | 1.95 | 11 | -0.23 | 2.5-9.9 | |
| CRS 2 | PR II | 3.05 | 10 | -0.24 | 1.0-4.3 | |
| CRS 3 | PR IV | 2.95 | 9 | -0.23 | 2.5-4.9 | |
| CRS 4 | PR IV | 0.10 | 8 | -0.26 | - | |

## Shear strength parameters

Anisotropic consolidated triaxial tests were executed with relative high values of the coefficient of earth pressure at rest, $K_0$'=0.8-0.9 to avoid sample disturbance during consolidation. Results from triaxial testing are interpreted both for Tresca failure criterion and the Mohr-Coulomb failure criterion. In addition, the pore pressure parameter D according to Janbu is interpreted. Compressive undrained shear strength is found in the range $s_u^C$=4.4-8.4 kN/$m^2$. In general the shear strength parameters indicate very soft clay, with high levels of excess pore pressure and contractive yield development.

*Table 4: Shear strength parameters from triaxial tests*

| Sample | Depth (m) | $K_0$'-consol. | $s_u^C$ (kN/$m^2$) | a(kN/$m^2$) | tgφ | D |
|---|---|---|---|---|---|---|
| PR II | 1.85 | 0.80 | 4.4 | 5 | 0.37 | -0.25 |
| PR II | 3.15 | 0.85 | 8.4 | 10 | 0.43 | -0.45 |
| PR IV | 2.85 | 0.90 | 6.5 | 5 | 0.47 | -0.44 |
| PR IV | 0.80 | 0.90 | 6.2 | 5 | 0.51 | 0.10 |

Falling cone tests, uniaxial tests and triaxial tests show increasing undrained shear strength with depth as normal for consolidated clays. A linear adaption to the three deepest tests may be expressed as:

$$\mathbf{s_u^C=0.42(\gamma'z+3)}$$

z=depth below seabed (m)
γ'=submerged weight (5 kN/$m^3$)

The above equation corresponds well with values of liquid limit and plasticity index as presented above. If anisotropic shear strength is required we suggest the following relations:

$$s_u^D/s_u^C=0.72\text{-}0.76$$
$$s_u^E/s_u^C=0.53\text{-}0.61$$

**Stiffness parameters**

Stiffness parameters are interpreted in terms of shear modulus $G_{50}$ from triaxial tests and dynamic shear modulus $G_{max}$ from bender element tests. $G_{50}$ is often applied in finite element programs such as PLAXIS and is interpreted from the secant value that intersects the τ-ε curve at $\tau=\frac{1}{2}\tau_{max}$. In addition two bender element tests are carried out to measure the shear wave propagation velocity, $v_s$. Consequently, maximum shear modulus is found by the equation:

$$G_{max}=\rho v_s^2$$

*Table 5: Stiffness parameters*

| Sample | Depth (m) | Test | $G_{50}$ (kN/m$^2$) | $\varepsilon_a$ (%) | $G_{max}$(kN/m$^2$) | $\varepsilon_a$ (%) |
|---|---|---|---|---|---|---|
| PR II | 1.85 | Triaxial 1 | 666 | 0.3 | | |
| PR II | 3.15 | Triaxial 2 | 79 | 2.3 | | |
| PR IV | 2.85 | Triaxial 3 | 1066 | 0.3 | | |
| PR IV | 0.80 | Triaxial 4 | 222 | 1.2 | | |
| PR IV | 2.60 | Bender element 1 | | | 3290 | 0.001 |
| PR II | 3.30 | Bender element 2 | | | 5860 | 0.001 |

It may be observed from table 5 that there are major variations in the $G_{50}$ –values that is in contrast to the uniform results from other tests. Bender element tests gave $G_{max}$ values that fit well with empirical values for normally consolidated high plastic clays, such as Larsson & Mulabdic /4/:

$$G_{max} = \left(\frac{208}{I_p} + 250\right) s_u$$

This equation fits the bender element best when $s_u$-values from falling cone apparatus are applied.

## BRIDGE ANCHORING

Since this survey must be regarded as an initial investigation as background for a conceptual study, we may conclude that a large number of uniform geotechnical data are retrieved. If the same properties of the clay layers down to 3-4 m will be recovered down to 15-20 m, we may conclude that these sub bottom conditions are suitable for suction anchors. In addition, properties of the Bjørnafjorden clay are comparable to other deep sea clays found in the Gulf of Guinea and the Gulf of Mexico. Suction anchors are installed in both these locations, and experience from these locations may be fruitful as comparison.

Some parameters for detailed design are not investigated in this survey, such as degradation due to cyclic loading and thixotropic properties. For a later and final investigation we may recommend the additional part of a program:

- Piezocone penetration tests in all proposed locations
- Sampling to bottom of clay layer in all proposed locations
- Cyclic triaxial tests
- Passive triaxial tests
- Direct simple shear tests

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# RE-FURBISHMENT OF THE BERGSØYSUND FLOATING BRIDGE (2011-2013)

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**ABSTRACT**

*The bridge is part of the primary road E 39 along the west coast of Norway, connecting the islands Bergsøy and Aspøy in Møre & Romsdal County close to the town Kristiansund. The superstructure was constructed during 1991-1992 at Aker Verdal in Norway. The bridge consists of a continuous 830 m steel frame and also has members of tubular steel. The superstructure floats on 7 pontoons constructed in high strength LWA concrete. The elliptical pontoons 34x20 meter with slip formed walls 6.1 - 7.0 meters high were constructed at the yard for Condeep oil platforms (Norwegian Contractors) in Stavanger. Uniquely the bridge only has anchoring at the endpoints taking transverse forces as compression or tension in the arch.*

*The steel superstructure was initially corrosion protected with Thermally Sprayed Zinc and a coating system. In 2011 a program for refurbishment of the bridge was initiated and this paper describes the investigations made and actions taken. Explanations for coating failure are given together with experiences with the use of TSZ and coatings.*

*The conditions of the pontoons have been controlled thoroughly at the periodic main inspection every 5.year. So far only minor damages from small boats have been observed on the outside concrete cover and there have not been any leakages into the pontoon compartments. The floating bridge transfers relatively small vertical loads from traffic loads to the abutments, but tidal variation causes significant vertical loads. In addition there are horizontal and transverse loads from currents, waves and wind. Neoprene rubber bearings in the abutments are designed to take care of the transverse reactions on the bridge arc. So far no damages have been observed on these neoprene bearings. Both compression and tension loads are induced in the arch bridge. These forces are directed into a flexible 12m tubular steel rod that is the connection between the bridge and the abutments. So far this has been a successfully design.*

# 1. HISTORY

The Bergsøysundet Bridge is one of two floating bridges constructed in Norway during the early nineties. The bridge is part of the primary road E 39 along the west coast of Norway, connecting the islands Bergsøy and Aspøy in Møre & Romsdal County close to the town of Kristiansund.

The superstructure was constructed during 1991-1992 at the large offshore yard (Aker Verdal) in Norway. The bridge consists of a continuous 830 m steel frame and also has members of tubular steel. The superstructure floats on 7 pontoons constructed in high strength LWA concrete. The elliptical pontoons 34x20 meter with slip formed walls 6.1 - 7.0 meters high were constructed at the yard for Condeep oil platforms (Norwegian Contractors) in Stavanger. Uniquely the bridge only has anchoring at the endpoints taking transverse forces as compression or tension in the arch.

All surface preparation and coating work of the steel was carried out indoors at the yard under controlled conditions. Prior to application the steel was degreased and then blast-cleaned with steel abrasives prior to application of the original coating system.

| **Surface Preparation** | **DFT (µm)** |
|---|---|
| Degreasing / freshwater washing | Alkaline |
| Blast-cleaning, acc. to ISO 8501-1 | Sa 3 |
| Roughness, acc. to ISO 8503 | Medium;<br>Segment 3 |
| Max. Chloride content | 20 mg/m$^2$ |

The steel superstructure consists of an orthotrop profile and tubular steel (nodes) and was initially coated with a coating system consisting of:

| **Original Coating System** | **DFT (µm)** |
|---|---|
| Thermally Sprayed Zinc *(TSZ)* | Min. 100 µm |
| Etch primer | 1 x 10 µm |
| Modified Alkyd / Chlorinated rubber - primer/<br>Modified Alkyd / Chlorinated rubber - intermediate coat | 3 x 50 µm |
| Silicone Alkyd Topcoat | 1 x 50 µm |

The system used for coating of the bridge is quite similar to the system used by the *Norwegian Public Road Administration (NPRA)* since the 1970'ies. The difference here being that the top coat had to be changed from an alkyd / chlorinated rubber to a silicon alkyd based topcoat due to the colour of the top-coat.

We have not been able to locate information with regards to differences in water permeability for the original NRPA topcoat and the silicon alkyd topcoat.

The deterioration of the paint system can be due to differences in the topcoat or the fact that the harsh environmental conditions, is the cause for this or a combination of these.

## 2. APPEARANCE OF THE BRIDGE

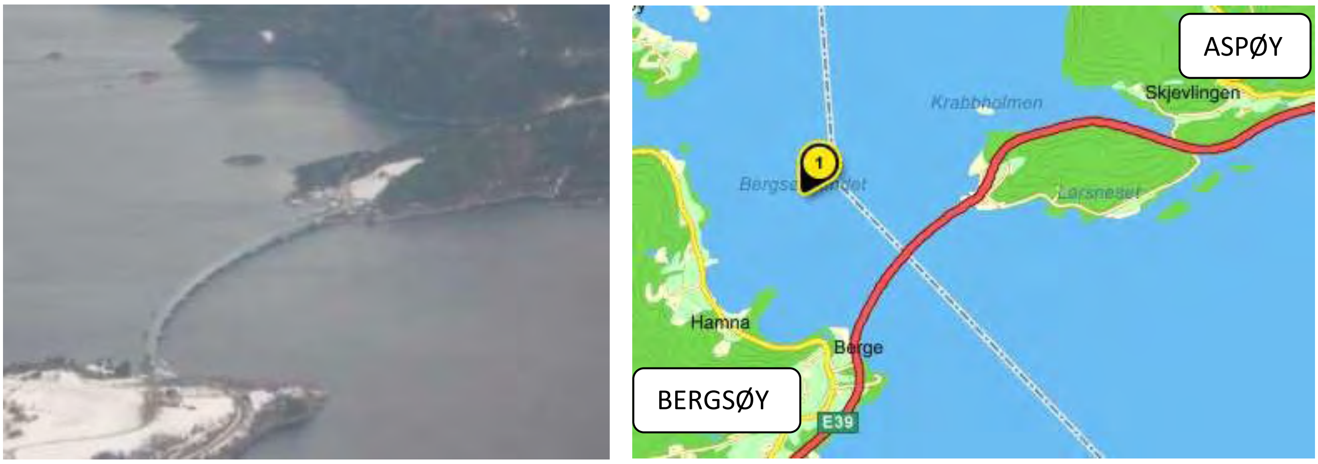


*Photo no. 1 Arial view / view of the Bergsøysund Bridge*


*Photo no. 2 The orthotrop steel structure*

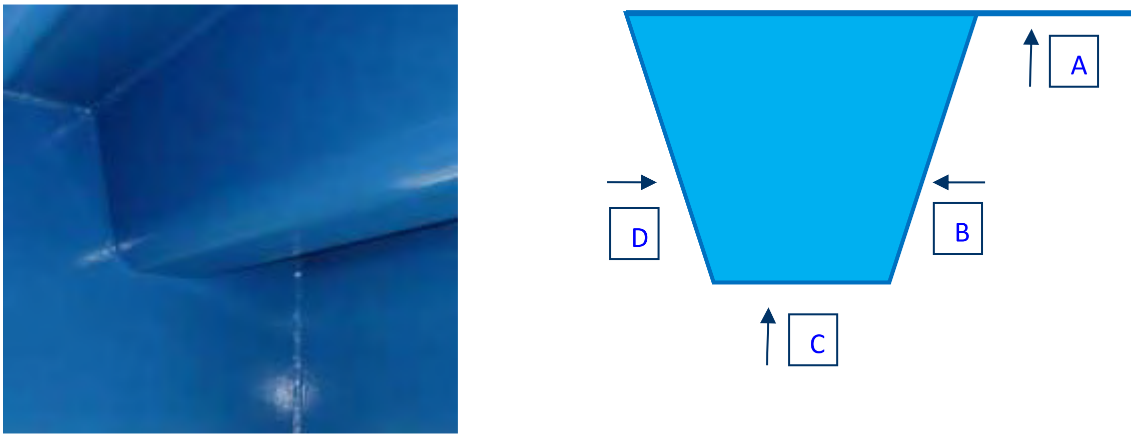


*Photo no. 3 Close-up of the trapez construction*

## 3. ENVIRONMENT

The location of the bridge deck is less than 10 m above MSL.

The environment surrounding the Bergsøysund Bridge is therefore extremely harsh – corresponding to corrosivity category C 5 M in accordance with ISO 12944. The saltwater collects under the deck of the bridge. Natural draft / ventilation will cause evaporation of the water and precipitation of salt on the surface (*Photo no. 4*).

During large periods of the year, the bridge has an environment with humidity higher than 80 % RH and a temperature above $0^{o}$C, which according to definition in ISO 9226, is the necessary environment to maintain corrosion.

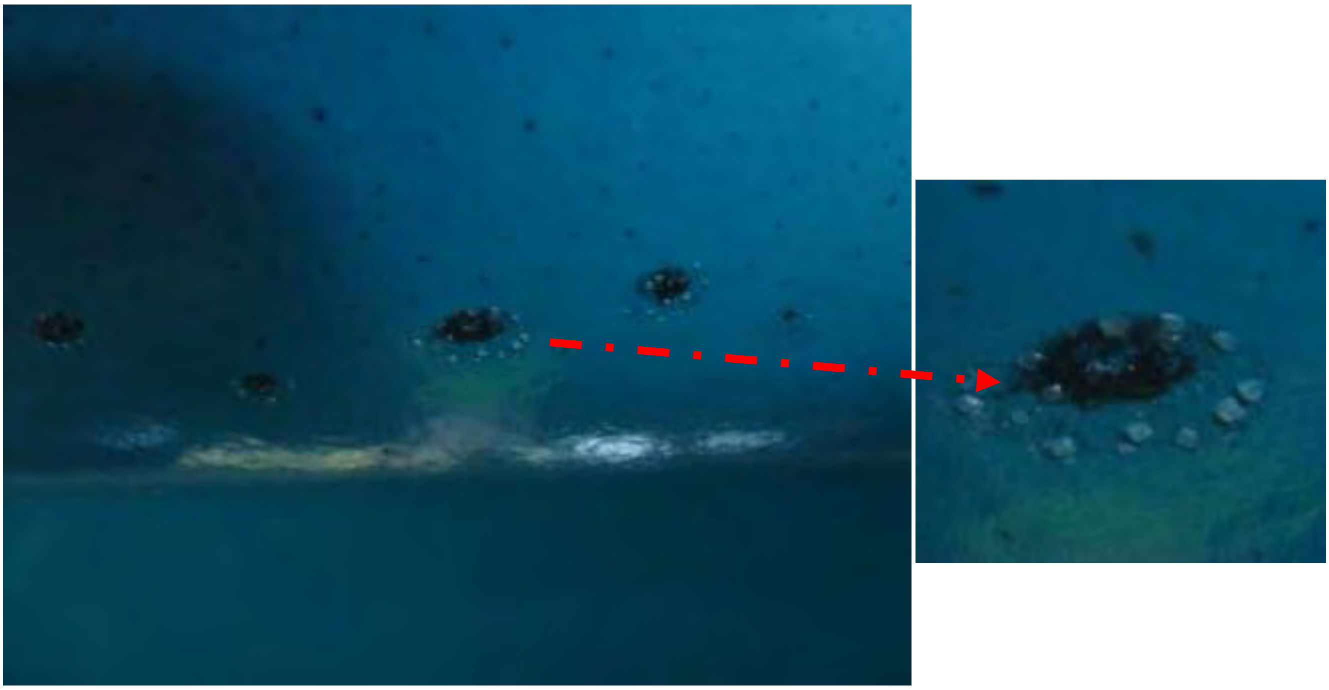

*Photo no. 4 Salt crystals in an area under a Trapez*

## 4. MAINTENANCE OF THE COATING SYSTEM

Maintenance of the Bridge has been carried out several times during the 20 years of service (1992 – 2011).

1st. major Maintenance was carried out: 2003
2nd. major Maintenance was carried out: 2008

The maintenance has been carried by use of a mobile lift, placed on the bridge and equipped with a flexible platform, and able to reach approximately halfway underneath the bridge (*Photo no. 5*).

| The following cleaning procedures were specified in advance of the maintenance: |
|---|
| • Alkaline degreasing and freshwater hosing<br>• Grinding with 3M Scotch- Brite pad<br>• Application of NPRA coating system (modified alkyd / CR), brush-application **or**<br>• Application of zinc epoxy, epoxy mastic and silicon alkyd topcoat |

After degreasing and fresh water cleaning the surface, a certain drying time prior to starting the mechanical cleaning had to be included.

The conditions in between the two operations may have resulted in new water soluble salts (from sea water) on the surface. Remaining salts on the interface between steel / original system and the new coating system, can cause osmotic blistering when condensed water is present on the surface.

*Photo no. 5 Maintenance work in progress*

## 5. COATING SYSTEM AFTER 22 YEARS OF PERFORMANCE

The surveys have revealed areas where the original NPRA system has deteriorated (blistered, white rust formation). Also observed was coating breakdown of maintenance coating from previous maintenance.

The condition of the coating system varies considerably over the steel structure. There are areas where the coating system seems to be in quite good shape, but nearby areas may reveal areas where the coating is either blistering or flaking from the substrate due to corrosion.

The DFT in areas of the original coating system is in most cases within the specified limits (min. 310 microns). The likely cause of the deterioration of the coating system is thought to be caused by the harsh environment.

The condition of the coating system after more than 20 years of service is seen in (*Photo no. 6* - *Photo no. 8*). There are numerous small “blisters” visible on the coating. White rust from corrosion of the TSZ penetrates the coating.

*Photo no. 6 White rust formation of the original coating system (22 years)*

*Photo no. 7 Penetration of corrosion products from TSZ through paint system (22 years)*

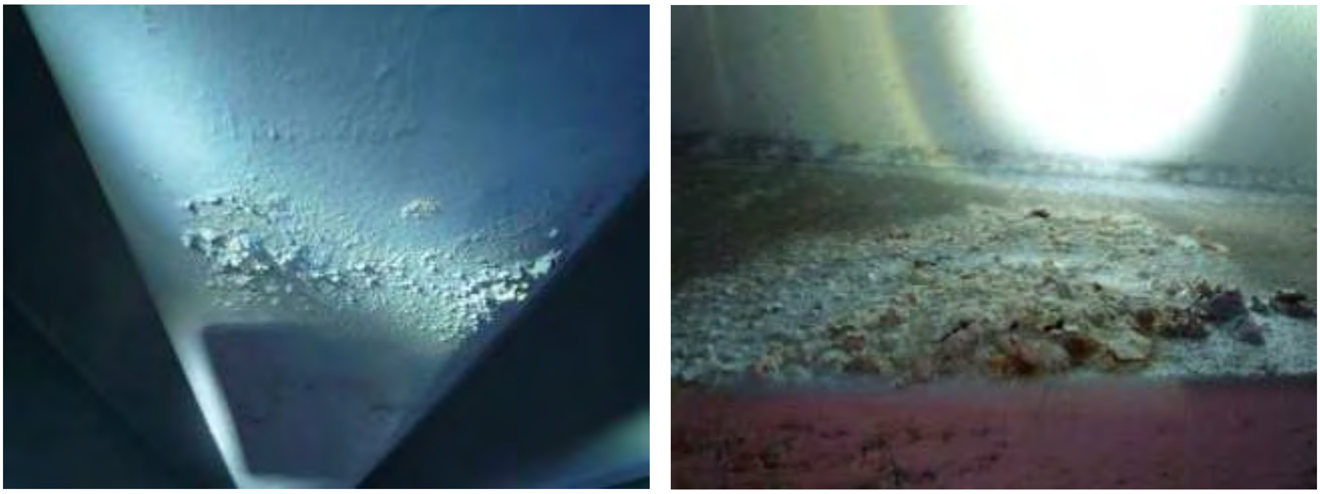

*Photo no. 8 Penetration of corrosion products / blistering of the TSZ / coating (22 years)*

## 6. REFURBISHMENT 2011-2013

Based on previous experience with the Duplex systems used by the NPRA, the main task for the refurbishment was careful removal of the original paint system by blast-cleaning to the TSZ layer and then application of the new paint system.

During the initial blast-cleaning trials, the paint system was removed to the TSZ using low blasting pressure and with a mineral abrasive, Olivine sand. The results showed considerable variations of the DFT of the TSZ in the blasted areas. DFT in the light grey areas the thickness of the TSZ was still after 20 years of performance close to the original values, but in areas close by with darker grey colour, there may be a reddish colour and the DFT may be as low as 10 microns. (*Photo no. 9)*

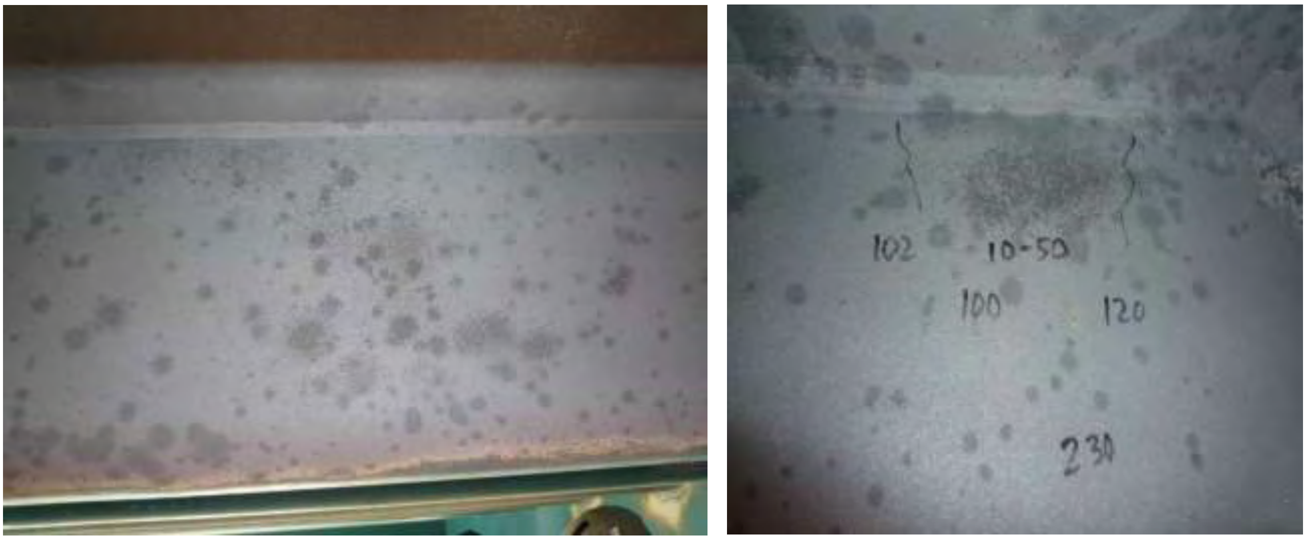


*Photo no. 9 Removal of coating system down to zinc coating. The dark, spotted areas have a DFT lower than the light grey areas due to corrosion after 22 yrs of service on the Bergsøysund Bridge*

## 7. REASONS FOR COATING FAILURE

The breakdown of the "original coating system" will be effected on the

- The top-coat was changed from the "original" alkyd /CR to an alkyd silicon type, which may have provided somewhat less long term performance compared to the "original" alkyd/CR used in general by the NPRA.
- Porosity within the TSZ may have caused pinholes within the coating system.
- During adverse weather conditions, salt spray will settle on more less all surfaces – precipitation of salt will be enable humidity on the surface and create an electrolyte.
- This causes the corrosion process to be more or less continuous and ongoing at more or less all the time (per definition of ISO 9226; corrosion will occur when T > $0^oC$ and RH > 80%).

This has resulted in deterioration of the steel, in a non-uniform matter – and has resulted in different corrosion different rates of the zinc. There are areas with no zinc is left on the steel surface and nearby areas with zinc. Re-application to increase the zinc thickness in areas with low DFT was considered too difficult to accomplish in a practical way.

As a result of this, the plans for maintenance of the bridge had to be changed; and all zinc is removed to the steel substrate and new TSZ and coating is applied. The coating system has been changed from an alkyd/CR system to epoxy/PUR – and the aim is that this system will provide long term protection of the steel structure.

As a result of this the progress for refurbishment maintenance of the bridge has slowed down, being more time consuming and will be more costly than the original system planned.

## 8. EXPERIENCES WITH THE USE OF TSZ AND COATINGS (DUPLEX SYSTEMS)

Knudsen et.al /[8]/ have investigated and discussed degradation , maintenance and life cycle costs for various coating systems used both for off-shore purposes as well as in corrosive coastal areas. Experiences with Duplex systems (Thermally sprayed Zn and a paint system) have been successfully used on coastal bridges for almost 40 years of service. Not only Norway uses or TSZ + coating for protection of their new bridges, more than 90% of all new bridges in the United Kingdom have thermally sprayed coatings.

The Forth Road Bridge was opened in 1964, and was thermally sprayed and painted at the time. Initially an etch primer was applied and then the surface was painted. The Forth Road Bridge is still going strong today. This is just one project of many that is testament to the metal spraying process in protecting steel structures from corrosion.

In connection with the corrosion protection of windmills both onshore and offshore, similar coating systems are widely used. There might be slight variations in the DFT of the thermally sprayed zinc coating – from 80 to 120 microns, but the paint system consisting of 250 microns for exterior protection/[9]/.

Test regimes carried out by Kuroda et. al /3/ also provide very good information as to performance of TSZ with a sealer performed very well in the sea air zone after 15 years of service.

## 9. THE FLOATING BRIDGE IN SERVICE

The public roads administration has only heard praises from drivers crossing the bridge. The bridge has no vertical curve and the arch has a long 1400m radius. The drivers have an overview over the total bridge length when passing the bridge. It has not been observed any traffic accidents on the bridge so far.

Consideration of tidal variations, wave heights, periods, and directions was of importance when designing the bridge. The bridge is placed in a sheltered fjord and is protected against the highest ocean waves. We have only observed a few storms in the fjord since 1992. On these occasions, like the storm “Narve” in January 2006, the bridge has shown to behave very calm with only minor accelerations and movements, even in high fjord waves. The bridge has not once been closed due to weather conditions.

The deformations from tidal variations are accommodated by the flexure of the bridge girder and changes in the draught of the end pontoons. On extreme high tides the end pontoons are pressed into the water and causes significant vertical forces in the upper horizontal rubber bearings in the abutments. This bridge was designed for a maximum of +- 2,0m tidal variation. In January 2005 we experienced the 25-years extreme high tide in the fjord, with a water level +1.8 m higher than a normal high tide. We then observed the bridge behaving in accordance of the principle bridge design in an extreme situation.

*Photo no. 10 Left: Storm "Narve" in 2006. Wind direction from south.*
*Photo no. 10 Right: Extreme high tide in 2005.*

From the start the bridge was instrumented for automatic leakage, unintentionally open hatches for safety of the bridge. This alarm system is connected to the alarm system in the region. Bridge movements and structural stress levels in critical areas were also monitored to confirm the design of the bridge. After ten years in service we observed corrosion in all switches on the hatches and on the signal wires in the original safety system and the monitoring system was starting to fail as well. We then focused on the safety system and replaced the alarm switches on all hatches to an inductive system and most of the original signal wires are changed to optical wires. Since then, the safety system has worked. The monitoring system was too difficult to repair because of lack of parts and the short lifespan on these software systems.

## 10. MAINTENANCE OF THE PONTOONS AND ABUTMENTS

The seven pontoons are basically similar in shape with length 34m, with 20m and height 6 or 7 meters and are made of high strength LWA concrete. The conditions of the pontoons have been controlled thoroughly at the periodic main inspection every 5.year. This is a visual inspection both over and under water, and includes in addition to corrosion control, inspection of the outer side of the pontoons for possible damages from collisions, control for leakages inside all nine compartments and control of the hatches and the alarm systems on the pontoons. So far we have observed only minor damages from small boats on the outside concrete cover and there have not been any leakages into the pontoon compartments.

A system for durability surveillance was installed and monitored since the start. Recordings the first ten years showed no sign of corrosion in accordance with the visual inspections. The corrosion sensors have not been recorded since 2000, but we are planning a new recording next year.

The floating bridge transfers relatively small vertical loads from traffic loads to the abutments, but tidal variation causes significant vertical loads. In addition there are horizontal and transverse loads from current, waves and wind. Neoprene rubber bearings in the abutments are designed to take care of the transverse reactions on the bridge arc. So far we have observed no damages on these neoprene bearing. Both compression and tension loads are induced in the arch bridge. These forces are directed into a flexible 12m tubular steel rod that is the connection between the bridge and the abutments. So far this has been a successfully design.

## 11. REFERENCES

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2003

# THE KEY TO EFFICIENT FERRY OPERATIONS

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## ABSTRACT

*A close look at a map over Norway shows that the Norwegian coastline is divided by long fjords and islands. Since approximately 80% of the Norwegian population live less than 10 km from the coastline, a good communication system is extremely important to overcome the obstacle that fjords and islands represent.*

*In this paper it will be shown that cost effective and environmental friendly ferries running frequently 24 hours a day, represent an efficient way of serving a country divided by fjords and islands.*

## SOME BACKGROUND INFORMATION

The map in fig. 1 shows that Norway is divided by long fjords and islands. The people of Norway are living close to the coastline, and approximately 80% of the Norwegian populations live less than 10 km from the coastline.

The length of the Norwegian coastline is 103 000 km including the coastline of islands and fjords. It is only Canada that has a longer coastline than Norway. As a comparison the Equator line is "only" 40 000 km long.

The fastest growing industries in Norway – the oil industry and the fish farming industry are located along the western and northern coastline of Norway.

All that underline the importance of good communication systems to overcome the obstacles that fjords and islands represents.


*Fig. 1 Map showing the Norwegian fjords and islands*

## 2 A SHORT HISTORICAL SURVEY

### *2.1 Number of domestic ferries and ferry connections*

At the moment 125 ferry connections are served by 150 ferries with a transport capacity ranging from 10 to 215 PCU (Private Car Units).

### *2.2 Traffic growth in the domestic ferry market*

The first car ferries were built in the beginning of 1920. They were often fishing boats converted into car carrying boats being able to transport 5- 6 private cars. But the normal way to carry goods and passengers on the fjords and to and from the islands was using boats that did not take cars. That was the prevailing transport form especially on the western and northern coast of Norway till the end of the 50-ties.

*Fig. 2 Example of one of the first car ferries built in Norway*

*Fig. 3 Example of combined goods and passenger boats used on the fjords in the 50-ties and 60-ties*

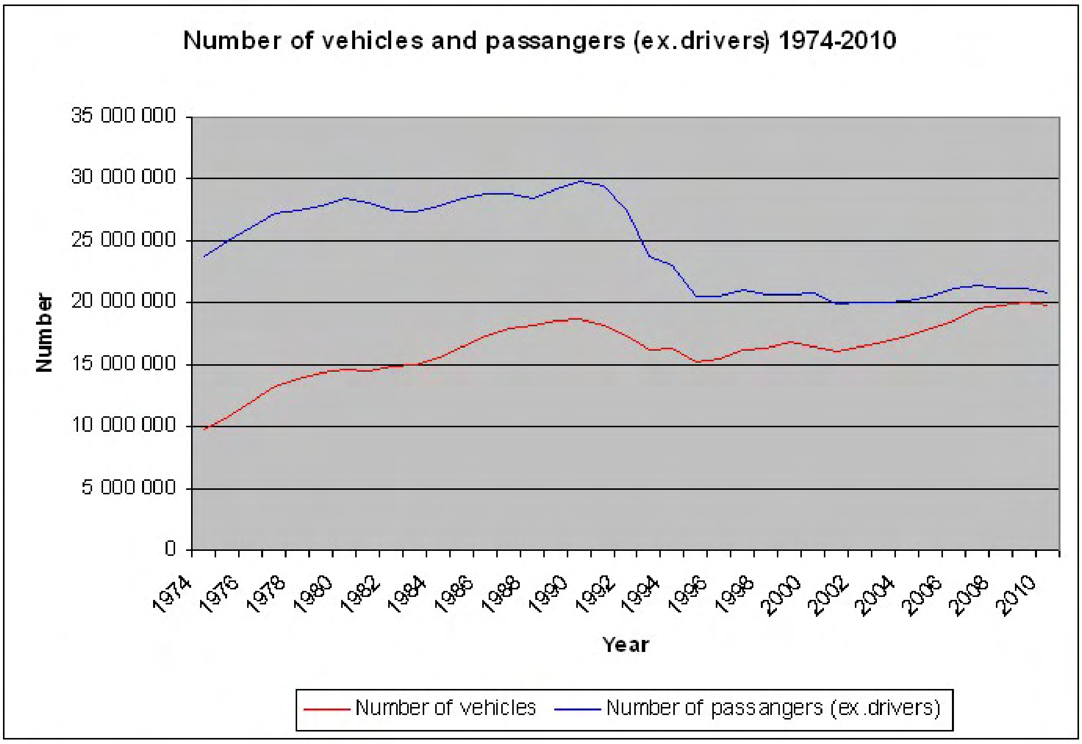


*Fig. 4 Traffic growth in the domestic ferry market 1974 – 2010*

In fig. 4 the growth in the domestic ferry market from 1974 to 2010 is indicated. This figure shows that the number of vehicles transported has more then doubled, but the number of passengers is reduced by 3 mill. in the same period. This shows that the number of cars has grown substantially in the same period and the number of passengers (exclusive drivers) pr car has fallen from 2.46 to 1.05.

It should be added that at the end of the 80-ties, during the 90-ties and also the last ten years, quite a lot of heavy traffic ferry connections have been replaced by fixed connection – bridges and tunnels. Since 1986 the number of ferry connections has been reduced from 161 to 125.

### *2.3 Ownership to Norwegian ferry companies*

In the beginning of last century a lot of local companies were established to serve local and regional areas with transport services for goods and passenger transport. The companies were locally owned and very often wholly or partly publicly owned – local and/or county councils. Those companies gradually turned into ferry companies as roads and infrastructure were developed.

The companies were mainly non profit organizations or giving a very small dividend to the owners. Their main goals were to be contributors to building transport infrastructure and transport supplies for local and regional areas.

This situation prevailed till the late 80-ties. In 1990 there were about 18 ferry companies in Norway. Today we have 4 independent ferry companies or owners constellations operating on the Norwegian coastline. The reason for this development will be explained later in this presentation.

## 3. THE KEY TO EFFECTIVE FERRY OPERATIONS

### *3.1 What do we mean by effective ferry operations*

In basic textbooks in economics efficiency is often defined as a situation where a service or commodity is produced at a lowest possible use of recourses (or costs) or a maximum number of a service or commodity is produced by a given number of resources (or costs).

This general definition does not help in assessing whether the ferry industry is effectively operated or not.

A more practical but commonly accepted definition of effective ferry operation can be:

- Ferry services must be produced at low costs – no barriers for new operators to the ferry market.
- A fare system that reflect the (marginal) cost of producing ferry services – and a man effective fare system that passengers find easy to use.
- A reliable service that run on time and with few cancellations.
- A safe service with few accidents.
- Effective terminal operations.
- Environmentally sustainable ferry operations.

Operations that fulfill those requirements can be said to be effective.

### *3.2 The concession system*

The Norwegian ferry industry is strictly regulated. Historically the ferry companies applied (to central or local government) for a concession or (license to) operate a ferry connection. The concession was normally given for 10 years.

To prevent the ferry companies from obtaining monopoly profit, both prices and production (number of crossings per day and seize of the ferries) were regulated by the public authorities.

The prices were normally fixed so low and the production so high that it was not possible to obtain positive profit from ferry operations. Therefore nearly as all ferry connections in Norway were subsidized by the public authorities.

Since the beginning of this century, legislation is changed. Now the operation of a ferry connection is based on tender. The ferry company that can operate a connection or a package of ferry connections with the lowest need for subsidy (lowest cost and/or the highest expectations when it comes to traffic and income) is given a concession/license to operate a connection, normally for 8-10 years. Still under the condition that prices is regulated by public authority and production is according to the condition in the tender.

By using tender as an instrument for selecting an operator for a certain ferry connection, the public authorities (and the taxpayers) are secured that the most effective operator will be operating a certain connection/certain connections.

By introducing the tender system also new operators that have not been in the ferry industry before, could take part in the competition.

This way of organizing the ferry industry secure efficiency in the domestic ferry industry.

Introduction of tender as a way of selecting the most cost effective operators, also lead to merging in the ferry industry – a way of reducing the risk of loosing the whole fortune if one should loose a tender competition.

### *3.3 The fare system for car ferry transportation*

As mentioned earlier in this paper, prices (fares) have been strictly regulated since the 70-ties. The fare system is known as “Riksregulativet for ferjetaksteer”.

All car ferry companies in Norway that receive subsidies from the Public Roads Administration (or County Authority), have been obliged to use the same fare system - the so called National Faresystem for Ferry Transportation (NFFT). (Riksregulativet for ferjetakster).

The NFFT consists of two elements - the distance of the journey (distance zone) and the length of the vehicle.

Each distance zone is 1 km so that a ferry journey between 14.0 and 14.99 km is placed in zone 14 and so on.

The NFFT classifies the length of the vehicles into 9 different categories – B2 to B10 - where B2 is cars up till 6 meters and B10 is vehicles between 19 and 22 meters.

The fares for passengers and motorcycles are also depending on the length of the journey.

The tickets are either collected/bought on board the ferries or in certain ticket offices (kiosks) on shore.

The Ministry of Transport and Communications is each year deciding the increase in ferry fares. The fare level is in that way politically decided. It is also politically expressed that it should not cost more to travel by ferries than it costs to drive the same distance by car. To a certain degree this is used as basic in the NFFT system; it costs more the longer the crossing, and it is also more expensive to carry long vehicles than short ones.

However, no attention is paid to the fact that total costs of driving a certain distance also include the value of travel time, not only the operating costs of a certain vehicle. When we include the value of travel time, it is more expensive to travel a certain distance by ferry than the same distance by road.

One can also buy a so called "value card" to achieve a discount on a ferry trip. These discount cards can be bought at the price of NOK 2,900 for small cars, NOK 11,500 for medium sized cars and NOK 21,800 for heavy vehicles and is valid in all ferry connections on the western coast of Norway giving a discount of 50%.

Passengers are given a discount of 17%.

It should also be mentioned that for 5 years a pilot fare project has taken place in the ferry connection Flakk – Rørvik .

This system is called AutoPASS - automatic ticketing for ferries – and is based on the concept that one only pays for cars being transported. – This system has only 3 length categories - less than 6 m, 6 – 12 m longer than 12 m.

The length of the vehicles is automatically read when they drive on board the ferries either by reading the information laying in AutoPASS chips bought in advance or by reading the license plates and later linking the numbers on the license plates to a national license register. Prepaid AutoPASS chips give you 50% discount and those without such chips are invoiced afterwards and have no discount.

Both systems have its advantages – the manual system with passenger payment and a relatively fine graded length payment for cars, secure that travelers (cars and passengers) willingness to pay is collected, and that everybody contribute to cover the cost of operating a ferry and the level of payment can be kept relatively low.

The AutoPASS system is more man effective and demands less crew on board the ferries and ashore to be operated. The AutPASS system for ferries is more like the systems we find at toll stations on roads and tunnels. That, together with increasing wage costs, will in my opinion lead to that the future fare system for ferries will be based on the concept that we find in the AutoPASS system.

The weakness or drawback with both the systems is that everybody that buys a “value card” or an AutoPASS chip in advance, obtain 50% discount on a ferry trip. There is no mechanism to spread the traffic or no peak load pricing in the fare system. That led to very low average capacity utilization in the domestic ferry industry in Norway. Based on experience from ferry operation in Norway, it may be stated that a capacity ceiling is reached when the average capacity utilization is 45% on a yearly basis.

### *3.4 A reliable service*

Ferry operations are reliable. In the Fjord1 company the degree of regularity obtained in 2012 was 99,52%. That means that out of 1000 planned departures less than 5 do not go as planned. Similar figures can likely be found for the rest of the Norwegian ferry industry.

### *3.5 A safe service with few accidents.*

There are few serious accidents in the Norwegian ferry industry. Fjord1 MRFtransported more than 500 million passengers and drivers the last 50 years without any fatal accidents. Similar patterns may be found at other ferry companies in Norway.

In some ferry connections there can be as many as 60 calls (arrivals and departures) at a ferry terminal per day. Each call represents a possibility for a hard landing that can cause damage on vehicles and passengers due to changing wind and sea current and even misjudgment from the crew. Fjord1 has therefore installed satellite surveillance of the ferries with voice warning in the wheelhouse if the ferries have to high speed when approaching the ferry terminal or if a ferry is on a wrong course. That is similar procedures that we can find in airplane cockpits to improve safety.

### *3.6 Ferry terminals and terminal time*

Since most of the fjord crossings take only 10 – 30 minutes, it is of great importance to focus on terminal solutions to reduce the terminal time and by that maximize the number of round trips pr day.

When The Norwegian Directorate of Public Roads – NDPR – (Vegdirektoratet) in 1960 -70 started the work to standardize the ferry fleet (especially double ended ferries), they also started a work to standardize the ferry terminal. In that way ferries could be used not only in one connection but being used in several connections of more or less same traffic volume. In that way the number of reserve ferries or supplementary ferries can be minimized.

The "state of the art" for the ferry terminals may be summarized in this way:

- No ferry terminal has personnel on shore to operate the terminal installations. The "ferry bridge" (the moveable bridge that connect the terminal to the ferry – adjustment for the tidewater) is wirelessly operated from the ferry by the crew on board the ferry as the ferry approaches the terminal.
- Some ferry terminals also have arrangements that automatically adjust the ferry bridge to the height of the ferry deck.
- The ferry bridges have one, two or three lanes depending on the size of the ferries that operate the different connections. That means that one, two or three cars can drive ashore and on board at the same time and that one can embark and disembark a ferry simultaneously.
- Normal terminal time (embarking and disembarking) for a 100 PCU ferry is less than 5 minutes.
- Pedestrians and cars are separated on terminals with high traffic.
- All ferry terminals are publicly owned so that change of operators, if an existing operator is losing a tender, can easily be done.


*Fig. 5. Ferry terminal serving a 100 PCU ferry*

### *4.7 Environmentally sustainable ferries – green ferries*

#### *4.7.1 Some background information*

In 1996 the Norwegian Parliament decided that Norwegian ferry operators should build two ferries using natural gas as energy producer. One of the ferries was meant to run on CNG (Compressed Natural Gas) and the other one on LNG (Liquefied Natural Gas). The reason for this decision was to obtain experience from more extensive use of the huge natural gas resources from the North Sea in the transport sector in general and in particular in the ferry sector. An additional aim was to reduce pollution from the transport sector.

NDPR gave Fjord1 MRF the task of building a LNG operated ferry. The CNG ferry project was later on postponed.

The gas ferry was design wise based on conventional double ended ferry.

The ferry carries 87 PCU and was certified for 300 passengers. The ferry was delivered in January 2000.

This world's first gas operated ferry (allowed to carry passenger) – "Glutra" – has later been converted and extended by a new section of 26 meters.
This world' first gas operated ferry has now been in operation for 13 years and sailed for more than 65.000 hours without any technical problems.

*Fig.6 The gas ferry "Glutra"*

*4.7.2 Environmental effects*

Based on theoretical considerations, one expected a reduction in $NO_x$ emissions of about 90% compared to a modern diesel electric ferry. Calculations made on board the ferry indicate a reduction of about 80%. Throughout a year that means a reduction of about 32 tons of $NO_x$ for that particular ferry of that seize.

The reduction in $CO_2$ was expected to be around 20%. Calculations made on board the ferry indicate a reduction of about 25-30% compared to a quite similar diesel electric ferry. Throughout a year that means a reduction of about 802 tons of $CO_2$.

*4.7.3 Further use of gas ferries*

**The Rogaland and Hordaland case**

- After winning a tender, Fjord1 ordered 5 gas ferries in 2004. 2 of the gas ferries were going to operate in the county of Rogaland and 3 in Hordaland – each ferry was designed to carry 212 PCU.
- The ferries were designed by LMG Marin in Bergen
- The ferries started to operate by turn of the year 2006/2007
- The 3 ferries in Hordaland are designed to operate at a speed of 21 knots, while the 2 ferries in Rogaland are disigned to operate at a speed of 17 knots
- The 21 knots ferries have 4 Rolls-Royce gas engines installed with a total output of 11.800 kw.
- The 17 knots ferries have 2 Rolls-Royce gas engines installed with a total output of 5.000 kw.
- Each ferry has 2 x125 $m^3$ gas tanks installed

*Fig.7 One of seven sister ferries operating in Rogaland and Hordaland*

**The Moldefjord case**

- In 2007 the Norwegian Directorate of Public Roads announced a tender parcel with 4 ferry connections in the Romsdal Region – one of the connections was supposed to be operated with 3 x120 PCU gas (LNG) ferries. The tender was supposed to start from 01.01.2010 for a period of 10 years.
- Fjord1 won the tender competition, and signed a contract with the Polish shipyard Remontowa for building of 3 LNG operated gas ferries in May 2007.
  - ”Moldefjord” – delivered in November 2009
  - “Fannefjord” – delivered in March 2010
  - “Romsdalsfjord” – delivered in July 2010
- The gas ferries were designed by LMG Marin. They are each equipped with 2 gas engines and one diesel engine. All connected to a common electric system running the propeller via frequency converters. The ferries can be operated on one gas engine only. Each engine will deliver 900 kw at full load.


*Fig 8. One of three sister ferries on The Moldefjord*

**The Flakk – Rørvik case**

- As a result of winning another tender competition, Fjord1 is operating a ferry connection with 3 gas ferries between Flakk – Rørvik close to Trondheim. The tender started 01.01.2011.
- One of the ferries is identical to ”Moldefjord”.
- ”Glutra” was converted and extended with a new section of 26 meters – increasing the capacity from 87 to 120 PCU.
- ”Tresfjord” a diesel electric 125 PCU ferry built in 1991 has been converted from diesel electric to gas electric operation. A new gas engine was installed running 2 converters (one for each electric engine running the propeller system in each end of the ferry). One can also choose to run the ferry on the diesel engine when overhauling the gas engine or the gas system.

*Fig. 9 “Tresfjord” converted from diesel electric to gas electric ferry*

**The Vestfjorden case**

After winning a tender, the ferry company Torghattten Nord ordered 4 gas ferries from the Polish yard Remontowa. The ferries varies in seize from 80 to 120 PCU and is built for open sea operations. They are designed to operate at a speed of 17 knots. The operation started 01.01.2013.

*Fig.10 Seagoing gas ferries for the Vestfjorden connection*

**Other cases**

Three more gas ferries have been delivered to Norwegian ferry operators and also ferries for passengers only are in operation in the Oslo basin.

*4.7.4 Distribution of gas*

In the last few years large quantities of natural gas especially in shale gas has been found. That has led to a fall in gas prices worldwide. On the domestic gas market in Norway – especially gas deliveries for the ferry industry –no such price fall has occurred. The gas prices are still closely linked to oil prices – market for future prices is immature.

That has to do with the fact that there are still few national suppliers of gas in Norway. Tank depots for gas along the Norwegian coastline (supplied by tankers) are very limited. Most of the supply to the ferry industry takes place from trucks that can be driving more than 500 km from their gas plants.

For the future tank depot along the coastline (supplied by small tankers) with local distribution by trucks will represent a flexible efficient solution for small scale distribution of gas to the ferries and other industries. That, together with more distributors, will most probably lead to a reduction in the gas prices to the final consumers (like the ferry industry).

To achieve such a solution, it is probably necessary to have some public involvement in a transition period.

*4.7.5 The experience so far with gas operated ferries*

The first ferry built for gas operation has now been running for 13 years and more than 65.000 hours and is still in operation. The experience so far is that this ferry has been just as reliable and economically to run as a conventional diesel operated ferry. The same experiences have been recorded for the other ferries built for gas operation independent of seize, speed and trading areas.

The manning of those ferries is the same as diesel operated ferries of same seize, and the noise level is lower than on a diesel operated ferry. Maintenance and other operational costs on gas ferries are at the same level as diesel operated ferries.

The investments in gas ferries are somewhat higher than in conventional diesel operated ferries – approximately 10- 15 % higher.

The effect on the environment is well documented. The reduction in $NO_x$ emission will be approximately 80- 85% and the reduction in $CO_2$ 25-30% compared to a conventional diesel operated ferry. There is no particle pollution from gas operated ferries. The noise level is very low.

## 5 SUMMARY

This paper has shown that the Norwegian ferry industry fulfill the requirements to be regarded as an effective industry when it comes to:

- The concession system
- The fare system
- Reliability
- Safe operations
- Effective ferry terminals
- Environmentally sustainable ferries – green ferries

But there is still some improvement to be done. The fare system should be changed so that more traffic is spread to off peak period – introduction of some sort of peak load pricing or off peak discount. That would lead to a higher average capacity utilization and we could postpone investment in increased capacity.

Furthermore the gas supply should be improved on the Norwegian market so that the ferry industry could benefit from the low prices one have on the international gas market and thereby speed up the transition from oil to gas as fuel for the ferry industry.

Future developments are likely to bring about:

- More use of gas as fuel for ferries
- AutoPass for ferries ticketing
- Use of battery as a substitute for oil and gas as fuel – as research project in the near future
- Hybrid solutions – both gas and battery as fuel on the same ferry
- Improved hull shape and propeller efficiency to reduce fuel consumption
- Extended use of satellite and electronic equipment to monitor ferry operations to make the ferry trip even more safe than today

## 6 CLOSING REMARKS

On the Norwegian coastline a great number of proposals are suggested for fixed fjord crossings replacing ferries with tunnels and bridges. More information on these projects will be presented at this conference. Most of those projects will require resources far beyond what will be available in the near future.

In 1948 Samuel Beckett wrote the famous play "Waiting for Godot" with two characters, Vladimir and Estragon, waiting endlessly and in vain for the arrival of someone named Godot that never showed up. Many of the fixed fjord crossing projects on the Norwegian coastline have resemblance to Vladimir and Estragon endlessly waiting for Godot.

While waiting for Godot – while waiting for fixed crossings, - within 1 – 2 years by using relatively small resources, the number of green ferries could be increased so that every major ferry connection could have a departure every 15 – 20 minutes.

And should Godot one day show up – should the bridges and tunnels one day be built – investment in green ferries represent no "sunk costs". Ferries will always have a value in alternative use.

In the near future, in my opinion, more attention should be paid to investments in roads between ferry connections and less to replace ferries with fixed connections.

And when one day when a sufficient level of knowledge and technological capacity is reached to build bridges and tunnels at acceptable costs and within the economic limits that a nation can use, I wish tunnels and bridges welcome.

# HOW CAN FERRIES BE A GOOD AND LASTING ALTERNATIVE IN CROSSING FJORDS?

*By Chief Commercial Officer Hallgeir Kleppe, Fjord1*
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## ABSTRACT

*Fjord1 AS is a ferry operator, serving passengers mainly on the Norwegian west cost between Stavanger and Trondheim. Leif Øverland, CEO in Fjord1 AS, has launched the idea of "free float" in the Norwegian ferry business. This means that the ferries connecting the highways should have a frequency in departures, capacity and opening hours that meets the expectations of customers in respect of ferry facilities going from A to B.*

*A bridge or a tunnel is regarded as the only really good alternative to traditional ferry operation. The total cost in maintaining the tunnel or the bridge can be quite significant in a life cycle cost view. How can the ferry business meet this challenge with economic, environmental and operational efficiency?*

*This article gives a review of how Norwegian ferries have developed during the last 30 years in respect to kind of fuel and fuel efficiency, size and organisations of crews, capacity, security, engines, propulsion, ticketing, need for docking time, design of ports, opening hours, operational speed, redundancy and so on.*

*During the last years, there have been significant investments in the fleet, but Norwegian ferries have still a high average age. An old fleet has higher operational cost and environmental emissions than a newer. The Fjord1 ferry fleet has an average age of almost 22 years. The age of an average PCU in Fjord1 is 15 years due to the fact that newer ferries are normally bigger ferries. This is most likely the same as for all ferries in Norway.*

*How will the access to sufficient and skilled crew affect the ferry business in the future? What are the instruments in recruiting and educating young people for our business?*

*What expectations and ambitions do Fjord1 have for the future, aiming for the most effective and seamless ferry - operation?*

*As an example the Fjord1 operations on E39 between Bergen and Stavanger will be presented where now 6 major (212 PCU) LNG ferries are in service. (Personal Car Unit - PCU– is a table used to convert bigger cars and trucks to normal sized cars. As an example, a truck that is 22 meters is calculated as 10,682 PCU)*

**CAPACITY AND FREQUENCY IN FERRY CROSSINGS**

The CEO in Fjord1, Mr Leif Øverland, has launched the idea of "free float" in the Norwegian ferry business. This means that the ferries connecting the highways should have a frequency in departures, capacity and opening hours that meets the expectations from customers in respect of ferry facilities going from A to B without needing a timetable. In fact, there should always be a ferry leaving or approaching the port in a very few minutes.

In some crossings among the 20 largest in Norway, this is in fact the situation today. At least in some parts of the operating time during 24 hours these crossings are operated with a frequency of one departure each 20 minutes. With 3 departures each hour there is in practical terms no waiting time and the customer can arrive at the port whenever he or she likes.

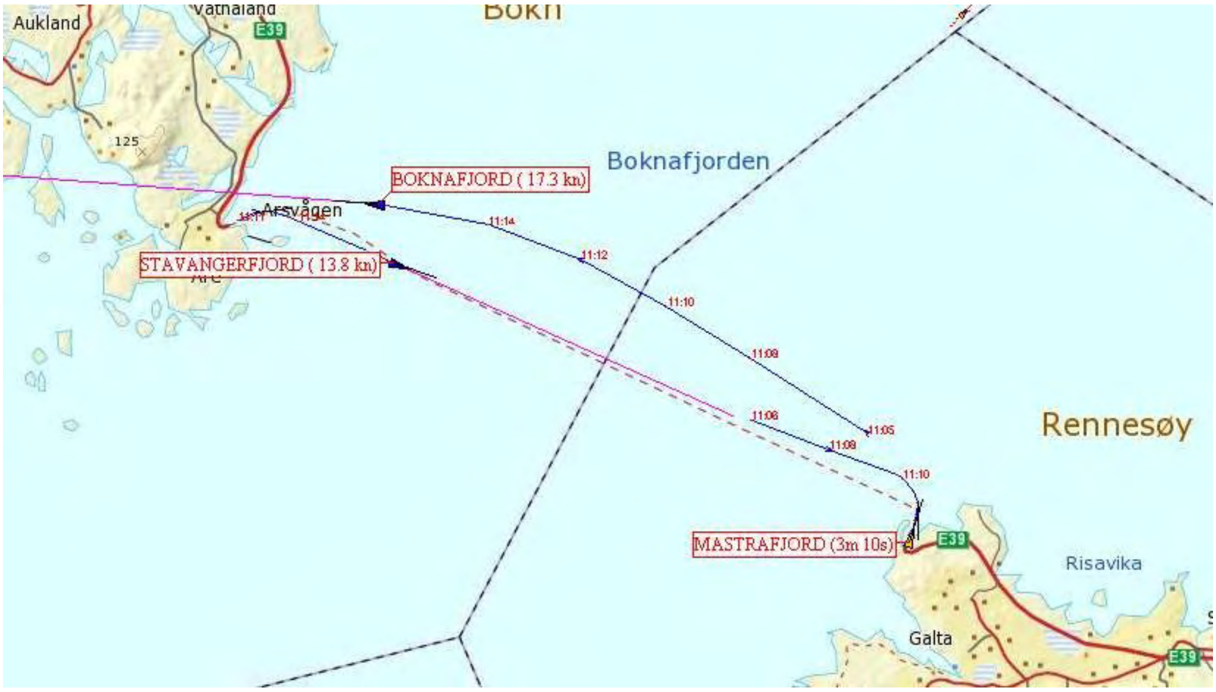


*Figure 1 Map showing location of ferry crossing between Arsvågen and Mortavika.*

On the map in Figure 1 above, an image from Norway's second biggest ferry connection between Arsvågen and Mortavika on the E39 highway between Bergen and Stavanger is shown. The ferry "Stavangerfjord" has just left the port in Arsvågen and the ferry "Boknafjord" is approaching the same port. The ferry "Mastrafjord" will leave her port in a few minutes and by then, "Stavangerfjord" will be not far from the same port area. The actual crossing time, included manoeuvring in port, in this line is 22 - 25 minutes and needed time in port to unload and load is 5 – 8 minutes. So each ferry has a new departure each 30 minutes.

**E39 - WHAT WILL IT TAKE TO INCREASE THE FERRY PERFORMANCE?**

Figure 2 shows that there are several ferry crossings on the E39 Highway between Kristiansand and Trondheim. All together there are 7 crossings and 17 ferries of different size and speed in firm and daily operation to cover this operation. There is also a need for replacement ferries to cover for the planned maintenance and disruptions that normally occurs from time to time during operation.


*Figure 2 Coastal Highway E 39*

The Norwegian ferry companies have their interest in delivering a good operation in accordance with signed contracts and also according to the need of the people and business activities in the region.

Two of the crossings on E39, Arsvågen – Mortavika and Oppedal – Lavik, are now already operated, or are decided to be operated, at a frequency of a departure each 20 minutes.

Waiting for the tunnels and the bridges to be built, there should be a plan to increase the capacity and frequency for also the remaining crossings over some time.

In this respect there are several factors that have to be considered.

**Halhjem - Sandvikvåg**

| Exact simulation - existing operation: | | |
|---|---|---|
| Distance in n.miles | 11,88 | → 22,00 KM |
| Average speed in Knots | 20,35 | |
| Time for manoeuvring | 4,00 | minutes |
| Unloading and loading | 6,00 | minutes |
| Depature each | **30** | minutes |
| Logic number of ferries | 3,000 | |
| Crossing and maneuvering | 39:01 | minutes |
| Actual number of ferries | **3** | |

| Exact simulation - new operation, alt 1: | | | Exact simulation - new operation alt 2: | | |
|---|---|---|---|---|---|
| Distance in n.miles | 11,88 | → 22,00 KM | Distance in n.miles | 11,88 | → 22,00 KM |
| Average speed in Knots | 24,56 | | Average speed in Knots | 17,59 | |
| Time for manoeuvring | 5,00 | minutes | Time for manoeuvring | 3,50 | minutes |
| Unloading and loading | 6,00 | minutes | Unloading and loading | 6,00 | minutes |
| Depature each | **20** | minutes | Depature each | **20** | minutes |
| Logic number of ferries | 4,000 | | Logic number of ferries | 5,000 | |
| Crossing and maneuvering | 34:01 | minutes | Crossing and maneuvering | 44:02 | minutes |
| Actual number of ferries | **4** | | Actual number of ferries | **5** | |
| **Increast frequency** | **50 %** | | **Increast frequency** | **50 %** | |
| **Reduced crossing time** | **05:00** | minutes | **Increased crossing time** | **05:00** | minutes |


*Figure 3 Example of calculated operation time, speed and number of ferries.*

Environmental, economic, operational, logistics and other questions should be asked and evaluated.  The relationship between speed and capacity, cost and emission on each ferry is significant in order to optimize the required number of ferries. The example in Figure 3 above shows the calculation in the operation and how crossing speed, manoeuvring time and unloading and loading time are combined.

Fjord1 operates the connection **Halhjem – Sandvikvåg** with 3 ferries, each with an operational speed of 20 – 21 knots. They produce a frequency of a departure each 30 minutes. To increase this frequency by having a departure each 20 minutes, either 4 ferries with a speed of 25 knots or 5 ferries with a speed of 18 knots must be operated in daily service. For environmental and technological reasons it is strongly recommended to choose the slower alternative.

*Figure 4 ferry serving the Halhjem – Sandvikvåg crossing*

In Fjord1, there is great scepticism to operate these huge double ended displacement ferries at 25 knots. In fact, the needed power to give an extra 4 knots would more than double the power in the engine installation. Also the propulsion equipment to give this level of speed for a double ended ferry, would be very challenging. Furthermore, an increase in the fuel consumption by significantly more than 100% for an increase in speed with 4 knots to 25 knots is expected.

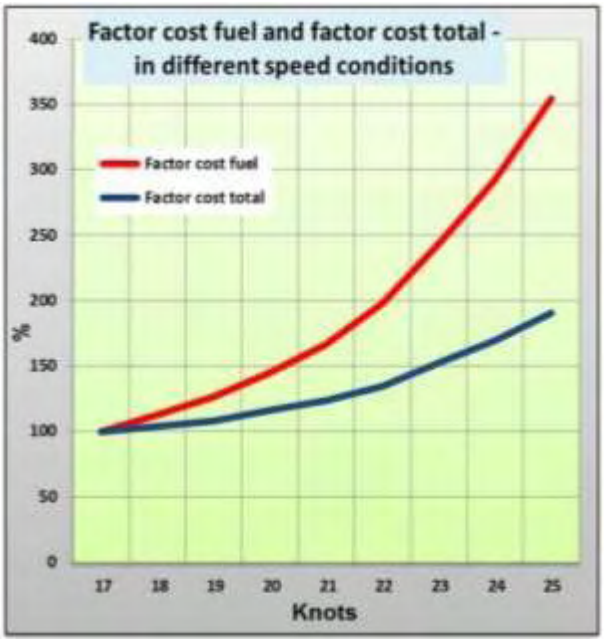


*Figure 5 Cost factors for the total ferry fleet at different speeds for Halhjem - Sandvikvåg*

As can be seen from Figure 5, the reduced cost because of a reduction in the logical number of ferries is not at all sufficient to cover for the increased fuel and maintenance expenses.
In general terms, the terminal facilities and more specific, the installation of an AutoPASS system, is a necessary prerequisite for an efficient ferry operation. AutoPASS means that an electronic chip in the car's window will be read electronically and the customer will be charged without stopping the vehicle while entering the ferry. In order to get an efficient use of an

AutoPASS system the authorities must simplify the fare system to make it work in a proper and easy way. Fjord1 are certain that a seamless operation and increased efficiency in departure frequency and payment facility, will lead to an increased satisfaction by the customer.

A reduction of the needed time in port is one of the most significant means to reduce the cost and also the needed number of ferries for a certain operation.

## FJORD1 HAS FOCUS ON SEVERAL ASPECTS THAT WOULD LEAD TO BETTER FERRY SATISFACTION:

- The ferries operating in a crossing should be of the same kind, size and speed in order to secure the optimal distribution of the traffic. This is important both for the security on the roads leading to the ferry and also for the fact that ferry operators, want to have the ferries most equal for security, maintenance and economic reasons.
- On major lines and crossings –at least in the medium term - there should be a departure each 20 minutes from 7 am to 7 pm.
- With a modified and adapted schedule during holidays and weekends, this operation and frequency should, be sufficient until the crossing is substituted by a tunnel or a bridge.

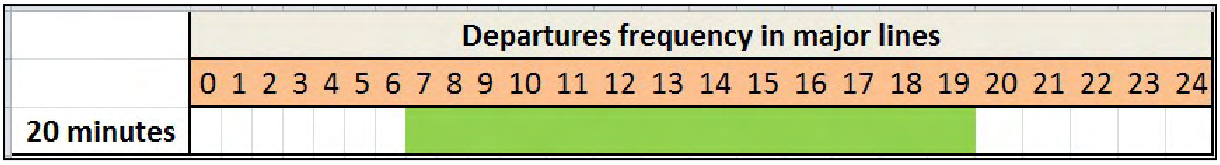


*Figure 6 Departure frequencies in major lines*

## THE DEVELOPMENT IN ONE SPECIFIC LINE THE LAST 30 YEARS: LAVIK – OPPEDAL ON THE NORWEGIAN WEST COAST:

In 30 years the number of departures from one port in the Oppedal - Lavik crossing, Figures 7 and 8, has increased in a dramatic way as there where 10 departures in 1985 – and in 2015 there will be 50 departures each day from each port. The offered capacity each day has been increased from 350 PCU to 6.000 PCU. To calculate percentages has no meaning in this case. In 1985 there was transported 170.700 PCU whilst in 2012 Fjord1 transported 1.015.000 PCU. That is a yearly average increase of 6.8%.


*Figure 7 Ferry crossing at Lavik - Oppedal*

| From Lavik | | | From Lavik | | | |
|---|---|---|---|---|---|---|
| 10 departures each day | | | 50 departures each day | | | |
| Capasity 350 PCU each day | | | Capasity 6.000 PCU each day | | | |
| 1 ferry a 35 PCU | | | 3 ferries a 120 PCU | | | |
| Time table 1984 | | | Time table 2015 | | | |
| | 06:00 | | 00:30 | 10:10 | 14:50 | 19:10 |
| | 07:40 | | 01:30 | 10:30 | 15:10 | 19:30 |
| | 09:25 | | 02:30 | 10:50 | 15:30 | 19:50 |
| | 10:45 | | 03:30 | 11:10 | 15:50 | 20:10 |
| | 12:35 | | 04:30 | 11:30 | 16:10 | 20:30 |
| | 14:45 | | 05:30 | 11:50 | 16:30 | 21:10 |
| | 16:00 | | 06:30 | 12:30 | 16:50 | 21:30 |
| | 17:45 | | 07:30 | 12:50 | 17:10 | 21:50 |
| | 19:15 | | 08:00 | 13:10 | 17:30 | 22:30 |
| | 20:40 | | 08:30 | 13:30 | 17:50 | 22:50 |
| | | | 09:00 | 13:50 | 18:10 | 23:30 |
| | | | 09:30 | 14:10 | 18:30 | |
| | | | 09:55 | 14:30 | 18:50 | |

*Figure 8 Changes in departure frequencies at Lavik*

## COSTS ASPECTS

The cost in operating ferries is quite predictable for the community as the contracts are awarded through public competition. Of course the Norwegian ferry operators have too low return on their invested capital. Reference is here made to the company Oslo Economics that has evaluated the results of the ferry competition these last years on behalf of the Norwegian Public Road Administration (NPRA) [1].

A bridge or a tunnel is regarded as the only really good alternative to traditional ferry operation. However, the total cost in ventilation, cleaning, maintaining and renewing the tunnels or the bridge after some years, can be quite significant, at least in a life cycle cost view. The priorities in the society, financing the new and very expensive fjord crossings - versus maintaining and improving the roads between the ferry ports should also be considered. E39 between Kristiansand and Trondheim has indeed a need for upgrading of existing infrastructure many places. Can the ferry operators meet this challenge with reliable, economic, environmental and operational efficiency? It will be significant to choose the most economic solutions for the community.

Ferries have an economical lifetime of some 30 – 40 years, are flexible and can be easily moved in order to adjust the production to the demand. When a tunnel or a bridge is insufficient to meet the demand, the consequences can be quite high for the community and it will normally take many years to adapt the capacity to the need.

**ENVIRONMENT**

The Norwegian ferries have had a significant development during the last years. Attention should be paid to the fact that both hulls, propulsion, engines, knowledge and attitude among the company and the crew has been developed and improved significantly. The company Multi Maritime AS in Førde and Fjord1 AS have launched specifications for a LNG Hybrid ferry back in 2012, Figure 9. This ferry will make it possible to carry out a full scale ferry operation in combination with the highest environmental results. .

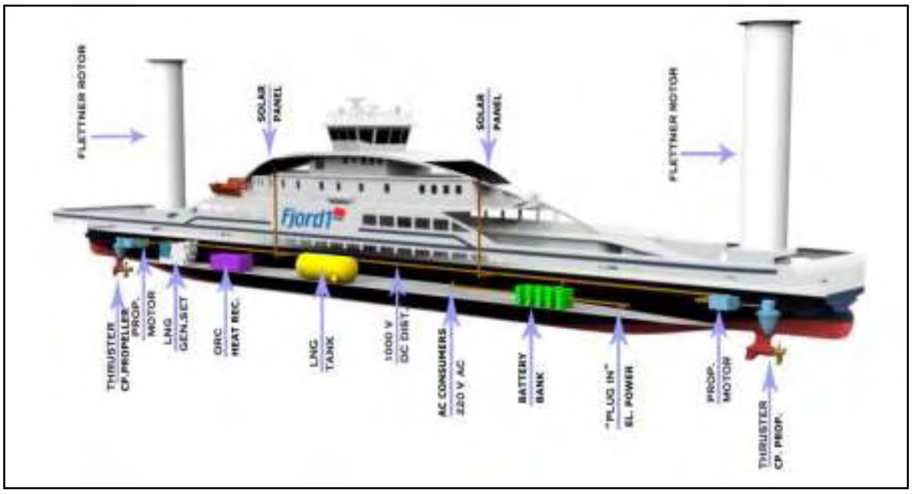


*Figure 9 Environment friendly ferry concept*

Also bear in mind that in periods, development and new buildings of ferries have contributed to technological development in large parts of Norwegian maritime industry.

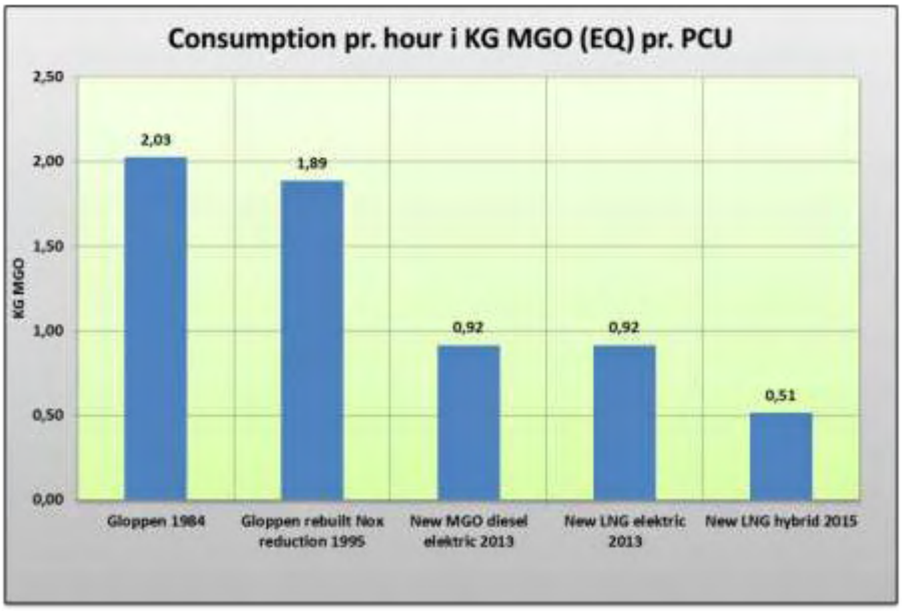


*Figure 10. Energy consumption in KG MGO per offered PCU facility*

Consumption pr. CPU pr. hour on the line Oppedal – Lavik has been reduced from 2.0 KG to 0.5 KG during a period of 30 years. That is a reduction of 75% and that is a quite dramatic improvement. The comparison in 2015 is based on the new LNG Hybrid ferry that Fjord1 offered to operate the Lavik – Oppedal line mention earlier. In this Fjord1 ferry, smaller LNG engines will be combined with a minor battery package. The batteries will be charged by surplus power from the LNG engines, ordinary electric power supply and also from solar energy. There is also a planned installation of modern sailing facilities – a Flettner rotor – that uses the wind energy to give an extra speed or even more reduced consumption.

The main case here is to have relatively small engines installed in the ferry – just enough to operate during sailing at sea. When operating in harbour or in bad weather, the battery package can be used to increase needed power. This is the same technology that can be seen in modern cars now, example the Toyota Prius. Unfortunately, this Multi Maritime / Fjord1 solution was not chosen by NPRA. Fjord1 is confident that this solution will be a very good one, probably the best, and hope to build this ferry quite soon.

The development in the ferry business in respect of **NOx emission**, have been through an even more amazing development, Figure 11. A new MGO diesel electric ferry will have a reduction of 80% and a LNG ferry – 97%.

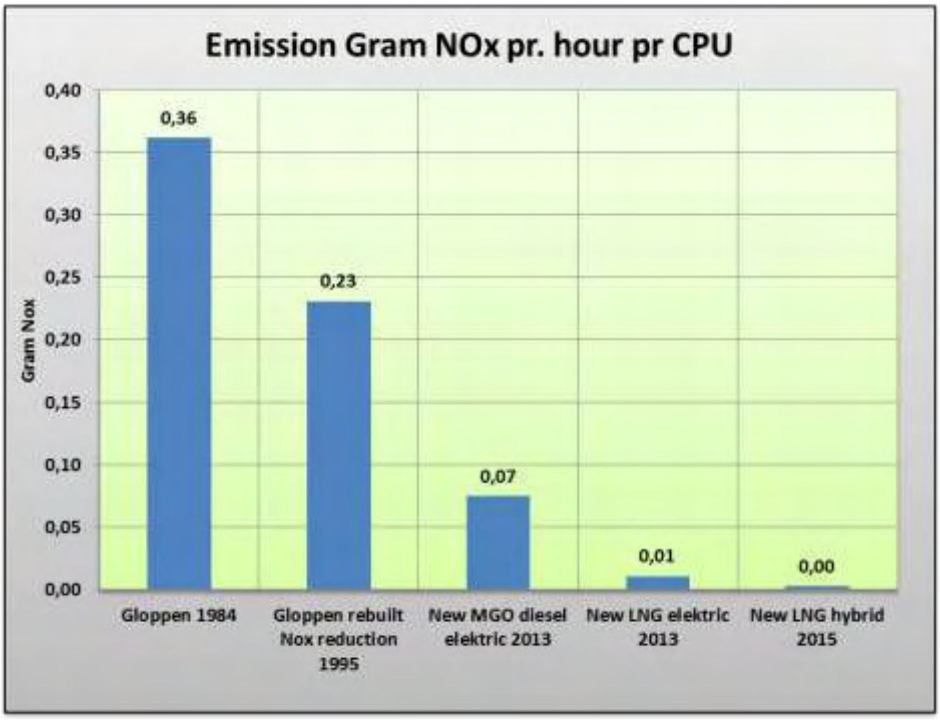


*Figure 11 Emission of $NO_x$*

The reduction in NOx emission is actually at the level of 99 % from the basic ferry Gloppen to the new Multi Maritime / Fjord1 LNG Hybrid ferry. In other words – the NOx emission from the newest types of ferries would be at a level that probably would compete with any other alternative when consideration to the life cycle, the emissions from the construction period and of course – the maintenance, is taken into account.

Also the **emission of $CO_2$** has been reduced during these last years, Figure 12. Reduced consumption and the type of fuel used – LNG, is the main factors to achieve fine results. The new Multi Maritime / Fjord1 LNG Hybrid ferry will contribute to a reduction in $CO_2$ emission of more than 80%.

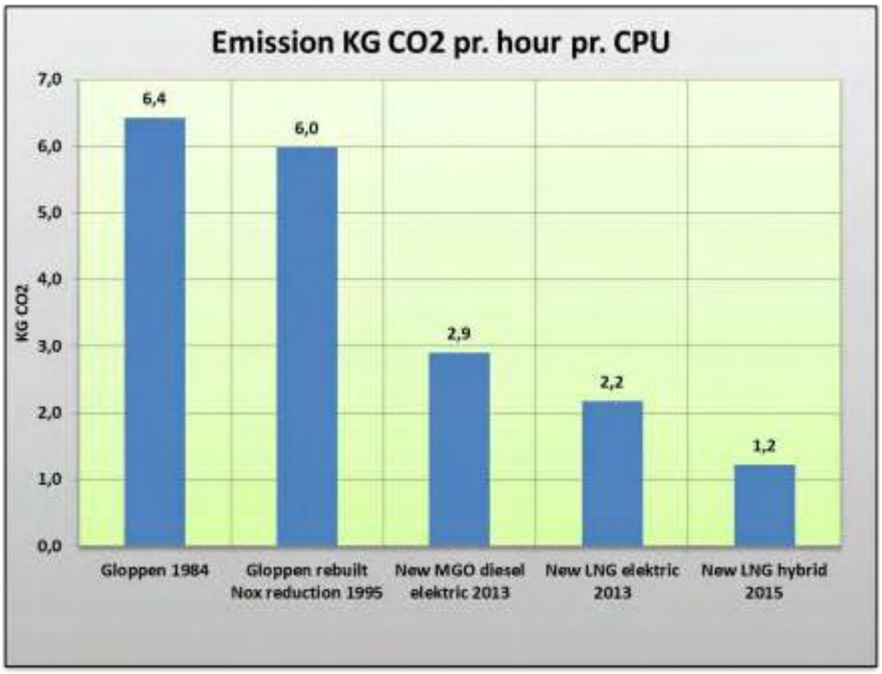


*Figure 12 Emission of $CO_2$*

In addition focus is also on the emission of **sulpha and particles** that in practical terms has been eliminated by the use of LNG.

## CREW RECRUITEMENT

Norwegian ferries has been through a significant development during the last 10 years in respect of kind of fuel and fuel efficiency, number employees in the crew, capacity, security, engines, propulsion, ticketing, need for docking time, design of ports, opening hours, operational speed, redundancy and so on.

*Figure 13 Ferry crew*

During the last years, there have been significant investments in our fleet, but Norwegian ferries have still a high average age. An old fleet has higher operational cost and environmental emissions than a newer. The Fjord1 ferry fleet has an average age of almost 22 years. The age of an average PCU in Fjord1 is 15 years. This is most likely the same as for all ferry companies here in Norway.

How will the access to sufficient and skilled crew affect the ferry business in the future. The ferry companies in general take a significant responsibility for training young people to be seamen. Through the ferry companies, the authorities finance this practical training.

However, there is still a high average age of people working on the ferries and there is a need for more recruiting activities.

**THE BENEFIT OF A BRAKE ON THE JOURNEY –**
**CAN AN EFFECT ON SECURITY ON THE ROADS BE ANTICIPATED?**

Surveys indicate that our customers enjoy a brake when driving long distance. But they want to have the rest on the ferry, not waiting for the ferry.

*Figure 14 Food brake in ferry cafeteria*

A ferry crossing bring people together, they can be offered traditional food and beverage and drive further on in a better shape and more awake because of the often needed rest.

The main focus in this article is that attention has to be made to the actual proven development of ferries and ferry operation, both because of the fact that it is necessary for the society, but also because a good ferry operation helps to build the country.

Waiting for the tunnels and bridges to be built, let us enjoy the ferry trip.

*Figure 15 Happy ferry crew and ferry in operation*

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# STRUCTURAL HEALTH MONITORING OF THE WUSU CABLE-STAYED BRIDGE

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## ABSTRACT

*In the recent past, there have been rapid advances in the development of structural health monitoring (SHM) of bridges. Especially for the large-scale bridges, implementing long-term SHM system is really an effective and time-saving way to detect the structural damage at early stages to prevent more severe damage from occurring. This paper gives the detailed information about the establishment of SHM system of the Wusu Bridge. Located in the Fuyuan delta, neighboring to the Sino-Russia border, the single-pylon, single-plane cable-stayed bridge acts as a significant crossing over the Wusu River and has an important geographic significance. Based on the finite element analysis, the sensor deployment is reasonably optimized. Correspondingly an SHM software was thus developed to monitor the working conditions of the sensors and the bridge structure. Despite the existence of data errors, the amended data can still be used to assess the real-time bridge structural condition. The contents could be used as a reference in health monitoring of bridges with similar structures.*

**Keywords:** Health monitoring, Cable-stayed bridge, Sensor deployment, FE analysis

## INTRODUCTION

Structural health monitoring (SHM) is a recently-developed technology for monitoring structural safety, integrity and performance. Compared with the artificial inspection, SHM realizes continuous and real-time inspection and damage detection with minimum labor involvement. An SHM system is a combination of the data collection and transfer, damage identification and condition assessment, providing a feasible approach for optimizing operational and maintenance activities for bridges. In the recent years, along with the booming construction of bridges with more complicated structures, SHM systems have been widely applied in newly-built long-span bridges. In China for some important bridges, the SHM systems were installed at the time of construction: e.g.,the TsingMa Bridge(774 sensors,1997), the SuTong Bridge(1440 sensors,2008), and the DongHai Bridge(561 sensors,2005) .

This paper introduces the design and installation of SHM system for the Wusu Bridge, and the analysis results based on the collected data. With this SHM system, the structural behaviors and the working conditions of the Wusu Bridge can be learned in real-time and amounts of data have been obtained to help making the safety assessment.

## BRIDGE CONFIGURATION

Completed in October, 2012, neighboring to the Sino-Russia border, the Wusu Bridge goes over the Wusu River (the Ussuri River), connecting the Fuyuan Delta to Heixiazi Island (or called Bolshoi Ussuriysky Island, a sedimentary island co-occupied by China and Russia). The bridge is a single-pylon cable-stayed bridge with the main span arrangement of 2×140m. The 26.5 meter wide bridge has been designed to carry a two-lane road in each directions. The total height of the concrete pylon is approximately 115.5m. 52 steel cables radiate out in one plane from different levels of the pylon in a harp-shaped arrangement. Each cable contains 163 strands of ø7 galvanized wire, which is protected by wax inside a double-layered PE sheath. The bridge deck is a 3. 55-meter-deep composite steel-concrete structure with large steel cantilevers in the transverse directions. The main spans provide a clearance of 13.5m at mean high water springs over the navigation channel [1]. The project also includes construction of approaches and major interchanges on each side of Wusu River. Its geographic position near the confluence of the Amur River and the Wusu River and next to the major Russian city of Khabarovsk, has given this bridge great strategic importance.

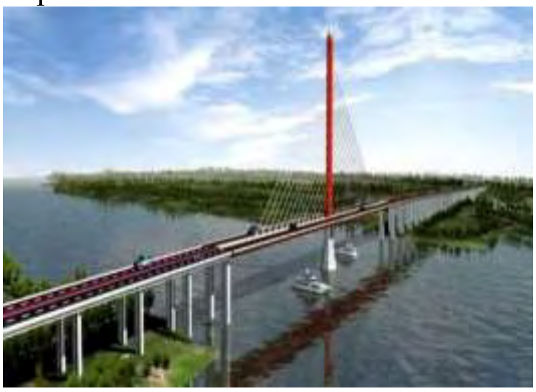

*Fig.1 The Wusu Bridge panorama*

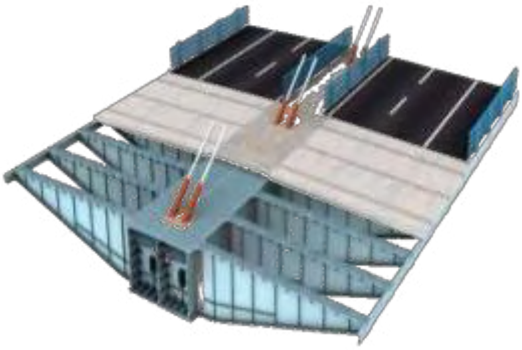

*Fig.2 Typical girder structure*

## WUSU SHM SYSTEM

The process flow chart of the Wusu SHM system is as follows:

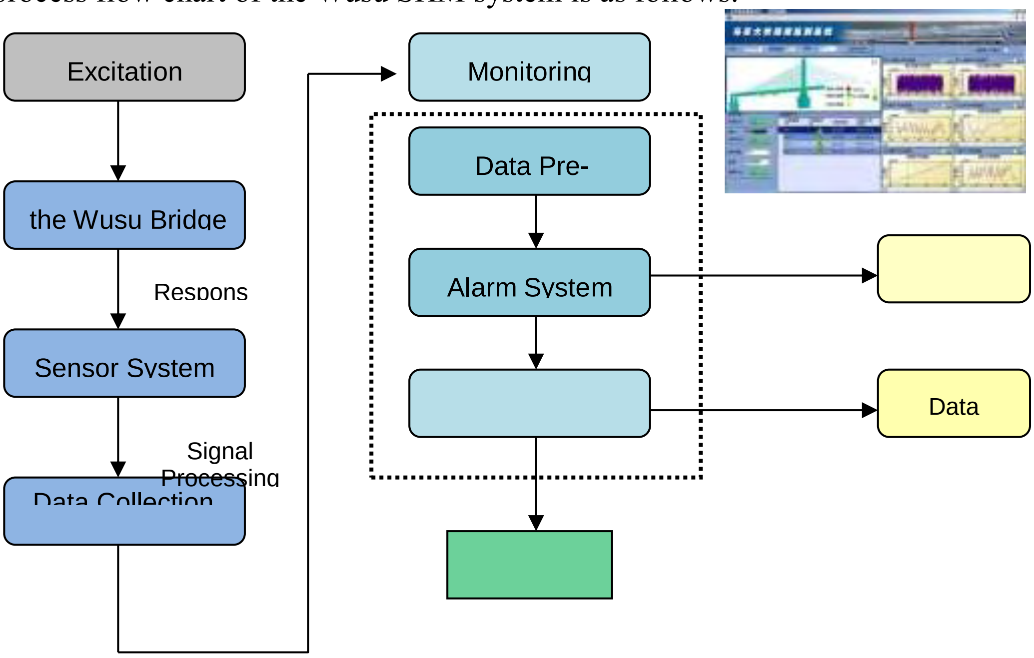


*Chart.1. The process flow chart of the Wusu SHM system*

## SENSOR SYSTEM

### Optimal sensor placement

Using minimum sensors to obtain the status information of the bridge structure is the goal to optimally allocating sensors. Among typical methodologies, genetic algorithms were proved to be an effective one and have been successfully used in some SHM projects. (e.g. the SHM system of the TsingMa Bridge). In this project, a 3D finite element model of the Wusu Bridge was firstly built and the structural responses of all nodes under live loads (wind load, vehicle load...) were comprehensively analyzed.

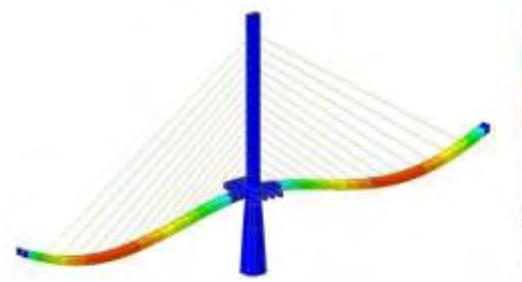

*Fig.3 Girder deflection under traffic load*

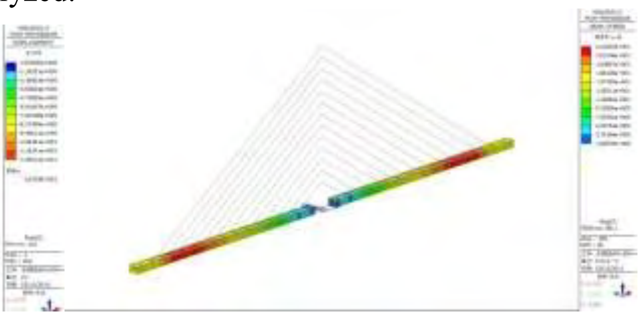

Fig.4 Calculated stress of steel box girder

Based on the FEM results, a genetic algorithm was used to generate the solution to sensor placement optimization on Matlab platform. And considering the symmetrical characteristics of the bridge structure, the total sensor number was initially determined as 76. However, the budget for this SHM system was pruned later on and the sensor number was eventually reduced to 61. With excellent characteristics such as sensitive to mechanical strain and temperature, immune to electrometric interference, low fiber loss and highly multiplexed distributed sensor arrays [2], FBG sensors (Fiber Bragg Grating Sensors) were chosen to measure the stress and temperature variations. A summary of the sensors used in the project is shown in Table 1.

*Table 1-Summary of the sensors*

| Monitoring Parameters | Sensor Type | Number |
|---|---|---|
| Temperature | FBG | 4 |
| Stress/Strain | FBG | 46 |
| Girder deflection(camber) | Pressure transmitter | 5 |
| Cable tension | Accelerometer | 4 |
| Girder horizontal displacement | FBG | 2 |

### Stress measuring points

For the steel box girder, 18 sensors were installed on its bottom and top plate; for the composite steel-concrete part, 8 sensors were mounted inside the box; for the cantilevers, 8 sensors were set up at L/2 and L/4 sections respectively. In addition, four sensors were deployed near the root of the main pylon. There were 46 stress sensors in total.

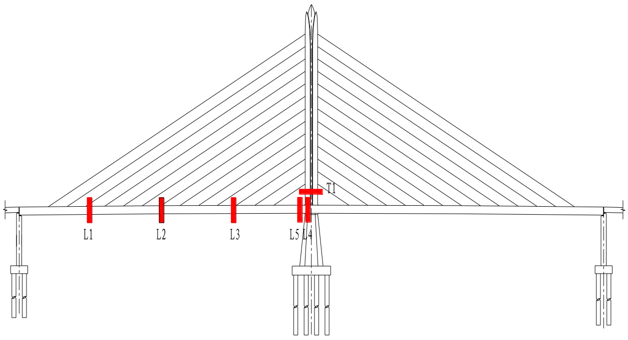


Fig.5 Measuring point locations

Fig.6 FBG stress sensor

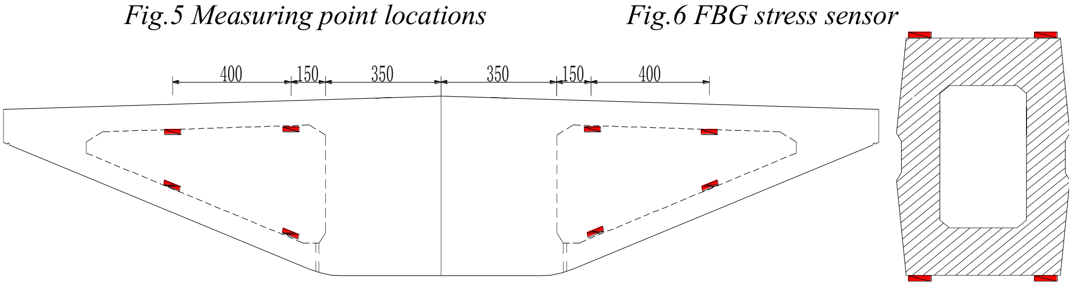


Fig.7 Sensor deployment at the steel-concrete part and the pylon root

**Camber measuring points**

ROUSEMONT pressure transmitter was chosen to measure the girder vertical displacement. The reference point was set inside the pylon. Four measuring points were set at the middle web and cantilever ends of box girders at 0.4L sections respectively, as illustrated in Fig.8. There were five measuring points in total.

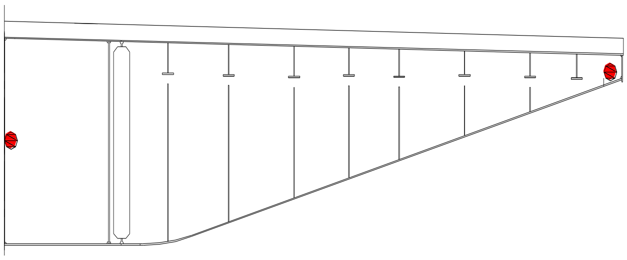

Fig.8 Measuring point layout of the girder camber

Fig.9 Pressure transmitter

**Temperature measuring points**

Three temperature measuring points were set inside the steel box at the mid-span section. Another temperature sensor was installed at the pylon root, as shown in Fig.10.

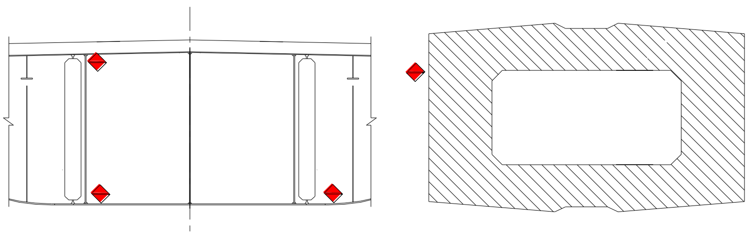

Fig.10 Temperature sensor layout

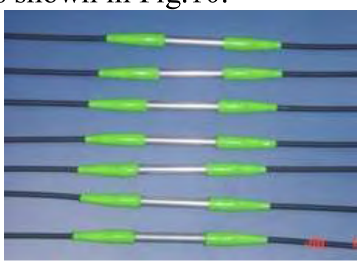

Fig.11 FBG temperature sensor

**Cable tension measurement**

Based on the numerical analysis, four main cables were selected to be the target ones for obtaining the cable tension, and one LANCE accelerometer was mounted on each cable, as seen in Fig.12 and Fig.13.

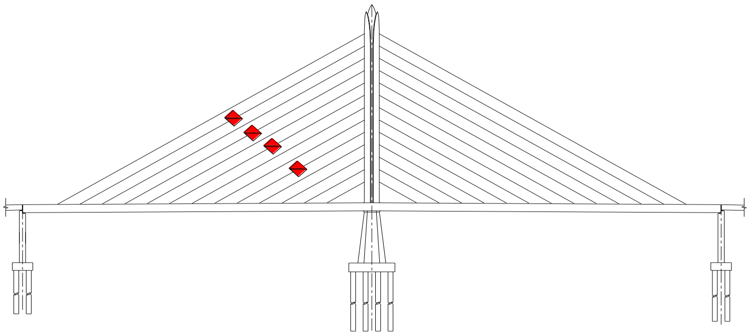

*Fig.12 Measurement points of the cable tension*

*Fig.13 Sensor installation*

Although some other methods (e.g. the magnetic flux method) have their own advantages, the well-known frequency method was eventually selected to measure the cable tension due to its simplicity and economy. Currently for the short cables（L<6m）, the errors brought in by this method cannot be ignored, however in this project, the shortest selected cable was about 36 meters long, and the precision of the measurements was supposed to be acceptable.

**Cable tension calculation**

Cables can be effectively modeled as taut-strings. Some empirical taut-string formulas, such as the ones proposed by Liu W.F. (2001) [3] and Zheng G. (2005) [4], have been widely used to calculate the cable tension based on the measured frequencies. In this project, the employed formula is as follows (Ren W.X. & Chen G. 2005) [5]:

$$\begin{cases} T = 3.342ml^2 f^2 - 45.191\dfrac{EI}{l^2};(0 \le \xi \le 18) \\ T = m(2lf - \dfrac{2.363}{l}\sqrt{\dfrac{EI}{m}})^2;(18 \le \xi \le 210) \\ T = 4ml^2 f^2;(\xi \ge 210) \end{cases} \qquad \xi = \sqrt{\frac{T}{EI}}\, l \ ,$$

where $T$ denotes the cable tension; $m$ is the unit cable mass; $l$ is the cable length and $EI$ represents the bending stiffness. Sagging and bending effects are considered in the formula.

## MONITORING SOFTWARE

The corresponding monitoring software was developed to help monitor the system status, including the functions of data collection, data management, intelligent analysis, pre-warning and basic structural safety assessment. Logged clients can directly learn about the historic and real-time data sent back by all sensors. And for the pre-warning and the safety assessment systems, Vibration-based damage identification method [6] and Hilbert-Huang transform method [7], combined with the FEM results, were compared and employed to determine the pre-warning threshold values of the measured data. However, since not all the structural damage could be identified by merely using the mathematical methods, and the sensor errors sometimes would cover the data variations induced by structural damage, the safety assessment of the bridge structural status has to be artificially made by professionals.

*Fig.14 Real-time data monitoring interface*

*Fig.15 Data collection & control interface*

**EXISTING PROBLEMS IN THE SHM SYSTEM:**

Sensor errors: Inevitably, some sensor errors occurred with the service time going on due to intrinsic or external factors. Generally it's difficult to distinguish some sensor errors from the actual structural responses. Both choosing durable and stable sensors and taking protective measures for sensors are equally important to the data reliability.

Data distortion: During the daily monitoring, it was commonly found that some stress data suddenly jumped to an abnormal value then return to normal. There are various factors contributing to this kind of data distortion, leading to false alarm when the distorted values exceed the threshold values. Thus it's necessary to add the intelligent data management function for the software to address this problem automatically.

Power failure: Two weeks after its starting to work, the SHM system experienced a one-day power failure, and the data of which period were thus missing. After the power was restored, the data gathered from some stress sensors had an obvious leap from the previous time, affecting the pre-warning function. The follow-up data management was therefore performed to reset the reference values.

**MEASURED DATA ANALYSIS**

Stress results: Four out of the 46 stress sensors were assumedly in failure judging from the long-term abnormal data they captured. And one stress sensor was severely damaged during the bridge construction. Fortunately the other sensors are still working in good condition and the obtained data seem normal and reasonable, indicating that no structural damage was detected until now.

Cable forces: Forces of the four cables were calculated based on the captured frequencies. It's noticed that the force of the shortest cable (L=36m) had a 17% difference from the designed value, those of the other cables were close to the corresponding designed values. Excluding the accelerometer problem, the in-situ boundary conditions of the shortest cable were probably not in accordance with the hypothetical ones in the formula (Ren W.X. & Chen G. 2005). And the constant parameters in the formula have a noticeable influence on the calculated results. Therefore for this cable, choosing another proper formula or modifying the constant parameters may be a proper solution.

Girder camber: since the first day the system worked, the measured camber data have varied between the -11.2cm and +4.4cm, while the corresponding scope resulting from FEM results ranged from 13.0cm to +4.89cm. It's assumed that the sensors have been working in good condition until now and the bridge has not suffered from severe overloads. Despite the results, it was observed that the liquid insider the pressure transmitter flowed relatively slow in winter (-30℃ on average in December), therefore long-term camber data could be more reliable than the real-time data.

Temperature data: The temperature data seemed reliable for that they were generally consistent with the data from local weather forecast, having differences approximately ± 3℃. Considering that the specific sensor positions might have different temperature from the air temperature. The measured data were acceptable.

## CONCLUSIONS AND RECOMMENDATIONS

This article introduces an SHM system established for the Wusu Bridge. The sensor layout and existing problems in this system are presented in detail. It's concluded that the sensor selection, installation and protection are significant to the data reliability. Additionally, using mathematical tools, like genetic algorithms, combining with the FE model, can effectively optimize the sensor allocations. Although some problems like the data distortion, sensor errors are not easy to be completely addressed, and the long-term stability and reliability of the system needs continuous observation, the SHM is still one of the best means available to monitor the large-span bridge structural behaviors and guide the maintenance and repair work.

## ACKNOWLEDGEMENTS

The installation of certain sensors was completed by the co-work of CCCC First Highway Consultants and Ningbo Shangong Center of Structural Monitoring & Control Engineering Co.,LTD. The authors are grateful to Ningbo Shangong for its technical support. The authors would also like to acknowledge the technicians from Xi'an CENTN Technology,Co.LTD, for their help to develop the SHM software, and the engineers from Xi'an Ruitong Highway and Bridge Science & Technology Co., LTD, for their providing significant data based on FEM analysis.

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# PRELIMINARY DESIGN AND COMPARISON OF SFT TUBE WITH DIFFERENT HIGH PERFORMANCE FIBER CONCRETE MATERIALS

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## ABSTRACT

*The submerged floating tunnel (SFT) is an underwater transport structure across the strait, river, lake and waterway. As SFT located in the complex environment, the materials for the building put forward very high demand on compressive strength, tensile strength, water resistance, durability and etc. In this case, the performance of the ordinary concrete and other traditional materials can not meet the requirements of the SFT. However, the high performance fiber concretes (HPFCs) with high tensile strength, high density and other characteristics, have the potential to be applied in SFT construction. In this paper, taking the external geometric, environment and load conditions of SFT in Qiandao Lake as an example, it was redesigned and analyzed by the different HPFC materials and parameters. From the view of impermeability and durability, it was explored that steel fiber concrete, glass fiber concrete and polypropylene fiber concrete were used in the feasibility of SFT. The economies of these materials included in composited sandwich material used in the SFT were also compared with each other. The results shown that high performance steel fiber concrete used in submerged floating tunnel has good security and economy. It may be the best selection for SFT construction.*

Keywords: *Submerged floating tunnel, tube design, steel fiber reinforced concrete, glass fiber reinforced concrete, polypropylene fiber reinforced concrete*

## INTRODUCTION

Submerged floating tunnel (SFT), known as 'Archimedes Bridge' in Italy, referred to as 'PDA' bridge, is composed of a suspended tubular structure, underwater cables (or pontoons), and shore structures (Ahrens D, 1997; Xiang, 2002)[1,2]. Since the working environment in the SFT is general very complex, the materials used in SFT construction are needed to meet some high standards, like the high tensile strength and good waterproof for outer shell, and excellent ability to resist fire and vehicle collision for inner shell. The waterproof is the key point in SFT construction, because the SFT is surrounded by water and the water leakage will produce huge security risks.

The currently selected materials of SFT tube (cable fixed) are mainly steel, reinforced concrete and multi-layer complex materials. Steel has high mechanical performances (such as tensile strength, resistance to fatigue, against impacts and ductility) and low weight, but it is expensive and vulnerable to corrosion. Besides, due to the small size of the structural elements, the

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[2] doctoral candidate

construction cost of underwater bases will increase substantially. Under the effect of high water pressure, steel structure is easy to local buckling. In order to increase the anti-buckling stability, the quantity of the stiffener and the thickness of steel plate should be increased, which will further lead to raise the construction cost. Multi-layer complex materials, like a “sandwich”: an internal layer made of steel, an intermediate layer made of concrete and an external layer made of aluminum which has high resistance to corrosion, sufficient strength and weight, and can prevent water seepage. The external layer is easily damaged and difficult to maintain. Therefore, multi-layer complex materials will have higher risk as the primary waterproof measure. Concrete has advantages of low cost, stabilizing weight against the Archimedes buoyancy to decrease the construction costs of underwater bases, good mechanical behavior in compression, but it shows poor behavior in tension and against permeability. It is necessary to improve the tensile strength, impact resistance, corrosion resistance and waterproof of concrete materials, or else the concrete is only used in mid-layer of SFT.

As we all know, the tensile strength and impact resistance of concrete are poor. So, in the past, the waterproof measures are necessary for concrete structures, which include coating waterproof, outer waterproof layer and concrete self-waterproof. Since 1990s, Holland, Denmark and other Nordic countries built a lot of immersed tunnels only with self-waterproof concrete, which gradually became the main material in structure design (Ning Maoquan, 2008). In recent years, many researchers proposed the performance of fiber reinforced concrete (FRC) improved by mixing some fibers(such as glass fiber, steel fiber etc.) into the ordinary concrete to meet the requirements of high tension strength, anti-permeability and good durability for concrete. Currently, the largest compressive strength of the fiber-mixed concrete has sometimes been up to 200Mpa, and the tensile strength has reached about 20Mpa. The density of fiber concrete is also higher than that of ordinary concrete. Hence, the fiber-mixed concretes are also called the high performance fiber concrete (HPFC).

The high performance fiber concrete (HPFC) generally divided into the glass fiber reinforced concrete, polypropylene fiber reinforced concrete, and steel fiber reinforced concrete, etc. This paper explored the feasibility of the high-performance fiber concrete in the SFT tube based on the principles and analysis process of SFT design.

## TUBE DESIGN PRINCIPLES AND PROCESS OF SFT

### Design principle

The tube function of SFT provides buoyancy and space for vehicles and pedestrians. The space should be enough big to float the whole SFT and bear self dead weight and traffic loads with cables together. The tube also plays a key role in maintaining the balance and stability of the whole structure. Therefore, the rational design of tube is directly related to safety and suitability of SFT. The tube design should follow the basic principles of security and economic on the base of comprehensively considering the load, collision, fire, gravity-buoyancy ratio, flow resistance performance, durability and anti-permeability and other factors. Besides, the interior space should also meet the clearance of road and rail transportation, ventilation, escape, anti-gushing, durability and some pipes pre-laying. As a structure to be floated in the water, SFT structure should also be consider the local land marine topography, geology, hydrology, meteorology, seismology, construction and operating conditions when it was designed.

The important design principles should be followed:

1. Gravity-Buoyancy ratio of tube should be less than 1.0, 0.5 to 0.8 is recommended.
2. Tube should have the enough tension and compression strength, stiffness and stability in the various stages of construction, floating movement and operation.
3. Tube shape should have good hydrodynamic force properties.
4. The whole structure should have high seismic fortification criterion.

### Design process

The design process of SFT is shown in Figure 1.

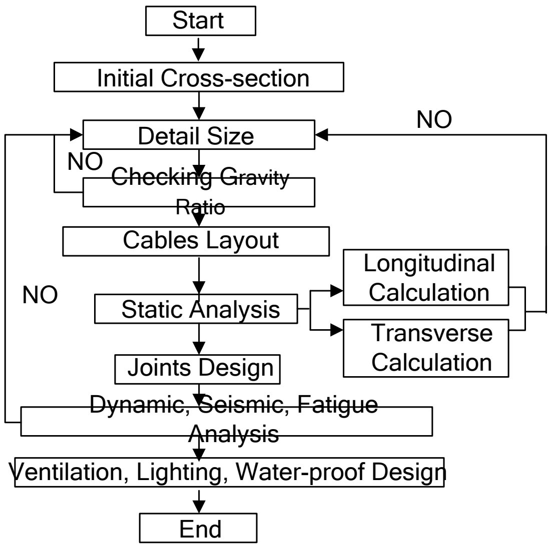



**Figure 1. Design process of SFT**

## CHARACTERISTICS OF HIGH PERFORMANCE FIBER CONCRETE (HPFC)

As stated above, mechanical performance and durability of concrete can be improved by adding a variety of fiber materials into concrete to form **HPFC**. According to mixed different fiber materials, there are the glass fiber reinforced concrete (GRC), the polypropylene fiber reinforced concrete (PPFC), and steel fiber reinforced concrete (SRC), hybrid fiber reinforced concrete (S-PPFC, steel- polypropylene fiber reinforced concrete) etc. These HPFCs have good compressive and some extent tension strength, anti-impact resistance, waterproof and durability. Compared with general reinforced concrete (RC), they have higher tensile strength and crack resistance. The properties of current HPFCs with different fiber and main mechanical parameters are partly listed in Table 1.

*Table 1. The properties of high performance fiber concrete with different fiber*

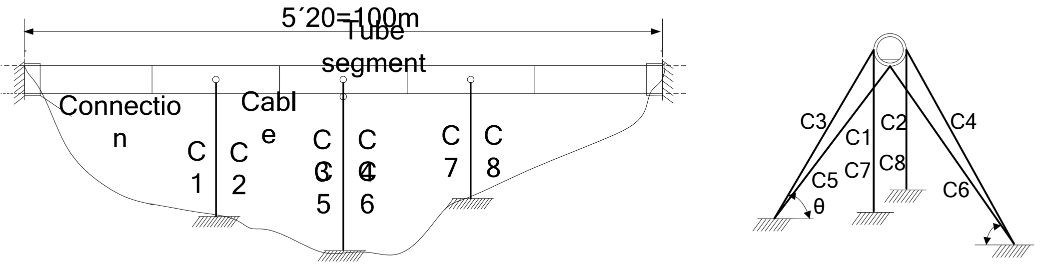
| Concrete material | Characteristics | Main mechanical parameters | Note |
|---|---|---|---|
| RC (C25) | — | $f_{cd}$=11.5Mpa, $f_{td}$=1.23Mpa, $E_c$=28.0Gpa,v=0.16 | — |
| SRC | Advantage: higher compressive strength and tensile strength, good flexural and impact toughness properties, strong anti-explosion and anti-permeability<br>Disadvantage: difficult to mix uniformly | $f_{cd}$=120Mpa, $f_{td}$=19.4Mpa, $E_c$=52.2Gpa,v=0.22 | high cost, need to ensure that steel fiber bond strength with concrete construction technology requirement: steam curing can increase significantly quality and performance of SRC |
| GRC | tensile, bending, shear, impact resistance, fatigue resistance and fracture toughness are significantly improved compared with RC | $f_{cd}$=23.3Mpa, $f_{td}$=2.16Mpa, $E_c$=29.9Gpa,v=0.24 | poor resistance to alkali, easy to embrittlement |
| PPFC | tensile strength, freeze-thaw resistance, anti-carbonation and fatigue resistance are all greatly improved, suitable for hydraulic structures | $f_{cd}$=20.5Mpa, $f_{td}$=2.20Mpa, $E_c$=31.5Gpa,v=0.20 | poor effect to late<br>shrinkage cracks and temperature cracks |
| S-PPFC | mechanical properties (strength and toughness) are determined according to mixed quantity of different fibers | $f_{cd}$=55.0Mpa, $f_{td}$=7.2Mpa, $E_c$=32Gpa,v=0.20 | avoid the negative effects of fiber mixing;<br>mixing should be sufficiently and uniformly |

## COMPARISON OF QIANDAO LAKE SFT TUBES DESIGN WITH DIFFERENT HPFCs

### Engineering background

Qiandao Lake is the famous scenic spot, located in Chunan, Hangzhou, Zhejiang, China. In order to develop tourism resource and construct the first prototype SFT in the world, scientists of the relative Universities and Company in China and Italy had been cooperated with each other for about 10 years. Joint Laboratory of Archimedes Bridge was established, the preliminary design and construction project of the prototype model was presented to provide demonstrations for the future design and construction of practical SFT (2007) [4].

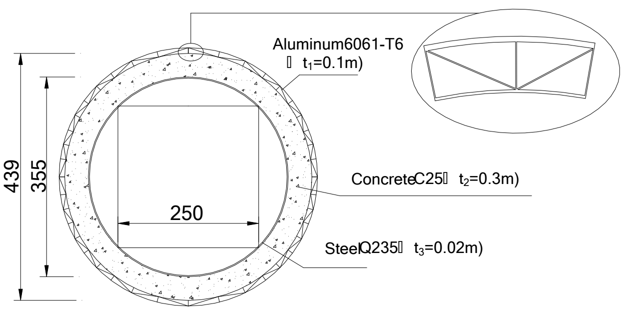
According to the Italian design of the prototype model, the total length of Qiandao Lake SFT is 100m, which consisted of five standard tube segment modules of 20m. There are four pairs of cables connected tubes to anchor foundation in the SFT, in which the middle two pairs cables are inclined, one pair cables on both sides are vertical. The arrangement of tube and cables is shown in Figure 2. The detailed design project and hydro-geological condition are seen in report by Archimedes Company etc. [4].




(a) Elevational drawing (b) End view drawing

*Figure 2. Layout of SFT Prototype in Qiandao Lake*

The tube cross-section was design for a multi-layer cross-section, which composited by different materials (internal steel plate layer with 2cm, intermediate concrete layer with 30cm and external aluminum frame layer with 10cm), like a "sandwich", as shown in Figure 3. Sandwich cross-section makes full use of various materials advantages, which external aluminium has good corrosion resistance and tightness under water environment, internal steel plate has good tensile strength, impact and fire resistance, the middle material adopt concrete to improve whole stiffness. But the processing technology of sandwich cross-section tube segment module is complex, the construction is difficult and the cost is high. In addition to these, the type of tube design may also be not suitable for strait connection way under the marine geology environment condition. It is necessary to explore and seek for the relative low cost and good performance material (such as good tensile strength, crack resistance, corrosion resistance and tightness etc. under water environment) to be used in the construction of SFT. The high performance fiber concretes may be the best selection.



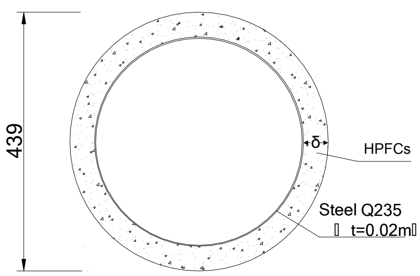




*Figure 3. Sandwich cross-section* *Figure 4 .Tubes section designed with different HPFCs*

**Analysis and discussion of tubes designed with different HPFCs**

In order to analyze and compare static behaviour of the tube structures made by the different HPFCs material, the Qiandao Lake SFT is taken as an engineering calculating example, of which the external geometric, environment and load conditions are same with the design project. The total length of 100m, the pair number of cables, the internal dimensions of circular cross-section and gravity-buoyancy ratio parameters remain unchanged. The Qiandao Lake SFT are respectively redesigned by GRC, PPFC, SRC, S-PPFC and so on. The key design parameters are shown in Table 2.

*Table 2. Section design of SFT with different HPFC material*

| **Material** | **Section type** | **Diameter (m)** | **Density (kg/m$^3$)** | **Gravity-buoyancy ratio** | **control indicator** |
|---|---|---|---|---|---|
| Aluminium, RC(C25) ; Steel | sandwich | 4.39 | 2800 for aluminium 2500 for concrete 7850 for steel | 0.8 | Crack width<0.2mm Aluminium stress<30~70Mpa |
| SRC | circle | 4.39 | 2500 | 0.8 | No crack |
| GRC | circle | 4.39 | 2500 | 0.8 | No crack |
| PPFC | circle | 4.39 | 2500 | 0.8 | No crack |
| S-PPFC | circle | 4.39 | 2500 | 0.8 | No crack |

The internal stresses and displacements of control cross-section in the SFT's built by different material tube were analyzed by using the general finite element method of ANSYS. For "sandwich" cross-section tubular, SFT structure is modeled by layered structural solid element (Solid 185) with three degrees of freedom at each node. The cable is simulated by element link180. The carriageway plate in tube segment is ignored. The boundary condition of tube is simulated as hinge connection. The model in all has 16086 nodes and 12008 elements. Typical computational model (sandwich cross-section) was shown in Figure 5. For SFT structure built by other HPFC materials, similar analysis model was also established. Hence, the tensile and compressive strength (or stresses) as well as stability of these SFT are checked according to analysis results of different material parameters and cross-section dimension etc under the action of same vehicle load and environment load. The reasonable thickness of circle shells with different materials is given from above analysis calculation. Based on detailed design and analysis, further estimation of their corresponding cost are gained. The results are listed in Table 3 to compare with each other.

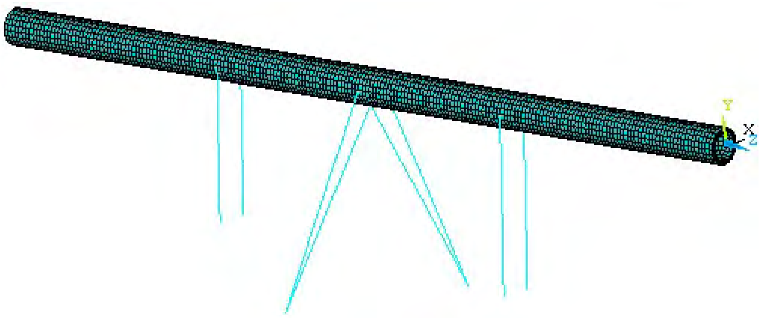


*Figure 5. Analysis model of SFT by finite element method*

As can be seen from Table 3, the cost of PPFC is the lowest and the cost of SRC is the highest, in addition, the cost of traditional sandwich cross-section design (C. M.) is much higher than any kind of high-performance fiber concrete. Therefore, the high-performance fiber concrete has a very good economy. Combined with comprehensive analysis of the characteristics of high-performance fiber concrete, S-PPFC not only has a high tensile and compressive strength, but also has good permeability and anti-fatigue properties to meet requirements of SFT construction. S-PPFC also has a better economy, and is recommended to use in SFT construction.

*Table 3. Calculation results of SFT with different HPFC material*

| Material | Section type | Tube Shell thickness δ (m) | Maximum tensile stresses (Mpa) | Maximum compressive stresses (Mpa) | Estimation Cost ($10^4$RMB/m) |
|---|---|---|---|---|---|
| C. M. | sandwich | 0.30 | 0.70 | 0.70 | 2.02 |
| SRC | circle | 0.10 | 2.76 | 2.76 | 1.25 |
| GRC | circle | 0.12 | 1.98 | 1.94 | 1.15 |
| PPFC | circle | 0.12 | 1.96 | 1.96 | 1.12 |
| S-PPFC | circle | 0.10 | 2.42 | 2.42 | 1.18 |

## CONCLUSIONS

SRC, GRC, PPFC and S-PPFC all have good tensile strength, crack resistance and impermeability, and their price is cheaper.

GRC has poor resistance to alkali and PPFC plays a poor effect to late shrinkage cracks and temperature cracks. They are not suitable for using in SFT construction directly. However, SRC and S-PPFC can be considered to use.

S-PPFC combines the advantages of SRC and PPFC, and has good tunability and economy. S-PPFC can be adapt to the underwater environment and is recommended for SFT construction.

## ACKNOWLEDGEMENT

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# THE INTEGRATED DESIGN OF STRUCTURAL HEALTH MONITORING SYSTEM WITH INSPECTION AND MAINTENANCE MANAGEMENT SYSTEM FOR QINGDAO BAY BRIDGE

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## ABSTRACT

*The Qingdao Bay Bridge (QBB), located in Shandong Province, China and completed in 2012, has an overall length of 28.047km. It is composed of navigation bridges, no-navigation bridges and land bridges. The total length of the navigation bridges and non-navigation bridges is 25.171 km. The navigation bridges are composed of three cable-supported bridges, i.e. (i) a twin-pylon steel-box-girder cable-stayed bridge, (ii) a single-pylon steel-box-girder cable-stayed bridge, and (iii) a single-tower self-anchored steel-box-girder suspension bridge. For such an enormous and complicated bridge structural system, traditional bridge inspection and maintenance approaches are however difficult to detect and identify the variation of structural condition in individual structural component effectively and precisely. The recent advancement of sensing and instrumentation technologies such as modern sensing technology, network communication, signal analysis and processing, data management, knowledge-based data mining, finite element modeling and simulation, etc., the structural health monitoring system (SHMS), which is the combination of all of these technologies, achieves the functions: (i) to valid assumptions and parameters adopted in the design and construction of the bridge , (ii) to provide structural for planning and scheduling of inspection and maintenance activities, (iii) to set alarms for abnormal loads, abnormal responses and structural damage/deterioration, (iv) to provide data and analysis tools for structural safety evaluation after disaster, and (v) to provide realistic data and information for revision/updating of structural design standards. The structural monitoring inspection and maintenance system on QBB consists of automatic data acquisition system, semi-automatic inspection-based maintenance management system, components rating system and safety evaluation system, integrating SHMS with inspection and maintenance management system.(I&MMS) The SHMS provides I&MMS with appropriate target and structural condition while the I&MMS reinforces and complements the SHMS with updated information on structural condition. The integrated system design not only ensures the reliability and safety of QBB under operation, but also prolongs the service life of QBB through cost-effective execution of maintenance works.*

**Keywords:** structural health monitoring, bridge inspection and management, components rating system, safety evaluation

## 1. INTRODUCTION

In recent years, the Ministry of Communications draws high attention in the construction of hghway bridges and a number of world-class level highway bridges have already been built and put forward in operation. According to current statistics, the total length of highways in China is 1.87 million km, and of which 13,376 km are made up of more than 320,000 number of bridges, in which 700 numbers of them are multi-span bridges with an overall bridge length exceeding 1 km. The traditional concept of “esteem for construction and contempt of maintenance” is very popular in bridge management works, and such illusive concept normally results in much less resource is deployed for bridge inspection and maintenance works. Under such a circumstance, the performance of bridges downgrades rapidly and their service life shortens significantly. Furthermore, the structural systems in long-span bridges are huge in scale and complex in structural arrangement, traditional approaches of bridge inspection and maintenance cannot identify and quantify correctly and precisely the structural health variations in individual structural components of long-span bridges.

The evolution of structural health monitoring technology for bridge management provides a new way of development. The application of structural health monitoring in the field of aviation, aerospace and mechanical engineering works is much earlier than that of the civil engineering works. Structural health monitoring adoped in civil engineering works is apparently starting in 1990s. Bridge due to its comparative simple in structural arrangement and clear in load paths, structural health monitoring is first introduced in bridge enghineering among civil engineering works. In China, most long-span bridges such as Tsing Ma Bridge in Hong Kong, Jiangyin Yangtze River Bridge and Sutong Bridge Yangtze River Bridge have been deployed with structural health monitoring systems.

The Qingdao Bay Bridge is composed of navigation marine bridges, non-navigation marine bridges and land bridges and has a overall length of 28.047km. The total length of the navigation and non-navigation marine bridges is 25.171 km. The navigation marine bridges are cable-supported bridges, i.e. (i) the twin-pylon steel box girder cable stayed bridge with a span configuration of 80m+90m+260m+90m+80m, (ii) the single-pylon steel box girder cable stayed bridge with a span configuration of 60m+120m+120m+60m, and (iii) the single-leg tower steel box girder self-anchored supsension bridge with a span configuration of 80m+190m+260m+80m. The deployment of an integrated structural health monitorng with inspection and maintenance management system (SHM-I&MMS) is particular essential for such an intensive and comlex structural systems as structural health monitoring system (SHMS) and inspection and maintenance management system (I&MMS) will work complementary and coorperatively to each other, i.e, SHMS provides target components/locations for inspection and maintenance and I&MMS provides information for updating and calibration of monitoring/evaluation criteria/indices.

## 2. OVERVIEW OF STRUCTURAL HEALTH MONITORING AND INSPECTION & MAINTENANCE MANAGEMENT

Structural health monitoring and structural safety evaluation works should be carried out with reference to the codified parameters and assumptions adopted in the design and construction of the relevant birdge works [1, 2 and 3]. A structural component which has no damage is referred to as a healthy structural component, which means that the structural component under loading (at serviceability limit state level or SLS level) behaves linear or has fully recovery capacity when the load is removed. Structural health monitoring refers to the usage of sensing instrumentation systems and relevent software tools to monitoring the structural performance of the bridge structural system in terms of loads and responses at the serviceability limit state. If the performance of a structural component under monitoring is exceeded, then structural safety evaluation should be carried out to evaluate whether damage is induced or to carry out prognostic analysis when the damge will be occurred at ultimate limit state level. Inspection normally refers to the visual inspection of those assessible structural components and such inspection is essential to determine the appearance of structural components. When problematic structural component or location is received from SHMS, detailed inspections can be planned and executed to investigate the details of actual structural conditons by equipments, load-trials, laboratory testing, where necessary. Upon completion of detailed inspections, corresponding maintenance actions can then be planned and scheduled for execution.

## 3. OPERATION AND ARCHITECTURE OF STRUCTURAL HEALTH MONITORING WITH INSPECTION & MAINTENANCE MANAGEMENT SYSTEM FOR QINGDAO BAY BRIDGE (SHM-I&MMS FOR QBB)

The SHM-I&MMS for QBB has the features of combining automatic data acquisition system (ADAS) and portable data acquisition system (PDAS). The ADAS is devised for the real-time monitoring of structural performance in terms of loads and responses under the serviceability limit state; whereas the PDAS is devised for routine bridge inspection.

Structural health monitoring is devised in three phases: (i) Phase 1 – if the monitoring result is below 75% of the monitoring criteria at SLS level, then the relevant structural component or location is considered as a healthy structural component or location and solely routine monitoring works will be carried out; (ii) Phase 2 – if the monitoring result is in-between the range of 75% and 100% of the monitoring criteria, then the structural component or location is considered have potential risk of exceeding the monitoring criteria and close monitoring of this structural componnet or location is required until the monitoring results following below 75% of the monitoring criteria; and (iii) Phase 3 – if the monitoirng result exceeds 100% of the monitoring criteria, then potential risk of damage may be ocuured in the structural component or location, and structural health evaluation should be executed in order to confirm the existence of damage.

The bridge inspection and maintenance management system should be devised in accordance with the codified approaches as outline in National Codes : (i) The Specification of Highway Bridges Maintenance, and (ii) The inspection and assessment of the load-carrying cpacity of Highway Bridges. All observation (or instrumentation) points and inspection items should be based on relevant tender specifications and/or recommedations from the bridge designers. Part of the permanent observation points and inspection items are incorporation in the SHMS as real-time monitoring.

The alarm for damage evaluation should be based on the results of regular inspections and structural health monitoring results. The alarm for damage evaluation should be based on three results: Result 1 – Damage Status Assessment basing on Regular Inspection, Result 2 – Damage Status Assessment basing on Monitoring Results, and Results 3 – Damage Staus Assessment basing on Combined Results of Regular Inspection and Monitoring. These results provide targets, structural conditions and time periods for execution of inspection and maintenance works. In order to fulfil the operation needs, the SHM-I&MMS for QBB is devised as four modular systems. These four modular systems are: (i) structural health monitoring system, (ii) Inspection and Maintenance Management System, (iii) Structural Health Evaluatio System, and (iv) Structural Health Rating System. The operation flow diagram of SHM-I&MMS for QBB is illustrated in Figure 1 below.

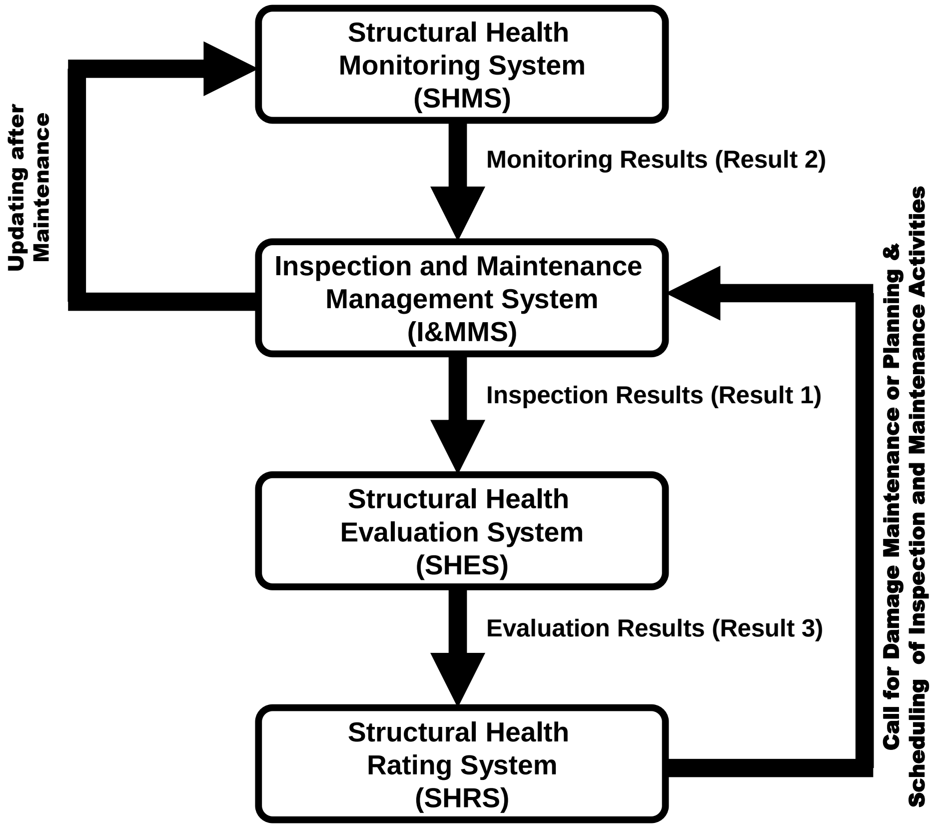


*Figure 1 Operation Flow Digram of the SHM-I&MMS for QBB*

## 4. SCOPE OF STRUCTURAL HEALTH MONITORING AND INSPECTION & MAINTENANCE MANAGEMENT

The scope of structural health monitoring is in general can be classified into four categories of physical and/or chemical quantities, namely, environmental loads and attacks, operation loads, structural features and structural responses. Environmental loads and attacks refer to the loads due to wind, temperature, seismic, corrosion, etc. Operation loads refer to loads due to highway traffics, ship impacting, scouring, settlement, etc. Structural features refere to influence lines (or responses at a certain point due to moving loads on brigde-deck), influence coefficients (or responses in global bridge due to movement of a foundation component), and global bridge dynamic characteristics (or modal frequencies, mode shapes, modal damping ratios and modal mass participation factors), and structural responses such as cable forces, bridge geometry profiles, stress distributions, remaining fatigue life in fatigue prone components, reactions in bearings, movements in expansion joints, etc.

The scope of inspection is in general referring to : (i) routine visual inspection for inspecting the appearance of accessible structural components, (ii) special inspection with appropriate equipment and facilities (including load-trials and laboratory tests, where necessary) for inspecting/determining the corrosion status and load carrying capacities of identified problematic componnets. The detailed scope of monitoring and inspection for Qingdao Bay Bridge is tabulated in Table 1 below.

*Table 1 Detailed Scope of Inspection for Qingdao Bay Bridge*

| Category | Monitoring/Inspection Items | Methods and Locations | Applications |
|---|---|---|---|
| Structural Performance | Dynamic Characteristics and Forces/Stresses in:<br>● Suspension Cables<br>● Suspenders<br>● Cable Bands<br>● Stay Cables | All cable-supported bridges:<br>● Real-time monitoring for key components<br>● Manual inspection for all components | ● comparing with design values<br>● comparing with monitoring criteria<br>● investigating the variation trends<br>● calibrating/updating cable dynamic features and forces |
| | Deformation:<br>● Key Location in Pylons/Tower<br>● Cable Anchorage Locations<br>● Key Locations in Deck-Girders<br>● Key Locations in Major Piers<br>● Key Articluations<br>● Dampers | All cable-supported bridges:<br>● Real-time monitoring for key components<br>● Manual inspection for all components | ● comparing with design values<br>● comparing with monitoring criteria<br>● investigating the variation trends<br>● correlation analysis and plots |
| | Dynamics:<br>● Ambient Vibrations<br>● Ship Impacting Vibrations<br>● Seismic Vibrations | All cable-supported bridges:<br>● Real-time monitoring for global bridge<br>● Bases of key piers, pylons or tower | ● modal extraction<br>● response spectrum |
| | Strains:<br>● Key Sections in Girders, Piers, Pylons, and Tower | All cable-supported bridges Real-time monitoring for key components | ● strain/stress histories<br>● strain/stress distribution<br>● fatigue-induced damage |
| | Structural Temperature:<br>● Effective Temperature<br>● Differential Temperature | All cable-supported bridge by Real-time monitoring for key components | ● effective temperature in decks, piers, pylons and towers, hence thermal movements<br>● differential temperature, hence thermal stress |
| | Scouring Depth | All bridges by Manual Inspection | ● evaluation of load resistance in foundations<br>● comparing with design values<br>● investigating the variation trends |
| | Structural Steel Weld Inspection | All steel bridges by Manual Inspection | ● detection of cracks in welding components |
| | Stresses and Deflections in Box-Girders | Load-trials | ● evaluation of load-carrying capacities |
| | Dynamic Characteristics | Field vibration testing | ● evaluation of load-carrying capacities<br>● extraction of global dynamic characteristics |
| Durability Performance | Corrosion | ● All bridges by Manual Inspection<br>● By regular checking of corrosion cells<br>● By exposure monitoring stations | ● Estimating the time for depassivation<br>● Estimate the growth of corrosion products<br>● Estimate the formation of first crack in concrete cover<br>● Estimate the time for the falling off of cracked concrete cover |
| | Concrete Strength | All bridges by Manual Inspection | ● Tracing concrete strength<br>● Establishing the random concrete strength model |
| | Depth of Carbonation | All bridges by Manual Inspection | ● Estimating the time for depassivation |
| | Relative Humidity inside Deck-Girders, Pylons and Tower | All cable-supported bridges by eal-time monitoring for key components | ● comparing with monitoring criteria |
| Service-ability | Bridge Geometric Profiles | All bridges by Manual Inspection | |
| | Appearance Inspection | All bridges by Manual Inspection | |
| Environments | Air Temperature | All cable-supported bridges:<br>Real-time monitoring for key components | ● determining the variation trends |
| | Wind Speeds and Wind Directions | Two cable-supported bridges:<br>Real-time monitoring for key components | ● mean and gust winds<br>● wind rose diagrams<br>● wind power spectrum and length scales |
| | Hydrology, Waves and Ices | All bridges by Manual Inspection | |

## 5. STRUCTURAL HEALTH MONITORING WITH INSPECTION AND MAINTENANCE MANAGEMENT FOR QINGDAO BAY BRIDGE (SHM-I&MMS)

As shown in Figure 1 that the SHM-I&MMS for QBB is composed of four modular systems (i.e., SHMS, SHES, SHRS and I&MMS). The details of system architecture of SHM-I&MMS for QBB is illustrated in Figure 2 below.

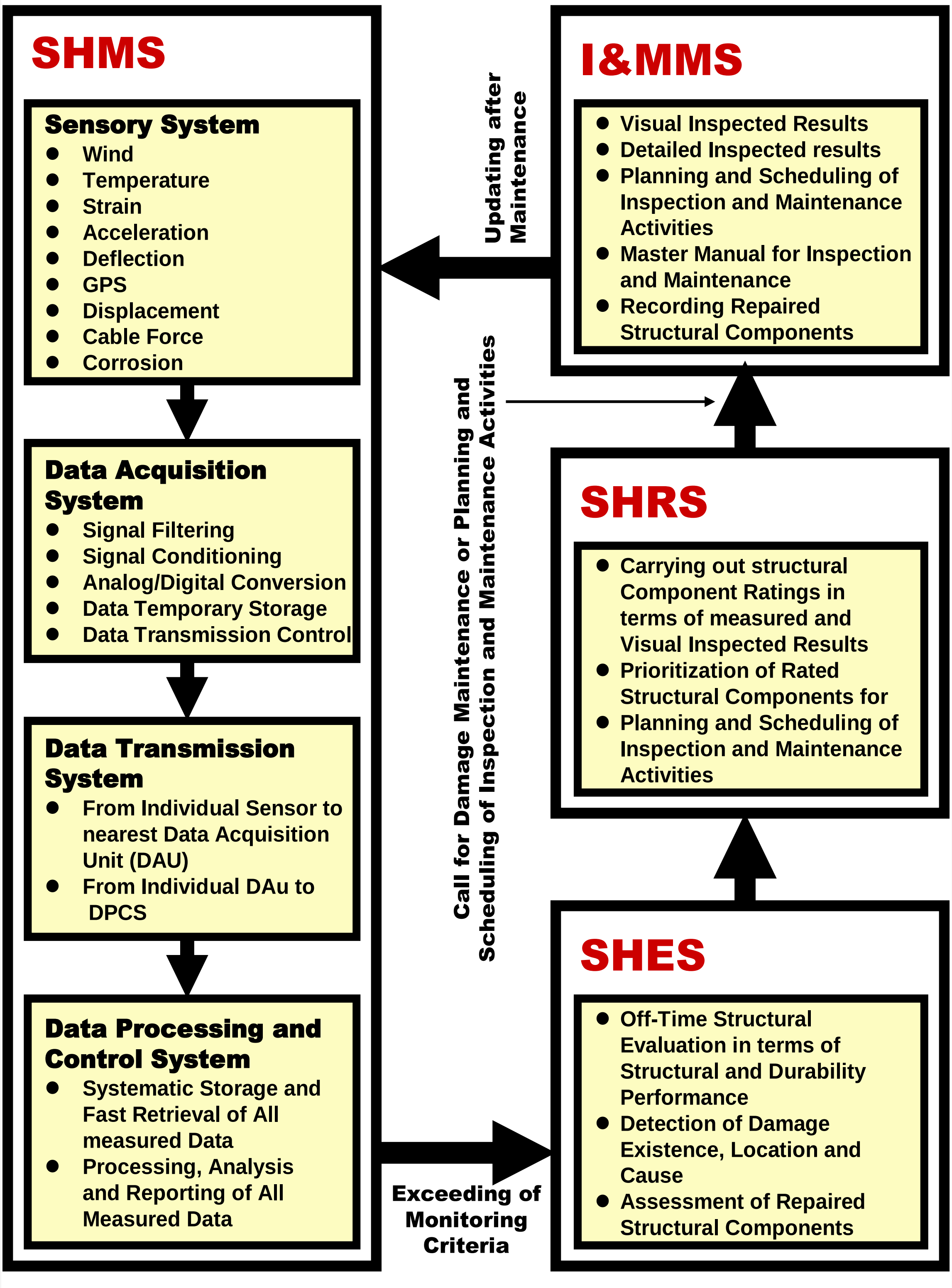


*Figure 2 The Detailed System Architecture of SHM-I&MMS for QBB*

### 5.1 Structural health Monitoring System (SHMS)

The SHMS is devised as an automatic system for processing, analysis and reporting of all types of measured data such as wind, temperature, strain, acceleration, deflection, displacement, corrosion, cable force, GPS, etc. It is the most importance system as it provides the objective observed results of the environmental variations of laods and attacks, and the structural and durability performance of the bridge structural system. Such data and information will combined with the inspection results (visual and special) for structural haelth evaluation of the bridge structural system. The SHMS is usually composed of four modular systems, namely, sensory system, data acquiston system, data transmission system and the data processing and control system. The arrangement of these four modular systems in SHMS is shown in Figure 2 (left hand side).

### 5.2 Structural health Evaluation System (SHES)

The SHES is called for action when the monitoring criteria are exceeded. The SHES is devised to carry out off-time structural evaluation of the bridge structural system basing on current measured data and structural conditions. The structural evaluation [4] is composed of two parts, namely, the load demand analysis and the laod resistance analysis.

The load demand analysis, basing on finite element software packages, will be carried out to investiagte whether damage has ocurred or not due to the detection of monitoring criteria exceedance. The types of structural analysis will be initial loads analysis (for confirmation of stress status basing on the as-built bridge geometry profile) , normal modes analysis (confirmation of as-built/current bridge dynamic features), influence analysis (confirmation of bridge staic features, static stability analysis (usually applying to highway and temperature loads and co-existing with permanent loads) and dynamic stability analysis (usually apply to wind or seismic or ship imapct loads and co-existing with permanent loads).

The load demand limits for stability analysis should be the respective serviceability limit state, ultimate strength limit state and the structural integrity limit state. The load resistance analysis, basing on codified approaches, will be carried out to estimate the resistance strength of key or problematic components. The resistance strength refers to the axial resistance, shearing resistance, moment resistance and torsional resistance of the structural component with the consideration of appropriate boundary condition. The resistance strength should also be computed to the three limit states as required in laod demand analysis. A comparison of the load demand and load reistance analyzed results at respective limit states will provide information to estimate the potential risk of damage induced on the problematic structural component.

### 5.3 Structural Health Rating System (SHRS)

The SHRS is devsied to carry out the ranking of the structural components used in planning and scheduling of bridge inspection and maintenance activities. In order to stored the rated information propoerly, an inventory system (or database) for system storage and fast retrieval of all relevant bridge information is required. The ranking of typical types of structural components are based on three types of ratings criticality ratings (by SHMS and SHES), vulnerability ratings (by SHMS and I&MMS) and structural condition ratings (by I&MMS). The detials of ratings could be found from References [5] and [6].

### 5.4 Inspection and Maintenance Management System (I&MMS)

The I&MMS is devised as a bridge inspection and maintenance management system (or central database) for systematic storage and fast retrieval of all types of inspection results, all maintenance results, master manuals for bridge inspection and maintenance. The inspection results include daily visual inspection, principle inspection, general inspection and special inspection with equipment/facilities, load-trials, laboratory tests, etc. All field inspected results will be stored in the central base through wireless cabling network.

## 6. SUMMARY

The SHM-I&MMS for QBB combines the works of real-time structural health monitoring system, near-real time safety evlaution system and bridge inspection and maintenance system to form an integrated system so that the SHMS provides I&MMS with appropriate target and structural condition while the I&MMS reinforces and complements the SHMS with updated information on structural condition. The integrated system design not only ensures the reliability and safety of QBB under operation, but also prolongs the service life of QBB through cost-effective execution of maintenance works.

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# NEW TYPES OF CONCRETE BOXED COFFERDAM FOR THE CONSTRUCTION OF OFFSHORE PILE CAPS

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## ABSTRACT

*In order to improve the corrosion resistance and speed up the construction progress of bridge construction, a new type concrete boxed cofferdam is used to replace the traditional steel boxes. This can improve the economic efficiency, reduce the risks of marine construction, reduce consumption of resources caused by construction and pollution of the marine construction. This technology was used in the construction of Qingdao Bay Bridge to save a large number of working hours and reduce costs about 51.86 million yuan. This paper introduces the construction technology of a new type of concrete boxed cofferdam and shows the mechanical properties of the boxed cofferdam. The research shows that the new technology can effectively reduce the stress of concrete boxed cofferdam caused by heat of hydration of concrete.*
***Key words:*** *concrete boxed cofferdam; capsule sealing; cap; bridge construction*

## INTRODUCTION

Qingdao Bay Bridge, following the East China Sea and Hangzhou Bay Bridge, is a world-class large-scale cross-sea bridge, whose length is up to 27 kilometers. It is one of the largest projects of Qingdao City during "Eleventh Five-Year" period. To ensure high standards, high quality, high efficient, successful completion and no accidents, the headquarters of Qingdao Bay Bridge construction combined the advanced experience and developed an underwater unsealed concrete boxed cofferdam for the sea pile cap construction process. At present, the construction of sea pile cap mainly uses the steel cofferdam and steel box with poured concrete sealed to create dry construction. In order to improve the corrosion resistance, speed up the construction progress, improve the economic efficiency and reduce the risk of marine construction, the consumption of resources caused by construction and the pollution to the marine enviroment caused by the construction, it is necessary to use concrete boxed cofferdam to optimize the pile cap construction technology.

At present, some bridge (*Sun Chengxin,2009. Yuan Tao, 2006. Jia Dongzhang,2007.Wang Bo,2008.Zhang Baoxing,2004*) constructions has already used the concrete box,but it is different with the unsealed concrete box. A brief description follows. Literature (*Xiang-Ping Huang, 2008*)introduced the analysis of the inner force on the boxes by the use of the infinite element model. Literature (*Xiang-Ping Huang, 2007*) analyzed the temperature stress of the concrete boxes on the East China Sea Bridge, revealed the reasons of cracks of the boxes and proposed the corresponding treatment. At the same time, it discussed the temperature stress of the thin-walled structure in the ocean. Literature (*Yaqing Liu, 2005*) gives a systematic introduction for the pile cap constrution of the East China Sea Bridge, including the choice of the construction statement for the pile cap construction. However, it is known that the concrete box cracked in the East China Sea Bridge (*Xiang-Ping Huang, 2007*). As a result of bad marine environment, the cracking may cause severe durability problem. Cracks would result in steel corrosion by the erosion of the choride, seriously affect the performance of the structure. Therefore, this article carries out a detailed research about construction process of the concrete boxes, and presents underwater unsealed concrete boxes (Fig.1).

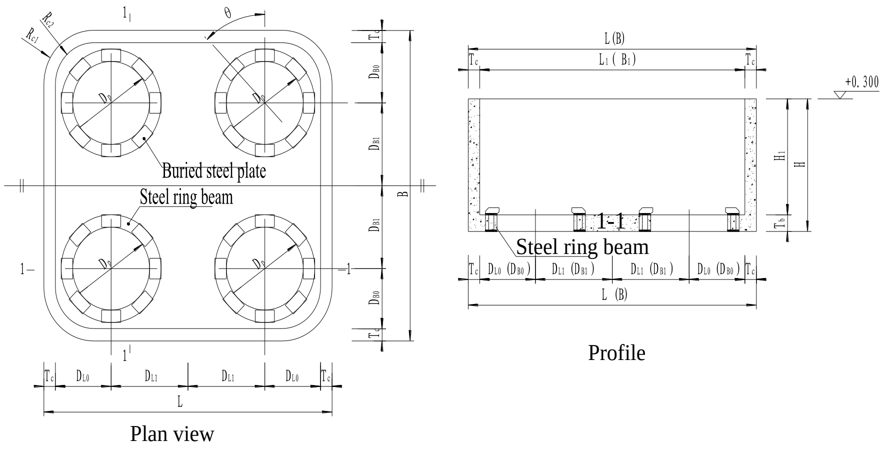



*Fig. 1 The structure of concrete box (Units :m)*

## CONSTRUCTION PROCESS

### Place model

According to construction requirements of caps, product the models. The number of the models inside and outside is four, the panel is steel with the thickness of 6mm.

*a) Model to be installed (out of the box)    b)Model to be installed(in the box)*

*Fig.2 Installation of the models of the sets of boxes*

*Fig.3    Preformed hole in the boxed bottom*

**Installation of reinforced concrete boxes**

The specific installation process as follows: Box shipments → maritime transport → field position (after barge then crane) →install the boxes.

In order to speed up the construction of concrete cofferdams four steel piles are welded with a well-shaped brace and cap, which can avoid the damage caused by the dash between concrete cofferdam and steel piles. The well-shaped caps were welded. After completing the preparations of the installation, a 600 t barge which has 50 t track was used to transport the concrete cofferdam and the hanger was set up at the same time. The main hanger which is used to install the box sets is composed of framed girder, cables and hangers. After examination crane, staffs leave the concrete cofferdam. They use the lifting crane to hook up concrete cofferdam and move the ship to the installation location (Fig.4).

*Fig.4 Installation of concrete cofferdam*

The concrete cofferdam is set up in accordance with the number of preformed holes of concrete cofferdam and the measured pile position. It is measured at the site that installation of a concrete cofferdam needs up to an hour in total. After installation the elevation of the top surface and the center position must meet the design standards. In order to carry out the underwater capsule sealing and welding system for a fixed conversion, the following work must be carried out .That is carrying out the anti-pressure system, adjusting and temporarily fixing the concrete box sets (Fig.5). Based on measurement at the site installing four sets of anti-pressure systems takes 3-4 hours.

*Fig.5 Anti-pressure system*

**Paste foam on the lateral wall of the boxes**

To prevent the lateral plate of the concrete boxes from cracking as a result of the lateral pressure of concrete pile cap, paste foam (2mm) on the place of sealing trip pasted. After cleaning concrete boxes, it is possible to paste foam on the lateral wall of the boxes.

**Capsule sealing at the bottom of boxes**

Capsule sealing process is the core technology and is never used in the history of bridge construction. Capsule sealing plays a important role for the construction of the boxe sets process.

According to preformed holes in the bottom plate (Φ1950mm) and the steel piles (Φ1800mm), a set of test equipment is designed and the tests are conducted more than two months. During tests more than 30 capsules with different thickness, size and raw materials ratio are used. Ultimately, it is successful. Based on the determined technical parameters and size from the result of the successful tests, more than 10 species raw materials are used to produce capsule after many processes (Fig.6)

*Fig.6 Rubber-tyred barrel capsule*

Before the boxes are shipped, the capsule must be produced in accordance with the technical requirements which is determined by the simulated test and up to standard. Then the capsule is installed in the preformed solt at the bottom plate of concrete cofferdam (Fig.7).

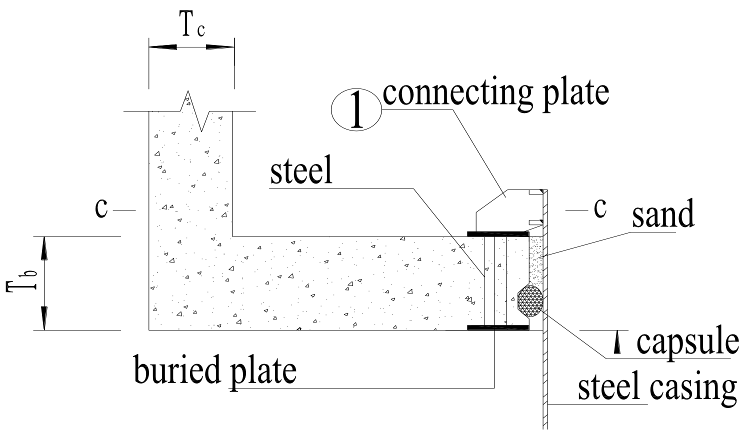



*Fig.7 Capsule installed in the preformed slot*

**Stress analysis of new boxed sets during the construction period**

In accordance with the layout of the cooling water pipes, establish the finite element model (Fig.8and 9).

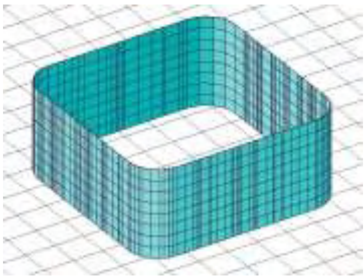


*Fig.8 Finite element model of foam*

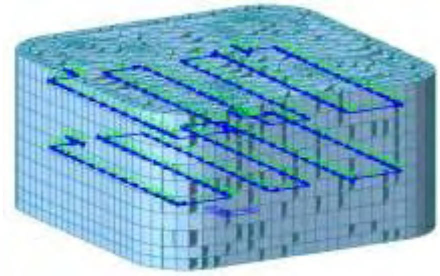


*Fig.9 Finite element model with consideration of cooled tube*

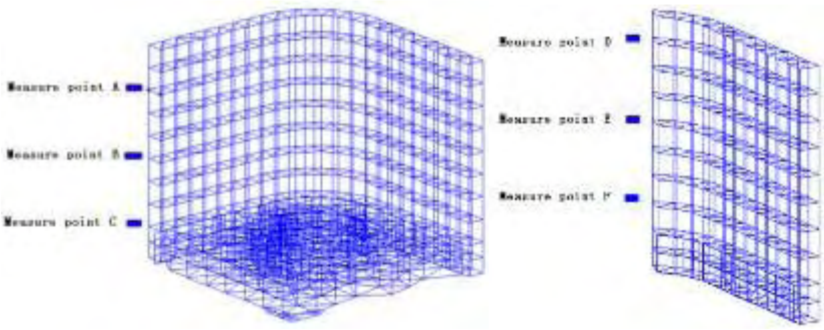
The main measured points are set in the middle of the lateral wall and the tapered edge, where four temperature sensors are put equidistantly from the top to the bottom.

Sensors buried at the scene are seen in the Fig.10.

*Fig.10 Sensors of boxes buried at the scene*

Calculated and measured data show that: the maximum principal stress appears at the bottom of the boxes and the interface of the piles. Although the interface exists the phenomenon of stress concentration, the site has reinforcement measures, such as ring beam, so the stress can be ingnored.

In order to further analyze the stress of the key positions, check the nodes of the key parts to analyze of time-stress. The cracking of the East China Sea Bridge suggest that cracking mainly occures in the lateral wall, so check the nodes of the key parts to analyze of time-stress.




*(1) Mid-span of box cofferdam* *(2) Fillet position of box cofferdam*

*Fig.11 Measure points*

Add up the max in all time- stress curve of the points, get the first principal stress among all the measuring point. The detail is as followed.

*Table1 Maximum stress and temperature of all measure point*

| Position | Max Stress(Mpa) | | Time(hr) | Max Temperature(℃) | | Time(hr) |
|---|---|---|---|---|---|---|
| | inner | outer | | inner | outer | |
| Measure point A | 0.6 | 2.8 | 80 | 35.3 | 28.2 | 240 |
| Measure point B | 0.8 | 2.6 | 70 | 35.2 | 28.3 | 260 |
| Measure point C | 0.8 | 1.9 | 60 | 33.5 | 28.1 | 240 |
| Measure point D | 0.75 | 0.1 | 80 | 33.7 | 26.7 | 240 |
| Measure point E | 1.1 | 0.5 | 80 | 31.9 | 27.3 | 230 |
| Measure point F | 1.9 | 1.7 | 80 | 30.5 | 27.5 | 220 |

Based on table 1, following conclusions can be drawn.

1) During the construction period, as a result of hydration heat and the own temperature difference, tensile stress appears on the lateral wall of the boxes. At about 80 hours the stress difference between medial wall and lateral wall is up to the max value (2MPa).

2) The first principal stress on the lateral wall of the box sets does not exceed the tensile stess of the concrete. Cracking of box sets does not occur during the construction period.

3) The mid-span of the lateral wall of the boxes, which is relatively worse than the tapered edge, indicates the tensile stress during the construction period.

4) During the construction of the box sets, the temperature difference between medial wall and lateral wall is 6°on average.

## CONCLUSIONS

1) Compared with the traditional steel boxes, the new type of concrete boxed cofferdam has anti-corrosion features on the cap in the marine environment. Additionally the construction period is shortened and the project cost is reduced effectively.

2) Capsule sealing process is the core technology and has never been used in the history of bridge construction. Capsule sealing plays an important role for the construction of the boxes sets process.

3) Compared with the traditional steel boxes, the new type of concrete boxed cofferdam added foam effectively reduces the stress of concrete boxed cofferdam which is caused by the cap hydration heat. This technology low the risk of cracking during the construction period.

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# THE KEY TECHNOLOGIES FOR DESIGN AND CONSTRUCTION OF A SINGLE COLUMN PYLON ON THE SEA ---- A SELF-ANCHORED SUSPENSION BRIDGE

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## ABSTRACT

*Qingdao Gulf Dagu River Main Channel Bridge is a self-anchored suspension bridge applying a single column pylon, an individual steel box girder connected by crossbeams* and a *spatial cable plane. The structure of the bridge is a novel and elegant design with excellent aesthetic effect. To ensure the structural safety and the capability of long-term service, many key factors have been taken into account. Factors such as the overall planning of structure and the harmony with the environment were considered through comparison-study in the design; the space-load of the self-anchored suspension bridge with a single column pylon and a spatial cable plane, and the earthquake resistance of the structure with a single column pylon were studied through theoretical analysis; key points such as the wind resistance of the* individual steel box girder connected by crossbeams *at sea, the stress mechanism and the design construction details of the steel anchoring box of the self-anchored suspension bridge were guaranteed through model tests. Because the geological conditions of the gulf are very complex and the ecological environment is fragile at the bridge site, a new type of auger drilling rig and a corresponding construction technology were invented during the construction period, the construction problems of the large-diameter and ultra-long pile foundation were solved under the complex marine geological conditions, and the ecological environment was protected. The large girder segments were hoisted into position by cranes. In order* to *ensure the safety of transportation and lifting, a new type of lifting tool was developed, and the versatility of the lifting tool was solved. A method of the loading of* large girder segments *onto the ships across obstacles was invented under tidal conditions, and the corresponding construction technology was developed.*

**Keywords**: Bridge, self-anchored suspension bridge, pylon, individual steel box girder , construction

## 1 INTRODUCTION

Qingdao Bay Bridge is the start of Qingdao - Lanzhou expressway, a Chinese national main road. It is the “Bridge Project” of “Expressway, Bridge and Tunnel Project” spanning from east to west over the sea in Qingdao municipal planning. It is also an important part of the main frame of the highway network in Shandong Province. As Qingdao Bay Bridge is not only a highway bridge, but also a part of the city roads and bridges, its geographical location is special. The bridge should meet the function of service, and should also be compatible with the local environment and terrain. The construction should try not to disturb the marine environment and marine aquaculture.

The bridge spans Jiaozhou Bay, and its location is an important scenic area of Qingdao City. According to the demands of urban planning, the bridge built should become a landscape or a landmark building, which put forward higher requirements for the selection of the bridge type.

Dagu River Channel Bridge is the largest of three Channel Bridges of Qingdao Bay Bridge, and it is also the most appropriate bridge which is designed to be a landscape structure and a landmark structure. In the design, a cable-stayed bridge, an arch bridge, a continuous beam bridge, a suspension bridge and other programs were compared. Through comparison-study, a four-span continuous self-anchored suspension bridge applying a single column pylon, an individual steel box girder connected by crossbeams and a spatial cable plane became the final selection of the construction program. The Bridge type was used for the first time in the world, and its bridge span was arranged as 80 +190 +260 +80 = 610m.

Subject to winds, waves, tides, the construction of Dagu River Channel Bridge had huge difficulties, accompanied by great risks. Affected by a high level of salinity and freeze, the corrosion situation is serious. The marine ecology is in a good cycle state, and the environmental requirements are high. The new structure and the special environment of the sea have brought new challenges to the design and construction. To achieve the design and construction of the new structure, series of research work has been carried out in these aspects, such as structural system design, the reasonable construction plan of the main beam at sea, the pile foundation construction program at sea , etc.

## 2 THE EXCELLENT AESTHETIC EFFECT AND THE REASONABLE STRESS OF THE BRIDGE

The single column pylon of Dagu River Channel Bridge is very high and magnificent. The pylon with a dumbbell-shaped cross-section is extremely like a sculpture. The main beam using an individual steel box girder connected by crossbeams is transparent and light. The cable system having a smooth shape, like a giant harp, plays a harmonious movement accompanied by the traffic. The overall bridge-type is light, elegant, which have a very rhythmic beauty. Double main cables of self-anchored suspension placed near the central reservation make the structure as a whole have not only a beautiful figure of a suspension bridge, but also a transparent effect like a cable-stayed bridge with a central cable-plane, fulfilling the organic integration of the structural design and the bridge landscape, and the perfect combination of strength and beauty. The styling is unique, and has a clear distinction with other domestic and foreign self-anchored suspension bridges.

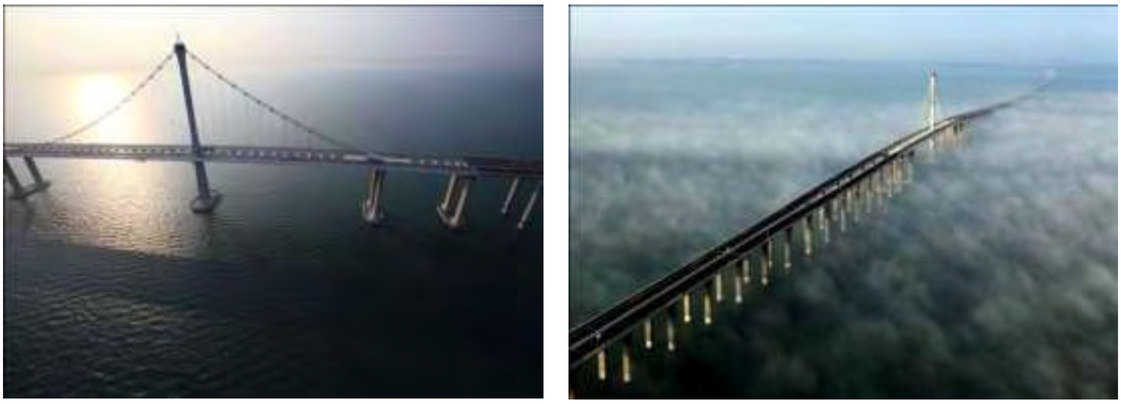

*Fig.1 Dagu River Channel Bridge after completion*

In support of a lot of detailed theoretical analysis and a large-scale model test research, the reasonable stress and the security of the innovative structural system was guaranteed. Through the aero-elastic model test of the full bridge, the wind resistance of the actual structure was examined. Through the three- components segment model test, the static and dynamic wind stability of the structure was studied. The combination scheme of the installation of the wind barriers and the elimination of the vortex-induced vibration not only will eliminate the vortex-induced vibration of the main beam which may occur in the marine environment, but also will ensure traffic safety in wind conditions. Through the large-scale static model test of the main cable anchorage structure of steel anchoring box, problems in design parameters and construction details of key parts of the steel box girder in which the main cable is anchored were solved.

## 3 THE INVENTION OF THE FOUR-POINT-HOISTING AND THREE-POINT-BALANCE LIFTING TOOL

The traditional method of construction is the short segmental assembling method of full framing and the incremental launching construction of less support. For the self- anchored suspension bridges at sea, the applicability of the above method is poor. Due to the long construction period, a great many field weld seams, the workload of the secondary anti-corrosion is heavy, which is detrimental to anti-corrosion of steel structure. Through the comparison of several schemes, the construction of Qingdao Bay Bridge adopt the large-segment hoisting construction method , taking advantage of the favorable condition of the usability of large floating cranes. The largest lifting weight of the floating crane can reach 2400t, and the lifting height can be more than 60m. According to the design, large segments of the main beam comprise the segments of the monolithic box girder and the segments of the separate box girder, and the types and forms of the segments are many. The vertical curve of the bridge in the facade will make the height difference of large segments larger. If the routine lifting tools were used, many kinds of lifting tools should be designed and segments could only be dropped in the flat slope after lifting girder. The adjustment of the slope after dropping girder segments was a heavy workload, and the construction risk increased. To complete the lifting of long segments of steel box girder, the four-point-hoisting and three-point-balance lifting tools using a cable-beam composite structure were developed（**Fig.2**）. The problem of the hoisting and installation of large segmental steel box girders with various forms, longitudinal slopes and transverse slopes were successfully resolved.

*Fig.2 The four-point-hoisting and three-point-balance lifting tool*

The four-point-hoisting and three-point-balance lifting tools make the force distribution of hoisting point clear, and the elevation of the four hanging points adjustable, which can avoid the redistribution of internal force caused by the deviation of the center of gravity of the girder segments. The connection between the beams adopt flexible cable, on the one hand, which makes the connecting operation convenient; on the other hand, makes the connection unrestricted basically by the angle difference, taking advantage of the flexibility of cables, and makes it possible to achieve the position adjustment of large segments of the steel girder in the air through the height adjustment of the hanging points.

The lifting tools invented in the construction were used for not only the hoisting construction of Dagu River Channel Bridge, but also the hoisting construction of the other two channel bridges, Cangkou Channel Bridge and Hongdao Channel Bridge， which cut the expenditure on the manufacture and changing different lifting tools to hoist different girder segments, improved the safety of the hoisting construction , speeded up the connection speed when lifting, and ensured the project schedule, and the economic and social benefits.

## 4 THE TECHNOLOGY OF THE LOADING OF LARGE GIRDER SEGMENTS ONTO THE SHIPS

Under conventional conditions, the loading of large girder segments onto the ships often adopts the more mature technologies, such as the slippage technology of setting tracks, the ro-ro technology and the loading through the floating crane hoisting. With regard to large segmental steel box girders of Dagu River Channel Bridge, the segments of the separate box-girder are overweight and extra-long, and the segments of the monolithic box-girder consisting of the upper and lower layers as a whole have great width and height dimensions. In addition, there is also a quay wall whose height is nearly 1m. In view of the above reasons, it was unable to adopt the conventional ro-ro shipment through laying tracks from the shore to the ship. At the same time, the transport ship changed in floating state with the tidal changed in the loading process. The loading cost of using a floating crane is quite high.

After careful study, the bow of the transport ship and the shore was relatively fixed. First, the steel box girder was moved utilizing the shore-based hydraulic system of the dolly car. After the cantilever end of the box girder was moved to the deck of the ship, the segment of the girder was move to the transport ship by the dolly car on board. Through the repeated relay transport using the shore-based dolly car and the dolly car on board, large segments were moved to the transport ship over obstacles.

At the same time, the ship floating state was adjusted by ship ballast water, which overcomes the problem of the height difference between the transport ship deck and the slipway ground due to tidal effects, fulfilling the loading of large steel box girders over obstacles in the tidal conditions. The whole loading process is shown in **Fig.3**.

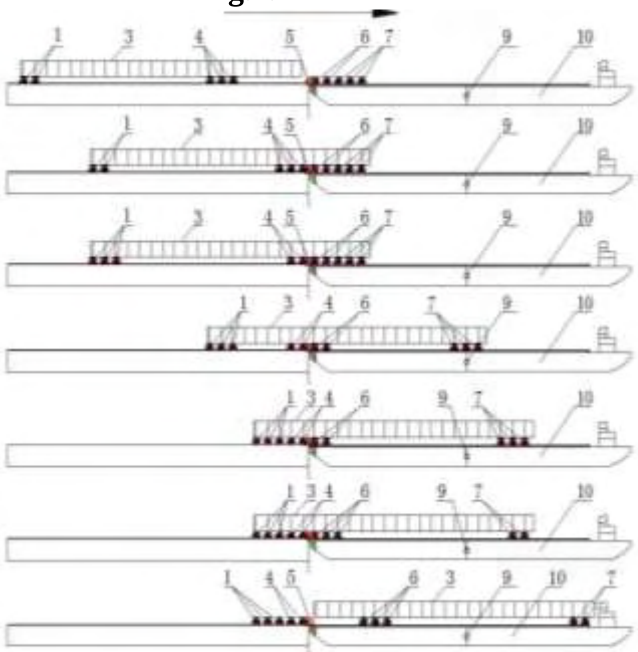

*Fig.3 the loading process of large girder segments onto the ship*

## 5 THE CONSTRUCTION TECHNOLOGY OF THE PILE FOUNDATION MEASURING 2.5m IN DIAMETER, AND 88m IN LENGTH AT SEA

There are a total of 24 piles of the main tower of Dagu river channel bridge, whose average length is 70m. The longest pile is up to 88m.There are a total of 2×19 piles of auxiliary piers, whose average length is 51m. There are a total of 2×19 piles of transition piers, whose average length is 51/37.5m. All the pile diameters are 2.5m. As the underwater strata at the bridge site are relatively complex, the hole-creating of the pile foundation can use the abrasion drill and the auger drill. The construction of the auger drilling has many advantages: the efficiency of the hole-creating is high; the stability of the hole wall is high; the hole trimming is thorough after the hole-creating; the noise of the construction is low; the emissions of the mud are less; the construction speed is fast; the construction method is conducive to the environmental protection. But the requirement of the construction platform is high.

Combined with the equipment manufacturers, the XR280 drilling rig was developed and improved. The maximum designed drilling diameter of the machine is 2.5m.The maximum drilling depth can reach 88 meters, which is the deepest in China. The performance of the XR280 drilling rig is advanced, and the operation is convenient. The technology has reached the international advanced level. According to different strata, different forms of drills were developed, as shown in **Fig.4**.

*Fig.4 different forms of drills*

In order to make wall-protecting slurry, bentonite and seawater was added into the steel casing pipe. In the process of drilling, the wall-protecting slurry was recycled to use after purification by purifier. The mix proportion of the wall-protecting slurry could be adjusted at any time, according to different strata. The operating torque was reduced effectively through the hierarchical reaming to accelerate the construction speed of the auger drilling.

The practice of the auger drilling construction proved that the construction process of the drilling and the pouring of piles was successful, and the quality of the holes was good in accordance with the studied construction technology. All the test indexes met the requirements of the specification and the design, demonstrating that the research achievements of the hole-creating technology of the pile foundations with large diameters at sea was feasible and the new construction technology obtained certifications.

The construction technology of the auger drilling with large diameters has been widely used, which greatly reduced the amount of the mud, speeded up the construction of the pile foundations and created very considerable economic benefits.

## 6 CONCLUSIONS

The Qingdao Gulf Dagu River Main Channel Bridge is a self-anchored suspension bridge applying a single column pylon, an individual steel box girder connected by crossbeams and a spatial central cable plane. It provides a new choice for the structure design and construction of the bridges at sea. The design idea of the organic integration of structural design and bridge landscape is helpful to design and build safe and beautiful bridge structures. The problem of the hoisting and installation of large segmental steel box girders with various forms, longitudinal slopes and transverse slopes were successfully resolved, using the four-point-hoisting and three-point-balance lifting tools invented in the construction. The continuous ro-ro shipment of overweight, extra-long, large steel box girders over obstacles in the tidal conditions was successfully realized.

The research achievements of the key technologies for the design and construction of Qingdao Bay Bridge can provide a useful reference for the construction of a sea bridge.

# SOIL AND BEDROCK CONDITIONS TO BE EXPECTED IN TALLINN – HELSINKI TUNNEL CONSTRUCTION

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## ABSTRACT

*The capital areas of Helsinki in Finland and Tallinn in Estonia have grown enormously during the last 20 years. The about 80 km wide Gulf of Finland separates the cities and restricts movement of people and goods. The idea of a tunnel between Tallinn and Helsinki was presented in early 1990's and has been a subject to a lively debate. The tunnel will be an extension of the future Rail Baltica railway, which is a project to improve north–south connections among European Union Member States. This project is already accepted by the Council of EU as a first priority EU project.*

*This paper discusses the bedrock construction conditions and the methods to be used in tunnel construction. The focus of this paper is to provide an overview of the geological and geotechnical properties of the construction environment and to describe the possible difficulties in building the world's longest undersea tunnel. The information is based on a co-operation project between the Geotechnical Division of Helsinki, the Geological Survey of Finland and the Geological Survey of Estonia in 2012.*

*The tunnel area is located at the border between the East European Platform and the Fennoscandian Shield. In the Helsinki area the exposed old Precambrian hard bedrock is overlain with a thin layer of loose Quaternary sediments. Near Tallinn the old crystalline basement meets the about 1.2 billion years younger sedimentary rocks. The tunneling project will be challenging especially in the area of its southern end due to limited experience of conditions near the interface between these two formations.*

*Possible methods for tunnelling are drilling and blasting techniques specific to hard rock conditions such as in Finland, or use of tunnel boring machines (TMB) as an alternative on Estonian site. Both methods are shortly described and compared in this paper.*

## INTRODUCTION

Taking in account traffic connections and passenger movements, Finland can be considered as an island. About 80 per cent of freight traffic leaving Finland nowadays goes by sea. The cities of Helsinki and Tallinn on both sides of the Gulf of Finland have grown enormously during 20 years and at the same time traffic by the sea has also increased. Today some 7.3 million passenger cross the sea each year, and the number is growing. Because of the importance of good connections from Finland to Europe, new ideas of logistics have arisen in the development of broader frameworks of accessibility of the Baltic Sea region. Rail Baltica is a project to develop a new railway connection from Warsaw to Tallinn. Only about 50 – 60 km strip of water obstructs a direct rail connection from Finland to Europe.

In the early 1990's, the Geological Survey of Finland published the geological maps of the Baltic Sea's Geology [1]. It was evident that old and hard crystalline bedrock continues below the Gulf of Finland and sinks gently under younger sediments in Estonia. This generated the idea of the possibility to construct an undersea tunnel from Finland to Estonia, especially taking into account good experience in developing long undersea tunnels in Finland. The idea was first proposed in 1992 at the Nordic Geotechnical Meeting (NGM92) in Aalborg [2].

The idea of the tunnel construction has been submitted and discussed in several contexts in seminars and congresses held by the experts of tunnel construction [2,3]. A feasibility study of a railway tunnel under the Gulf of Finland was submitted by Anttikoski in 2007 [4] A Helsinki-Tallinn Feasibility Study Working Group was established in 2008 on the basis of the meeting of the mayors of the cities of Helsinki and Tallinn. Later in 2010, a Euregio Helsinki -Tallinn Transport and Planning Scenarios Project (H-TTransPlan) was developed. The final report of the project was published at the beginning of 2012 [5].

During the years 2011 – 2013, debate about the tunnel construction has been intensive. Critical voices have been strong and several negative statements have been given. At the beginning, the tunnel idea initiators did not receive attention from politicians or transport planning authorities [6]. Nowadays the tunnel alternative is considered as a good alternative for the transport of people and goods. For the rapidly developing area several economic and technical facts favoring the tunnel alternatives can be proven.

Besides the financial and functional feasibility studies, also technical feasibility has been assessed [2, 4], but it has been in a minor role. This paper contains a review of what we know of the construction suitability of the bedrock in Finland and Estonia and what kind of technical challenges are to be expected for the construction.

## TUNNEL ALTERNATIVES AND TECHNICAL SOLUTIONS

Two alternatives for (Figure 1) excavated rock tunnels under the Gulf of Finland [4] are proposed. In both options, the track would link with the Muuga coastal railway on the Narva main road in Maardu, Estonia. On the Finnish side, the Porkkala track option would link with the coastal railway in Jorvas in the municipality of Kirkkonummi. The Pasila track option would link with the main railway line after the Pasila depot. The undersea tunnels will be railway tunnels. The lengths of the undersea tunnel section are 58 km and 73 km, respectively, and about 12 – 15 km is required for the other tunnel sections. The tunnel system would use two track tunnels with cross-sections of about 80 square meters, and one possible maintenance tunnel (Figure 2).

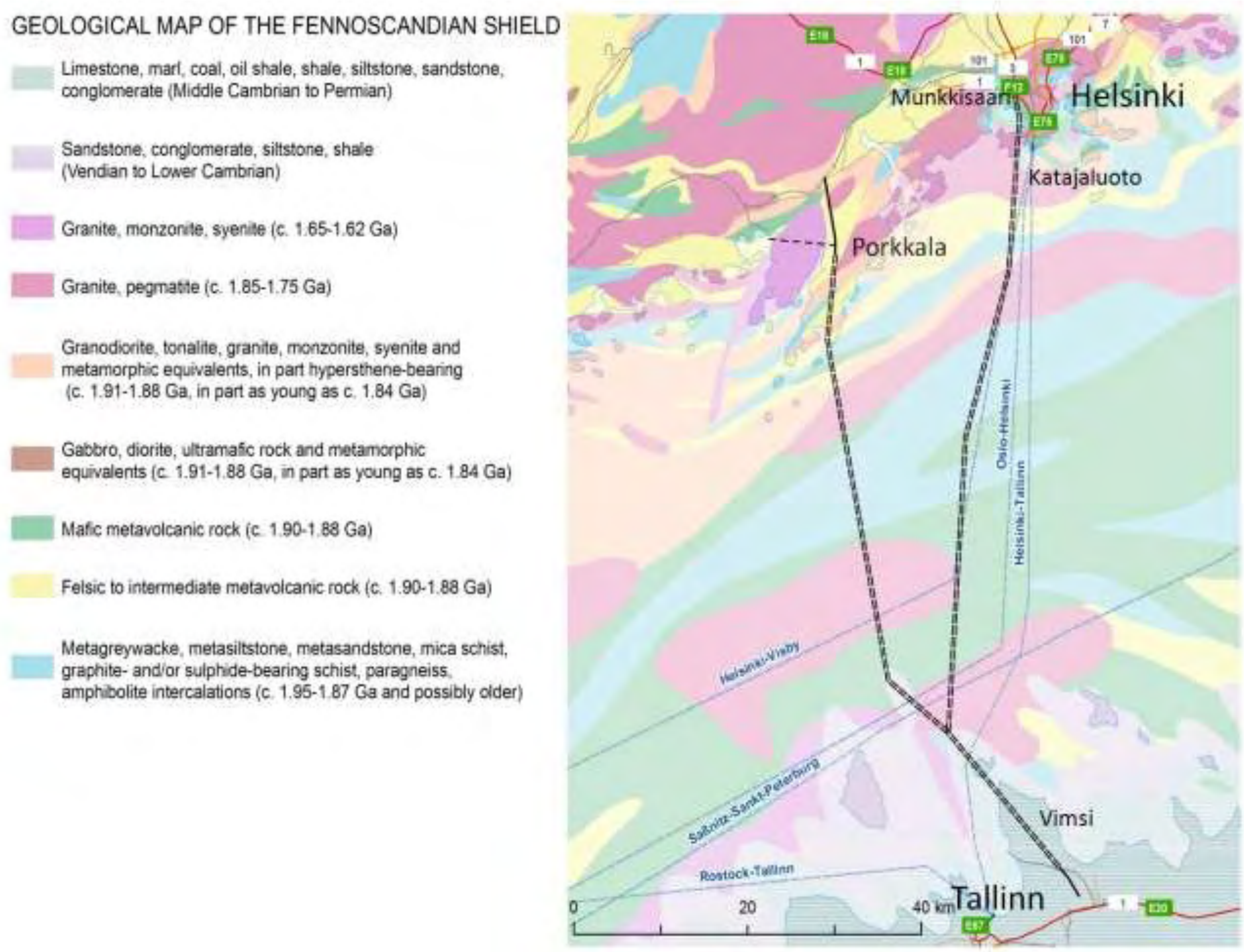


*Figure 1. Proposed two tunnel alternatives from Finland to Estonia, 85 km long Helsinki – Tallinn connection and 70 km long Kirkkonummi – Tallinn tunnel [4] on a geological map of Fennoscandian shield [1] .*

The maximum tunnel longitudinal inclination could be about 1.2 – 2.0 %. The depth of the Gulf of Finland is 90 – 100 meters. The lowest point of the tunnels is estimated to be about 220 meters below the sea [4]. This is based on predictions of the depth of the hard bedrock surface.

The tunnel lines are planned to go through islands and shallows at the Gulf of Finland. At sea, entrances to the work tunnel and work bases would be built on islands along the route. In addition, manmade islands with service shafts to the tunnels would be created on sea banks, where water depth is less than 10 m. Service shafts and ventilation exhaust systems will be constructed on the man-made islands. The islands will offer possibilities for house maintenance facilities [4]. The islands would serve also as wind power stations.

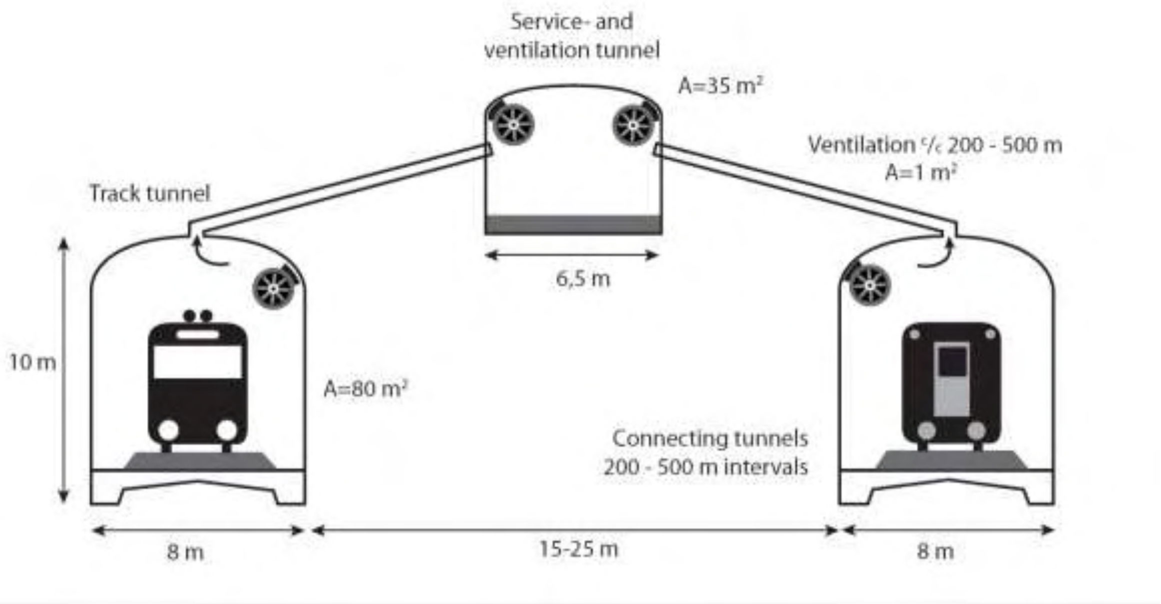


*Figure 2. Submitted cross-section of the rock tunnel under the sea [4].*

## INVESTIGATION AND REVIEWS OF THE GEOLOGICAL DATA

Geological data of the Finnish area is obtained mainly based on mapping made in the coastal area and islands. The geology of the Precambrian in the surroundings of the Gulf of Finland is given by Koistinen [1]. More detailed data is gathered in some undersea sewage tunnel projects reaching a few kilometers to the sea. Also seismo-acoustic profiling and echo soundings are made, but the information of the depth of the crystalline bedrock is not available in this description. The general features of the bedrock can also be estimated from the aerogeophysical small scale maps.

The description of investigation and geological setting of the Estonian area is based on the report made in 2012 by the Geological Survey of Estonia (EGK) for the City of Helsinki and Geological Survey of Finland (GTK) [7]. In that work, the data was collected from different databases of a predetermined area within the Estonian Exclusive Economic Zone (Figure 3). On the basis of the data, a 3D –model (Figure 4) of the main geological units was compiled and explanation of the physical properties of the soil and bedrock units was given [7].

On the Estonian side, the marine geological data are based mostly on marine geological mapping in the scale 1:200,000. Data for the Tallinn Bay is mostly interpreted with data from islands and mainland drill cores. Wells reaching the Precambrian crystalline basement were drilled on some islands (Naissaar, Prangli, Aegna). Low-frequency seismo-acoustic continuous profiling (0-450 Hz) with echo sounding (24 kHz) was used as geophysical method in mapping. Distance between profiling and sampling was ca. 2 km.

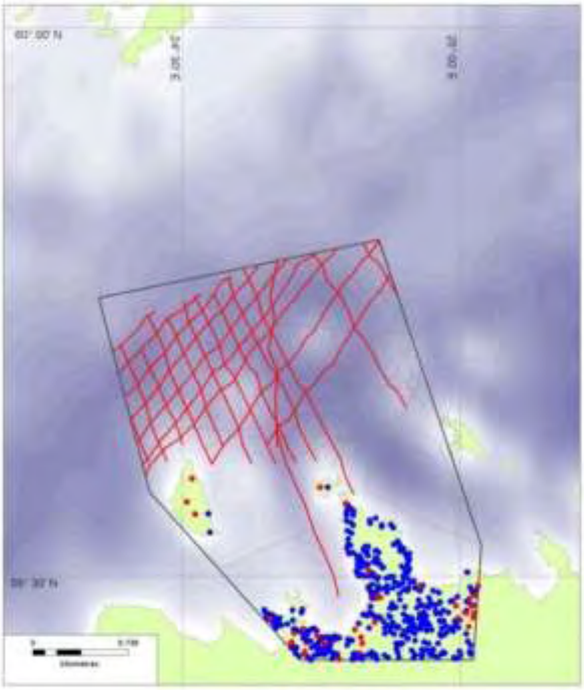

*Figure 3. The data used in compiling the 3D model of the Estonian site [7]. Lines – seismoacustical profiles, bold lines –seismoacustical profiles used in this project, dots – drill holes (red ones reach the crystalline basement).*

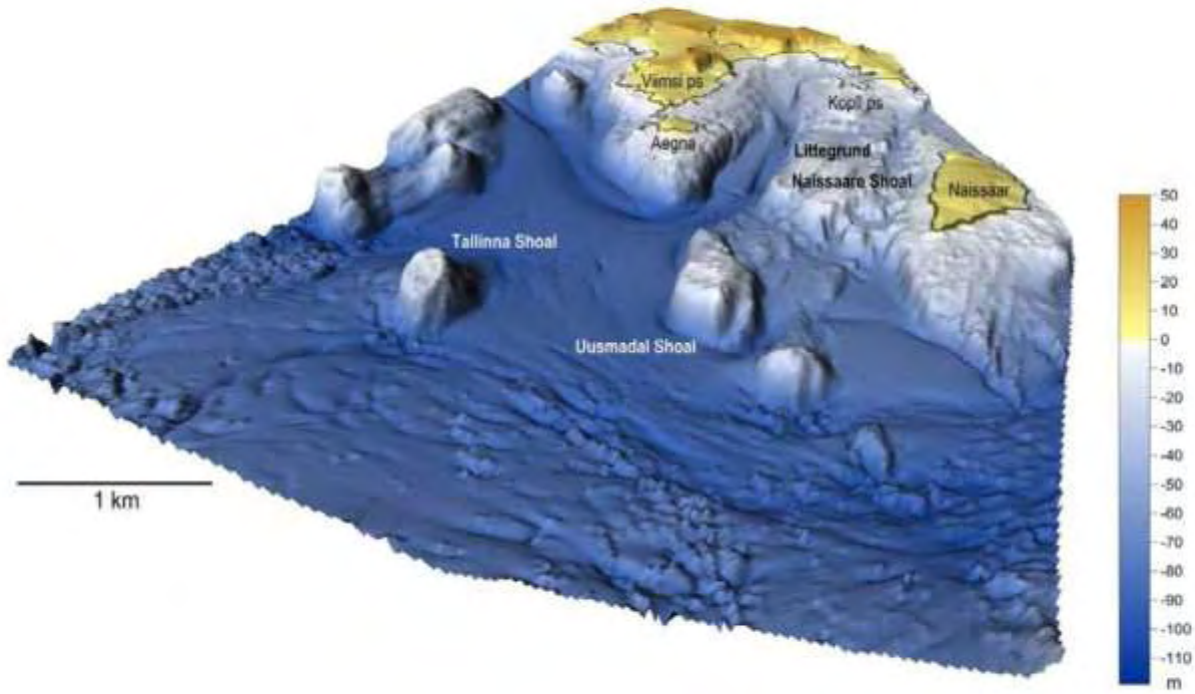


*Figure 4. The 3D surface of the Quaternary sediments of the Estonian site according to the compiled 3d-model [7].*

**Geological setting**

The Gulf of Finland is 50 - 60 km wide. Along the north coast (i.e. Finnish coast) the 1.8 – 1.9 Ga old Precambrian crystalline basement is exposed. It forms a sedimentation basement for much younger Ediacaran (Vendian) sediments in Estonia. The sedimentary cover comprises rocks belonging to Ediacaran (Vendian), Cambrian and Ordovician systems (bedrock), and loose Quternary deposits (i.e. Estonian coast). The sedimentary cover in the Tallinn area reaches up to 200 meters in thickness. Bedding in the sedimentary cover is close to horizontal (in average, dipping southwards 2–3 m/km), but there are some linear dislocations (Figure 4 [7]).

The study area is located between the border of the East European Platform and the Fennoscandian Shield (Figure 1). The geological section consists of two principal elements in the platform area, the Precambrian crystalline basement and sedimentary cover. The crystalline basement contains younger formations of the Subjotnian rapakivi granites and remnants of Jotnian sediments and diabases. The whole crystalline basement is eroded quite flat during long lasting continental erosion and dips gently to the south below Ediacaran rocks at the depth of about 130 – 140 meters below sea level near the coast of Estonia (Figure 5).

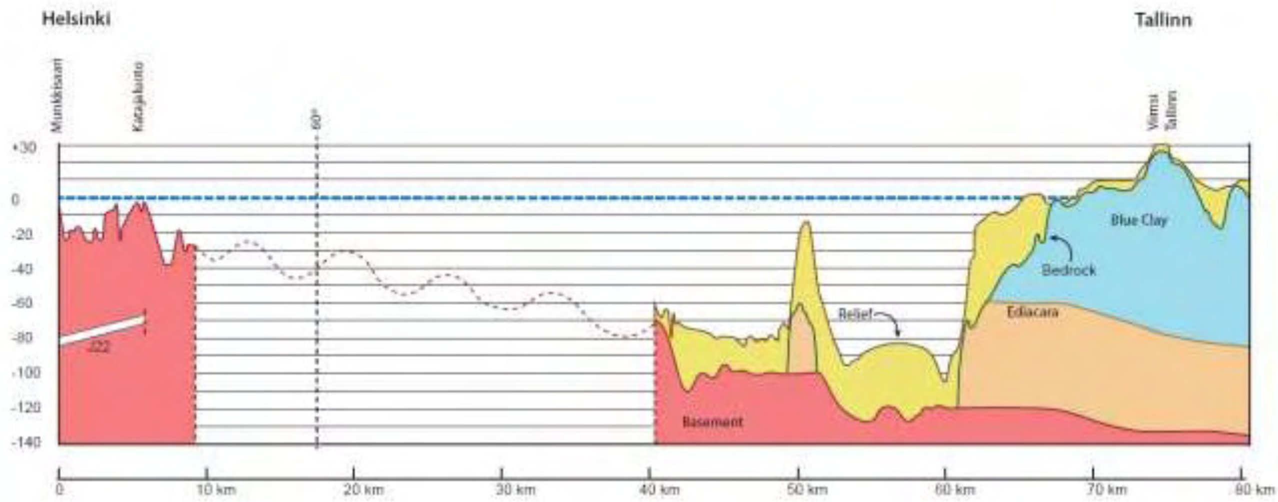


*Figure 5. A cross-section through the Gulf of Finland from Helsinki, Munkkisaari to Tallinn, Vimsi (Figure 1) according to the compiled 3D-model [7] and the data of Geotechnical Division of the City of Helsinki. J22 is the cleaned wastewater outlet tunnel, which was built in the 1980s and extends from the Viikinmäki wastewater treatment plant to the area south of Katajaluoto. The tunnel measures 17 km, of which 8 km are in the sea area.*

**The Precambrian crystalline basement** occurs on the Finnish side in the islands of the varying and broken coast. No observations of the bedrock are made in deeper water areas. In the undersea sewage tunnels excavated some kilometers to the sea, typical gneisses and granitic rocks are seen [1]. The geology of the crystalline basement does not change in this area dramatically, according to available small-scale geophysical maps. On the Estonian side, crystalline basement is represented by different complexes of metamorphic and intrusive rocks. The crystalline basement has been subjected to intense denudation during pre-Ediacaran time.

Therefore, the crystalline basement is covered by a 1–20 m thick weathering crust under the sedimentary rock cover and its surface is quite smooth. The weathering crust is absent in the sea bottom areas, where only postglacial loose deposits cover the crystalline basement, and the surface of the crystalline basement is uneven (such as roche moutonnée features).

The crystalline basement is characterized by the stratified Jägala complex of intercalating sillimanite - cordierite and biotite gneisses, felsic, intermediate and mafic metavolcanites, and leucocratic gneisses [7]. The physical-mechanical properties are illustrated in Table 1.

Intrusive rocks are represented by the Naissaar and Neeme rapakivi massifs. They consist mainly of porphyritic rapakivi granite of homogenous texture. Physical-mechanical properties of the rapakivi granites of the Neeme Massif are seen in Table 1.

*Table 1. Physical-mechanical properties of different formations along the Tallinn-Helsinki tunnel [8,9]. 1= volumetric weight, 2= compressive strength, 3= porosity, 4= P-wave velocity ca. 6000-6500 m/s, 5= thickness of formation.*

| | | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|---|
| | | G/Cm3 | Mpa | % | m/2 | m |
| Precambrian gneisses | A | 2.65 - 2.75 | 110 - 240 | 0.1 - 0.2 | 6000 - 6300 | |
| Rapakivi granites | B | 2.65 | 100 - 200 | 0.1 | 6000 - 6500 | |
| Weathered crystalline besement | C | 2.0 - 2.6 | 1 - 100 | 1 - 20 | 2000 - 5000 | 1 - 20 |
| Sandstones | D | 2.0 - 2.3 | 1 - 25 | 10 - 20 | 2000 - 3000 | n. 60 |
| Siltstone | E | 2.25 - 2.35 | 5 - 25 | 10- 15 | 2500 - 3500 | 1 - 2 |
| Sandstones | F | 2.1 - 2.2 | 1 - 5 Mpa | 20 - 25 | 2500 - 3000 | 15 |
| Blue clay | G | 2.3 - 2.4 | 2 - 4 Mpa | 8 - 10 | 2000 - 2500 | 45 |
| Limestone | H | 2.55 - 2.65 | 100 - 150 | 0.1 - 5.5 | 4000- 5500 | 20 |
| Glauconite sandstone | I | 1.95 - 2.1 | 1 - 20 | 1 - 10 | 2500 - 3000 | 2 |
| Alum shale | J | 1.9 - 2.0 | 40 - 50 | 1 - 10 | 3500 - 4000 | 3.5 |
| Sandstones | K | 2.1 - 2.8 | 1 - 40 | 1 - 20 | 2500 - 3500 | 3- 8 |
| Quaternary loose sediments | L | 1.5 - 2.2 | < 1 | 10 - 30 | 1500 - 2000 | 0 - 60 |
| A= Rocks of Jägala complex, B= Naissaar and Neeme rapakivi, C= Weathered crystalline rocks. D= Ediacaran sandstones, E= Ediacaran siltstone, F= Cambrian quartzose sandstones of the Tiskre formation, G= Cambrian blue clay of the Lontova formation, H= Ordovician limestone, I= Ordovician glauconite sandstone of the Leetse formation, J= Ordovician alum shale of the Türisalu formation, K= Ordovician sandstones of the Kallavere and Ülgase formations, L= Quaternary deposits | | | | | | |

**The Ediacaran system** is represented mainly by weakly cemented sandstones with relatively thin (up to 1–2 m) layers of varicolored (red-brown-grey) clayey siltstones of the Kroodi formation. Composition and the physical-mechanical properties of the Ediacaran rocks depend from this composition. For the sandstones, the properties are illustrated in Table 1.

**The Cambrian system** is represented mainly by its lower series (thickness ca. 100 m), which consists of siliciclastic rocks (clay-, silt- and sandstones). The upper part of the Cambrian section crops out up to the thickness of 20 m at the foot of the Cambrian–Ordovician escarpment of the

Baltic Klint. The outcropping section includes an up to 15 m thick layer of weakly and moderately cemented fine-grained quartzose sandstones of the Tiskre formation and up to 5 m thick sequence of blue clays of the Lükati formation. Physical-mechanical properties of the rocks are given in Table 1**.**

Blue clays are the oldest (ca. 530 Ma) clays in the world that have retained the plastic properties of clay (Table 1), although being more consolidated than young glacial clays**.** The complete succession of blue clay is ca. 60 m thick and consists of (from top to bottom): ca. 15 m thick blue clay stratum with quartzose sandstone interlayers of the Lükati formation and ca. 45 m thick stratum of pure blue clay of the Lontova formation. The lower part of the Lontova stage is represented by a more than 20 m thick stratum of the Sämi member, where there is a ca. 10 m thick bed of weakly cemented quartzose sandstone inside of a ca. 10 m thick sandy blue clay layer. The content of blue clay in the Sämi member decreases and the content of sandstones increase towards the west. By composition, two complexes can be distinguished inside the Cambrian system: upper – sandstones of the Tiskre formation, and lower – blue clays.

**The Ordovician system**, covering the Baltic Klint limestone plateau in the southern part of the area, is represented mostly by up to 20 m thick stratum of Middle-Ordovician carbonate rocks (limestones). The physical-mechanical properties of the Ordovician limestone are quite stable (Table 1). Lower Ordovician is represented mostly by more than 10 m thick stratum (from top to bottom): ca. 2 m glauconite sandstone of the Leetse formation; ca. 3.5 m alum shale (graptolite argillite) of the Türisalu formation; 3–8 m thick stratum of quartzose sandstones with phosphatic brachiopod detritus of the Kallavere and Ülgase formations. Physical-mechanical properties are varying (Table 1).

**The Quaternary deposits** cover most of the area. Their thickness varies between some tens of centimetres on alvars and up to 150 meters in buried valleys (bedrock surface elevation -135 m. b.s.l. in well no. 174), reaching in places up to the crystalline basement and thus cutting through the complete bedrock complex. Merivälja, Ülemiste, Kopli and Harku buried valleys can be observed from east to west, and they extend tens of meters into the crystalline basement. They are filled with till and loose silt, sand and gravel deposits.

The thickness of the Quaternary deposits is 20–60 m on the sea bottom. They consist of up to five strata (from top to bottom): contemporary marine deposits (mud) – up to 15 m; post-glacial deposits (clays) – up to 5 m; late-glacial (Baltic Ice Lake) deposits (varved clays) – up to 20 m; glacial deposits (till) – up to 60 m.

## CONSTRUCTION SUITABILITY

The Quaternary sediments are water-saturated loose and soft deposits and pose a challenge for the tunnel penetration [7]. In buried valleys, the Quaternary sediment thickness may reach up to 150 m, With the high groundwater pressures it is a construction environment that has to be avoided. Supposedly also the rocks of the Ordovician system stay outside of the tunnel project. The blue clay stratum is a steady aquitard and good environment for tunnel penetration. The

Ediacaran water-saturated silt- and sandstones, reaching up to 60 m in thickness, are an important source of water supply for the Tallinn city and its surroundings, and one of the main challenges for tunnel penetration. The crystalline basement consisting of very hard solid rocks is a firm and protected environment for the tunnel constructions. In Finland, several large projects have been developed in these rocks.

In this study, the geological succession is divided into seven complexes, according to geotechnical and hydrogeological properties of rocks and deposits, which mostly coincide with the litostratigraphical units (Table 2).

*Table 2. Physical-mechanical properties of the rocks divided into 8 groups [7]. Complexes from 2 to 6 represent the sedimentary rocks, collectively called bedrock. Bedrock is, partly or completely, eroded by later processes.)*

| Complex no. | Thickness m | Properties tbl. 1 | Construction conditions | Tunnel km incl. 1.5 % | Formation |
|---|---|---|---|---|---|
| 1 | 0 - 60 | L | Very difficult | outside tunnel | Quaternary deposits; |
| 2 | 20 | H | Good | outside tunnel | Ordovician limestones; |
| 3 | 10 | J,I | Very challenging | outside tunnel | Lower-Ordovician alum shale and glauconitic sandstone |
| 4 | 15 + 10 | I,K | Very challenging | 2 | Lower-Cambrian and Lower-Ordovician sandstones; |
| 5 | 60 | G | Good | 4 | Blue clays (Lükati and upper part of Lontova formations); |
| 6 | 60 | D, E | Very challenging | 4 | Ediacaran silt- and sandstones (Kroodi formation); |
| 7 | 15 | C | Challenging | 1 | Weathered crust of basement |
| 8 | (km) | A, B | Very good | 63 | Precambrian basement metamorphic and igneous rocks |

## CONCLUSIONS

Helsinki – Tallinn tunnel will be constructed in varying rock conditions. Most of the tunnel will lie in hard crystalline Precambrian bedrock starting from the Finnish site. Close to the south coast of the Gulf of Finland the tunnel will rise to younger Ediacaran and Cambrian rocks. The physical and mechanical properties are quite different from that of crystalline bedrock. The tunnelling will be very challenging in an about 6 km long segment in very porous and water-leaking sand- and siltstones. Also the possibly weathered surface of the Precambrian crystalline basement in contact with Ediacaran sediments might be challenging. In water-leaking and weak rock sequences the tunnel can be excavated by tunnel boring machine (TBM) with reinforcing the tunnel by concrete-steel lining.

Most of the tunnel will lie in the old Precambrian crystalline gneisses and granitoids, along about a 63 km long stretch in the Helsinki City alternative. Because of missing investigations, it is uncertain to predict the construction suitability. But experiences on the Finnish side of the Gulf of Finland have shown that the conditions are usually very good. In the old bedrock, fracture and weakness zones are usual, but normally they are not very wide or insurmountable.

For the tunnelling under the sea, the most important issue is to maintain the rock roof. Therefore, in the future lots of investigation to locate the bedrock surface are required. That will be done with seismo-acustic sounding in the first phase and by drilling in the second phase. Weakness zones also have to be located. For this, geophysics and interpretation of sea bottom (bedrock) relief maps will be useful methods.

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# BRIDGE CROSSING THE TRONDHEIMSFJORD

**Technical solution and benefits to society**

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## ABSTRACT

*The Trondheim fjord is approximately 130 km long. The Trondheim metropolitan area, with about 200,000 residents, is located on the south side, midway into the fjord. Fosen is a geographically large area north of the fjord with about 25 000 inhabitants. The distance between Trondheim and the center of gravity of the population of Fosen, is about 40 km by road and ferry. The ferry distance is 7.5 km. The alternative is to drive around the fjord, a distance of about 160 km. The ferry is used by most people. The average daily traffic is about 2,900 cars. It is slightly less than the top three ferry connections on the E39, but more than the other E39-connections.*

*The ferry crosses where the fjord is narrowest. A bridge should be in approximately the same area. If construction is approximately linear, but adapted to topography, it will be about 6.7 km long. The water depth of the fjord along the south side, running for about 1 km, is between 50-100 meters. In that area it is possible to construct a fixed bridge, most likely a cable-stayed bridge. Over the rest of the fjord the depth is about 500 meters to the bottom of the sea and about 1000 meters to the rock. Soils are largely assumed to be clay, at least at the top. In this part, a floating bridge with sideways anchoring is the most appropriate solution.*

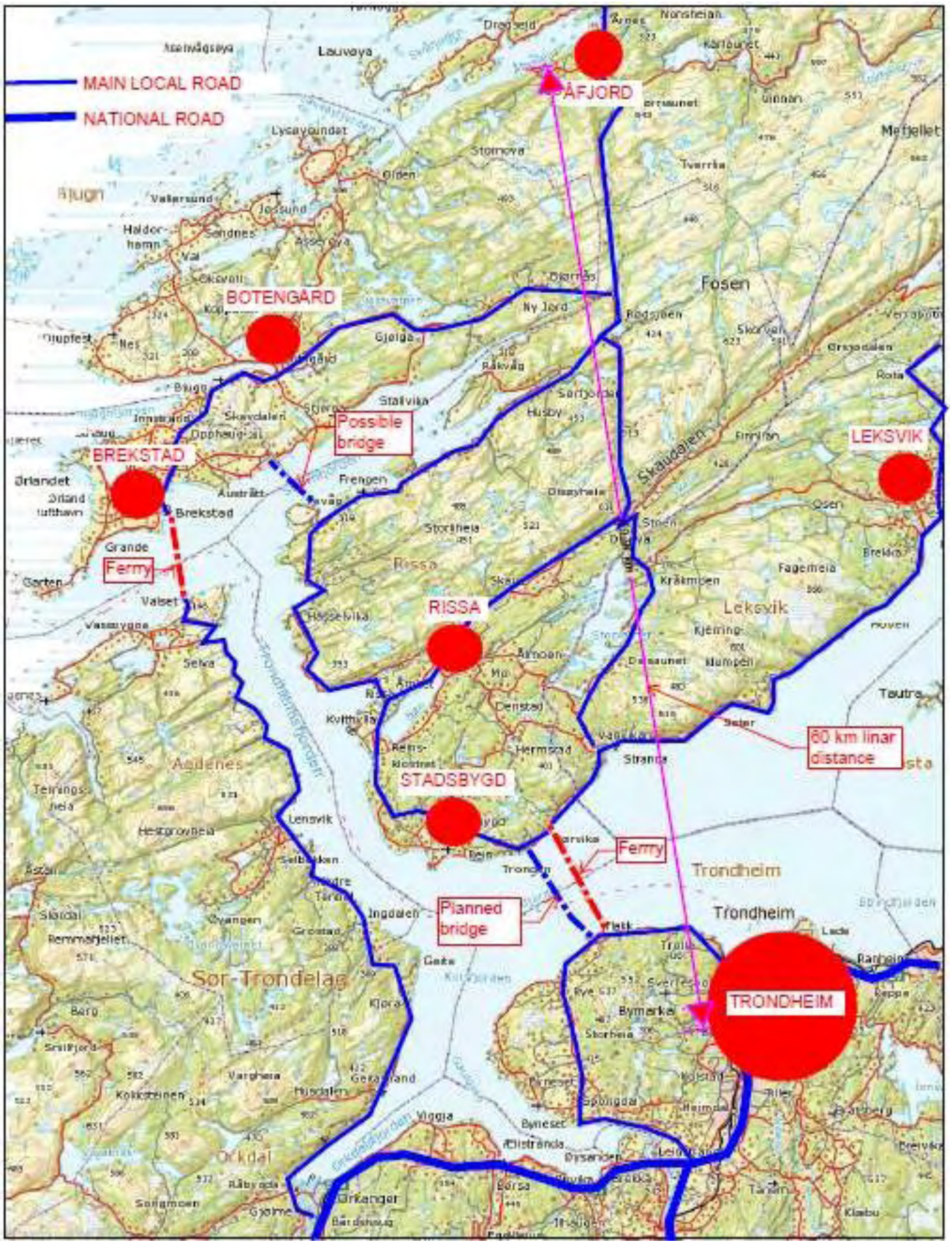
*Rough estimates are made regarding costs and benefits. Cost is estimated between 8 and 15 billion NOK. The expected value is 11.5 billion. It is envisaged that users benefit and saved ferry costs estimates will be 350 million NOK per year at current traffic rates and a possible doubling some years after completing the project. The project is likely to have positive benefits to society calculated according to current estimation methods.*

*The environmental impact of the project in the operational situation will probably be very positive. Today's traffic will spend 14 % of ferry and High Speed Vessel fuel currently used on the stretch. Traffic will redistribute to a route with shorter driving distance. That will also provide significantly reduced fuel consumption. But the bridge will probably also create new traffic as a result of lower generalized driving costs. The net environmental impact of changes in traffic volume is not calculated, but it can be both positive and negative.*

## GEOGRAPHY AND POPULATION

The Trondheimsfjord is today crossed by ferry between Rørvik and Flakk. This is the fifth largest ferry connection in Norway. It is used mainly by travelers between Fosen and the

Trondheim area, but is also the main connection between Fosen and destinations south of Trondheim.




*Figur 1 Roads and towns on Fosen.*

About 25 000 people live in Fosen while about 250 000 live in the Trondheim area. The nearest part of Fosen is located about 20 km from central Trondheim with 7.5 km of that distance being covered by ferry. Other villages or towns are located at distances of 40-100 km using the existing roads.

If the ferry were to be replaced by a bridge, one could achieve a journey time of less than one hour between all the larger towns on Fosen and the city of Trondheim. It will provide a time gain of ½ hour during the daytime and somewhat more in the evenings and at night time.

Figure 1 shows the towns of Fosen, the most important main roads and the two ferry routes that cross the fjord. Linear distance between Åfjorden and Trondheim is about 60 km. The current road distance to Brekstad (where Norway's main military air base is located) is about 100 km, including the ferry distance. This can be shortened to about 65 km, but it requires an additional bridge crossing at Stjørfjord. The latter can probably be built with relatively traditional technology, e.g. suspension bridge with a main span of approximately 1400 meters and total length of about 2800 meters.

## TUNNEL CROSSING UNDER FJORD IS HARDLY FEASIBLE

Trondheim fjord is wider than most fjords of western Norway. In the area of shortest distance between Fosen and Trondheim the fjord varies from 6.4 to 20 km wide. Where the fjord is narrow, it is about 500 meters deep with a further 600 meters to reach bedrock. This means that a tunnel under the fjord is not a question of where it is narrowest.

The possibility of a tunnel in an area north-east of Trondheim was investigated by SINTEF some years ago. East of Trondheim the rock level is higher. In principle it could be possible to find a tunnel solution, but the tube will be about 40 km long. The depth will be more than 600 meters. Figure 2 shows the depth to bedrock in different parts of the fjord

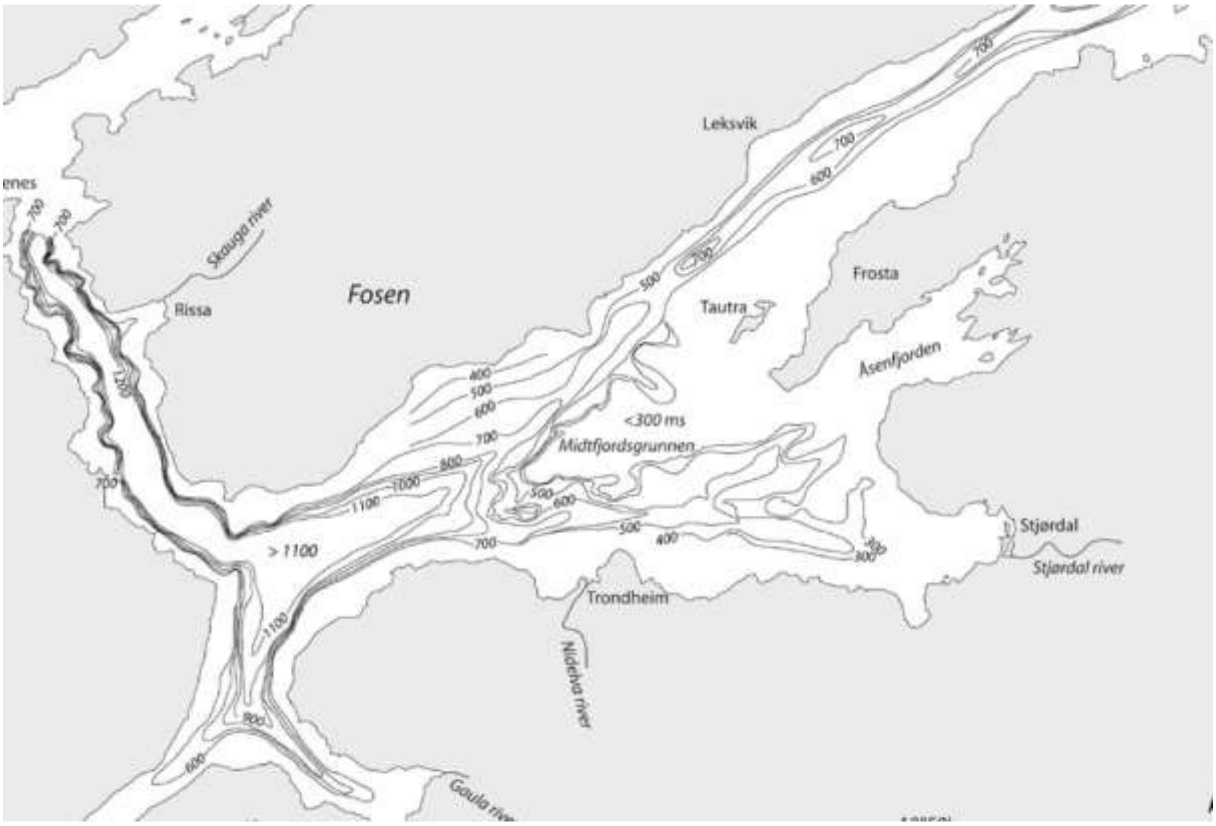



*Figure 2 Depth to bedrock in the Trondheimsfjord. SINTEF, ref. 1.*

It is perhaps technically possible to build a tunnel at depths greater than 600 meters, but it has not been done before. The deepest sections will need to be located relatively close to land on both sides. This means that the requirement for maximum gradient of 5%, will result in up to 10 km extra length on each side to raise height differences. Total distance between Trondheim and the traffic junction north of the fjord (Vanvikan) will be about 50 km compared with 25 km today, ferry distance inclusive. Travel time gains from such a solution would be modest. This is the main reason why we have focused our attention on a bridge where the fjord is narrowest and the travel distance on main routes is the shortest.

## BRIDGE ADAPTED TO SHIP REQUIREMENTS

The requirements of the largest ships that sail along the coast of Norway and visit Trondheim also need to be accommodated. Sailing heights to be accommodated are still determined by the structures built in the fjord (offshore yard in Verdal). These require a height of 90 meters. Larger structures could be built, but have not yet been made. 90 Meter clear height is spacious and will allow all shipping. 70 Meters is a normal maximum. Ref. 2

The fairway should normally have a width of at least 200 meters. In a fjord with significant traffic where towing is also part of the traffic, it is desirable to have a wider fairway.

There are few vessels in use on the Norwegian coast with a draft deeper than 12-14 meters. Some offshore rigs may have a draft of 16-17 meters. The largest cargo ships in international traffic may have a draft of up to about 20 meters, but this is not relevant to the Trondheim Fjord. The large scale oil tankers with a draft of more than 20 meters will also not be catered for.

Basically, it is assumed that a 90 meter free sailing height, a width of at least 200 meters combined with a depth of at least 12 meters will be sufficient to meet all ordinary ships requirements. Desired depth of 16-20 meters will be met if possible.

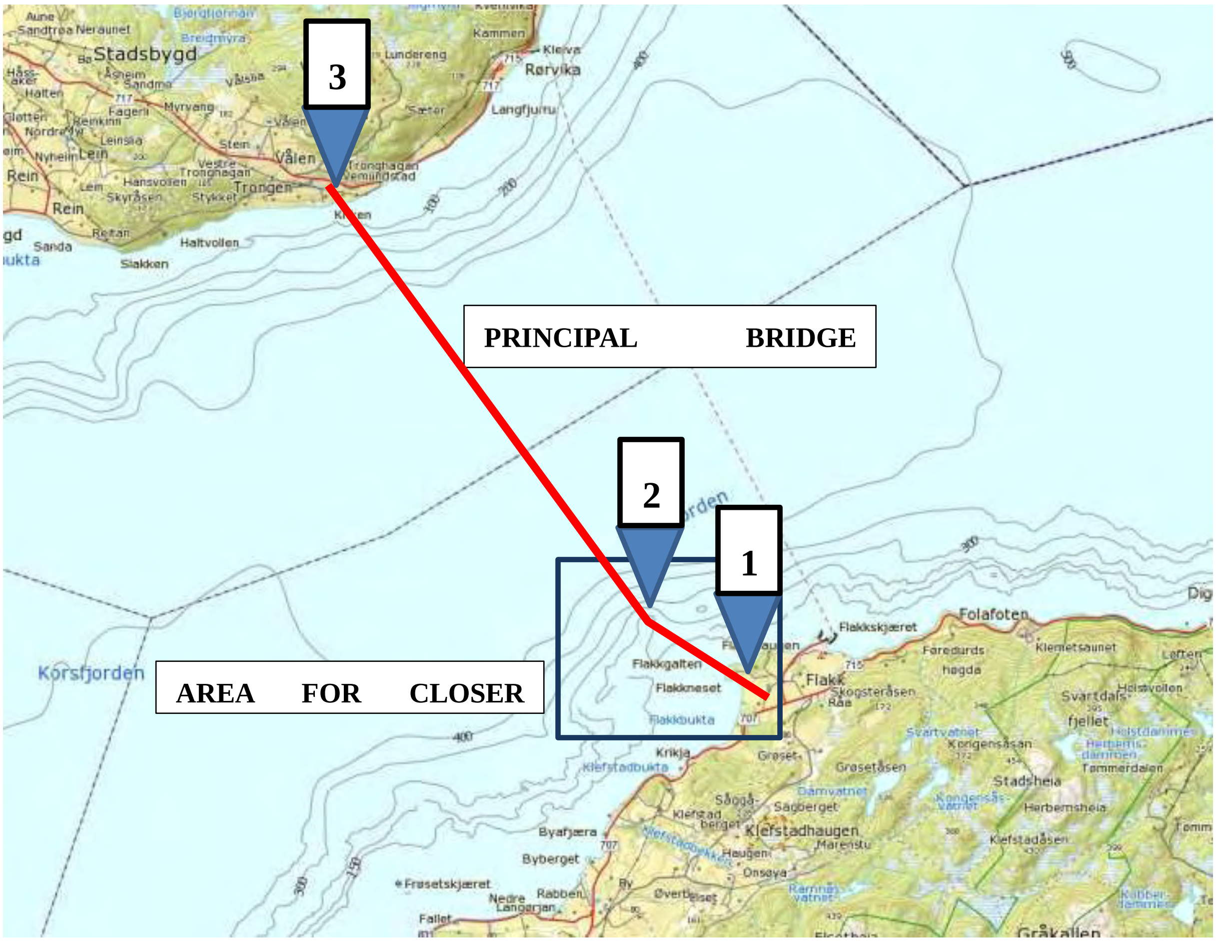
## WIND AND WATER CURRENT

The nearest meteorological stations are at Orland and Trondheim. There is reason to believe that relevant wind strength will be covered by the results of these monitoring stations. During the last 10 years Ørland has had a maximum recorded wind speeds of 29 m/s. The maximum observed in Trondheim during the same period is 14 m/s. These winds are well below the values that must be followed in the design of structures.

Strong winds can cause a bridge to be closed because cars can be swept onto the guardrail. This is also an issue on mountain passes in Norway, especially during winter on slippery roads. Ørland has wind speeds of above 20 m/sec for 0.1% of the time. Beyond this speed, the bridge might need to be closed. This indicates that a bridge will rarely need to be closed due to strong wind.

There are significant differences in tidal water in the fjord. There are relatively strong water currents occurring twice a day. With the outgoing tide, the current moves westward along the Fosen coast. The direction is the opposite with the incoming tide. On the Flakk (south) side, the pattern is more complicated.

## AS SHORT A BRIDGE AS POSSIBLE




*Figur 3 Principal bridge location*

At its narrowest, the fjord has a width of about 6.5 km. It is considered possible to place ordinary space built structures at depths of up to about 100 meters. This will normally be based on the foundation of rock. Optimistically considered, this still leaves 5.1 km between the possible permanent structures. As the longest practical length of suspension is about 2 km, only floating structures of some form are therefore conceivable. Another option would be to build floating towers which are lowered onto the seabed as has been done in the North Sea. But the largest construction of this kind made to date is the Troll platform which is sited on the depth of 302 meters and is 472 meters high in total. The depth of the Trondheimsfjord is 500 meters.

Based on the analyses that the Norwegian Public Roads Administration have made in connection with plans for E39 (without ferries) (ref.3), we have concluded that a "traditional" floating bridge with a length of approx. 5.7 km is the most appropriate and interesting solution. This would be constructed between the support on an underwater reef (marked 2) and land on the north side (marked 3) as shown in Figure 3. Point 2 is on an underwater mountain reef with a length of several hundred meters and depth of about 55 meters.

Between land on the south side of the fjord and the underwater reef, there is 1.1 to 1.3 km with a maximum depth of less than 97 meters. In this section it is possible to construct several types of structures. A cable stayed bridge with one tower on Flakkskjæret (point 1, rock above sea level) and span of 700 meters to either side, could be a solution. A cable stayed bridge with two towers could be another solution. A tunnel solution in one form or another is also conceivable.

## FLOATING BRIDGE

A pontoon bridge with a length of 5.7 kilometers or more has not previously been constructed. The longest in Norway has a floating portion of 1246 meters. Both ends are anchored in rock onshore, but without additional anchors. The arc of the floating bridge deals with the horizontal forces. The longest in the world is more than 2 km long, with side anchors in shallow water.

It is believed to be necessary to use side anchoring in one or other form. Basically, we assume it will be pontoons at 150-200 meter intervals, with every second pontoon being anchored to the bottom on both sides, ie 300-400 meters between each anchorage.

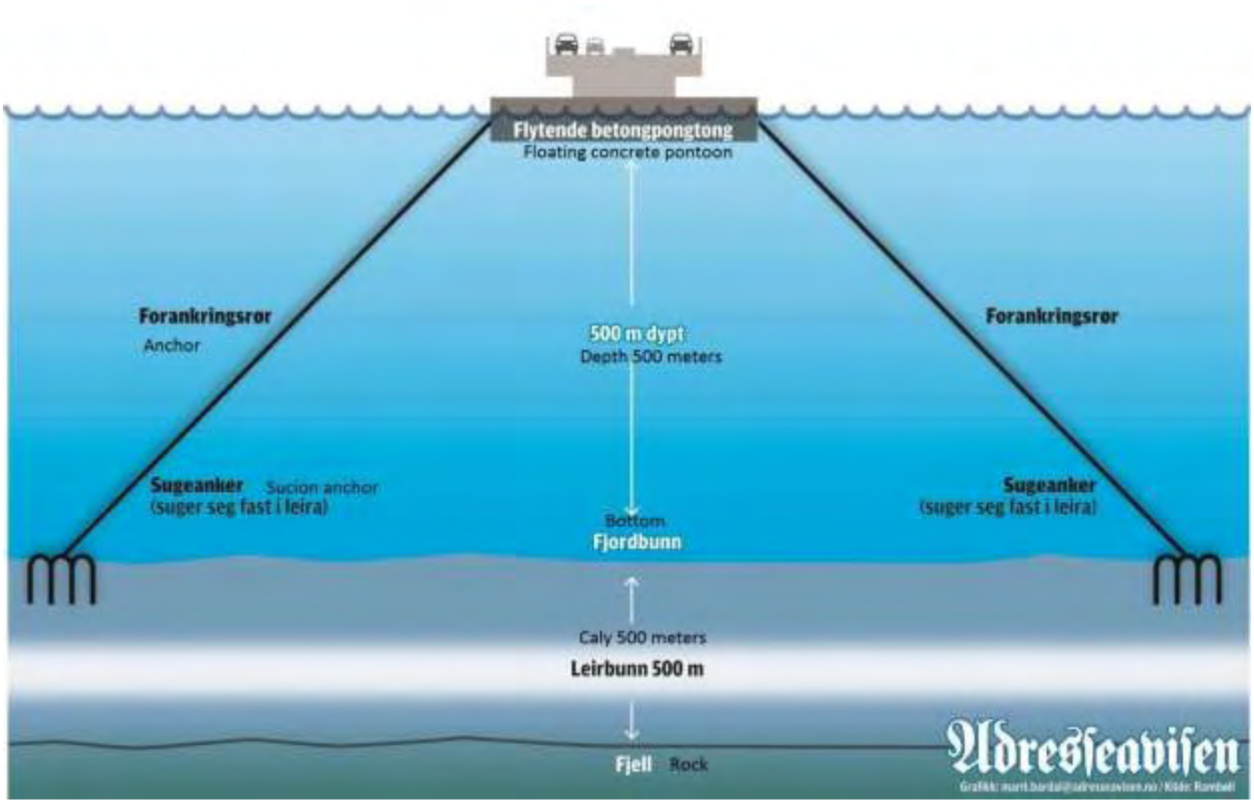



*Figure 4 Principle of side anchored pontoon*

Figure 4 shows the anchor solution in principle. The demonstration of suction anchors is demonstrated. This is an anchor solution which is widely used with platforms in the North Sea. The bottom of the Trondheim fjord is supposed to be well suited for the use of this type of anchor. Each tether is about 700 meters long. They are made with the same density as water so there will be no deflection. There are several solutions to address this problem.Ref.3.

**ORDINARY BRIDGE**

There are in principle many solutions for a solid bridge between the southern bank and the shallow rock at – 55 meter, about 1000 meters off the south bank of the fjord. In principle it is also possible to imagine tunnel solutions, a mountain tunnel, submerged tunnel or combinations of the above. Figure 5 illustrate the assumed most appropriate solution.

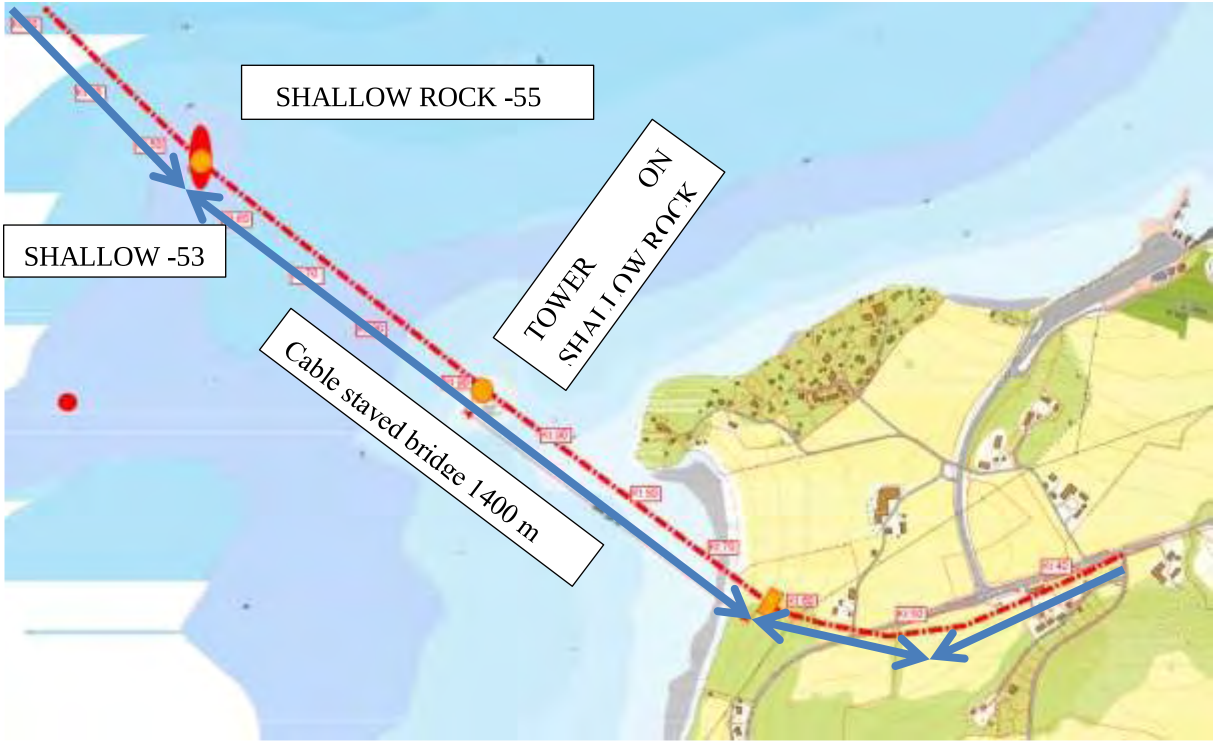



*Figure 5 Road alignment close to south bank.*

So far it is assumed the bridge is the most appropriate. The maximum depth is 97 meters in the area between the shallow reef and the bank. The shallow reef is approximately 800 meters long, running perpendicular to the direction of the bridge. It has at least two distinct peaks, both at about contour line -55. The north peak, which has the best location in terms of the shortest total bridge length, after investigation, was found to have rock on the top with only small amounts of soil above.

About 300 meters from the shore is shallow rock that rises a few meters above the top water level. It is suitable for the placement of a bridge tower, e.g. for a 1400 meters long cable stayed bridge. But it is also possible to imagine several other types of bridge solutions.

It is possible to establish a fairway with clear height of 90 meters and width of 200 meters. It is natural to have the highest point between land and reef closest to land. The water depth is limited to about 10 meters, but it has little practical significance for current use. On the other side of the tower there will be a 600 meter wide fairway with height varying from 60 to 90 meters. It should be comfortable for all current ships.

## INVESTMENT COSTS

A detailed cost estimates is not done, just an estimate based on the unit costs indicated by E39-assessments, ref 3. It is based on the following assumed unit costs:

- Cable stayed bridge, 2.2 million NOK per meter
- Floating bridge, 1,5 million NOK per meter
- Viaduct bridge, 0,4 million NOK per meter
- Roads, 0.1 million per meter

Based on these assumptions, the total cost of the fjord crossing project will be approximately 11.5 billion NOK. This is an estimate that would normally have an uncertainty of + - 40%.
In addition to the investment costs are annual operating and maintenance costs for the bridge. These have not yet been calculated.

Road users will experience a higher car costs. Since the bridge will remain at approximately the same location as the current ferry route, the increased distance per car is about 7.5 km. This gives an additional annual car cost of NOK 19 million for current traffic and 27 million at 40% traffic growth.

For society, it is possible that other indirect costs will appear. There may be longer sailing routes, aesthetics problems, environmental conflicts, conflicts with other interests etc. We do not anticipate major problems with the above, in this particular case, but they have not yet been analyzed.

## BENEFIT FOR SOCIETY

Such projects tend to introduce many beneficial elements to the communities. Some of these are included in the calculation methods used in Norway. For other elements, there is a discussion about how they should be specified. It is, however, recognized that previous methods used have underestimated the benefits of improved transport solutions. Ref.4.

The benefits of replacing the ferry with a bridge contain the following major components:

1. No operating and investment costs for ferries
2. The value of reduced travel time costs for ferry passengers.
3. The benefit of increased traffic
4. Transport opportunity available all hours of the year.

5. Regional economic effects, increased value of the resources in the affected area.
6. Reduced environmental costs, pollution to the air due to reduced energy consumption.

Point 1 can be easily calculated. It represents 85 million NOK annually at current traffic level. The amount will change proportionally with traffic.

To calculate point 2, there are established Norwegian computational methods. These are presently under revision. The "old" methods has been used. Time gain per road user can be set at 32 minutes plus hidden latency. Average hourly cost of road users can be set at 155 NOK. Today motorists will annually save time worth 200 million.

Other new items require a more thorough discussion.

## INCREASED TRAFFIC

In those cases where fast road link replaces a ferry, there will be an immediate increase in traffic. If toll costs are about the same as the ferry ticket, it is normal to expect a traffic growth of about 40%. This is often interpreted as the newly created traffic, i.e. traveling as was previously not carried by other routes or to other destinations. This interpretation is wrong. To use this simple assumption will considerably reduce the calculated economic benefit of the new bridge Flakk – Rørvik.

A new road solution providing shorter travel time within a network of roads, will in principle attract these types of "new" traffic:

1. Traveling previously carried by other routes. This effect can be great in a network, and small when alternative routes has not easily accessible. The result is usually reduced traffic as a total.
2. Traveling that was previously done by other modes of transport. This effect is generally small where car and bus have little competition from other modes.
3. Travel that previously had other destinations. This effect might be large. This can both increase and decrease traffic as a total. New destinations can be more attractive than the old. That increases total traffic. But they may also have the advantage of reduced distance. This reduces traffic.
4. New travel activity occurs as a result of reduced travel costs on the stretch. The result is net increased traffic.

The ferry crossing Flakk - Rørvik is competing with at least three other travelling routes between Fosen and Trondheim. Some use the ferry Brekstad - Ørland and car via Orkanger, others use the car via Skarnsundet bridge/Innherred and finally a high speed vessel Trondheim – Vanvikan, combined with bus or car on Fosen.

The first two options do have about the same travel time as the use of car via ferry Flakk-Rørvik. They will therefore receive the same benefits from the new bridge as current users of the ferry.

240 000 ferry travelers per year changing to the bridge have been estimated. Those traveling via Skarnsundet today will also have a reduced traveling distance by car. Reduced ferry cost Brekstad - Ørland due to traffic reduction is hardly expected and therefore not included in the calculations. Community benefit is estimated at 32 million NOK per year, considering all current additional road traffic between Fosen and Trondheim.

Current travellers using the fast speed vessel Trondheim - Vanvikan can use a bus across the bridge. They will save travel time compared with today’s situation, but not as much as the current ferry passengers. Current traffic is 200,000 passengers per year. This provides hourly frequency on busses with 18 departures per day. This is better than the current boat frequency. Net benefit by saving travel time for passengers, saved speedboat costs and increased service costs for bus provide a net benefit to society of about 28 million NOK per year.

## NEW DESTINATIONS AND NEW TRAFFIC

The bridge will make it more attractive for people from Trondheim region to travel to Fosen, as commuters or for recreational purposes. This will partly be travel replacing travel elsewhere with similar opportunities. It may in sum create more traffic, but also possibly less.

Similarly, people from Fosen will increase traveling to Trondheim, particularly as commuters and users of the opportunities in Trondheim, within commerce, culture and social networks. This will most likely create more traffic, but that is also the purpose! People on Fosen get easier access to activities that add value for them and thus to society.

It is extremely difficult to distinguish between real newly created traffic from traffic that would otherwise have gone to different, but less valuable destinations. Considering the new created traffic, it is reasonable to set the value of society benefit per journey to half the value of existing traffic journeys. New transport users appreciate improved transport opportunity less than previous users. The consequence is less society benefit when making calculations for new users.

Possible dimension of the traffic increase Trondheim - Fosen after establishment of the bridge might be estimated by comparing areas with the same road distance to Trondheim, but with shorter journey times. Table 1 compares the ratios of passenger travel between sites along the E6 south of Trondheim with that of Fosen.

*Table 1 Travle frequencies between Trondheim and surrounding region. Journys/day.*

| Area | Distanse from Trondheim city | Number of inhabitants 2009 | Car | Bus, train or boat | SUM |
|---|---|---|---|---|---|
| Melhus | 15 - 50 km | 14500 | 1,12 | 0,12 | 1,24 |
| Midtre Gauldal | 50 - 80 km | 5900 | 0,40 | 0,06 | 0,45 |
| Fosen | 20 - 100(70) km | 25000 | 0,18 | 0,04 | 0,22 |

Fosen cannot expect to have the same travel frequency as Melhus. Most of the buildings in Melhus are located 20-30 km from the city of Trondheim. But with the bridge established, it is least could expectable to have a similar travel frequency as between Trondheim and Midtre Gauldal.

The calculation of the effect of the first year is based on the assumption that the travel rate key figure will increase by a quarter of the difference between Fosen and Middle Gauldal. This provides new traffic of about 460,000 passenger journeys. Some of this is really new, and some results from changing destinations. Benefit to society per journey is set at half of the estimated benefit of current ferry users. Calculated societal benefits are 28 million NOK for the first year.

## CALCULATED SOCIAL BENEFIT FROM BRIDGE FLAKK - RØRVIK

Calculated benefit to society for the new bridge is calculated to be 351 million in the first year. It is the sum of saved by reduced ferry and high speed ferry costs by 110 million as well as time benefit to road users of 267 million. The sum of these benefit figures is reduced for car expenses increasing by 16 million and bus service costs increasing by 10 million.

Today's passengers number on the ferry about 1.6 million per year. Along with traffic being transferred from other routes, this will provide bridge traffic of 2.1 million passenger journeys in the first year. In addition the bridge will receive approximately 0.5 million passenger journeys from other destinations and created new traffic. This calculated estimate which will be studied in detail at a later stage. The estimate corresponds to an average daily traffic of 4,400 vehicles.

It is unclear how this figure will evolve over time. The building of the bridge is likely to facilitate greater population growth in Fosen than the average for the Trondheim area as a total. In the last 20 years, all the neighborhoods of Trondheim, except Fosen, have enjoyed strong growth. A bridge will turn this picture around. It would not be unrealistic to expect that growth that doubles traffic would add benefit to society. Annual benefit of 700 million NOK per year within a period of 15-20 years might be realistic. Residents and businesses will benefit from the spacious areas that will be available on Fosen. It will reduce development pressures southward and eastward of Trondheim. A lot of people will regarded this as a significant advantage.

## OTHER BENEFITS TO SOCIETY

The value of continuously available transportation all day and night, is controversial. It is included in the current numerical calculation methods by calculating the effect of hidden latency. Hidden latency contributes only with small figures in the calculation. In the same category is the factor reliability. At the traffic peaks you have the risk of lack of ferry capacity. The ferry may also have problems in bad weather conditions. Technical failure occurs with reduced capacity and frequency as a result. Estimation of these effects have not been attempted.

Better Traffic access to a geographic area will normally increase the value in use of the resources available in the area. The main resource is the existing labor and the infrastructure that serves labor. Residents in Fosen will become more effective participants in the relatively large labor market in the Trondheim area. This will increase specialization, innovation and thus power the value of labor in the whole area. Property values in Fosen will increase while they may be reduced in Trondheim since land supply will increase sharply.

Ferry and speedboat are using much fuel per passenger travelling a kilometer compared to car or bus for the same distance. In average situations the ratio is one to ten. This is also important for air pollution. But in normal situations the sea distance is the shortest, so when boat is replaced by car, the total change in fuel consumption is not that high.

The current ferry and high-speed traffic that will cease, has a fuel consumption of approximately 4100 tons per year. Increased car and bus traffic due to bridge will, consume approximately 560 tons per year. Annual fuel consumption is therefore reduced by about 3500 tons. This corresponds to about 11 000 tons of $CO_2$. In relation to the global environment, the bridge is a very good project considering operation costs and $CO_2$.

## PRESENT VALUE OF BENEFIT TO SOCIETY AND RECOMMENDATION.

The economic calculations in Norway have until recent years used a 4.5% interest rate and a 40 year economic lifespan for roads by discounting future costs and benefits to present value. It is proposed that one reduce the interest rate to 4%. Other countries use 3.5% or lower in the corresponding calculations.

A bridge over the Trondheimfjord will have a technical lifetime of 100 years. One should certainly consider how the combination of interest rate, lifespan and period of calculation affect the present value of the calculated benefit. Table 2 shows the present value of 350 million per year using 15, 25, 40 and 100 year lifespans and calculation periods with different discount rates. Table 3 shows the same for 700 million per year.

*Table 2 Present value for 350 mill/year. Bill. NOK*

| Economic lifespan | Interest | | | |
|---|---|---|---|---|
| | 3,5 % | 4,0 % | 4,5 % | 7,0 % |
| 15 | 4,0 | 3,9 | 3,8 | 3,2 |
| 25 | 5,8 | 5,5 | 5,2 | 4,1 |
| 40 | 7,5 | 6,9 | 6,4 | 4,7 |
| 100 | 9,7 | 8,6 | 7,7 | 5,0 |

*Table 3 Present value for700 mill/år. Bill. NOK*

| Economic lifespan | Interest | | | |
|---|---|---|---|---|
| | 3,5 % | 4,0 % | 4,5 % | 7,0 % |
| 15 | 8,1 | 7,8 | 7,5 | 6,4 |
| 25 | 11,5 | 10,9 | 10,4 | 8,2 |
| 40 | 14,9 | 13,9 | 12,9 | 9,3 |
| 100 | 19,4 | 17,2 | 15,4 | 10,0 |

With an annual benefit to society of 350 million NOK per year, the project will not be of financial benefit in relation to calculated investment cost of 11.5 billion NOK in any of the demonstrated interest / lifespan combinations. With annual benefit of 700 million, the project will be economically viable with a lifespan of 40 years or more and the discount rate of 4.5% or lower.

Operation and maintenance costs of the bridge are not included in the calculations.. It also lacks other cost components, but they are probably relatively modest. On the other hand, the value of a continuously available transportation facility is not included. Other benefits may also occur as discussed.

It is not unlikely that the project is economically viable. Most Norwegian transport projects calculated in the same way are considerably less profitable than this project. The majority of projects having received funding from the government in the last 10-20 years may be in that category. They are implemented despite the lack of profitability calculated.

New computational methods are likely being introduced. They will give better profitability results than that of the old calculation method.

There might be a strong economic reason to bring the project forward as quickly as possible.

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# COMBINED FLOATING BRIDGES AND SUBMERGED TUNNEL

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## ABSTRACT

*Norway is a country with many wide and deep fjords along its west coast, calling for special bridge crossing solutions for replacement of the numerous ferries along its important E39 coastal highway. The most challenging one in this respect is the 3700 meter wide and 1250 meter deep Sognefjord, which therefore was chosen by The Norwegian Public Roads Administration as a test object.*

*This paper presents one of the solutions, the main purpose of which should be to make unrestricted two-lane vehicle traffic possible, with separate lanes for bicycles and pedestrians. The bridge should further allow all traffic to and from the very long inside fjord to cross it, during the summer season also the new giant-sized tourist ships. The design requirements for the bridge crossing included therefore a 400 meters wide, 20 meters deep, and 70 meters high passage for the ships.*

*The present solution is, as shown in plan- view on the attached* ***Sketch No. 1****, a combination of floating elements without any connection to the fjord bottom. From both shores it is circular floating bridges supported on pontoons and starting from shore abutments.*
*They are at their outer end connected to and supported by a monolithic structure consisting of two cylindrical concrete platforms which are rigidly interconnected through a straight submerged concrete box structure, in this design with its top deck placed at 25 meters depth for ships passage. Thus, this bridge, with its partly circular and partly straight longitudinal axis, forms a continuous structure consisting of a chain of hinged or rigidly interconnected elements, which separately all are monolithic structures.*

## DESIGN – AND OPERATIONAL CONSIDERATIONS

Whether hinged or rigidly interconnected, this bridge structure will in both cases theoretically be stable against symmetrical current and wind forces from its concave side, but that will, depending upon the type of connection between its elements, not necessarily be so against symmetrical forces from its convex side, nor against anti-metrical wind and current, which is known to occur at this bridge site like for many Norwegian fjord bridges.

Permanent horizontal anchoring has therefore been considered as necessary for the convex side of the bridge, and has also been included for the concave side. This anchoring shall then run from the submerged outer ends of both platforms, towards onshore winch stations. With an optimal angle against the bridge axis of 45 degrees, the horizontal length of the anchoring chains or steel tubes will exceed 2 kilometers. In case of chains, hanging as catenaries, it would be considerably longer, depending upon the degree of pre-tensioning . Intermittent support on chain pontoons, often used in the offshore industry, would on the other side reduce the total length and also make chain anchoring more efficient.

Proof of necessity or not of such an expensive anchoring system, and detailed design of it, requires complex calculations, involving wind forces on the superstructures, current - and wave forces on the submerged parts, and upon the elastic properties of all elements involved. The alternative to horizontal anchoring would be to increase the cross section dimensions and strength of the floating bridge super-structures.

One possible solution for the cross section would then be two separate steel boxes, interconnected by means of steel trusses. Calculations and structural details for the floating bridge structures on the whole have, however, been left out here. In principle the design will be similar to the one applied for the two existing floating bridges in Norway, which are referred to in the following under structural details.

Reverting to anchoring again, steel tubes instead of chains would be a more efficient and cheaper solution. Vertical anchoring of offshore platforms in the North Sea with such tubes has been used for at least two of them. This author knew of no reference project using them for horizontal anchoring, but considers it feasible with welded steel pipes, having about neutral buoyancy, and with suspended weights and floaters for final regulation of the vertical profile. It has to be deep enough for avoiding collision with ships which may have left or drifted off from the official sailing channel. Dropped anchors and fishing gear are other risk factors which have to be taken into consideration.

On **Sketch No. 1**, this anchoring is shown for both sides of the bridge at its final location. It would firstly be required during the final construction works there on the two platforms, including the ballasting of them down to their final depth, and subsequently during the hooking on operations for the two floating bridges, which shall be towed in full length to this bridge site.

## ROAD TRAFFIC

The road traffic shall, as shown on the vertical section of the attached **Sketch No. 2**, pass from one towards the other floating bridge deck, both at around 13 meters above sea level. This shall be achieved through circular spiral turns inside the first platform, down to the traffic deck at approximate elevation minus 32, passing along the 400 meters long submerged tube, and from there up to the deck of the second floating bridge through similar spiral turns.

With an external platform diameter of 110 meters, chosen as the maximum possible to be built in a Norwegian dry dock, as shown on **Sketch No. 4,** this leads to a 47 meter centerline radius for the inner vehicle lane, and two 360 + 90 degree turns will then be required for keeping its gradient within 6,25 %.

The two 450 cm wide vehicle lanes, having opposite traffic, are as shown on the radial section on **Sketch No.2** separated by a 50 cm partition, with a height permitting vehicles to cross over and turn around in a case of an emergency. The outer lane will have a guard rail against the fire- and condense protecting insulation.
For pedestrians a 100 cm wide sidewalk with guardrail against the vehicle traffic is placed inside the inner vehicle lane and adjacent to the fire-protection wall, which will have fire-safe emergency escape doors spaced at 250 meters. For bicycle traffic and also serving as a fire escape route, a 240 cm wide lane follows behind that wall.
An additional space of 120 cm width towards the inner circular platform wall, all around, is intended for water pipes, ventilation canals, power cables, for possible lifts, etc. It will also serve as working space for pre-tensioning, anchoring and grouting of cables in the platforms inner wall, which for the outer wall has to be done on the outside, and in both cases with anchoring at pilasters, as common for silos.
Both concrete side-walls in the vehicle traffic zone, and probably also the underside of the traffic decks will need fire-protecting insulation.

**SHIPS TRAFFIC**

Considering the size of the newest visiting tourist ships, over 350 m length, up to 50 m width, with a cruising speed at the bridge site of up to 20 knots, it was evident that the passage had to be in the middle of the fjord, and the width has on this background been increased from the initial 250 to 400 meters. The height requirement of 70 meters, or possibly more, is an important restriction for some of the alternative solutions, but obviously not for this one.

**CONSTRUCTION PROCEDURES**

The central monolithic structures, consisting of one platform and one half of the submerged tube bridge, are as shown on **Sketch No. 4** planned to be prefabricated in two dock settings in the Hanøy dry dock near Bergen. That will be with only part of the platform height cast but with the tube-halves complete. They will then be ready for towage out of the dry dock and to a sheltered area near the bridge site for mating of the two halves.
This mating will be done with help of tugboats and some temporary land anchoring, with a freeboard of about 2.5 meters for the 200 m half length of the submersible structure, sufficient to avoid wave over-washing of the decks also during the following completion works on the platforms. The jointing details will in principle be similar to the one which is being used in immersed tunnel construction.

Then follows short towage to the final bridge site where the by then 620 m long central monolith shall be anchored up before being submerged to its final draft.

The floating bridge elements can be pre-fabricated in much smaller docks, towed to a sheltered area near the bridge site, and interconnected there to full length, as demonstrated and described for the two existing Norwegian floating bridges, at Bergsøysund and Salhus (Nordhordland), and then finally towed to and hooked on to the anchored-up central monolith and to the land abutments.

## STRUCTURAL DETAILS

For the floating bridges, reference is above made to the two above-mentioned existing ones, for which pamphlets are assumed to be exhibited at the Symposium.
Both have performed well over the about 20 years of service. Investigation of the Bergsøy-sund bridge after 20 years, showed however, some corrosion on the steel superstructure, now being under repair. This experience will have to be taken into consideration for the Sognefjord floating bridges through increased height above water and a new type corrosion protection.

For the two Sognefjord floating bridges, the span length will as shown on the sketches be about 1634 m, exceeding the 1246 m long Nordhordland bridge with nearly one third, which with increased superstructure cross section is considered feasible.

The two circular transition platforms in the monolith, shown through vertical- and radial sections on **Sketch No.2**, and on top deck plan on **Sketch No 3**, will have many semblances and shared experiences with some of the offshore platforms built for the North Sea.

A major difference is, however, that these two will have both external and internal cylindrical walls, between which the road traffic passes on spiral concrete deck ramps as shown on **Sketch No.2.** Like for the 400 m long tube, the platforms will have a bottom story with compartments for ballasting and leakage control, while also being part of the structural system.

At this stage the rather complicated calculations requiered for documentation of the structural fitness and cooperation between these two cylindrical shells, one subject to external and one to internal water pressure, have only been realized in principle. At the outset, the traffic deck spiral is their only connection., but there is, as can be seen from **Sketch No.2** ample height space for additional bracings in form of slabs and walls. These will also, as mentioned under the following risk considerations improve decisively the floating ability of a platform, if and when damaged from a ships collision.

The 400 meter long concrete tube, connecting the platforms, has been given a preliminary width of 70 meters plus local adaption to the platforms. Only a minor part of this width will be occupied by traffic lanes. It is therefore dimensioned out of assumed structural requirements, without detailed strength calculations.

A bottom story with walls and compartments is considered necessary for these reasons as well as for ballasting, and with compartments for draft adjustment and leakage pumping. Regarding ballasting it should be mentioned that while it for the submerged box can be done with water only, it will for each of the platforms take some 40000 m3 of water-saturated sand, placed in their cellars and in their traffic sections under the traffic ramps, to bring the platforms down to their final vertical floating position.

## TECHNICAL SYSTEMS

Like for any floating structure, and in this case with a complex road- and ships traffic system, a number of mechanical, electrical and electronic systems have to be included for ballasting, leakage control, ventilation and de-icing, fire- and road traffic control and rescue service, ships communication, navigation signals, etc.
Such systems have not been detailed at this preliminary stage, but for the case of a serious accident like a ships collision, toll bars have been foreseen to be lowered automatically at the abutments, stopping new vehicles from entering the bridge. Clearing of existing traffic on the bridge should not take more than about 5 minutes, unless in case of severe local damage to the structures.

## RISK CONSIDERATIONS

Regarding risks, giant waves created by major mountain land-slides, a possibility in Norwegian fjords, may also occur in The Sognefjord, but there at a sufficient distance, so as not making it critical for this bridge-site.

The next type of a serious accident would be a collision between one of the platforms and a big tourist ship sailing at speed, given as up to 20 knots. Its probability should be very small, considering the passage width of 400 meters and an advanced signals system. For a detailed design, advanced studies involving maritime- and structural expertise would be called for. The following is thus only preliminary guessing.
If it still happened, it is assumed to be against one of the platforms. The platform in question plus part of the submerged bridge could together with the ship move a sufficient distance to sever the adjacent floating bridge span from its platform connection. While the local damage to that bridge span might be total, major damage to the other spans could be limited and loss of lives on them avoided.

As for the platforms, however, the leakage made in one of them from a colliding ship might at the outset after some time make it tip over and finally sink, drawing also the submerged bridge structure and the other platform down, but probably only after full evacuation from all bridge parts, and without major loss of human life. Such leakage could also happen from a quite different cause, which after the 22. July 2011terror incident in Oslo not can be neglected, a truck with a ton of ammonium nitrate exploding somewhere under the water line.

An easily feasible and efficient countermeasure against collision damage will be floating fenders around the platforms, to a depth covering the level of a ships bulbe, with collapsible elements and with the fender structure itself designed for plastic deformation up to full collapse in order to reduce the damage on both bridge and ship, while also serving as a temporary seal reducing a leakage in the outer platform wall. At this stage, however, no detailing has been done, only an indication on **Sketch No. 5.**

A different countermeasure against the consequences of an explosion as well as of a collision is also feasible, but then in form of a rather drastic technical solution, based on the air pressure created through sinking movement of the platform, under steel roof structures (domes), spanning over the 78 meter diameter central area. In order to reduce the circumferential stresses, this area would then be divided by vertical and cylindrical steel walls with diameter 39 m into inner and outer rooms, with those vertical walls horizontally braced at their top by the roof structure, at their foot horizontally from the outer wall and vertically by inclined steel columns from ballast deck level at elevation minus 40. This is shown on the attached **Sketch No. 5.**

As these roof structures also would have to be designed against normal loads like wind, snow and own weight, a detailed design becomes relatively complex. The construction costs could also only be justified by a high enough estimated risk for total loss of both platforms and their connecting submerged bridge.

Compression of the air volume between water surface and roof would then slow down sinking of the platform. In combination with buoyancy from rooms remaining air-filled during leakage water streaming into the lower traffic volumes, this could help providing sufficient time for a temporary sealing of the leakage area, using materials from the fender, and subsequently to be followed by a proper underwater concrete repair.

A completely different issue is that a ship, badly damaged after a collision with one of the platforms and with a high number of passengers, may represent a more serious risk for loss of lives than the bridge with its small number of people on it. This should therefore primarily be a concern for the Norwegian pilot service and the cruise ship companies. In this connection it may be mentioned that there will as shown on **Sketch No. 3,** be space for helicopter landing on the platform roofs.

## DESIGN ADVANTAGES- AND DISADVANTAGES

In addition to the independence of height restrictions for the ships traffic, the shore road connections will for the present design be the simplest and least costly possible. Regarding feasibility of the construction process including the marine operations, this author has, based on own experience, no doubts. Detailed cost calculations have not been made. The total concrete volume , somewhat above 200000 m3, gives an indication, while estimation of the marine operations has to be done by one of the major companies in that field of work and will also depend upon seasons and ongoing offshore activities at the time in question.
The operational costs, not including maintenance which is a different issue, may, however, for this design be on the negative side in comparison with more “static” structures.

## SOLUTION FOR HIGHER TRAFFIC INTENSITY

For the E39 traffic at Sognefjorden, more than 2 vehicle lanes plus a lane for bicyclists and pedestrians is out of question. The design presented here can, however, for other fjord crossings be adapted also for twice that traffic capacity, without doubling the total costs. Sketches have not been included here, but the principal changes for each of the main elements are as follows:

Platforms

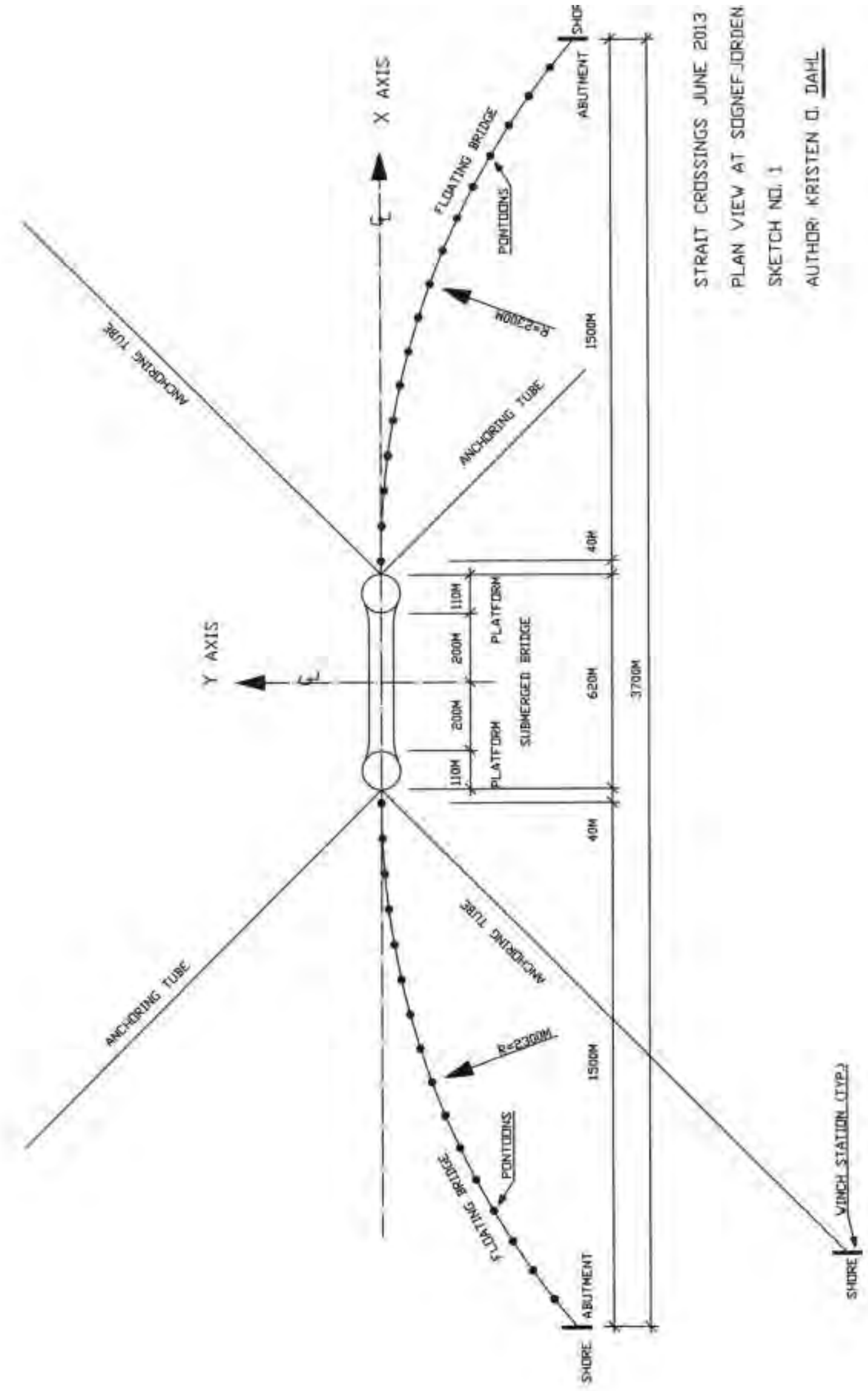
It will then be two pairs of two platforms with the same outer diameter of 110 m as before, and largely with identical design except that each pair will be cast monolithically in one dock setting and at the bridge site turned 90 degrees perpendicularly to the bridge axis. Each of these 4 platforms will cost slightly less than one of the original ones, and thus represent a certain saving for the doubled traffic solution.

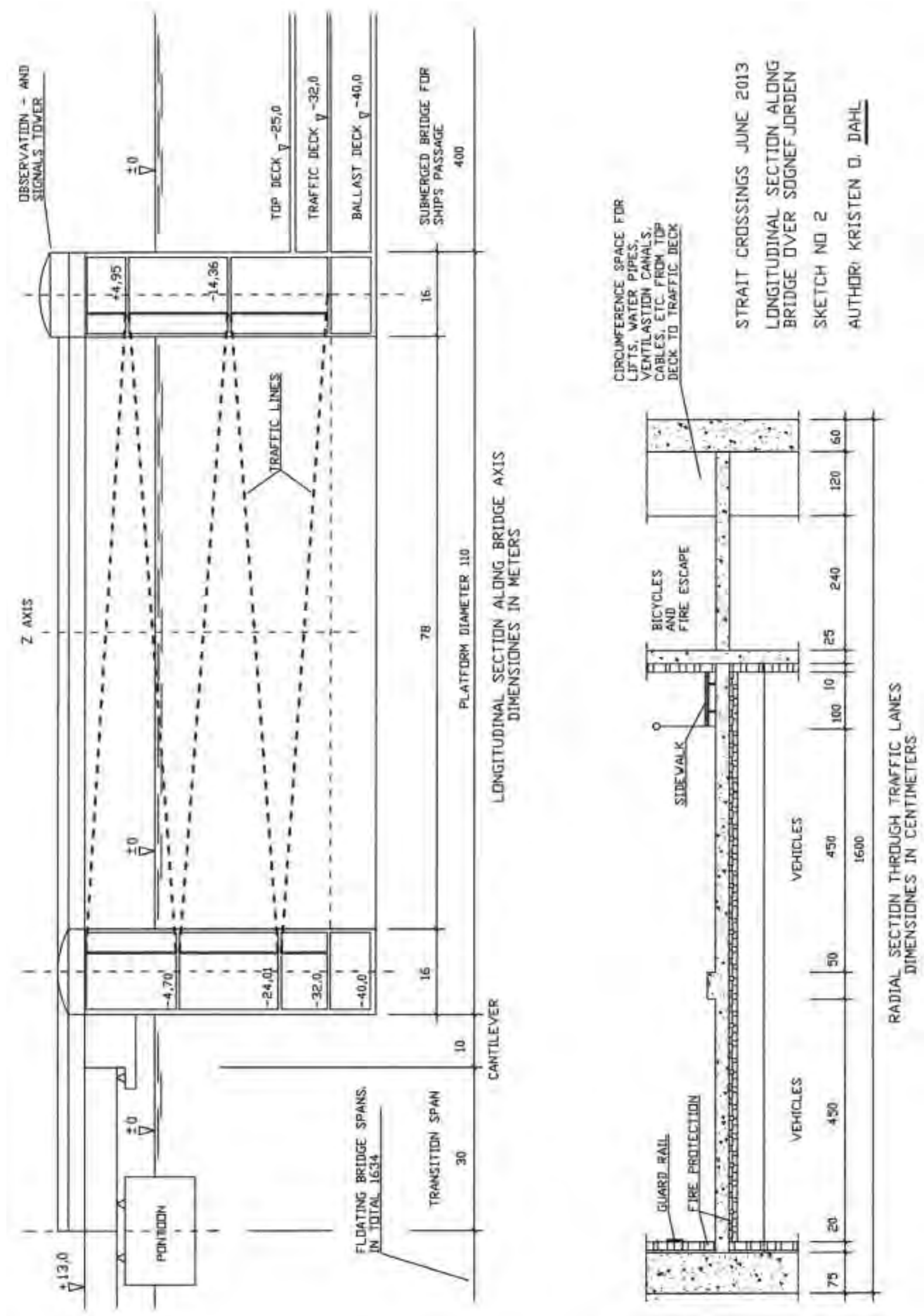
Submerged bridges

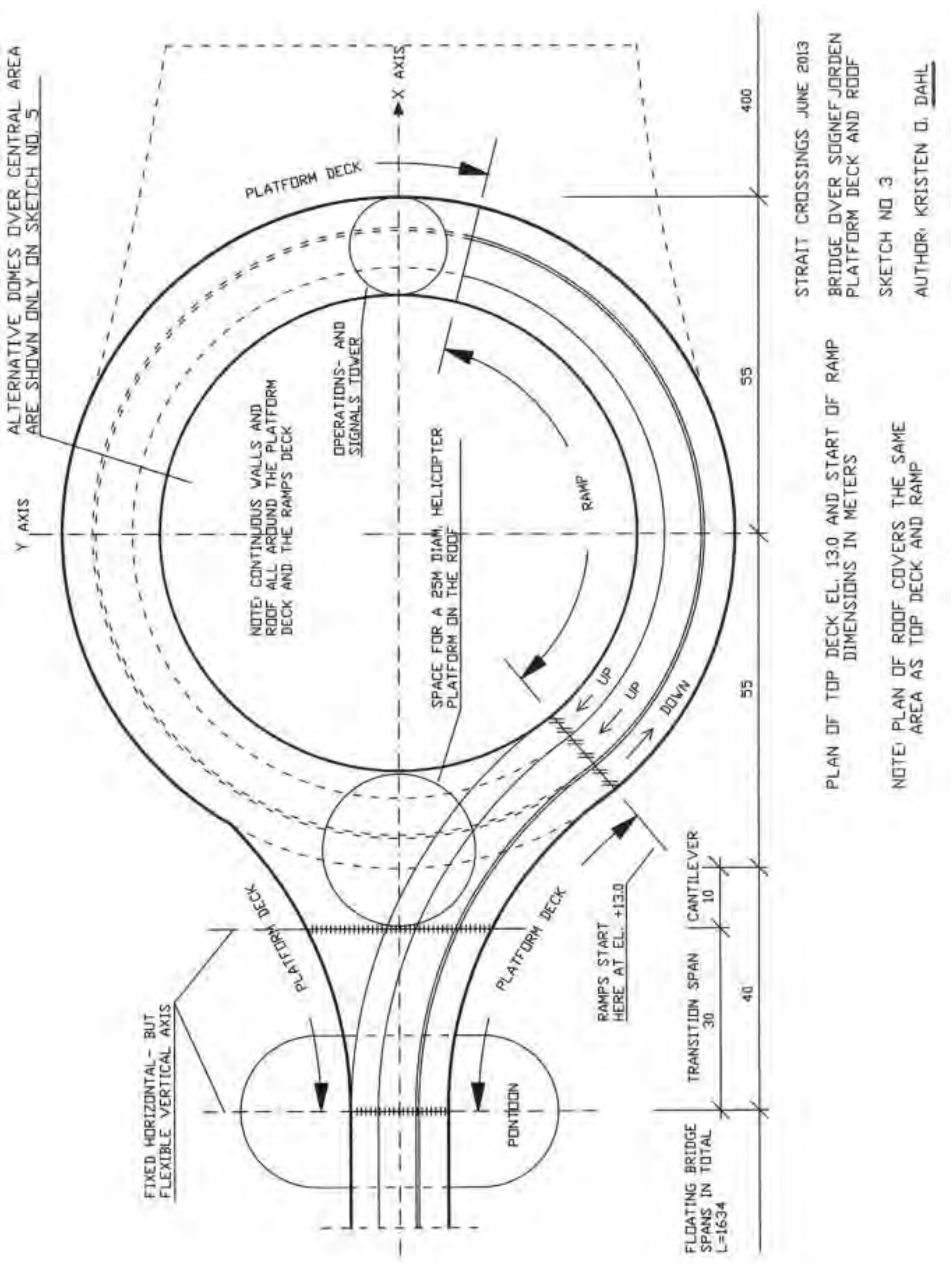
They will be combined into one, and with exception of its connection to the platforms, have the same width of 70 m as the single one before. Casting of it in the dock will take 2 settings, and mating also 2 operations, firstly between the two half lengths and then between the combined new 400 m long floating structure and the two pairs of twin platforms. With the reduction on part of concrete volumes in the submerged bridge, this will also represent a considerable saving for the doubled traffic solution.

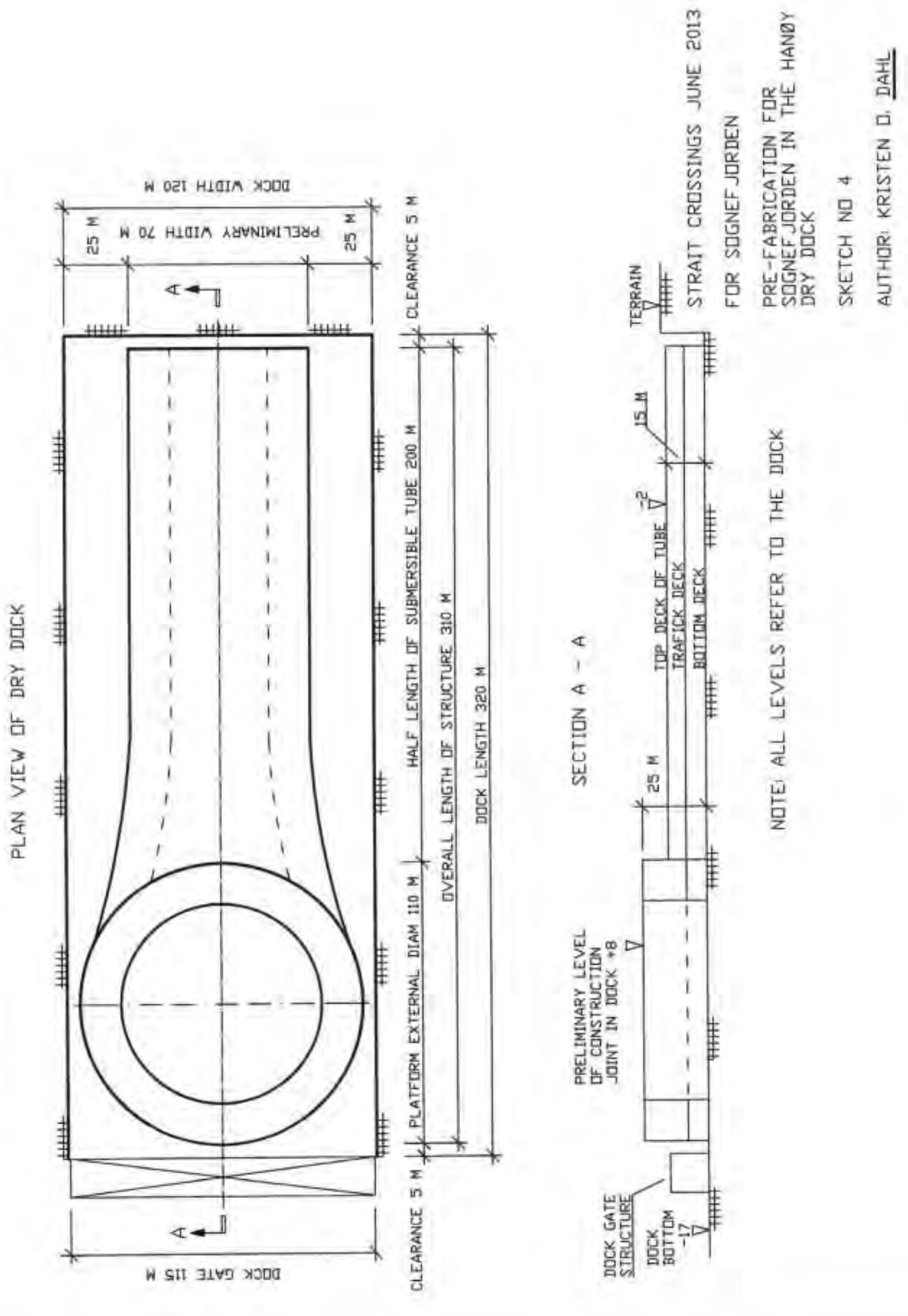
Floating bridges

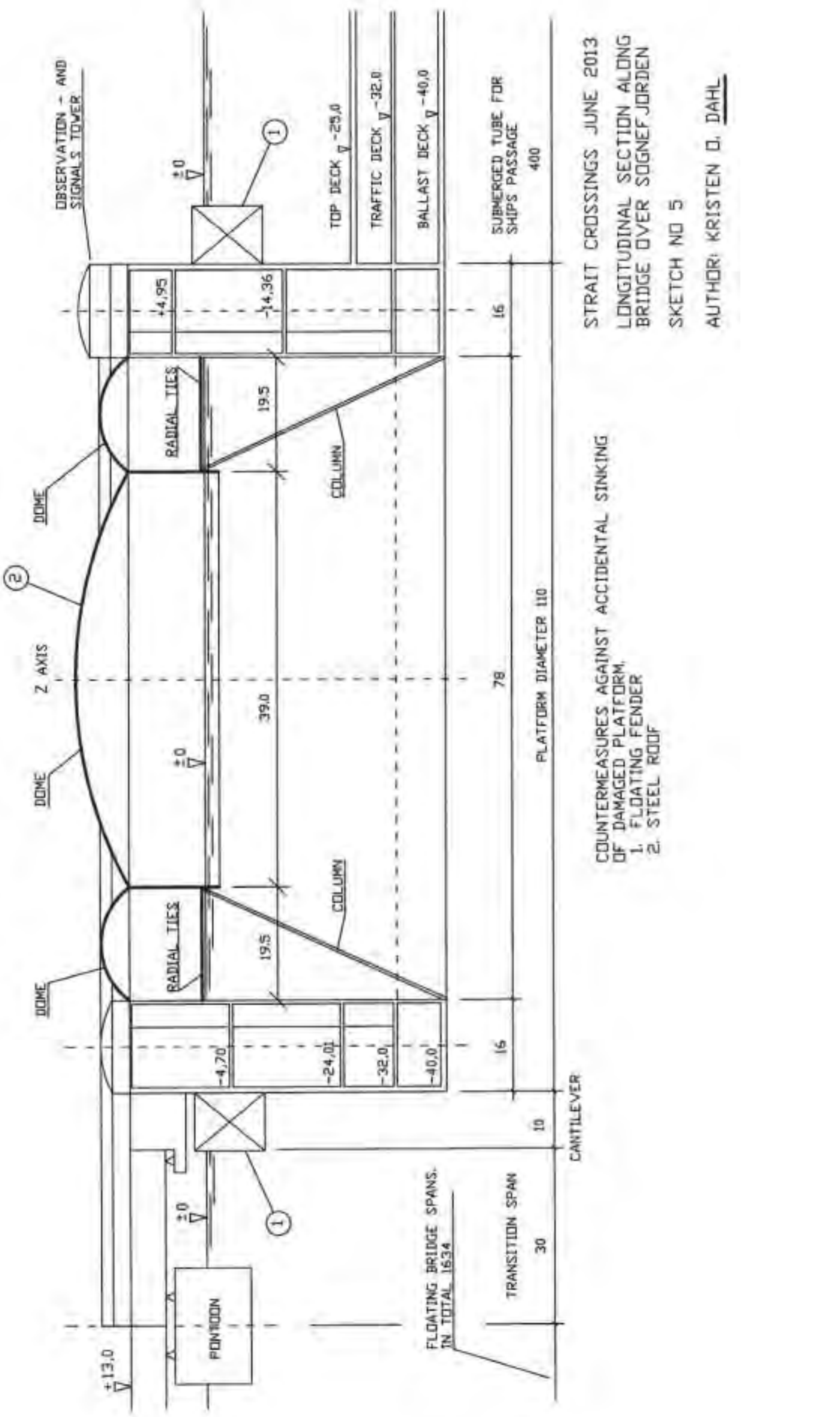
For them there will be two alternatives. Either to be combined into one monolithic box structure for all 4 vehicle lanes and 2 bicycle- and pedestrian lanes, or they can as separate cross sections for the major part of their length be combined through perpendicular cross connections to a Vierendel type structure, and in both alternatives share the same enlarged pontoons. There is then also the possibility that the increased rigidity and strength of their superstructure can eliminate the need for horizontal anchoring. On the whole the cost will be considerably less than doubled for this doubled traffic solution.


STRAIT CROSSINGS JUNE 2013
PLAN VIEW AT SOGNEFJORDEN
SKETCH NO. 1
AUTHOR: KRISTEN O. DAHL
X AXIS
Y AXIS
FLOATING BRIDGE
PONTOONS
R=2300M
ANCHORING TUBE
ABUTMENT
SHORE
1500M
40M
620M
3700M
110M
200M
PLATFORM
SUBMERGED BRIDGE
WINCH STATION (TYP.)


OBSERVATION - AND SIGNALS TOWER
±0
TOP DECK -25,0
TRAFFIC DECK -32,0
BALLAST DECK -40,0
SUBMERGED BRIDGE FOR SHIPS PASSAGE
400
+4,95
-14,36
TRAFFIC LINES
Z AXIS
16
78
16
PLATFORM DIAMETER 110
LONGITUDINAL SECTION ALONG BRIDGE AXIS
DIMENSIONES IN METERS
-4,70
-24,0
-32,0
-40,0
10
CANTILEVER
FLOATING BRIDGE SPANS. IN TOTAL 1634
TRANSITION SPAN
30
PONTOON
+13,0
CIRCUMFERENCE SPACE FOR LIFTS, WATER PIPES, VENTILASTION CANALS, CABLES, ETC. FROM TOP DECK TO TRAFFIC DECK
BICYCLES AND FIRE ESCAPE
SIDEWALK
VEHICLES
VEHICLES
GUARD RAIL
FIRE PROTECTION
60
120
240
25
10
100
450
50
450
20
75
1600
RADIAL SECTION THROUGH TRAFFIC LANES
DIMENSIONES IN CENTIMETERS
STRAIT CROSSINGS JUNE 2013
LONGITUDINAL SECTION ALONG BRIDGE OVER SOGNEFJORDEN
SKETCH NO 2
AUTHOR: KRISTEN D. DAHL


ALTERNATIVE DOMES OVER CENTRAL AREA ARE SHOWN ONLY ON SKETCH NO. 5
Y AXIS
X AXIS
PLATFORM DECK
NOTE: CONTINUOUS WALLS AND ROOF ALL AROUND THE PLATFORM DECK AND THE RAMPS DECK
OPERATIONS- AND SIGNALS TOWER
SPACE FOR A 25M DIAM. HELICOPTER PLATFORM ON THE ROOF
RAMP
UP
UP
DOWN
PLATFORM DECK
PLATFORM DECK
PONTOON
FIXED HORIZONTAL- BUT FLEXIBLE VERTICAL AXIS
RAMPS START HERE AT EL. +13.0
FLOATING BRIDGE SPANS IN TOTAL L=1634
TRANSITION SPAN 30
CANTILEVER 10
40
55
55
400
PLAN OF TOP DECK EL. 13.0 AND START OF RAMP
DIMENSIONS IN METERS
NOTE: PLAN OF ROOF COVERS THE SAME AREA AS TOP DECK AND RAMP
STRAIT CROSSINGS JUNE 2013
BRIDGE OVER SOGNEFJORDEN
PLATFORM DECK AND ROOF
SKETCH NO 3
AUTHOR: KRISTEN O. DAHL


PLAN VIEW OF DRY DOCK
DOCK WIDTH 120 M
25 M
PRELIMINARY WIDTH 70 M
25 M
CLEARANCE 5 M
HALF LENGTH OF SUBMERSIBLE TUBE 200 M
PLATFORM EXTERNAL DIAM 110 M
CLEARANCE 5 M
OVERALL LENGTH OF STRUCTURE 310 M
DOCK LENGTH 320 M
DOCK GATE 115 M
SECTION A – A
TERRAIN
15 M
-2
TOP DECK OF TUBE
TRAFICK DECK
BOTTOM DECK
25 M
PRELIMINARY LEVEL
OF CONSTRUCTION
JOINT IN DOCK +8
DOCK GATE
STRUCTURE
DOCK
BOTTOM
-17
NOTE: ALL LEVELS REFER TO THE DOCK
STRAIT CROSSINGS JUNE 2013
FOR SOGNEFJORDEN
PRE-FABRICATION FOR
SOGNEFJORDEN IN THE HANØY
DRY DOCK
SKETCH NO 4
AUTHOR: KRISTEN D. DAHL


STRAIT CROSSINGS JUNE 2013
LONGITUDINAL SECTION ALONG BRIDGE OVER SOGNEFJORDEN
SKETCH NO 5
AUTHOR: KRISTEN D. DAHL
COUNTERMEASURES AGAINST ACCIDENTAL SINKING OF DAMAGED PLATFORM.
1. FLOATING FENDER
2. STEEL ROOF
OBSERVATION - AND SIGNALS TOWER
TOP DECK -25,0
TRAFFIC DECK -32,0
BALLAST DECK -40,0
SUBMERGED TUBE FOR SHIPS PASSAGE
400
DOME
RADIAL TIES
COLUMN
Z AXIS
PLATFORM DIAMETER 110
PONTOON
FLOATING BRIDGE SPANS. IN TOTAL 1634
TRANSITION SPAN
CANTILEVER

# GLOBAL RESPONSE OF SUBMERGED FLOATING TUNNEL AGAINST UNDERWATER EXPLOSION

By

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## ABSTRACT

*Submerged floating tunnels (SFT) for transportation must be designed to secure passengers' safety in case of accidental events. Underwater explosion is the most critical event to long submerged structures. It generates shock waves propagating into the surrounding fluid to conflict impulse loading to SFTs. Extreme impulse loading makes critical damage to the body of SFT and may break it into several pieces. To design a safe SFT against underwater explosion, its behavior must be investigated and clarified. In this paper, first, shock waves and impulse pressures generated by an explosive away from the SFT are expressed by the formulas coming from the experimental results. Second, the SFT tethered by mooring lines and anchors is modeled as a simply supported uniform beam with continuous elastic springs. Finite element analysis for the beam model subjected to the impulse loading from the experimental formulas is conducted by using a commercial code so that the response of the SFT can be investigated. For design purpose, theoretical analysis is also conducted for the same model. Impulse loading along the SFT is assumed as piecewise linear, whose initial peak decays linearly. Based on this assumption, simple equations to show the response of the SFT due to underwater explosion attack were deduced. Time dependent displacements were calculated by these equations. The calculated results were compared with the previous numerical results and proved to give good agreement with them. Theoretical analysis using the simple equations yields solutions so rapidly that they can be applied for parametric study of the response of SFT against underwater explosion during initial design stage. Also guidelines for design of SFT can be proposed; thanks to these simple equations.*

## INTRODUCTION

Submerged floating tunnel(SFT) is a floating transport infrastructure which maintains the balance with exceeding buoyancy in the water by the tension of mooring system. A long-distance SFT is appropriate to a railway system in terms of safety, but to make it transport infrastructure for railway operation, safety verification under various load conditions is necessary. Load which may be working on SFT includes wave load, tidal load, seismic load and further load in emergency situation would possibly give rise to fatal damage[1]. Included in such loads in emergency situation are earthquake, collision and underwater explosion. Particularly the load by underwater explosion, among others, may cause a critical risk which could split the floating structure into two pieces as seen from the sinking of Cheonan Naval Vessel 3 years ago in Korea

[2]. Before SFT is widely used as transport infrastructure linking the lands in the coming days, design in preparation for such risk is a must. This study is intended to develop the analysis method for designing the structure in dealing with such fatal risk while carrying out the conceptual design of SFT. The shock pressure induced from under explosion on SFT was calculated based on empirical formulas and the response was analyzed based on beam theory. Analysis results were compared with the calculation using finite element analysis approach.

## UNDERWATER EXPLOSION

When explosive is burst up in the water, shock wave with high pressure spreads in the water, which is followed by spherical bubble caused by high pressure explosion. Fig. 1 shows shock wave, bubble pulse pressure and bubble growth development over the time. Initial speed of shock wave by underwater explosion spreads very fast at early stage in spherical wave form and the speed gets reduced to the level of sound speed as getting distance from the explosion point. As shock wave spreads, peak pressure is reduced while waveform is extended. Thus the pressure of shock wave in the water is represented as follows;

$$P(R,W,t) = P_m(R,W)exp\left[\frac{-(t-t_0)}{\theta(R,W)}\right] \tag{1}$$

$$P_m(R,W) = K\left(\frac{W^{1/3}}{R}\right)^{\alpha} \tag{2}$$

$$\theta(R,W) = W^{1/3}K'\left(\frac{W^{1/3}}{R}\right)^{\alpha'} \tag{3}$$

where, $P(R,W,t) =$ shock pressure
$P_m(R,W) =$ peak pressure
$R =$ distance between body and explosive
$W =$ weight of explosive
$K,\ K' =$ explosive constant
$\alpha,\ \alpha' =$ explosive index

Bubble pulse following the shock wave as shown in Fig. 1 is the pressure waveform discharged when bubble generated by gaseous product is expanded, which has lower pressure than shock wave but very slow. Bubble with high pressure and temperature at early stage pushes the water toward outside the spherical wave and stops by inertia beyond pressure equilibrium point and when water pressure around bubble exceeds the pressure inside bubble, bubble tends to contract.

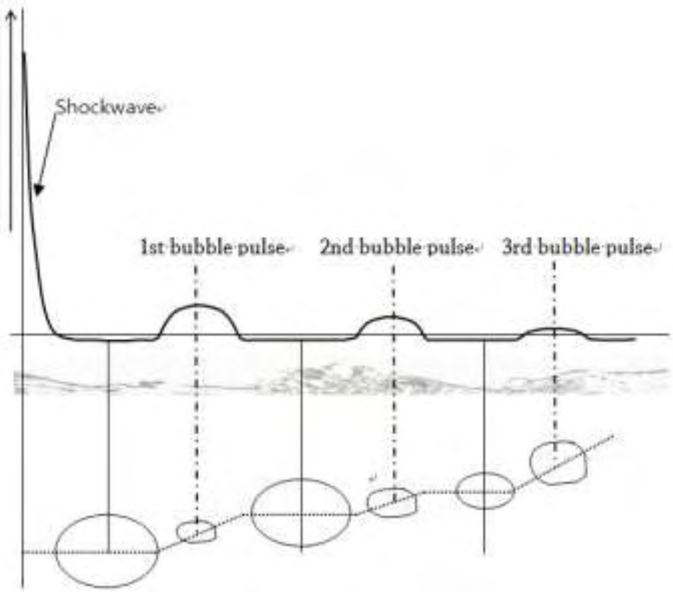


*Fig.1 Shock wave generated by underwater explosion*

**SUBMERGED FLOATING RAILWAY**

The concept of submerged floating railway for high speed rail operation is illustrated in Fig. 2. It is designed to sustain the tunnel structure which tends to rise by buoyancy with tension force of mooring system. For complete water-tightness, outer and inner shells are made of steel materials and to maintain the buoyancy at certain level the space between shells is filled with concrete. Inner shell also contributes to achieving the dual watertight system. A module is fabricated at intervals of 100m as shown in Fig. 3 and outer shell is fastened with bolts on free surface and pre-assembled before assembling completely in the water. Fixed-type structure for evacuation and ventilation is prepared at certain interval [3]. Mooring system is linked to gravity concrete block on sea-bed so as to convey the tension force to deal with exceeding buoyancy. Fixed jackets are designed to be placed at every 10km for passenger evacuation and ventilation purpose.

*Fig. 2 Submerged floating railway shaped as pipe line*

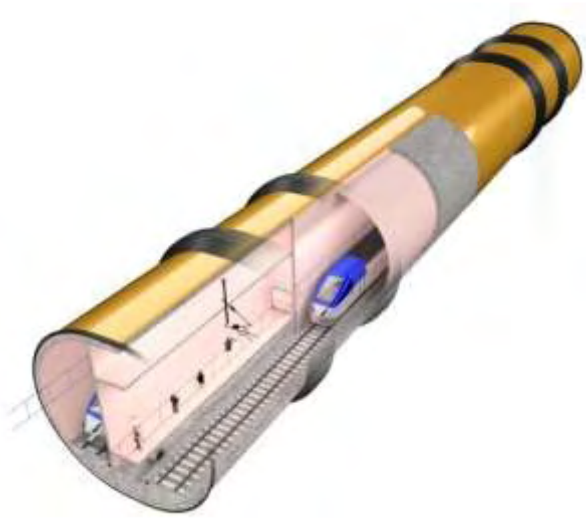

*Fig. 3 One unit of tunnel structure for submerged floating railway*

## RESPONSE ANALYSIS MODEL BY UNDERWATER EXPLOSION

When underwater explosion occurs by accident or on purpose by terrorist, submerged floating railway is fatally affected by impulse pressure of shock wave. To secure the safety in preparation for underwater explosion, structural response of submerged floating railway under shock pressure was analyzed in theoretical method.

The body of submerged floating railway is in the form that the tunnel on uniform section supported by mooring line is continuously connected between fixed jackets. Since the body has uniform section and the interval of fixed jacket is longer than the width of the body, the body can be idealized to beam, mooring line to elastic support and fixed jackets to supporting boundary. In case of underwater explosion under the body as shown in Fig. 4, behavior of the body is identified with response of a simply supported beam with elastic springs subjected to impulse pressure.

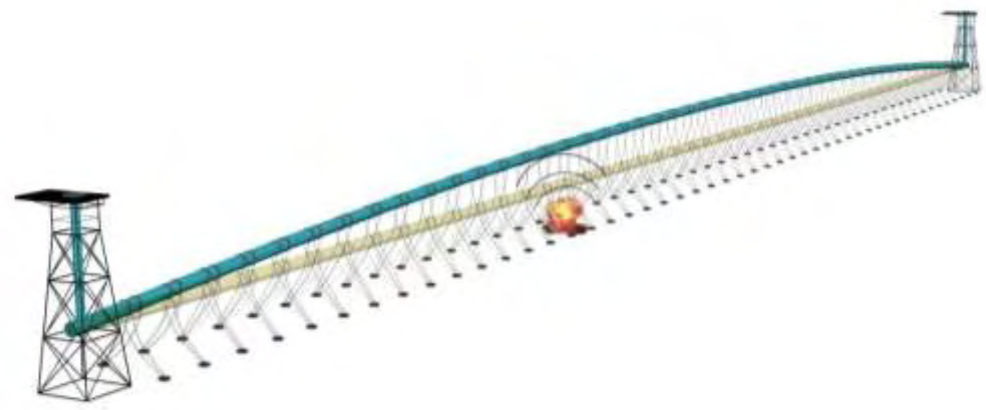

*Fig. 4 The body of submerged floating railway attacked by underwater explosion*

Shock pressure by underwater explosion, as indicated in Fig. 1 and Equation (1), varies depending on exponential function. To simplify the analysis, however, it may be idealized into piecewise linear functions as indicated in Fig. 5.

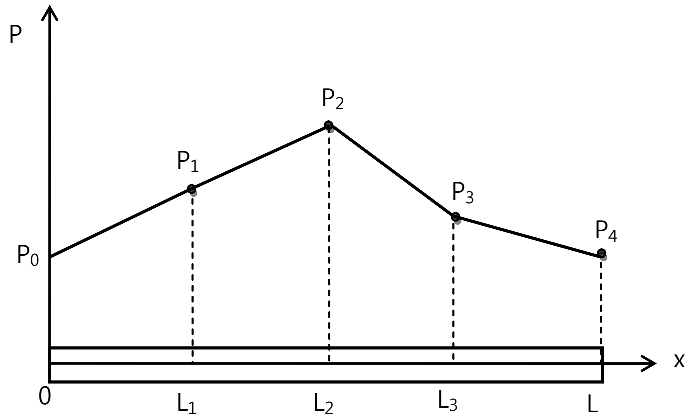


*Fig. 5 Beam subjected to partial linear load*

Fig. 5 is represented by equations as follows:

$$0 < x < l_1, \qquad p(x) = \frac{P_1 - P_2}{l_1} x + p_0 \tag{4}$$

$$l_1 < x < l_2, \qquad p(x) = \frac{P_2 - P_1}{l_2 - l_1} x + \frac{-P_2 l_1 + P l_2}{l_2 - l_1} \tag{5}$$

$$l_2 < x < l_3, \qquad p(x) = \frac{P_3 - P_2}{l_3 - l_2} x + \frac{-P_3 l_2 + P_2 l_3}{l_3 - l_2} \tag{6}$$

$$l_3 < x < l, \qquad p(x) = \frac{P_4 - P_3}{l - l_3} x + \frac{-P_4 l_3 + P_3 l}{l - l_3} \tag{7}$$

where, $p(x) =$ pressure along beam length

The pressure over the time is assumed to decrease linearly as indicated in equations below.

$$0 < t < t_1, \qquad p_1(x,t) = p(x)(1 - \frac{t}{t_1}) \tag{8}$$

$$t_1 < t, \qquad p_1(x,t) = 0 \tag{9}$$

where, $p_1(x,t) =$ pressure at a time on a point
$t_1 =$ time on zero pressure

Governing equation of the beam is as follows[4];

$$EI\frac{\partial^4 w}{\partial x^4} + m\frac{\partial^2 w}{\partial t^2} + kw = p_1(x,t) \tag{10}$$

where, $w$ = deformation of beam
$EI$ = bending rigidity of beam
$m$ = mass of beam per unit length
$k$ = spring constant per unit length

Deformation of the beam which is simply supported on both ends is expanded with sinusoidal series satisfying the boundary condition which is represented by the following equation.

$$w = \sum_{\mathrm{n}=1}^{\infty} sin\frac{n\pi x}{l} q_n(t) \tag{11}$$

Substituting equations (8), (9) and (11) into equation (10) and multiplying both sides by $sin\frac{m\pi x}{l}$ which is then integrated, the followings are obtained.

$$\frac{d^2 q_n(t)}{dt^2} + \omega_n{}^2 q_n(t) = R_n\left(1 - \frac{t}{t_1}\right) \tag{12}$$

$$\begin{aligned} R_n = \frac{2}{M}\Big[ & -\frac{1}{n\pi}P_1 cos\frac{n\pi l_1}{l} + \frac{P_0}{\pi} + (P_1 - P_0)\frac{1}{n^2\pi^2}\frac{l}{l_1}sin\frac{n\pi}{l}l_1 - P_1 \\ & -P_2\frac{1}{n\pi}cos\frac{n\pi}{l}l_2 + P_1\frac{1}{n\pi}cos\frac{n\pi}{l}l_1 + \frac{P_2 - P_1}{n^2\pi^2}\cdot\frac{l}{l_2 - l_1}\left(sin\frac{n\pi}{l}l_2 - sin\frac{n\pi}{l}l_1\right) \\ & -P_3\frac{1}{n\pi}cos\frac{n\pi}{l}l_3 + P_2\frac{1}{n\pi}cos\frac{n\pi}{l}l_2 + \frac{P_3 - P_2}{n^2\pi^2}\cdot\frac{l}{l_3 - l_2}\left(sin\frac{n\pi}{l}l_3 - sin\frac{n\pi}{l}l_2\right) \\ & -P_4\frac{1}{n\pi}cos\, n\pi + P_3\frac{1}{n\pi}cos\frac{n\pi}{l}l_3 + \frac{P_3 - P_2}{n^2\pi^2}\cdot\frac{l}{l_3 - l_2}sin\frac{n\pi}{l}l_3\Big] \end{aligned} \tag{13}$$

$$\omega_n^2 = \left[\frac{EI}{m}\left(\frac{n\pi}{l}\right)^4 + \frac{k}{m}\right] \tag{14}$$

Solution of equation (12) can be obtained as follows:

$$0 < t < t_1,\ q_n(t) = \frac{R_n}{\omega_n{}^2}(1 - cos\,\omega_n t) + \frac{R_n}{\omega_n{}^3 t_1}(sin\,\omega_n t - \omega_n t) \tag{15}$$

$$t_1 < t,\ q_n(t) = -\frac{R_n}{\omega_n{}^2}cos\,\omega_n t + \frac{R_n}{\omega_n{}^3 t_1}(sin\,\omega_n t - sin\,\omega_n t(t - t_1)\big] \tag{16}$$

## RESULT OF RESPONSE ANALYSIS

The response of the body of submerged floating railway under shock pressure is expressed by equations (11) to (15). Fig. 6 shows the analysis model in under explosion. The shape and distribution of shock pressure by underwater explosion is shown in Figs 7 and 8. Fig. 7 shows the shock pressure variation over the time according to equation (1) and for simplifying analysis, it was idealized in triangular pulse as indicated in equations (8) and (9). Fig. 8 is the result of idealizing the distribution of shock pressure lengthwise based on the center of the body as shown in equations (4) to (7)

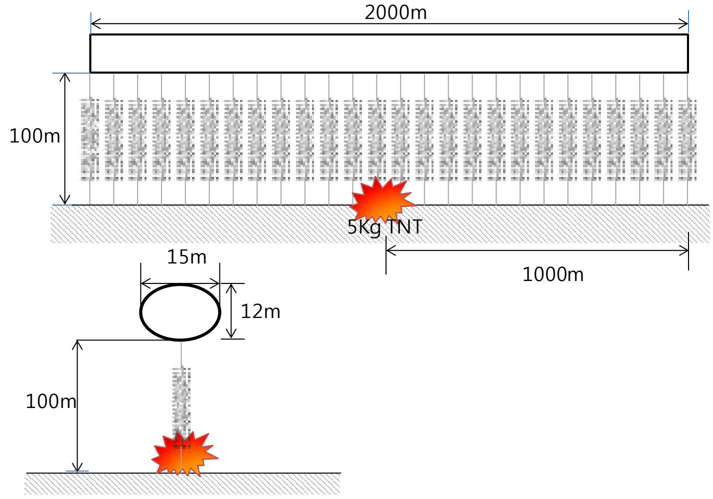


*Fig. 6 Analysis model of submerged floating railway subjected to underwater explosion*

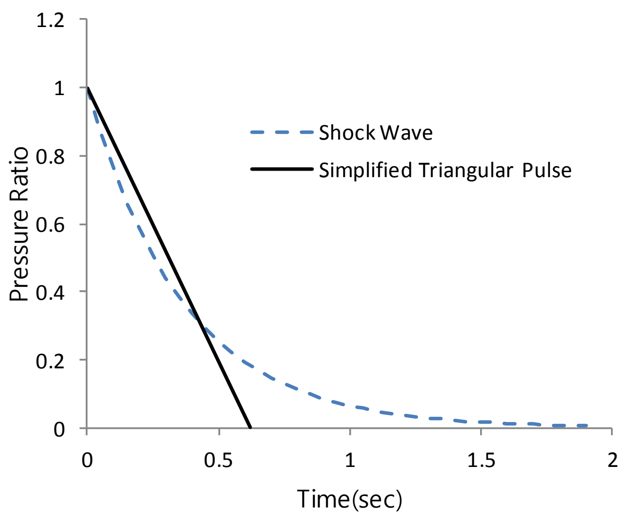


*Fig. 7 Variation of shock wave over the time*

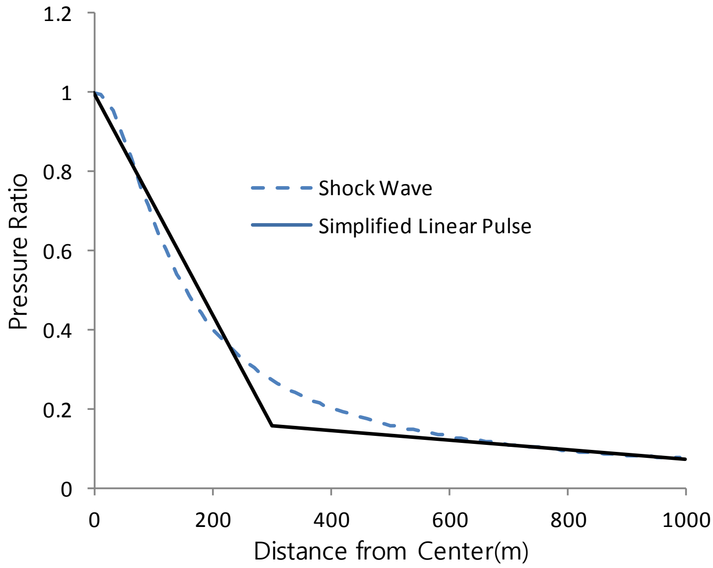


*Fig. 8 Distribution of shock pressure lengthwise*

The model for finite element analysis is shown in Fig. 9 below. Modeling the body with beam element and modeling the mooring system with distributed spring element were carried out[5]. The data for finite element analysis is indicated in Table 1

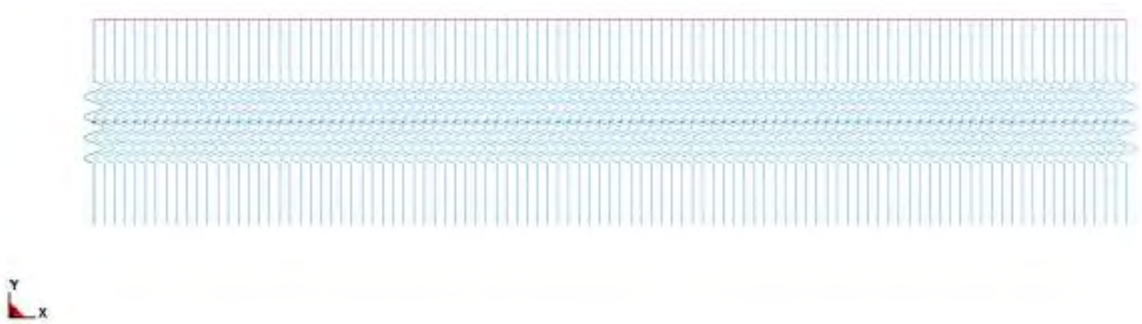


*Fig. 9 Finite element analysis model(100 beam elements and 101 spring elements)*

*Table 1 Data for analysis*

| Item | Value |
|---|---|
| Bending rigidity, $EI$(N·m$^2$) | $6.7\times10^{12}$ |
| Mass per unit length (including virtual mass), $m$(kg/m) | $1.4\times10^{5}$ |
| Stiffness of mooring system, $k$(N) | $4.8\times10^{5}$ |
| Length of the body (m) | 2,000 |
| Diameter of the body (m) | 15 |
| Weight of explosive (kg) | 4 |
| Distance between the body and explosive (m) | 100 |
| Explosive constant , $K$ | 52.5 |
| Explosive constant , $K'$ | 0.084 |
| Explosive index, $\alpha$ | 1.13 |
| Explosive index, $\alpha'$ | -0.23 |
| Maximum shock pressure (Pa) | $5.3\times10^{5}$ |
| Maximum shock load (N/m) | $7.93\times10^{6}$ |

When calculating the variation of displacement at the center where maximum displacement occurred, the result is shown in Fig. 10. Fig. 11 shows the distribution of displacement lengthwise at certain time. Viewing Fig. 10 and 11, the result from finite element analysis and theoretical analysis is coincident.

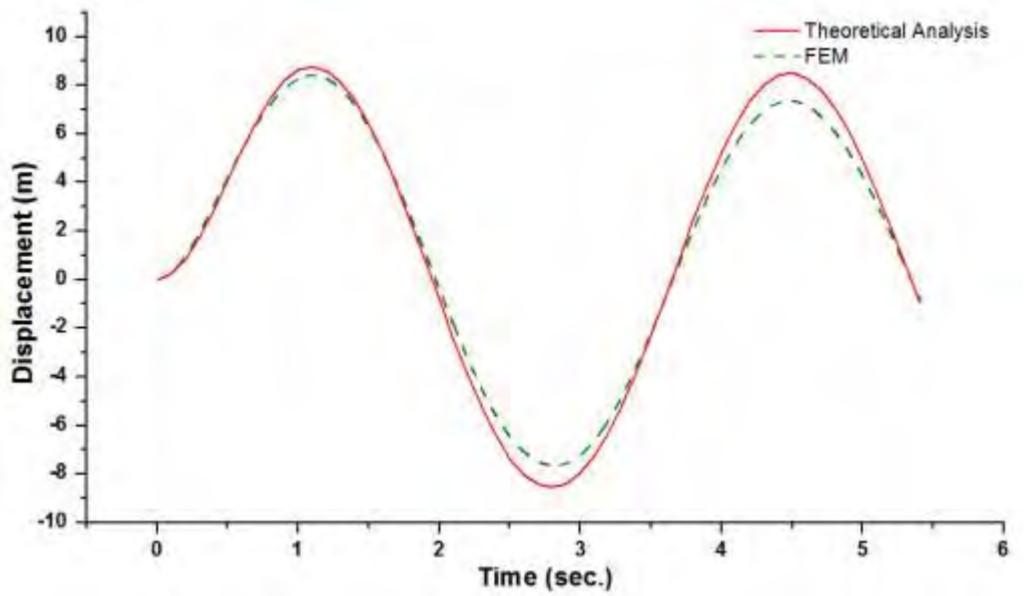


*Fig. 10 Variation at the center of the body over the time*

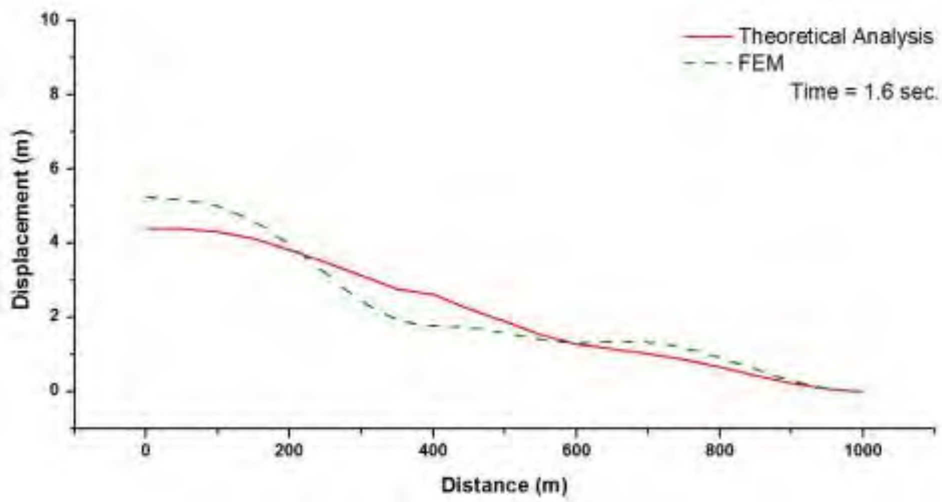


*Fig. 11 Displacement of body lengthwise*

## SIMPLIFIED RESPONSE ANALYSIS

The load caused by underwater explosion works on the body in the form of impulse pressure. First natural frequency mode of the body of submerged floating railway is shown in Fig. 12 which is the same as the case when n=1 in equation (14)

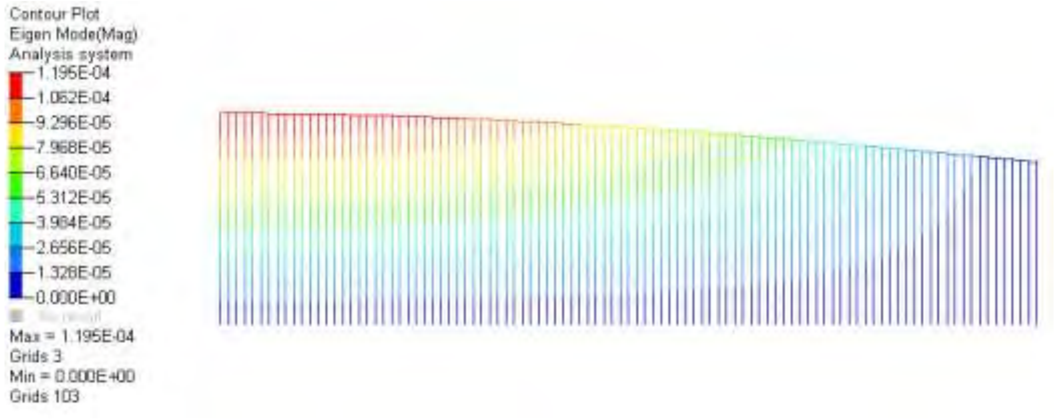


*Fig. 12 First natural frequency mode of the body*

Viewing equation (14), as the length of the body is getting extended, natural frequency of the whole body is dominated by the stiffness of mooring system while the body is acting as mass alone. Maximum displacement at the center of the body can be obtained using dynamic response factor indicated in Fig. 13 which is measured by one-degree-of-freedom vibrator [4]. Calculation results for the same model shown in Fig. 6 are presented as Table 2. As indicated in Table 2, maximum response could be obtained within 6% tolerance using one dimensional shock model.

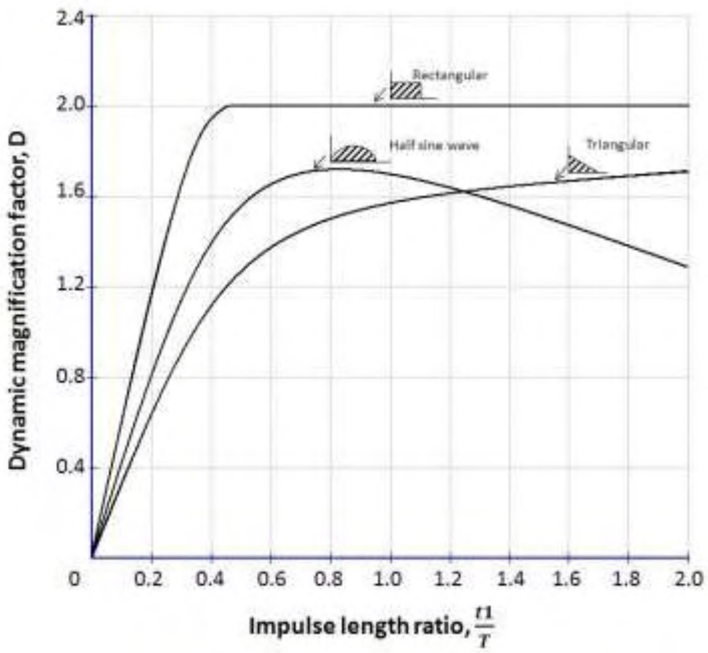


*Fig. 13 Dynamic response factor by shock load*

*Table 2 Calculation of maximum displacement to dynamic response factor*

| Item | Theoretic analysis | FEM | One dimensional model |
|---|---|---|---|
| Natural vibration frequency, T(sec) | 3.39 | 3.39 | 3.39 |
| Pulse duration time t1(sec) | 0.62 | exponentially decay) | 0.62 |
| Dynamic Magnification Factor | - | - | 0.55 |
| Max displacement (m) | 8.63 | 8.60 | 9.09 |

## CONCLUSION

In this study, response analysis of the body of submerged floating railway to underwater explosion was carried out in an effort to secure the safety of a long-distance submerged floating railway. Shock pressure by underwater explosion was assumed to be piecewise linear and theoretical solution of the governing equation was obtained for the simply supported beam with elastic springs under impulse pressure. Calculation result was compared with the result of finite element analysis. As a result of comparing, the calculated displacement was acceptably coincident. When the length of the body is extended and natural frequency of the body itself is much smaller than that of mooring system accordingly, response of the body could be obtained using one dimensional model considering the mooring system only. Theoretical simple solution is very useful for repeated analysis of design parameters needed at early design stage.

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# THE PAST, PRESENT, AND FUTURE OF THE SEIKAN TUNNEL

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**ABSTRACT**

*The Seikan Tunnel is an extremely long undersea tunnel with a length of 53.85 km that links Hokkaido and Honshu, which are two of the main islands of Japan. It is currently the second longest tunnel in the world. The construction history of the Seikan Tunnel is one of fighting and overcoming groundwater. This tunnel is maintained by several state-of-the-art engineering technologies. In this article, we briefly introduce the past, present, and future of the Seikan Tunnel from an engineering viewpoint and other points of interest.*

## 1 OVERVIEW AND HISTORY

### *1.1 Overview*

Although the concept of the Seikan Tunnel was initially developed before World War II, the actual geological survey started after the war. The *Toyamaru* ferry sinking in 1954 produced an impetus for early implementation of the project; ten years later in 1964, construction began on the inclined shaft at Yoshioka, Hokkaido. In March 1985, the main tunnel was finally completed after many collapses. Operation of the line between Hakodate and Aomori had to wait until March 13, 1988, for completion of the Seikan Tunnel to link Hokkaido and Honshu, which are two of the main islands of Japan, by land. The tunnel, which has a total length of 53.85 km and was excavated 240 m below sea level, took 40 years to complete.

### *1.2 Road to completion*

The construction history of the Seikan Tunnel is one of fighting and overcoming groundwater because about half of the tunnel was excavated beneath the sea. At the time, there were no well-tried models for such construction. Before the main tunnel, inclined shafts, and pilot tunnel were built, tunnelling methods were developed, and geological surveys were conducted. The service tunnel was then excavated followed by the main tunnel. The project was successfully completed through the enlisting of new technologies to battle with the extreme conditions. Such new technologies included laser surveys, the world's longest horizontal boring of 2150 m; a grouting water glass–concrete mixture which uses high-pressure pumps to strengthen weak bedrock; and shotcrete to stabilize the bedrock after excavation. In addition to the above technologies, the rock bolt method, which is a novel technology wherein rock bolts are hammered into the bedrock radially, was also applied to an actual tunnel construction project for the first time.

*Table 1 Outline of Seikan Tunnel*

| | |
|---|---|
| **• Route** | |
| Starting point | Hamana, Imabetsu-machi, Aomori Prefecture |
| Finishing point | Yunosato, Shiriuchi-cho, Hokkaido |
| **• Length** | 53.85 km (23.30 km undersea, 30.55 km under land ) |
| **• Tunnel design standard** | |
| Minimum radius of curvature | 6,500 m |
| Maximum gradient | 12/1,000 |
| Minimum overhead earth thickness (undersea) | 100 m |
| Maximum water depth over tunnel | 140 m |
| Tunnel cross section | Double-track Shinkansen-type |
| **• Track structure** | Triple-rail concrete slab track that allows both ordinary and Shinkansen trains to run in the future |

*Table 2 History of Seikan Tunnel*

| | |
|---|---|
| 1946/4 | Geological survey begins |
| 1954/9 | *Toyamaru* ferry sinks in Tsugaru Strait |
| 1964/5 | Inclined shaft excavation begins at Yoshioka (Hokkaido side) |
| 1966/3 | Inclined shaft excavation begins at Tappi (Honshu side) |
| 1967/3 | Pilot tunnel excavation (Hokkaido side) begins |
| 1968/12 | Service tunnel excavation (Hokkaido side) begins |
| 1970/1 | Pilot tunnel excavation (Honshu side) begins |
| 1970/7 | Service tunnel excavation (Honshu side) begins |
| 1971/9 | Main construction begins |
| 1979/9 | Service tunnel is completed at Tappi |
| 1980/3 | Service tunnel is completed at Yoshioka |
| 1983/1 | Pilot tunnel is completed |
| 1985/3 | Main tunnel is completed |
| 1987/11 | Seikan Tunnel is completed |

## 2 STRUCTURE OF THE SEIKAN TUNNEL

Figs.1 and 2 illustrate the sectional view of the Seikan Tunnel. The starting point of the Seikan Tunnel is Hamana in Imabetsu, Aomori prefecture, and its end point is Yunosato in Shiriuchi, Hokkaido. The total length of 53.85 km exceeds that of Channel Tunnel. Part of the tunnel was constructed below sea level with a minimum earth covering of 100 m. It is 4.3 km longer than the shortest route of the Tsugaru Strait (19 km) because the tunnel was excavated along a shallow route. The cross section of the tunnel is a horseshoe shape capable of supporting a double-track Shinkansen line: 9.7 m wide and 7.85 m high. The triple-rail concrete slab track installed on the concrete panels, which can hold three rails, allows the operation of both a Shinkansen line and the existing line in the future.

There is no horizontal element in this tunnel to drain the constantly flowing groundwater. The water is drained to the lowest point near the centre of the tunnel and channelled up over the land by pumps to three drainage areas. However, the steepest gradient of 12/1000 will allow high-speed Shinkansen operation in the future. The world’s first undersea stations – Yoshioka Kaitei and Tappi Kaitei – also serve as emergency stations.

The main structures of the Seikan Tunnel are introduced briefly below.

### *2.1 Vertical shaft*

During construction, machinery, materials, and tunnel workers were carried by elevators in shafts. Currently, these shafts are used as gas exhaustion ducts.

### *2.2 Main tunnel*

This is the railway tunnel: 7.85 m high and 9.7 m wide. It can fully house a three-story building.

### *2.3 Inclined shaft*

There are two major inclined shafts: one at Yoshioka and another at Tappi. They have a gradient of about 14° going down to the bottom of the sea. During the construction period, they were used for geological surveys, mucking, draining, and carrying tunnel workers, machinery and materials. They are currently used as air ducts for ventilation.

### *2.4 Pilot shaft*

This tunnel was excavated before any of the other tunnels. It was used for surveys on marine geology and groundwater flow and to examine and develop excavation methods in preparation for the construction of the service and main tunnels. Its current roles are draining and ventilation.

### *2.5 Service tunnel*

The service tunnel was constructed 30 m from the main tunnel to increase construction speed. The connecting galleries excavated before the main tunnel allowed the cutting blades to be

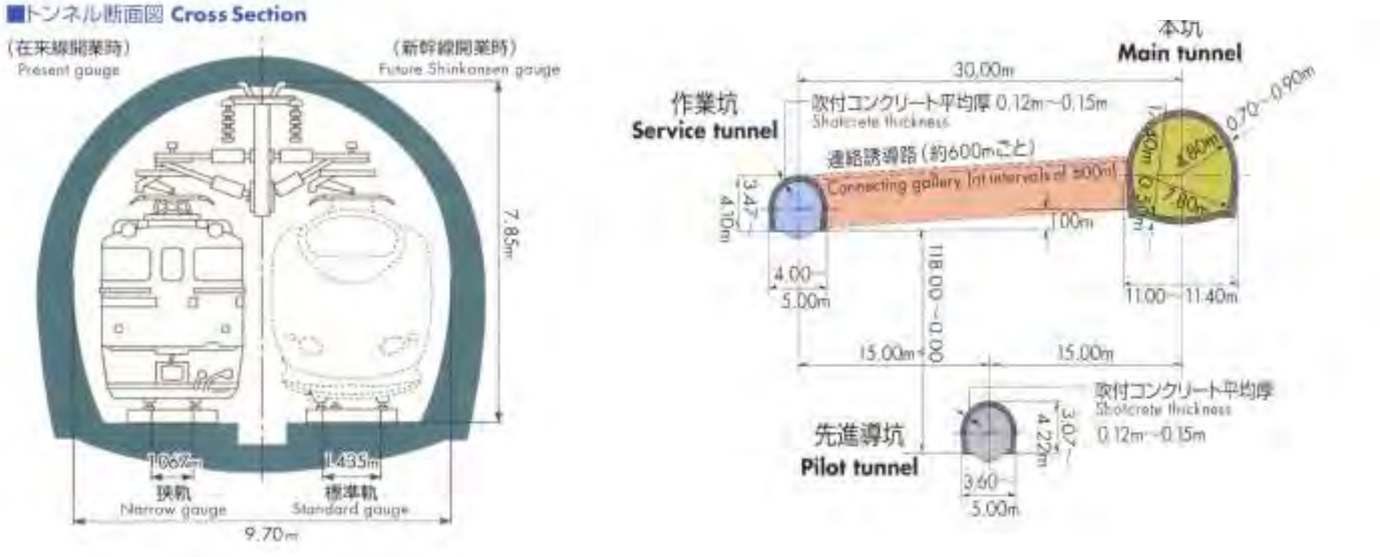


*Fig. 1 Cross-sectional view of the Seikan Tunnel*

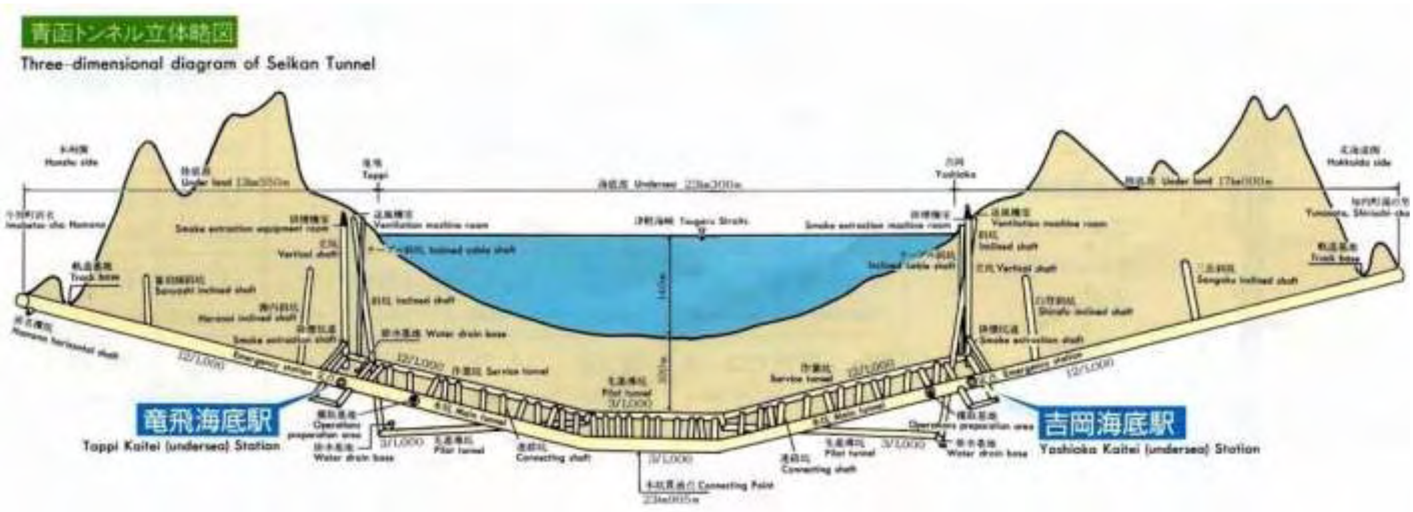


*Fig. 2 Three-dimensional diagram of the Seikan* Tunnel

increased. It was also used for mucking and to carry machinery and materials. Currently, it is used for maintenance and allows for bicycle and motor traffic.

## *3* SYSTEMS

### *3.1 Control centre*

Operation of all trains running on the line between Hakodate and Aomori is managed centrally at the Hakodate Control Centre. The location of each train and the state of equipment are accurately displayed on a panel, and the locomotive driver can communicate with the dispatcher at the Centre by radio transmission at any time. Each disaster prevention apparatus, which ensures tunnel safety, sends information automatically to the Centre. In emergencies, the train stops automatically, and the dispatcher at the Centre can manually operate the fire extinguisher, gas exhaust equipment, and lighting. A large volume of data is transmitted through optical fibre cables, a product of the latest technology. In the event of a malfunction in the optical fibre cable, a backup system of microwaves can be operated.

### *3.2 Disaster prevention system*

Various state-of-the-art disaster prevention systems have been constructed within the Seikan Tunnel to ensure passenger safety. Yoshioka and Tappi Kaitei (Undersea) Stations are notable for their role in emergencies. If the fire detector inside the tunnel detects abnormally high temperatures, the trains stop immediately at the nearest station. The passengers are guided from the platform via the connecting gallery to a fire shelter, where ventilation is ensured. The whole area of the escapeway is equipped with lighting, 75 TV cameras, and a public address system with 173 speakers. Communications are directed by the Control Centre. A train fire can be extinguished with water from fire hydrants capable of providing water at 7 t/min for 40 min. In addition to the four routes provided by electric companies (two routes each for Hakodate and Aomori), a private power generation system has been introduced to ensure a power supply in emergencies. If an earthquake of a certain magnitude should occur, all trains in the tunnel stop automatically. The Control Centre assesses conditions and instructs locomotive drivers to resume operations only after confirming the tunnel’s reliability.

## *4* SUMMARY

Currently, construction on the Hokkaido Shinkansen rail line linking Shin-Aomori and Shin-Hakodate (provisional name) is underway. The opening of the Hokkaido Shinkansen Line should significantly enhance the convenience of travellers going between Hokkaido and Honshu, particularly between Hokkaido and the Tohoku region. Thus, the role of the Seikan Tunnel will be more important in the future. In this conference, we will present the maintenance of the Seikan Tunnel in addition to the structural systems shown in this article.

## ACKNOWLEDGEMENT

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# SIMPLIFIED COLLISION ANALYSIS METHOD FOR SUBMERGED FLOATING RAILWAY USING THEORY OF BEAM WITH ELASTIC FOUNDATION

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## ABSTRACT

*Submerged floating railway is an innovative tunnel infrastructure passing through the deep sea independent of wave and wind so that high speed train can run. It doesn't depend on water depth and is cost effective due to modular construction on land. The construction period can be reduced drastically. In this paper, a concept design of submerged floating railway is introduced and a method to analyze structural behavior of the body in case of collision with a submarine is proposed for securing safety. Theory of beam with elastic foundation was used to calculate equivalent mass of the body so that the perfect elastic collision could be applied to calculate the collision velocity. Maximum deformation and bending moment were analyzed based on energy conservation. For verification of the analysis results, collision analysis using finite element analysis code was made. Comparison with the results showed that this simplified collision analysis method gives accurate deformation and bending moment enough to be used for actual estimation in initial design stage.*

***Keywords* :** Collision, Elastic foundation, Energy conservation, Equivalent mass, Submarine, Submerged floating railway, Theory of beam

## 1. INTORDUCTION

Submerged floating railway system is a railway system running within floating tunnel supported by buoyancy, which can be built independently of water depth. Since it is pre-fabricated on land before assembling on sea, construction cost and time could be reduced [1]. The effect by wave or wind can be significantly reduced as tunnel goes down deeper, submerged floating tunnel maintains the stability without impact by external agitation. While immersed tunnel is designed to immerse tunnel structure by gravity and fix it to the sea-bed [2], submerged floating railway usually has tunnel structure with greater buoyancy than gravity so that the rising force is constrained by moored vessel, maintaining a balance.

The concept of submerged floating railway has long been under study since it was proposed and in Japan and Italy, it's developed to basic design stage but has yet to be realized in any countries [3] An underwater tunnel crossing the inland lake has recently been implemented in China, which however has yet to be put on track for construction [4]. There might be a number of reasons behind the delay with submerged floating railway, which however is rather attributable to passenger's fear of safety in emergency than any technical challenges. Thus verification of the safety in emergency and passenger safety are more than important in realizing submerged floating railway [5]

There might be a several emergency scenarios threatening the life safety under the water and in Korea which is surrounded by the sea where the military force is concentrated, collision by submarine is one of the potential and also the vessels sailing on sea. Depth over 20 m can reduce such risks. Collision with submarine could be avoided by detection using sound wave, but it's necessary to verify the safety against collision in case of emergency such as failure of warning system. This study is intended to suggest the conceptual design of submerged floating railway as well as propose the fundamental technology of collision-prevention design, just in case. To prove the structure design of the enclosure in timely manner, simplified collision analysis method is proposed and the applicability is verified in a way comparing with numerical analysis result.

## 2. DESIGN SUMMARY OF SUBMERGED FLOATING RAILWAY

A pipeline-shaped composite structure in Fig 1 is the typical structure with the sufficient buoyancy and strength required in running under the water. In this study, conceptual design of submerged floating railway was applied to the sea with maximum depth of 120m at Honam-Jeju area to evaluate the safety and feasibility of the Submerged Floating Tunnel (SFT). For complete waterproof effect, enclosure was formed with steel metallic material and to maintain the buoyancy at certain level inside the enclosure, it's filled with concrete. Internal plate using steel metallic material was formed internally to achieve the dual waterproof system. A module with length of 100 m was fabricated and external plate was fastened with bolts on water surface and pre-assembled before assembling completely in water. Fixed-type jacket for evacuation and ventilation was prepared at certain interval [1]

The structure has the buoyancy exceeding gravity and thus exceeding buoyancy is balanced by the tension of mooring. Among various materials and structures which could be used for mooring, the chain which has been widely used for offshore construction was used to fix it to the sea-bed. Mooring chain functions to convey the tension by buoyancy to sea-bed and is fixed to sea-bed in various methods. The chain could be connected by driving the pile into seabed as traditional method or the method to offset the tension by gravity using a large concrete block is also available. Among the advanced methods is suction pile. Anchoring method into sea-bed may be adopted taking into account of difficulties with construction depending on depth, submarine topography and cost, but gravity anchoring method using concrete block has advantage in terms of cost and construction efficiency. Based on concept design of submerged floating railway at Honam ~ Jeju, 4 mooring cables per 100m-module considering the buoyancy-gravity ratio of the body and stationary structure at every 10km, considering passenger evacuation and ventilation, were considered [1]

*Fig. 1 Submerged floating railway shaped as pipe line*

*Fig. 2 Section of submerged floating railway*

## 3. COLLISION ANALYSIS MODEL

To carry out the analysis in a simple way in the event of collision by submarine as illustrated in Fig 3, one dimensional perfect elastic collision was conducted. The modeling of submarine with lumped mass was first carried out, which was followed by modeling the submerged floating railway in a simple way using the concept of equivalent mass and equivalent rigidity. Mooring line maintaining the balance with initial tension against exceeding buoyancy is capable of resisting with tension (increased initial tension) and compression (decreased initial tension) to deal with the variation in load of the body, and as designed with the steel showing linear elastic characteristic, it could be idealized using elastic support spring.

Structural behavior of submerged floating railway which continuously supports the buoyancy of the body by mooring line at certain interval could be idealized by the beam with infinite length on elastic support. When concentrated load is working on elastic supported-continuous beam, deflection of the beam is calculated by the following equation [6]

$$w(x) = \frac{P\beta}{2k} e^{-\beta x}(cos\beta x + sin\beta x) \tag{1}$$

where, $\beta = \left(\frac{k}{4EI}\right)^{1/4}$

$w(x) =$ deflection of beam

$P =$ concentrated load

$k =$ spring modulus of elastic support per unit length

$EI =$ bending stiffness of beam

*Fig. 3 Collision with a submarine in deep water*

Variation depending on maximum speed of the body after collision could be assumed as follows according to variable separation principle, referring to equation (1)

$$v = V_0 e^{-\beta x}(cos\beta x + sin\beta x) \tag{2}$$

where, $v =$ traveling speed of the body

$V_0 =$ max traveling speed of the body

Equivalent mass is obtained by calculating kinetic energy of the body. That is,

$$E_t = \frac{1}{2} M_t V_0^2 = \frac{1}{2}\int_0^\infty 2m_t \left\{V_0 e^{-\beta k}(cos\beta x + sin\beta x)\right\}^2 dx \tag{3}$$

where, $E_t =$ kinetic energy of the body

$M_t =$ equivalent mass of the body

$m_t =$ mass per unit length of the body

As the behavior of the body and submarine in water is indicated, mass includes added mass. Equivalent mass is calculated from equation (3)

$$M_t = \frac{3}{2\beta} m_t \tag{4}$$

In the event of collision between the body and submarine, momentum conservation law is expressed as follows in case of elastic collision.

$$M_s v_{si} = M_s v_{sf} - M_t V_0 \tag{5}$$

where, $M_s =$ mass of submarine

$v_{si} =$ speed of submarine before collision

$v_{sf} =$ speed of submarine after collision

When applying energy conservation law, the following equation is made

$$\frac{1}{2} M_s v_{si}^2 = \frac{1}{2} M_s v_{sf}^2 + \frac{1}{2} M_t V_0^2 \tag{6}$$

The speed of the body after collision with submarine is calculated from equation (5) and (6)

$$V_0 = \frac{M_s}{\frac{M_s}{2} + M_t} v_{si} \tag{7}$$

When idealizing the whole body to elastic spring at the point where collision with submarine was occurred, equivalent rigidity modulus has the following relationship.

$$w(0) = \delta = \frac{P\beta}{2k} = \frac{P}{K_{eq}} \tag{8}$$

where, $\delta =$ maximum displace of the body

$K_{eq} =$ Equivalent rigidity of the body where collision with submarine was occurred

Equivalent rigidity from equation (8) is expressed as follows.

$$K_{eq} = \frac{2k}{\beta} \tag{9}$$

On assumption that kinetic energy of the body is stored in the form of elastic energy after collision according to energy conservation law, deformation is calculated as follows.

$$\delta = \sqrt{\frac{3m_t}{k}} \frac{1}{1 + \frac{3m_t}{M_s \beta}} v_{si} \tag{11}$$

Bending moment of the section of the body is calculated by following equation.

$$M_B = EIw(x)" = 2EI\delta\beta^2 e^{-\beta x}(-cos\beta x + sin\beta x) \tag{12}$$

where, $M_B =$ bending moment of the body

## 4. COLLISION SIMULATION USING FINITE ELEMENT METHOD

Before conducting a non-linear analysis, feasibility of Equation (1) which is the theoretic analysis for concentrated load on elastic-supported beam was verified. Displacement by concentrated load on infinite beam was calculated which was then compared with the result of finite element method. Modeling of the body in elastic state was conducted with beam element as indicated in Fig 4 while modeling the mooring line with spring element. Modeling all bodies between stationary structure was carried out and sectional characteristics and major data are indicated in Table 1. The submarine used for collision test was Navy's main marine, Sohnwonil-grade (displacement 1,800ton, maximum speed 20knots=10m/sec) Analysis was carried out using linear and non-linear finite element analysis software LS-DYNA [7]

To verify the feasibility of Equation (11) after elastic analysis, collision simulation is conducted. Simulation using non-linear finite element analysis in case that the submarine traveling at certain speed hit the body is performed to identify the behavior of the body. Modeling the collision part which was fabricated with steel material was conducted to enhance the accuracy of the analysis. Virtual mass was assumed to be 50% of displacement, given the round section [8]

Non-linear collision analysis considering plastic deformation must be effective numerical analysis in incorporating real effect, which is however takes extended time for detail modeling and analysis. Because the assumption idealizing the body of submerged floating railway as elastic-supported beam is not able to simulate the local strain and failure behavior by impact load occurred during collision as well as is not able to consider the dispersion of collision energy by plastic strain behavior of steel structure of the submarine, it has the limit in predicting the actual collision behavior. Despite of such limits, simplified collision analysis method is able to predict the collision response of the body accurately to some extent through the simple equation using only core data from concept design stage, which is very useful. When design parameters are changeable at early design stage, collision performance of the body could be immediately evaluated and thus the result of this study would make meaningful commitment.

As detail design of the body of submerged floating railway is further developed and structure design of the submarine is made available, more detail collision analysis including plastic deformation behavior is expected to continue.

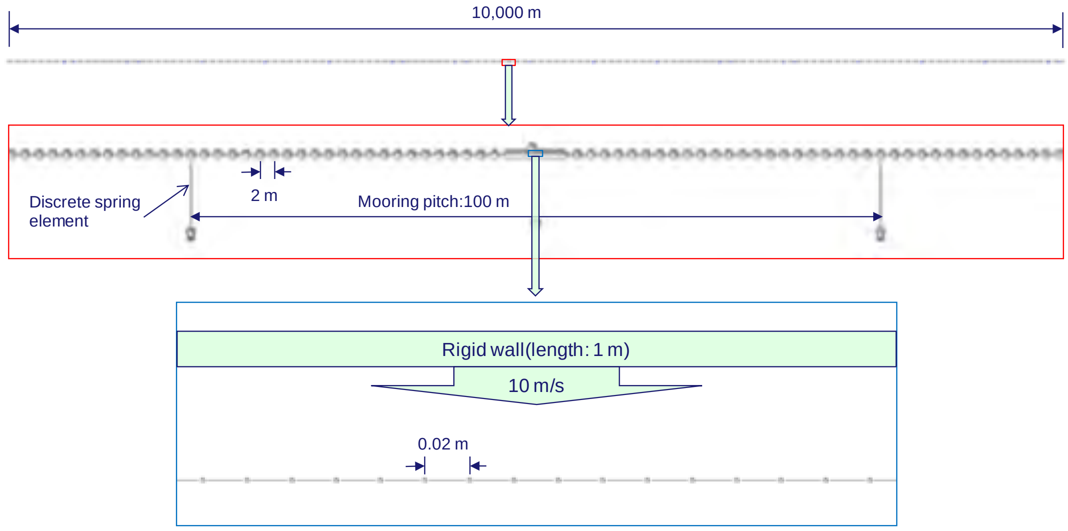


*Fig. 4 Finite element model for collision analysis*

*Table 1 Data for collision analysis*

| Item | Value |
|---|---|
| Beam cross-section geometry | Outer diameter of a tube: 14,800 mm<br>Inner diameter of a tube : 14,722 mm<br>Young's modulus: 210 GPa<br>Equivalent density: 44,092 kg/m$^3$ |
| Beam element type | Hughes-Liu with cross section integration |
| Bending stiffness | 10,429 x 10$^9$ Nm$^2$ |
| Spring constant for mooring chains | 48.3 MN/m |
| No. of beam elements | 10,992 |
| No. of nodes | 10,998 |
| Mass and added mass | Mass of a impacting barrier 50% up : 2,700 ton<br>Equivalent density of a beam: added the water mass inside a tube : 139,188.4 kg/m$^3$ |

## 6. COMPARISION OF ANALYSIS RESULT

As a result of conducting finite element analysis at static load state, comparison with Equation (1) which is the theoretic analysis result is made as Fig 5. The error of displacement at static load state was indicated 1.1% or less. Very little effect of boundary condition appeared when bearing point is considerably distanced, and acceptable result would possibly be obtained when using analysis result of the beam with infinite length.

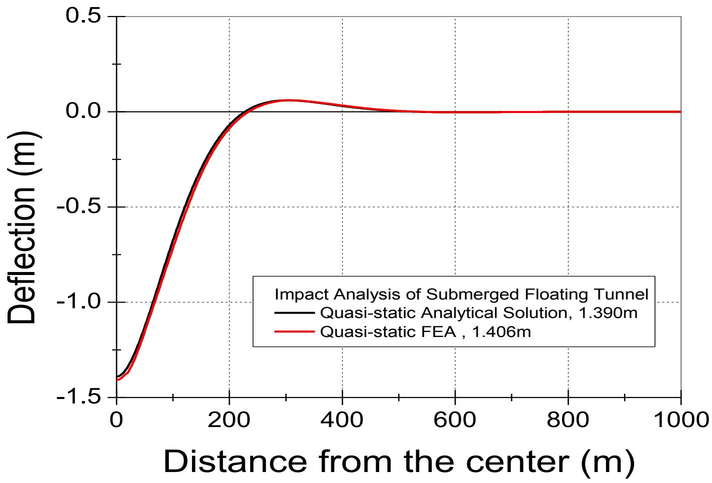


*Fig. 5 Quasi-static deflections of body of tunnel*

In case of collision, displacement distribution of the body as a result of finite element analysis is as shown in Fig 6. Comparison between the result of simplified analysis in Equation (11) and the result of finite element analysis is as Fig 7 showing maximum displacement at the center was within 21% which is acceptable. According to comparing bending moment in Equation (12) with the result of collision simulation, the result is as Table 2. The moment at collision point showed the error of 18%, which was attributable to either simplification of the body into one dimensional concentrated mass or displacement function for theoretical analysis which was not able to completely realize the wave vibration by actual collision. However the result was reliable enough to predict the displacement at early design stage and bending moment in timely manner.


*Fig. 6 Contour of deformation by finite element analysis*

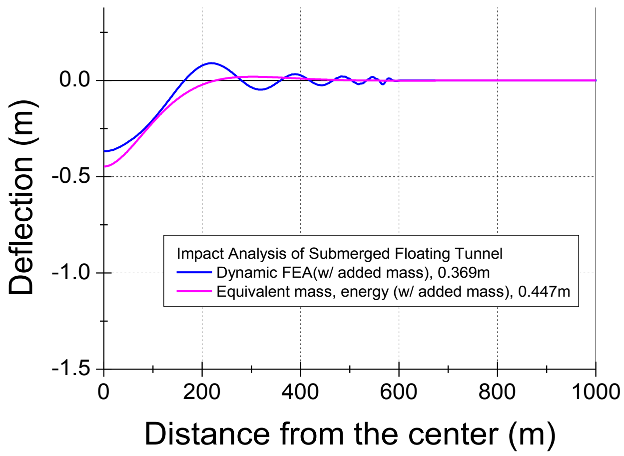


*Fig. 7 Comparison of deflections in body of tunnel after collision*

*Table 2 Comparison of bending moments at center of body after collision*

| BM at center by analytic solution | BM at center by FEM | Difference |
|---|---|---|
| $1.003\times10^{9}$ N-m | $1.227\times10^{9}$ N-m | 18.2 % |

## 7. CONCLUSION

Submerged floating railway is innovative transport infrastructure which is relatively independent of water depth comparing to undersea tunnel and is built within a short time at low cost. In this study, the method to verify the safety in preparation for collision with submarine or falling stuff which is considered potential threat to submerged floating railway, using theory of beam with elastic foundation, is proposed. Equivalent mass of the body of submerged floating railway was calculated and the speed in case of complete elastic collision with submarine to one dimensional model was calculated and then strain of the body and maximum bending moment was calculated according to energy conservation law. As a result of conducting finite element analysis using beam element, the error of the result of simplified analysis proposed in this study and displacement of static load was within 1.1% and collision load displacement 21% and bending moment was 18%. When it comes to static load displacement, the body bearing point was distanced considerably and thus very few boundary effect appeared, and acceptable result was could be obtained by using analysis of the beam with infinite length alone. The error with collision load seemed to be attributable to incomplete incorporation of wave vibration by simplified model, but simplified equation proposed in this study could be used in predicting the strain and maximum bending moment of the body in timely manner at early design stage.

Among the limits in this study is plastic deformation by collision which was not considered but repeated analysis to deal with changing design parameters is one of the advantages. In a bid to supplement the limits in this study, collision analysis considering non-linear plastic deformation behavior through detail modeling upon available of detail design of the body and data on submarine will be carried out.

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# NEW CHALLENGES FOR THE FIRE SAFETY IN SUBMERGED FLOATING TUNNELS

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**ABSTRACT**

*The new E39 between Trondheim and Kristiansand is one of the most challenging road projects in the world. The use of submerged floating tunnels will challenge the safety concepts for fires. In order to come up with effective and reliable solutions for these tunnels, it is necessary to apply the latest knowledge from large infrastructure projects and research. In the paper a sum up of the most recent research that have been published on large-scale fire tests in tunnels, both with and without active mitigation systems and with different materials in construction, is presented.*

## INTRODUCTION

The concepts of the submerged floating tunnel (SFT) have been on the drawing board for a long time, or since Norway came close to build the world's first SFT in the 1980s. It was a two-lane SFT road connection adopted for a crossing of the Høgsfjord toward the city of Stavanger. It was at the eleventh hour that the local county council decided an undersea rock tunnel at a different location and not the Høgsfjord SFT as promoted by the Norwegian Roads Administration (NRA) [1]. The concept is therefore no futuristic novelty but a serious engineering challenge in different parts of the world. In recent years it is Norwegian engineers that have invested more in developing the technology. For Norway, the concept provides the best option for providing fixed links across the country's deep fjords, at locations where the divide is either too deep for deep undersea rock tunnels or too wide or exposed for long high bridges or floating bridges [1].

SFT are positioned at about 25-30 m below the water surface, connected to short landfall tunnels at either side, and secured in some way to counteract buoyancy and the movement of tides, currents and waves. From an economical point of view a SFT should [1]:

- be much shorter than a deep undersea rock tunnel or a high level suspension bridge;
- be less expensive to build than the deep mined rock tunnel or high level bridge option;
- consume less materials than the alternatives;
- take less time to construct.

The most common alternative in Norway has been the undersea rock tunnels. About 30 undersea rock tunnels have been built since 1980 with a total length of slightly over one hundred kilometers. The greatest drawback of undersea rock tunnels is that the deeper below the water the longer the tunnel. To pass under some of the deep fjords in Norway, the connections to the surface either side must extend several kilometers to control the incline and decline gradients. To limit the length of these approaches, the Norwegians have pushed gradients to 10% in some cases, as the maximum for vehicular tunnels. Such steep down gradients are hard to control for drivers and create standstills on the steep up gradients due to slow traffic on the up-grades. The NRA is considering to limit on the gradients of 5% which is in compliance with the European Tunnel Directive [2]. Therefore SFT becomes an interesting alternative for NRA.

Internationally, fire problems in long and steep tunnels have not been focused on simply because there are not many tunnels that are as steep as the ones found in Norway. This has historically been treated as a specific problem for Norwegian road tunnels. The fire accident in the Glazier tunnel in Kaprun in 2001 [3] changed slightly the view of such fires. The effects of the slope became obvious to those who studied and investigated this accident. The fire started in a short a funicular carriage running from Kaprun to the mountain Kitzsteinhorn having 3300 meters of track inside a tunnel angled at 30 degrees. The majority of the escapers went in the wrong uphill direction and were captured by the smoke and toxic gases as the flow inside the tunnel accelerated due to buoyancy forces. This remarkable incident teaches us many lessons about fire dynamics and risks in steep and long tunnels. The driving force for the smoke to spread is the density difference multiplied by the height difference between the location of the fire and the outlet where the smoke will be transported to. Due to the buoyancy forces the ventilation flow will be controlled by this height difference and not the height difference of the portals.

There have been at least four fire incidents in road tunnels in Norway in recent years with large vehicles involved. Two of these fires occurred in the steep undersea Oslofjordtunnel in 2011, one in March and one in June. The Oslofjordtunnel is 7,2 km long with the steepest slope of 7 %. In the June fire, which was more serious, a Heavy Goods Vehicle trailer with some cellulosic material combined with some plastic material started to burn. The fire became quite violent and was extinguished within one hour. The fire demonstrated well the problems with this type of fires in long and steep tunnels, i.e. evacuation, smoke spread and rescue. Over 30 people were trapped in the smoke for a long time before they were rescued and dozen of them were send to hospital for further medical investigations.

When future SFT are to be built on the E39 road network, it is important to focus on these specific problems that may rise. The protection of the people inside these tunnels, have to be secured by build twin tube systems with regular cross-passages. The construction must resist long fire durations, otherwise there will be a risk for loss of the tunnel as water may be introduced into the tunnel due to failure in the construction.

The SP Technical Research Institute of Sweden (SP) and the Norwegian Research Institute SINTEF – NBL are two world leading institutes working on fire safety engineering and research in the field of tunnel fire safety. These two research institute have a long experience in this field. The purpose is of this paper is to share the experience from the work of these institutes and increase understanding of what fire poses for threats in SFT. The methods that are required to examine practical solutions are on a large scale. This requires cutting-edge national and international knowledge, including experience and technologies from large scale testing. The large scale tests performed by SP and NBL in tunnels are unique and have been used in many research studies and infrastructure developments. They provide new models of fire development, fire growth rates, influence of ventilation and mitigation systems such as fixed water spray systems. In the following different fields of fire safety in tunnels are discussed from a SFT point of view.

## VEHICLE FIRES IN TUNNELS

SP and NBL have been involved in the performance of large scale tunnel experiments involving road vehicles. Knowledge about how fires develop in different type of vehicles is vital in order to understand the consequences of a fire in a SFT. It not only gives an idea of how fast the fires grow, but also how large they become in form of MW and the expected duration. The two of the most important large scale test series that have been performed and that SP and NBL were involved are the EUREKA EU499 tests [4] and the Runehamar tests in 2003 [5-7]. These tests have provided fire safety engineers with very important data, namely the HRR from different types of road vehicles. These tests have resulted in many publications on heat release rates (HRR) in tunnels. The most recent one is the summary carried out by Ingason and Lönnermark [8], which gives a good overview of different vehicle fires.

These tests show that HRR depends on many factors including the infrastructure of the tunnel, the type of vehicles and their contents as well as the ventilation conditions. Furthermore, separations between vehicles or ‘fuel packages’ are very important in relation to spread as well as the nature of the content of each vehicle or fuel package. The geometrical arrangements and proximities of fuels within a fuel package would also be expected to significantly affect spread. The growth rate and time to reach peak HRR will vary depending on the test setup. In a tunnel the "heat trapping" and surrounding wall re-radiation will tend to reduce the time to reach peek HRR.

Experience from large tunnel fires incidents show that the HRR is the most important parameter for evaluation of the hazardous situation for evacuation [9]. The HRR is also a key parameter used in the design of tunnel safety systems and in considering the structural strength of a tunnel. The design parameters usually involve tabulated peak HRR values in MW [10, 11]. In the following the available data on HRR for different vehicle fires is presented. This is necessary information in order to make an accurate fire engineering analysis of a SFT.

### Passenger cars

The HRRs data presented in Figure 1 vary from 1.5 MW to about 9 MW for single cars, but the majority of the tests show HRR values less than 5 MW. When two cars are involved the maximum HRR varies between 3.5 and 10 MW and with three cars 16 MW. PIARC [10] proposes a maximum HRR for one small passenger car to be 2.5 MW and for one large passenger car to be 5 MW and NFPA 502 [12] with cars 5 – 10 MW. Based on the data presented here these appear to be reasonable values. There is a great variety in the time to reach peak-HRR, between 10 and 55 minutes. A t-square fire growth rate corresponding to a fast curve covers most of the fire growth rates [8]. In many of these tests, the maximum occurs very late depending on how the windows break and what ventilation conditions are dominating. One should bear in mind that these tests are in mostly carried out in car parks, where very little longitudinal ventilation exists. Increased longitudinal ventilation may increase the fire spread between multiple vehicles, and thereby increase the maximum heat release rate and shorten the time to maximum (peak) heat release rate. Surrounding walls that are heated up also tend to shorten this time.

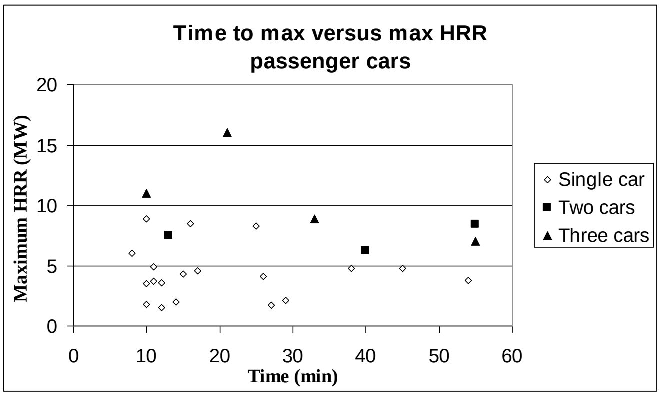


*Figure 1 A summary of time to reach maximum versus maximum HRR for single, two and three cars respectively.*

**Buses**

In Figure 2, numerous HRRs as a function of time are given for buses. This is both large scale tests in tunnels and in laboratory.

In 2008, Axelson et al. [13] presented a large-scale experiment with a modern coach (Volvo) with 49 seats which was carried out in SP's fire hall. The authors of the report estimated that the maximum HRR could have been as much as 25 MW if the bus had been allowed to continue to burn. A bus test was carried out in the Shimizu Tunnel using a sprinkler system [14]. The peak HRR was estimated to be 30 MW (HRR estimated by two different methods). A bus fire occurred in a new bus in the Ekeberg tunnel in Oslo, Norway in 1996 [15] , which appears to have developed in a similar way to that in the Eureka 499 bus test [16] shown in Figure 2. This occurred despite the large differences in ages and types of buses. These tests show that we may expect a very rapid fire development in modern buses (t-square fire growth rate of ultra fast [17] is shown in figure 2), and the highest HRR should be at least 25 – 30 MW, if not more. Fire tests in tunnels tend to result in faster growth which in turn can result in slightly higher peak HRR.

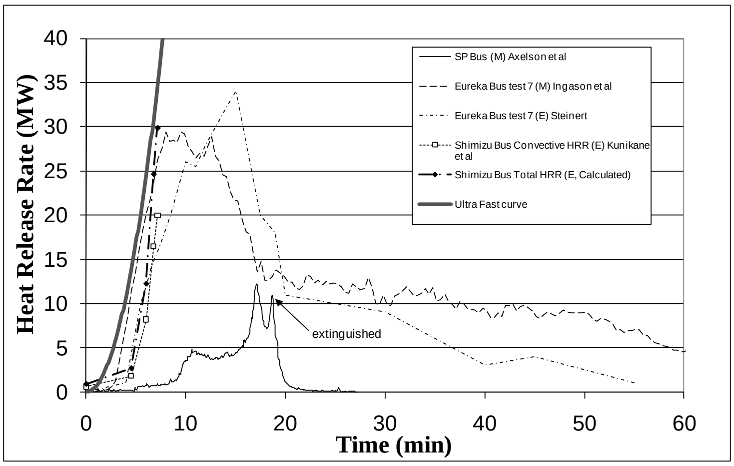


*Figure 2 Heat release rates measured for buses [8].*

**Heavy Goods Vehicles (HGV)**

The first HGV test was performed in 1992 in the EUREKA 499 [4], the second in the Mont Blanc tunnel 2000 [18], the third in the Second Benelux tunnel [19] in the Netherlands in 2001 and most recently in the Runehamar tunnel [5, 6]. In the EUREKA 499 tunnel test programme a real HGV loaded with mixed furniture was burned using varying longitudinal velocity. The second test in the EUREKA 499 series was conducted using a simulated HGV-trailer load (mock-up). In the Mont Blanc tunnel test, a HGV (truck and a trailer) similar to that which generated the fire in 1999 but with a much smaller amount of transported goods was conducted [18, 20]. In the Second Benelux tunnel test series [19] HGV trailer mock-ups were used. Standardized wood pallets were arranged in two different configurations with different longitudinal velocities. In the Runehamar test series [5, 6], four large-scale tests, each with a mock-up of a HGV-trailer in a road tunnel, were performed.

The test results presented in figure 3 show that the peak HRRs from all tests conducted on HGVs or HGV trailer mock-ups are in the range of 13 MW to 202 MW depending on the fire load, ventilation etc. The time to reach peak HRR is in the range of 10 to 20 minutes. The fire duration is less than one hour for all the HGV trailer tests. The t-square fire growth rate of ultra-fast covers most of the HGV fires found in the literature, except for the Runehamar tests, which grow faster than ultra-fast.

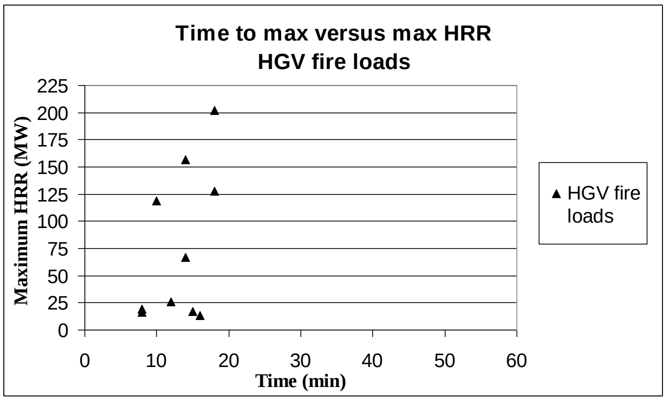


*Figure 3 A summary of time to reach maximum versus maximum HRR for HGV fire loads.*

As can be seen in Table 1, from the NFPA 502 table A.11.5.1 for road tunnels, these values appear to fit well to the results presented earlier. This means the using the NFPA502 table A.11.5.1 [12] for design of SFT is recommended.

*Table 1 Fire Data for Typical Vehicles (from NFPA 502 table A.11.5.1 [12])*

| Vehicles | Peak HRR (MW) | Time to Peak HRR (min) |
|---|---|---|
| Passenger cars | 5-10 | 0-30 |
| Multiple passenger cars (2-4) | 10-20 | 13 – 55 |
| Bus | 20 - 30 | 7 – 10 |
| Heavy Goods Truck | 70 – 200 | 10 – 18 |
| Tanker | 200 – 300 | - |

## VENTILATION SYSTEMS

In case of a fire, the tunnel ventilation system generally needs to change to the fire ventilation mode. The objective of a ventilation system is to control of smoke flow and mitigates the effect of fire and smoke, to aid evacuation and rescue service fighting the fire. To reduce the cost and also simplify the structure construction, the fire ventilation system should always combine with the normal ventilation system. The fire ventilation modes can be categorized into longitudinal ventilation, point extraction ventilation (or semi-transverse), full transverse and combined fire ventilation. Further, special constructions and ventilation systems could be designed to mitigate the effect of fire and smoke flows, such as cross-passages and rescue stations inside the tunnels.

In case of a fire, the longitudinal ventilation system is designed to create a longitudinal flow to prevent the smoke backlayering produced by the fire. Therefore at least upstream of the fire, the tunnel users are relatively safe. However, the tunnel users located downstream of fire are exposed to the fire and smoke flow. In Norway the longitudinal ventilation is used in most road tunnels, both steep and long as well as flat and short. For road tunnels, they are assumed to be able to escape out of the tunnel by driving their vehicles. However, the case could be completely different when they get trapped in a queue and cannot escape. Therefore in such cases, the longitudinal ventilation is not good enough to assure the safety of tunnel users located downstream of the fire.

However, a basic longitudinal ventilation system needs no extra space for ducts but only for the jet fans and thus it provides a very cheap solution for tunnel ventilation. This is the main reason why they are still popular worldwide.

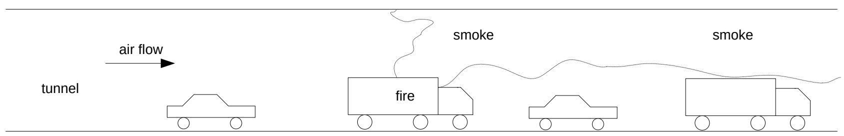


*Figure 4 Fire ventilation in a tunnel with longitudinal ventilation.*

The most important parameters for a tunnel with longitudinal ventilation are the critical velocity and the backlayering length. The critical velocity is defined as the minimum longitudinal ventilation velocity to prevent reverse flow of smoke from a fire in the tunnel (no backlayering).

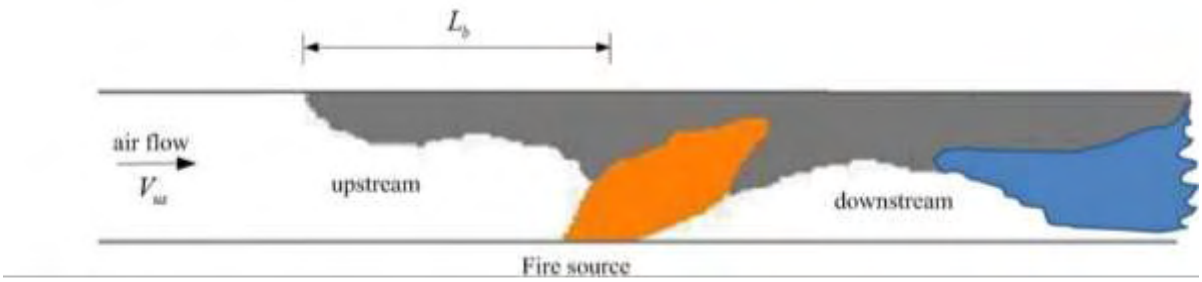


*Figure 5 Fire ventilation in a tunnel with longitudinal ventilation and backlayering.*

The backlayering length, $L_b$, is defined as the length of the reversed smoke flow upstream of the fire when the ventilation velocity is lower than the critical velocity (see figure 5).
In a longitudinally ventilated tunnel, fresh air with a velocity not lower than the critical velocity at the designed heat release rate is expected to prevent smoke reverse flow, which means that the tunnel is free of smoke upstream of the fire site. However, smoke stratification downstream of the fire may not persist, as the ventilation velocity is too high. For a large fire that covers more or less the whole cross section of the tunnel it will be difficult to maintain a stratification, as shown in figure 5.

In steep SFT considerations has to be made if the fire occurs in the sloped part of the tunnel. Depending on if the flow should be reversed or directed against the inclination it is important to take into account the possible resistance due to buoyancy effects. The design fires in Table 1 should be used for the design of the ventilation system in a SFT assuming that no water based fixed fire fighting systems (FFFS) is used. If a FFFS systems is used the design fire should be modified, i.e. lowered from the values given in Table 1. Which value should be used should be based on an engineering analysis.

## WATER BASED FIXED FIRE FIGHTING SYSTEMS (FFFS)

FFFS in tunnels are used to prevent and control fires from developing. These systems can be subdivided into deluge water spray systems and deluge water mist systems, both with and without the use of foam additives. All of these systems have been applied to tunnels, although deluge water based systems without additives represent the vast majority of the installed systems. There are also other type of systems, such as a high expansion foam system (Hi-Ex) and compressed air foam (CAF).

Deluge water spray systems consist of open nozzles/sprinkler heads attached to pipework at the tunnel ceiling or corner. The pipework consists of mains pipes, manifold pipes, feed mains and branch pipes. Water flows into the feed main and branch pipes and discharges from the open sprinklers. The branch piping is divided into deluge zones, typically 25 m to 50 m in length, each served by its own deluge valve. An independent fire detection system that is capable of locating a fire accurately is required, so that the deluge valve serving the zone where the fire is located can be released. Standard water spray nozzles, which typically require a minimum operating pressure of 1.5 - 5 bar, are used and they discharge a uniform pattern of water droplets over the protected area with droplet sizes less than 1 - 2 mm in diameter. The water discharge density over the length of the deluge zone or predefined area is in the range of 6 – 12 mm/min. Tests with fires having potential free burning heat release rates in the order of 25 MW to 140 MW have been undertaken with deluge water spray systems.

The most suitable length of the deluge zones must be based on the width of the tunnel and the capacity of the water supply. Large zones will reduce the number of control valves but place a higher total water demand [21]. The typical application rates and zone sizes can result in flow demands in the range of 7500 to 15000 liters per minute, which can have a significant impact on supply and drainage system requirements [22]. This is important to consider in a SFT as the water has to be pumped back to the mainland. An SFT can be compared to a floating ferry, where water introduced into the systems is always a problem.

There are other more robust options developed by for example the Swedish Traffic Administration. The system consists of a single pipe in the centre line of the tunnel ceiling, fitted with two extended coverage nozzles (large K-factor nozzles) directed horizontally towards each of the tunnel walls. The entire cross section is covered with only one pipe. Long sections of 50 m are used and they are designed for delivering 10 mm/min without the use of any additives to the water. The deluge water spray system is combined with the fire hydrant system, reducing the need of water mains in the tunnel to only one.

The main purpose of the system is to limit the fire size and prevent fire spread during the time of evacuation in congested traffic situations. The system can be manually operated from the Traffic Control Centre based on detection by CCTV, or from the tunnel escape routes where the deluge valves are located. The system also starts automatically if a heat sensing cable detects high temperatures from a fire.

The water mist systems are fundamentally similar to deluge water spray systems, i.e. the pipework consists of a water-filled mains pipe, manifold, deluge valves, dry feed main and branch pipes to which the nozzles are attached. Water mist deluge systems may vary with respect to their working pressures, i.e. low pressure and high pressure systems. The low pressure water mist system is to produce a fog (or mist) of small water droplets at a nozzle pressure of 3-10 bar. The high pressure water mist system produces a fog (or mist) with a mix of different sizes of water droplets at a nozzle pressure of 60-120 bar.

The total water flow rate per 25-m zone for low pressure systems (without additives) is in the range of 221- 683 l/min, and for high pressure systems, 140 - 550 l/min. The discharge rate for low pressure systems is in the range of 1.1 - 3.3 mm/min (l/min/m$^2$) and for high pressure systems 0.5 - 2.3 mm/min. Note that the design application densities are based on a density per unit area of coverage (mm/min). They are sometimes converted to another measures often used when discussing water mist systems, namely a volumetric density expressed as a flow rate per volume ($l/min/m^3$) by simply dividing mm/min by the ceiling height of the tunnel in meters.

The water mist systems use significantly less water than deluge water spray systems. On the other hand they require significantly higher pressures, especially for high pressure water mist systems. As a result, pipes, tanks and pump capacities can be smaller, and the water demand be lowered. The large-scale fire suppression tests described within this paper were intended to simulate a fire in the trailer of a heavy goods freight truck on a ro-ro deck. Typically, the height of these decks is very low and vehicles are tightly packed. Consequently, the nozzles that were tested were positioned quite close to the top of the simulated trailer.

Large-scale fire tests with FFFS have showed satisfactory performance of mitigating fire in heavy goods vehicles. Fire in the fuel array in these tests had the potential to grow to 100 MW or larger if uncontrolled. The FFFS are typically activated when the fire was between 15 and 20 MW and growing. The water mist system prevented the fires from growing to their full potential – generally reducing the peak heat release rates to 50% of their maximum potential.

Arvidson [23] presented very interesting tests carried out in a fire laboratory simulating a ro-ro deck situation on a ferry. The tests really show the difference in using water spray and water mist systems for fires with open cargo (water hit directly the fire) and covered cargo (water hits a cover above the fire). The tests where the fires were **fully exposed to the water spray (no cover)** show that there is a clear relationship between the level of performance and the water application rate. A discharge density of 15 mm/min provided immediate fire suppression, 10 mm/min fire suppression, and 5 mm/min fire control. However, improvements in performance were documented with a higher system operating pressure and associated smaller water droplets, i.e. when 10 mm/min was applied at 4,9 bar instead of 1,2 bar.

The high-pressure water mist system tested provided fire control at a discharge density of 5,8 mm/min, but not to the level that was achieved with the water spray system at 5 mm/min. Tests at 3,75 mm/min and 4,6 mm/min, respectively, provided no fire control and had to be terminated. Arvidson [23] concluded that for a water spray or water mist system to successfully suppress a fire in ordinary combustibles, the droplets must be capable of penetrating the fire plume to reach the burning fuel surface. In SFT, however, the goal may be to protect the construction through thermal management, i.e. is cooling the structure and to prevent fire spread between vehicles.

For the fires where the **fire was shielded from direct water application (cover)**, the tested systems had a limited effect on the total heat release rate and the associated total energy, as almost all combustible material was consumed in the tests. The most efficient reduction of the convective heat release rate and the associated convective energy during the water spray system tests was demonstrated with an application rate of 10 mm/min at an operating pressure of 4,9 bar. The tests also showed that if the roof of a real vehicle burns through, water from the water spray system will have access to the fire and the performance will be significantly improved, even if the burn-through area is small.

The high-pressure water mist system provided an improved reduction of the convective heat release rate and the associated convective energy as compared to the water spray system of the shielded fire. However, no improved reduction of the total heat release rate and the associated total energy was documented, i.e., the ability to reduce the actual heat release rate was not enhanced. The effect of entrainment of water droplets from the section upstream of the fire is not represented in the test setup described here. In high ventilation flow this may become very important.

Using a water mist system with lower total water demand may be an alternative for SFT if the goal is to protect the construction (good thermal management). On the other hand, using 10 mm/min deluge water spray system may obtain a better control of the fire development, especially if activated early. The consideration of total amount of water becomes a leading question due to Archimedes principle on floating things.

## PASSIVE FIRE PROTECTION

The original intent of the tunnel lining system was not as a fire protection system but primarily to act as drip shields and to provide a visual aesthetic. Today, the main design consideration for tunnel lining systems is how much thermal protection they can provide for the primary tunnel structure. Modern tunnels are generally made of concrete linings in a rock, but what solutions will be used in future SFT is not clear to the authors. Concrete has excellent structural properties at ambient temperatures, but may fail when subjected to high temperatures and rapid heating as in fire conditions. The main problem is spalling of the concrete. This may risk the construction as the risk for leakage becomes evident. Spalling is an explosive failure which commonly occurs when concrete is rapidly heated and steep temperature gradients are formed in the bulk material, leading to thermal stresses and high pore pressures in the material.

Passive protection systems for tunnels are generally either a bolted on panel system or a sprayed on mortar system. Research is on-going into the performance in fire of concrete containing various forms of fibres, which have been claimed to resist spalling and improve the fire resisting properties of concrete. If a suitable additive can be identified, this may have a considerable impact on the way that concrete tunnel structures are made. For example good results are found from full scale fire tests on shot concrete with added PP-fiber (two hour RWS fire).

There is an increasing tendency to consider fire suppression systems as structural protection systems, and trade-offs have been widely discussed. The main reason is reducing or removing the requirement for a passive thermal barrier system if a suppression system is being installed. In a SFT tunnel there is no doubt that if installing a FFFS this should be considered. This can reduce the total investment cost considerably. The main goal with a SFT is that no hole should occur; otherwise the SFT as an entire construction will collapse.

## CONCLUSION

In the paper a sum up of the most recent research that have been published on large-scale fire tests in tunnels using different road vehicles, ventilation systems, active mitigation systems and different passive construction protection, is presented. Aspects from this knowledge are transformed and discussed for SFT, in order to give some guidelines on fire protection for future constructions of such systems.

Design fires for ventilation systems in a SFT without FFFS should be obtained from NFPA502. If FFFS is to be considered, two aspects should be focused on, the total water demand, as SFT are to be regarded as floating ships, and a trade of on passive fire protection should be considered as it may reduce weight. If a FFFS systems is used the design fire should be modified, i.e. lowered from the values given in Table 1. Which value should be used should be based on an engineering analysis. In order to secure the safety of evacuees a twin tube system is proposed with regular cross-passages of 100 m or 150 m distance, depending on the fire safety engineering analysis using a ultra fast t-square fire growth rate curve. There is a need for further research on many aspects of the fire safety in SFT. This concerns both fire ventilation, active fire protection, evacuation and passive fire protection.

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# WHAT IS THE REAL COST OF A RESCUE TUNNEL?

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## ABSTRACT

*No doubt, construction of a rescue tunnel is an extra cost to subsea road and railway tunnels. However, in addition to the safety, a rescue tunnel opens for extra opportunities both during construction as well as operation of the main tunnel. In particular during excavation of subsea tunnels, the rescue tunnel can serve as transport tunnel during excavation, opening the main tunnel for an early access for the permanent installations such as road pavement, drain system, electrical installations, ventilation and safety equipment etc. Furthermore, with an optimal cross section, the construction time of the rescue tunnel might be minimised giving early access to the real challenges and time consuming processes like crossing fault zones and other complex geological features in particular at the fjord bottom. In some cases there might also be possible to reduce the tunnel length and/or tunnel inclination by reducing the rock overburden and accepting more heavy rock support, maybe ground freezing as well. The heavy rock support might be executed from the rescue tunnel, removing the time consuming rock support from the critical path on the time schedule for the project. During the operation of the tunnel the benefits with a rescue tunnel are obvious with respect to logistics, less traffic interruptions and H&S matters for the maintenance personnel - and last, but not least, the safety for users and rescue personnel in case of an accident. Further optimising and LCC analysis are required to get the correct answer and real picture of the feasibility of the concept.*


## INTRODUCTION

The concept with parallel rescue tunnels is of interest for long subsea single tube road and railway tunnels where there is no possibility for separate adits or access tunnels.
The rescue tunnels will be of great use and have great value both during construction and operation stages as well as for upgrading and rehabilitation of subsea tunnels.
During the construction stage, the rescue tunnels represent an extra cost in the project. Considering the benefits during operation and in case of accidents in the tunnel, the picture might be different. In addition, there are also several opportunities during the construction stage if the rescue tunnel is utilized as transport tunnel for muck and equipment, and as pilot tunnel through geological challenges and features like fault zones and sections with shallow rock overburden.

## WHEN ARE RESQUE TUNNELS REQUIRED?

Requirements for emergency exits in Norwegian road and railway tunnels are given in the Norwegian Public Roads Administration (NPRA), Manual No. 021 for road tunnels and the safety regulations for railway tunnels. For road tunnels in class D (average annual daily traffic, AADT >

7500 vehicles), emergency exits are required for each 500 m in tunnels longer than 500 m (Figure 1). Since the tunnel openings in each end represent an emergency exit, extra emergency exits are required for tunnels longer than 1 km.

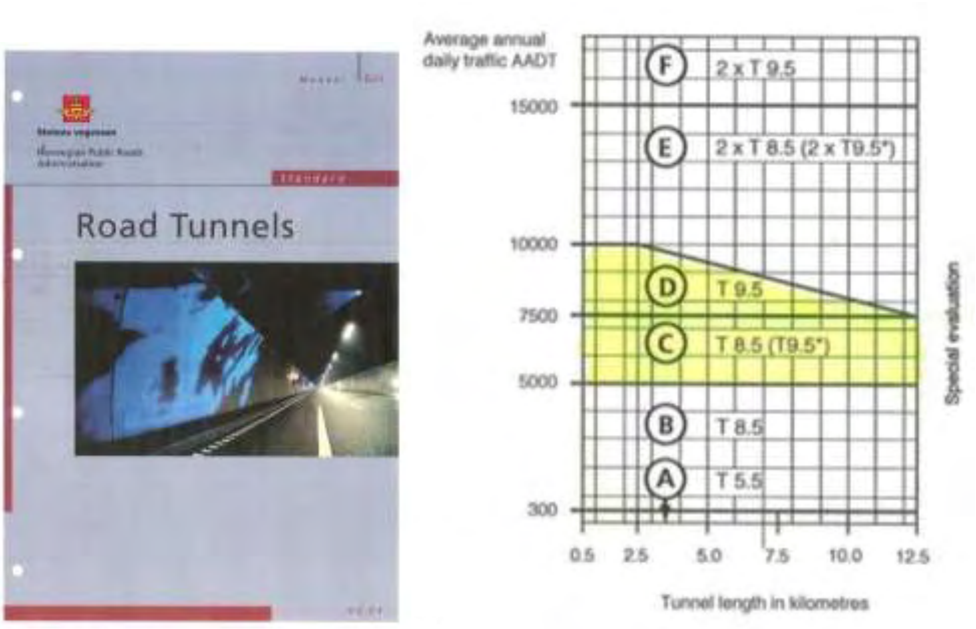


*Figure 1 Tunnel Classes from the NPRA Manual No. 021*

If risk analysis proves an acceptable safety without emergency exits, this might be acceptable for road tunnels in class C with length up to 10 km. For road tunnels in class D, emergency exits are required for each 500 m according to Manual No. 021 (Figure 2).

*Table 5.1 Measures to ensure minimum safety level in tunnels*

| ● Obligatory<br>○ Evaluated | **TUNNEL CATEGORY** | | | | | | **NOTES** |
|---|---|---|---|---|---|---|---|
| | A | B | C | D | E | F | |
| SAFETY MEASURES | | | | | | | |
| Emergency lay-bys | | ● | ● | ● | ● | ● | See chapter 4 Geometrical design |
| Turning points | | ● | ● | ● | | | See chapter 4 Geometrical design |
| Escape possibility by foot | | | | | ● | ● | Every 250 m (see 4.7) |
| Emergency exits | | | | ● | | | Requires either an exit to open air or a separate evacuation tunnel with cross connections for tunnel category D (number of vehicles per lane > 7500) and for tunnels longer than 10 km tunnel category C (see 5.1). Exits for every 500 m (see 4.7) |

*Figure 2 Requirements for emergency exits (Translated from Manual No. 021)*

For railway tunnels, distance between emergency exits must be less than 1000 m which means that emergency exits are required for all railway tunnels longer than 2000 m.

## PARALLEL RESQUE TUNNELS OPEN SEVERAL OPPORTUNITIES

Rescue tunnels along and parallel to the main tunnel opens several opportunities both during construction, operation and emergency operations in case of accidents in the tunnel. The opportunities are presented in the following sections.

### The Construction Stage

For subsea tunnels, the construction stage will be long since there is no separate access possibility along the tunnel route except from both ends. The excavation will be «single face» construction from both sides of the strait or fjord. With time consuming operations like grouting ahead of the tunnel face, excavation and permanent rock support, the construction time before breakthrough will be long.
By using the future rescue tunnel as a pilot and construction tunnel with high priority on the progress, the rescue/construction tunnel opens for several opportunities like:

- *Adits from the rescue tunnel to the main tunnel opens for excavation from two extra tunnel faces (Figure 3)*
- *If the rescue tunnel is used for transport during construction, permanent installations can start in sections ready for installations in the main tunnel (Figure 4)*
- *With and optimal cross section on the rescue tunnel, the rescue tunnel might be used as pilot tunnel through fault zones (Figure 5)*

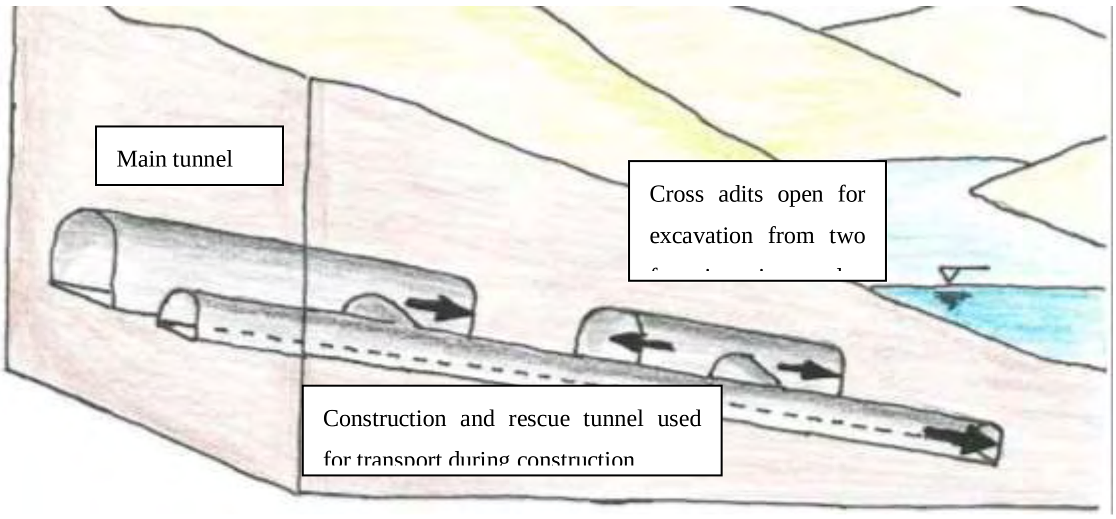


*Figure 3 Cross connecting tunnels open for excavation from extra tunnel faces. Transport of tunnel muck goes in the construction/pilot tunnel*

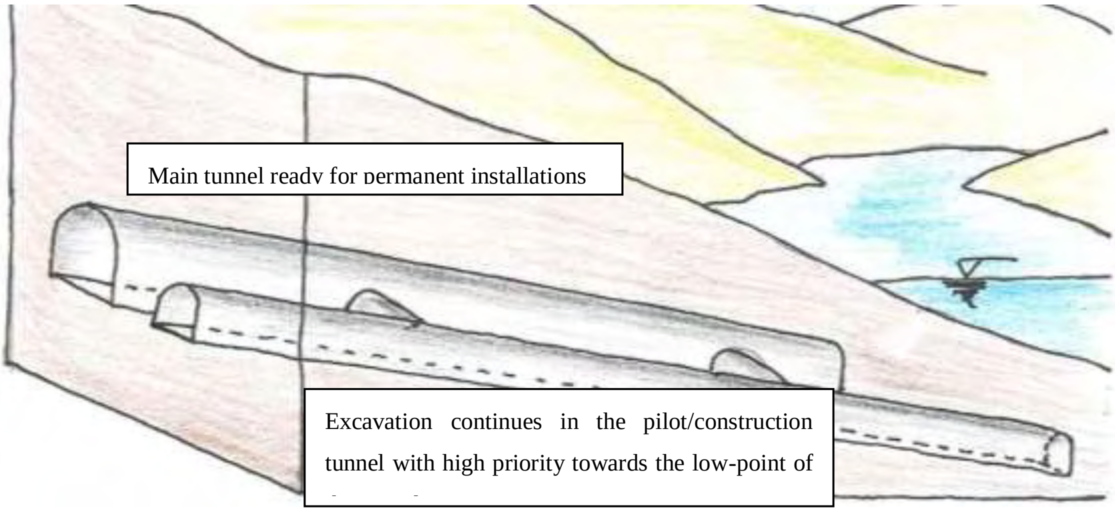


*Figure 4 Sections of the main tunnel prepared and ready for permanent installations while excavation continue in the main and construction tunnels towards low-point*

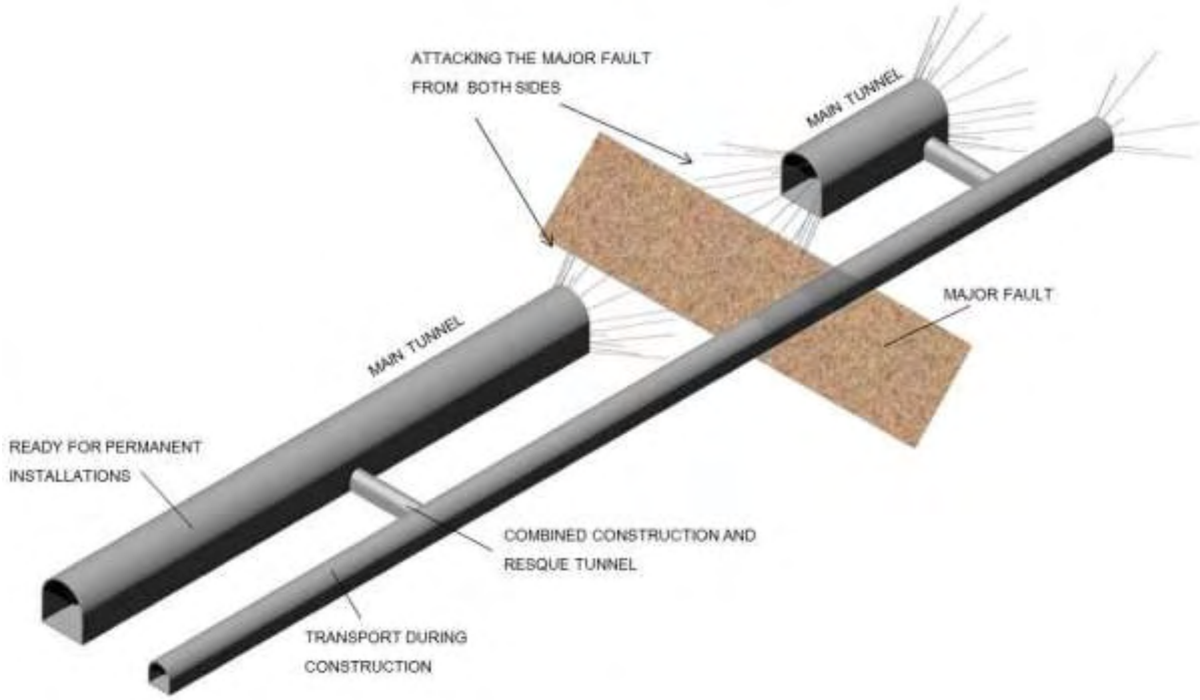


*Figure 5 Priority sequence for excavation of the pilot tunnel through the major fault and preparation for the main tunnel from both sides of the major fault*

The concept of a separate construction tunnel is not new. The idea was considered during a feasibility study of crossing the Hjeltefjord in Western Norway with a 9.5 km long pipeline tunnel for oil and gas from the North Sea (Figure 6). The study, which was a part of the “Gullpipe Transportation System” was carried out in 1983 by the Norwegian consulting company Grøner AS for Statoil as the Client. From seismic investigations and profiling, a 1 km wide low velocity zone, weak or soft rock, was identified (Figure 6). With early operation and financial income from the pipeline system as a goal, there was great focus on high performance on tunneling and early access for permanent installations.

*Table 1 Key data from the Hjeltefjord feasibility study*

| Option no. | Tunnel layout | Construction method | Construction time weeks (1983) |
|---|---|---|---|
| 1 | Single tunnel | Drill and Blast | 142 |
| 2 | Double bore from west side (left in figure 6) | Drill and blast in both slopes, TBM through the soft rock through the middle part (soft rock) | 136 |
| 3 | Double bore in both slopes | Drill and Blast in both slopes, TBM through the middle part (soft rock) | - |
| 4 | Single tunnel | TBM from one end | - |
| 5 | Single tunnel | TBM from both ends | 122 |
| 6 | Single tunnel | Drill and blast from both ends, TBM thorough the middle part (soft rock) | 150 |

For the tunnel excavation 6 different alternatives were assessed (Table 1): In options 2 and 3, Table 1, separate tunnel for transport of tunnel muck from one or both sides was evaluated in order to release the pipeline tunnel from transport and other construction logistics and prepare for permanent installations as soon as possible. As shown in table 1, single tube TBM tunnel from both sides has, not surprisingly, the shortest construction time with 122 weeks of construction, while option 2 with double bore from west side is a good number two with 136 weeks of construction time (capacity figures from 1983).

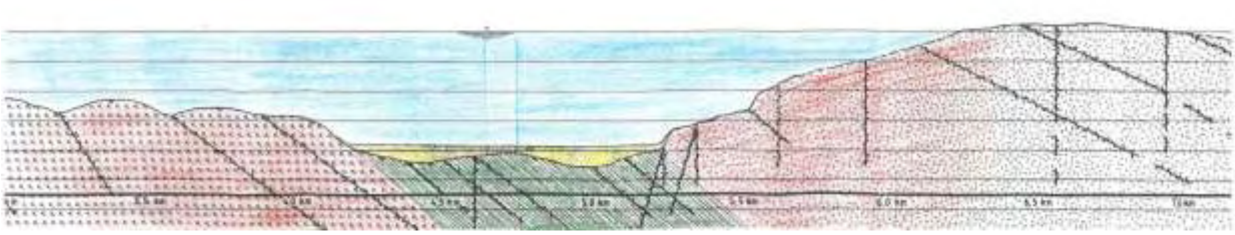

*Figure 6Profile of the Hjeltefjorden pipeline tunnel*

Another example is the 4.6 km long Storberget railway tunnel in Vestfold, Southern Norway, presently under construction. During the design stage, there were three separate emergency exits and rescue tunnels, each of them branching off in southern direction. One of these had the purpose as adit for construction of the main tunnel (Figure 7). By rotating two of these in east-west direction and parallel to the main tunnel, these rescue tunnels can serve as transport tunnels during the construction with adit each 500 m opening for several construction faces in the main tunnel. I addition, the access road system, number of tunnel openings and portals has been reduced to one instead of three. Furthermore, cost for road maintenance has considerably been reduced.

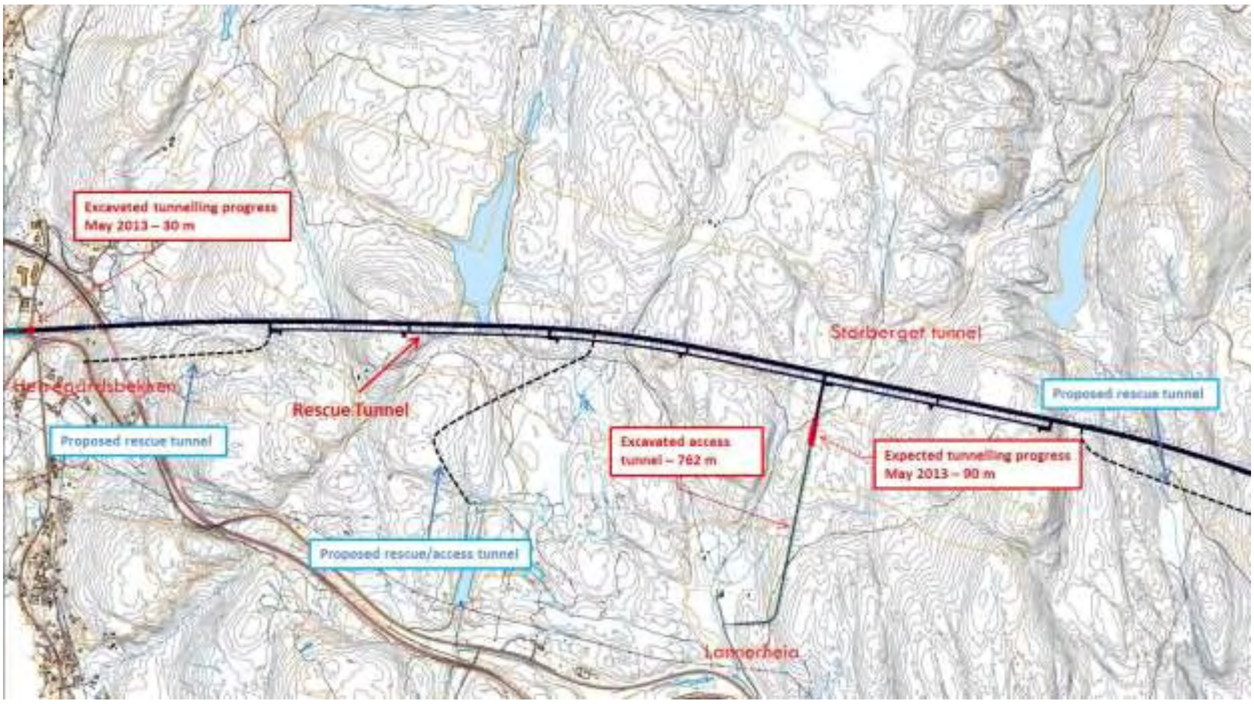


*Figure 7The Storberget railway tunnel*

**Operational stage**

A separate operation and rescue tunnel has obvious benefits during the operational stage as per example:

- *Access and storage of equipment during maintenance*
- *Possibilities for upgrading according to new standards and safety requirements*
- *As bypass tunnel during maintenance and upgrading*
- *Tunnel for cables and service equipment*
- *Improved overall safety of the tunnel*
- *Improved H&S aspects for personnel during maintenance, evacuation and rescue*
- *Potential income from other users*
- *The tunnel can be used by pedestrians and cyclists*

As an example, upgrading of the single tube subsea road tunnels near Ålesund, Northwest Norway, might be mentioned. During upgrading of the tunnels, closing of traffic was required for a long time due to safety requirements. If a separate maintenance and rescue tunnel had been available, the rescue tunnel could have been used as bypass and storage, and sections of the main tunnel could have been open for traffic during rehabilitation.

**OTHER ASPECTS**

There is a continuous development and stronger demand for improved safety in road and railway tunnels, as per example reduced distance between emergency exits and rescue/evacuation openings. If a parallel operation and rescue tunnel exists, it is very simple to establish additional cross connecting tunnels.

The new EU requirements have introduced a maximum of 5% inclination in road tunnels. In many of the Norwegian subsea tunnels, the inclination is steeper. Furthermore, as a rule of thumb, a minimum rock overburden of 50 m is required in Norwegian subsea tunnels. For subsea tunnels, the EU requirements will cause longer tunnels and considerable increase of the construction and maintenance costs (Figure 8).

If a future rescue tunnel was used as pilot tunnel during construction, a reduction of the rock overburden to 30 m could be considered by compensating with heavy rock support in the critical sections.

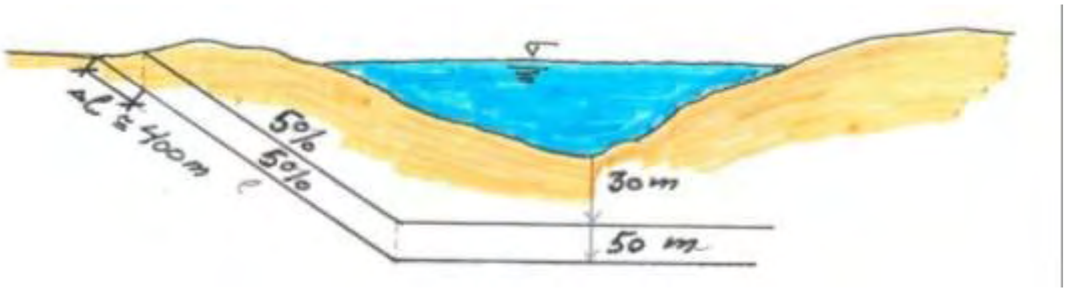


*Figure 8Reduced tunnel length by reduction of rock overburden from 50 m to 30 m and EU requirements of 5% inclination*

The reduction of tunnel length in each end is approximately 400 m by reducing the rock overburden from 50 m to 30 m (Figure 8). With a unit cost of €20.000 – 25.000 per meter tunnel, the total savings will be approximately €8 – 10 million in each end of the tunnel, or a total of €16 – 20 million in cost savings. By investing a part of the savings in crossing the deepest part with heavy rock support, ground freezing if required, the reduced rock overburden from 50 to 30 m might be technically and economically feasible.

The technical execution of a subsea tunnel with reduced rock overburden, crossing fault zone is illustrated in Figure 5. The construction sequence might be as follows:

- *The pilot tunnel is excavated with high priority and performance down to the critical sections and sections with overburden less than 50 m*
- *Preparatory work such as probe drillings, grouting and support ahead of the tunnel face, ground freezing if necessary, is performed in critical sections*
- *The pilot tunnel is excavated through the critical sections*
- *Preparatory support is performed for the main tunnel from both sides of the critical section*
- *In the meantime, excavation of the main tunnel can continue on the other side of the critical section by using the pilot (and future rescue) tunnel for transport of tunnel muck and other logistics*

In Norway, the Frøya tunnel and the Oslofjord tunnel are good examples on major geological challenges and features where the concept of separate construction of a future rescue tunnel could have been used during the construction stage and crossing of the complex geology at the deepest point.

As mentioned, operation and rescue tunnels can be used for other purposes such as cables and other infrastructure, pedestrians and bicyclers. By renting out to potential user, the operation and rescue tunnel represents an extra income to the project.

## CONCLUSION

No doubt, construction of a separate operation and rescue tunnel, represent an extra cost for the project. With a T5.5 cross section (cross section area 43 $m^2$) the unit cost will be around €4000-5000 per meter tunnel, depending on geological conditions, extent and requirements for water and frost protection.

By optimizing the cross section and construction of an operation and rescue tunnel as well as effectively utilizing the tunnel as pilot and transport tunnel during construction, both time and cost might be saved during construction. Another potential benefit is to reduce the rock overburden, compensated by increased rock support, the tunnel length might be reduced and the EU requirements of max 5% inclination could be satisfied.

The benefits during operation, maintenance, upgrading of standards and safety as well as the overall safety with ventilation, evacuation and rescue aspects in case of accidents are obvious if a parallel rescue tunnel exists.

Fire accidents in road and railway tunnels are among the worst case scenarios (Figure 9).

*Figure 9Fire accident in the Seljestad road tunnel West Norway*

Fire situations in tunnels has been described by rescued people, nearly no visibility, a mixture of pedestrians and vehicles, both in panic to escape, lost feeling of direction and orientation, dense smoke etc. Benefits in emergency exit and rescue tunnels in such cases cannot be evaluated in money but less injured people and saved human lives.

To calculate the construction costs for subsea tunnels, there is a lot of information and experience available. To calculate cost savings and benefits during operation and maintenance, tunnels safety in general and safety during rescue operations is more complicated if not impossible.

The design lifetime of a modern road tunnel is 100 years. Experience shows that rehabilitation of road tunnels starts at 20 years of operation. This is not only due to corrosion, wear and tear, but also development and stronger requirements on improved standards, tunnel comfort and safety. Today, there are no exact figures available for the real cost savings with a rescue tunnel. With 100 years design lifetime, the author is however convinced that a separate operation and rescue tunnel represents cost savings in the long run of long and single tube subsea tunnels.

# THE BJORØY TUNNEL – BLASTING ON THE SEABED ABOVE THE TUNNEL RUNNING THROUGH THE "BJORØYZONE"(JURASSIC SEDIMENTS IN THE FRACTURE ZONE)

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## ABSTRACT

*The Bjorøy tunnel is a subsea road tunnel, situated near by Bergen city, on the west coast of Norway. In Norway we have around 32 subsea road tunnels and more will be built in coming years. After the tragic accident, when the ship Rockness hit a reef just above the Bjorøy tunnel, and several people were killed, it was decided that this reef and surrounding rock should be removed. The responsible authority for the work was Kystverket (The Norwegian Coastal Administration (NCA)) and the work was done by a Finnish contractor; Wasa Dredging Ab.*
*Totally 65 000 tfm3 rock was removed on a 1200m$^2$ seabed area and carried away by boat, to be used for widening a harbour 10km away from the site. The total work was completed in 5 weeks' time.*
*Briefly said, the Bjorøy zone is about 10 to 15 m wide where open vertical cracks are filled with sand and silt down to the tunnel level, approximately 70 m below sea level. The main zone of sand and silt has a thickness of about 2 m, but with fragments of rock of Jurassic age. There are few cracks in the zone and elsewhere in the rock mass. Because 10 – 15 m of the tunnel is built through the zone and since only a 1 m thick concrete lining was cast around the whole profile when the tunnel was built in 1994-1996, it was decided to take special precautions during the work to avoid a total breakdown of the tunnel. The job was done through cooperation between the NCA, the Finnish contractor, NPRA, Fjell municipality, Bergen municipality, the fire brigade and the Norwegian Navy.*

*Special arrangements were made in order to be in the forefront if critical conditions should arise.*
- *The maximum vibrations from the blasting - not to be above 25mm/sek.*
- *Checking the tunnel profile after each round - rock and concrete*
- *Monitoring the leakage into the tunnel*
- *A ferry was kept on hold in case of emergency*
- *Boats were provided to transport people to and from the island in case the tunnel had to be closed.*
- *The fire brigade had a fire-engine with all necessary equipment stationed on the island throughout the blasting period.*

## 1. INTRODUCTION

The first Norwegian subsea tunnel was opened in 1983. Subsequently 31 other subsea tunnels have been built. As a result, in the spring of 2013 a total of 32 tunnels with a total length of more than 150 km are open to traffic. Several other subsea tunnels are also in the process of being planned, including tunnels of up to 27 kilometres length.

Generally speaking, building costs for subsea tunnels have been reduced over the years. However, costs vary a great deal from project to project. Operation and maintenance costs also vary considerably. Costs for reinvestment and equipment are particularly high. Water ingress has diminished over time, so that the need for pumping leakage water has been reduced.


*Fig.1.Map showing the planning area marked in red.*

*Fig2. The entrance to the Bjorøy tunnel.*

## 2. FACTS ABOUT THE BJORØY TUNNEL

a. Opened in 1996.
b. Traffic in 2012: 1500 cars per year - 7% heavy heavy trucks.
c. Depth under sea level: 80 m.
d. Length: 2000 m.
e. Water leakage: 24 hours messuring: Normally 600 l/min. In rainy seasons: 900 l/m.
f. The Bjorøy zone with Jurassic sediments.
g. The pumping chamber is laying 4 meters under the bottom of the tunnel.

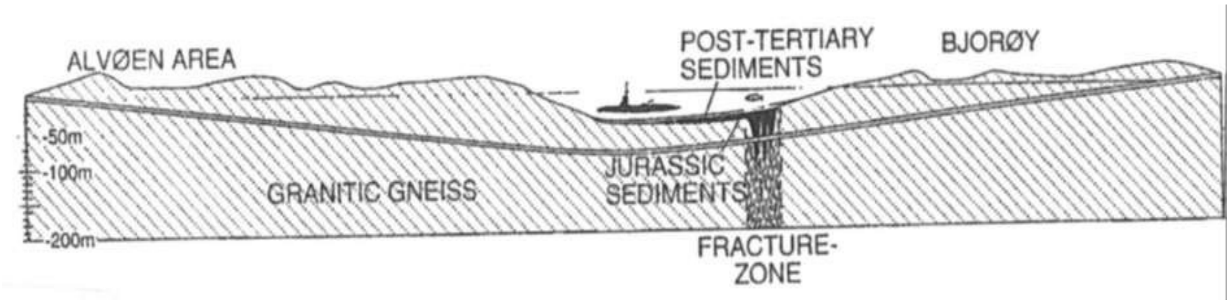


Fig.3. Longitudinal section of the Bjorøy tunnel

## 3. THE CAUSE OF THE WORK

After the tragic accident, when the ship Rockness hit a reef just above the Bjorøy tunnel, and several people were killed, it was decided that this reef and surrounding rock should be removed to improve the fairway in the strait called Vatlestraumen.

Construction work was carried out in the period of June – December 2012. To improve navigational conditions in the fairway of Vatlestraumen, The Norwegian Coastal Administration removed the underwater reef at Revskolten lighthouse. The responsible authority for the work was Kystverket (The Norwegian Coastal Administration (NCA)) and the work was carried out by a Finnish contractor; Wasa Dredging Ab.

Totally 65 000 tfm$^3$ rock was removed on a 1200 m$^2$ seabed and carried away by boat, to be used for widening a harbor 10 km away from the site. The total work was completed in 5 weeks' time. Briefly said, the Bjorøy zone is about 10 to 15 m wide where open vertical cracks are filled with sand and silt down to the tunnel level, approximately 70 m below sea level. The main zone of sand and silt has a thickness of about 2 m, but with fragments of rock of Jurassic age. There are few cracks both in the zone and elsewhere in the rock mass. Because 10 – 15 m of the tunnel is built through the zone and since only a 1 m thick concrete lining was cast around the whole profile when the tunnel was built in 1994-1996, it was decided to take special precautions during the work to avoid a total breakdown of the tunnel.

## 4. CHALLENGES AND ISSUES

Multiconsult AS was commissioned to prepare a ROS-analysis of the project. The area is located in / up the fairway in Vatlestrømmen and the task was to increase the safety for ship traffic. The project aim was to obtain greater sailing depths in a wider corridor for safe ship passages.

The following topics are considered to represent the greatest risks:
• Establishment of a rig site and construction phase
• Blasting under water

Fig.4.The vibration Meters

• Vibrations
  Limits for vibration from blasting according to NS8140-2012 was for an area in the Bjorøy tunnel set to v = 25 mm / s.
• Stability in Bjorøy tunnel
• Traffic in the tunnel Bjorøy
• Emergency vehicles
• Transportation of the excavated rock through an area with numerous ship passings.

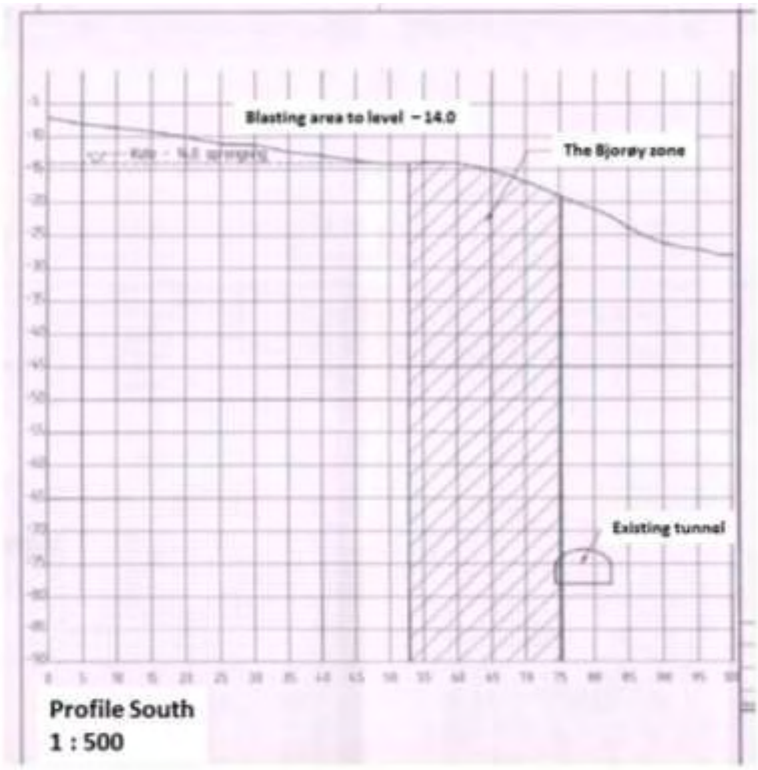


*Fig.5 The distance from the blasting area to the tunnel*

## 5. FACTS ABOUT THE WORK

a. Risk Analyses was done for
   i. The Bjorøy tunnel
   ii. The Bjorøy society
   iii. The blasting on the seabed
b. Safety evaluations for both the tunnel and the society on the Bjorøy Island was performed.
c. Traffic regulations through the tunnel was implemented when doing the blastings.
d. Traffic regulations on the sea through the Vatlestraumen were implemented when the work was carried out.
e. Plan
   i. The drilling, blast, transport of stone
   ii. The safety in the tunnel
   iii. The safety for the population at Bjorøy
f. Execution
   i. The drilling, blast, transport of stone was done during 5 weeks
   ii. Only stop in traffic on sea and in the tunnel, when blasting and inspecting the tunnel – 30 – 60 minutes 2 times a day.
   iii. No other problems for the population at Bjorøy.
g. Results
   i. The blasting and all other work was done according to the demand and the plan.

*Fig.6. The excavating machine.*

*Fig.7. The drilling rig and the excavator.*

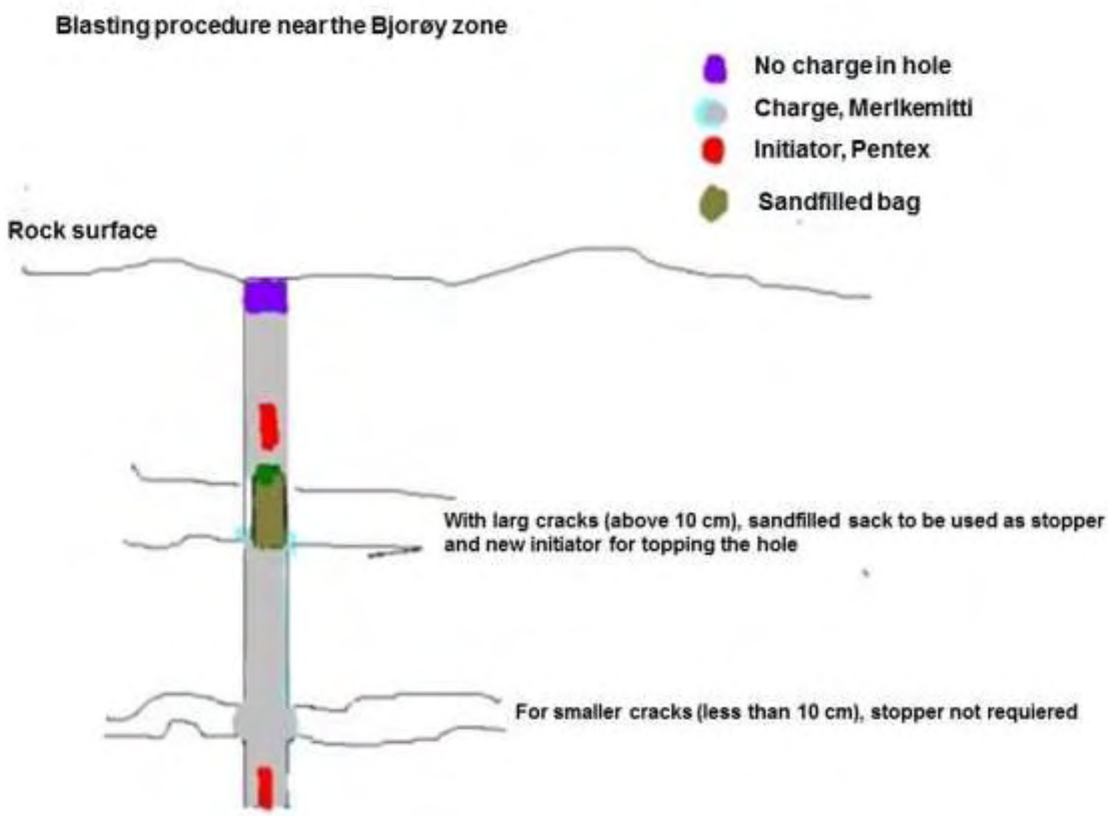


*Fig.8. Description of blasting procedure*

## 6. CONSTRUCTION WORK

Revskolten reef was dredged to a depth of 14 metres, and 65.0000 cubic metres of rock were removed from an area of 12.000 square metres. After the construction work was completed, the new sailing width was increased to 250 metres. The construction work area was temporarily marked by buoys.

During the work period, dredged mass was transported on a regular basis by barge to the landfill area at Kleppestø, located approximately 5 nautical miles north of Vatlestraumen.

Vatlestraumen was open for commercial traffic in the construction period, but was closed in shorter periods during the blasting work. The blasting work was taking place between 10.00 and 14.00 hours. Fedje VTS was providing detailed information on blast schedules to incoming commercial traffic, so that vessels could choose an alternative route if needed.

Prior to blasting, a guard boat was patrolling the area to ensure that there was no leisure traffic in the area. Vessels had to take notice that the fairway would become considerably narrower during the construction work. Vessels that were en route in the waters should proceed with caution and follow navigational markings in the area.

A navigational warning of the construction work was issued via the national navigational warning system NAVCO and Notices to Mariners (Efs). Vessels had to continuously monitor and maintain listening watch on VHF channel 16. All vessels sailing in Vatlestraumen should proceed with caution, and follow navigational markings and information provided by Fedje VTS.

**Tunnel inspection in connection with the blast near Revtangen lantern.**

NCA performed blasting at Revtangen lantern in Vatlestrumen to make the sailing lane deeper in a local area. The site is close Bjorøy tunnel, with the shortest distance of approx. 60 meters. Blasting and possible consequences for Bjorøy tunnel was considered by NGI(Norwegian Geotechnical Institute) in a report dated 16 September 2008. Multiconsult has performed a risk and vulnerability analysis for the project documented in a report dated 14 January 2011.

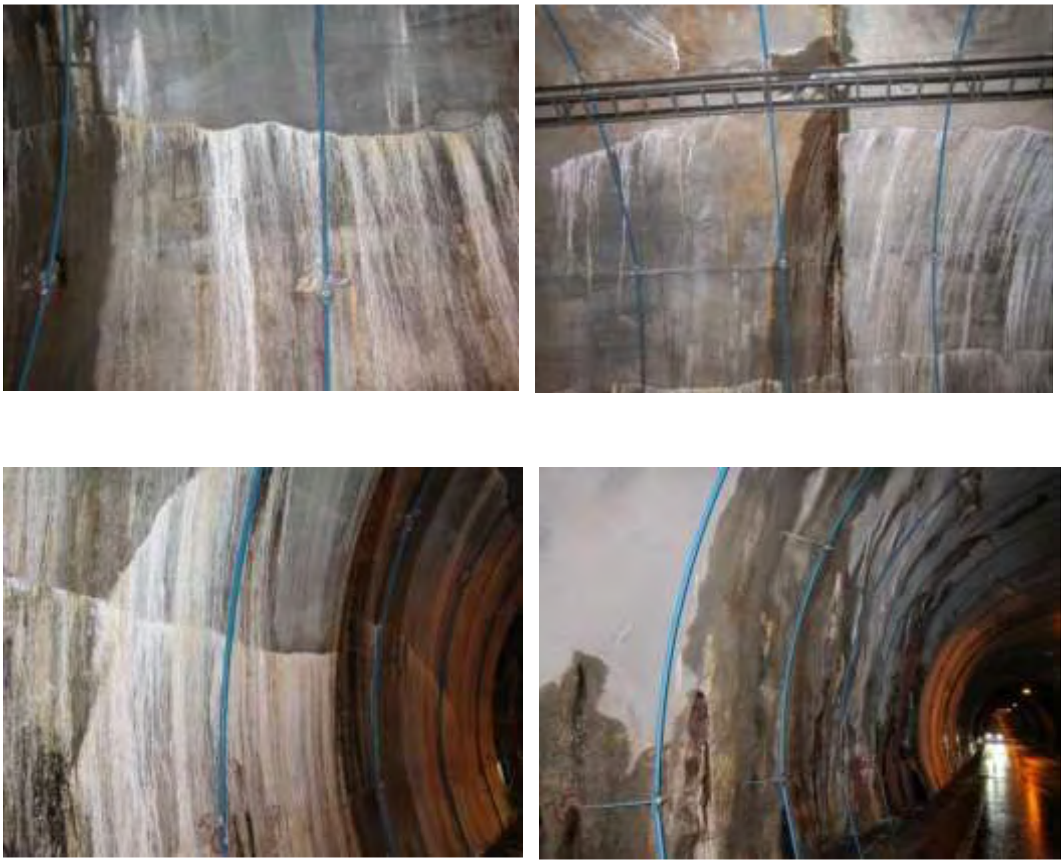

*Fig.9. The concreted Bjorøyzone showing the leakage of seawater and earlier injections*

In line with recommendations the blasting was monitored with regular inspections by an engineering geologist. Quick visual inspections were performed after each round. In addition, a thorough weekly inspection was carried out. As a follow-up the WG TUNNELSEALING was dismantled at the most critical section so that any changes in the stability conditions / leakage ratio could easily be observed. A concentrated single leak (approximately 10-15 l / min) was recorded behind the WG TUNNELSEALING about 50 meters past the safety lining on the Bjorøy side. After the initial inflow of water no increase in leakage levels have been observed of either total leakage measured at the pumping station, or at visually inspected leakage points.

The WG TUNNELSEALING was dismantled for a distance of approximately 70 meters on both sides of the lining. There was no visible new damage in the lining in the form of cracks or fissures. Some leakage was detected in some of the construction joints. Efflorescence, probably of carbonate could be seen along old cracks in the lining. On both sides of the concrete lining reinforcement measures had been implemented with bolts and shotcrete. In one area the upper part of the lining was secured with reinforced shotcrete arches (estimated thickness of 25 cm). The lower part was only secured with bolts and shotcrete with an estimated thickness of 8-10 cm.

No significant new damages (cracks / fissures) or signs of deformation in the sprayed concrete were observed. In the lower part of the lining a limited area was detected where contact with the rock mass was missing. However, there were no signs of cracks or fissures in this area, and there are no reinforcing bolts here. It was therefore assumed that the missing contact was not a recent development. Some precipitates were observed in the sprayed areas, both white, presumably carbonates and brown precipitates of suspected iron compounds.

Primarily CombiCoat bolts seem to have been applied in the bolted area. Visible bolt ends, supporting plates, nuts and washers do not yet appear to be significantly corroded. It appears that some holes have been drilled without inserting bolts (for reasons unknown). In some of these holes some leakage has been observed. There were also some other leakage points, both on the upper and lower part of the safety lining.

The need for closing the tunnel was only during blasting operations and for immediate inspection after each blast. One estimate was that this would take from 30-60 minutes each time. During the planning stage it was considered that the blasting operations would not influence the tunnel in a harmful way. The brief closures would therefore have little consequences for users. If, on the other hand, the tunnel would have to be closed for a long time for unexpected reasons, the consequences could be disastrous.


*Fig.10.Ship passing the aerea of blasting in Vatlestraumen.*

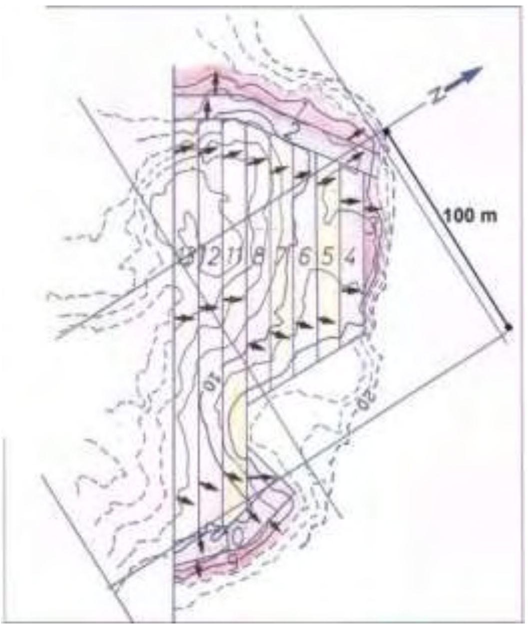


*Fig.11. The surface area at water depths between 2 and 4 m is in the range l 000 m$^2$. Water depths down to 4 m represent areas with approximately 2000 m$^2$.*

## 7. CONCLUSION

The blasting work has not led to detectable changes in leakage conditions or stability conditions in the tunnel. The implemented security measures appear to be intact. This applies both to the cast lining, shotcrete and bolts.

It was agreed that the consultant should take samples of the linings for property analyses in order to investigate the quality of the shotcrete and concrete lining in the critical area before the WG-TUNNELSEALING was reinstalled. The samples for measuring the compressive strength for both the concrete and shotcrete proved to be satisfactory. A check on the thickness of sprayed concrete was also recommended.

It may be possible to seal some of the concentrated leakage by injection of epoxy or the like, but how much improvement can be achieved is uncertain. Probably it will be marginal relative to the total leakage. Still this could possibly be considered as test.

Supplementary security measures were considered if damage was found, or changes observed in connection with the remaining blasting. So far no damage has been detected, so the blasting may be considered as successfully executed.

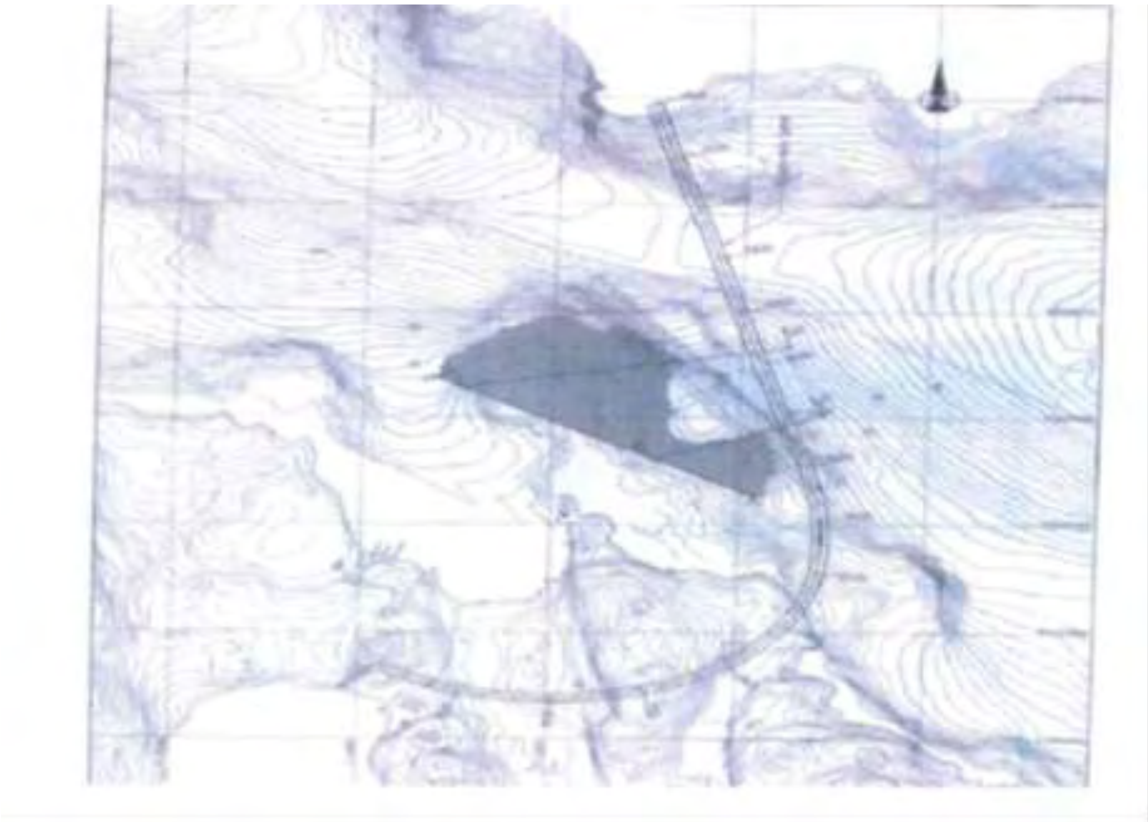

*Fig.12. The Bjorøytunnel near to the blasting aerea.*

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# BRIDGE CREATES NEW OPPORTUNITIES FOR FOSEN AND TRONDHEIM

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## ABSTRACT

*The population in the Trondheim urban area has grown by 28% over the last 20 years to approximately 200,000 inhabitants. During the same period, the population of Fosen has remained stable at 25,000. There is less distance between Trondheim and the surrounding areas in Fosen than the distance to the growing hinterland of Trondheim - Stjørdal, Malvik, Melhus, Skaun, and Orkdal. We argue that the main difference lies in the road versus the fjord and ferry connection and that consequently there is a correlation between transport time, transport mode, economic development, job creation and population growth.*

*Trondheim has had and still has a strategy to densify urban development within the existing urban structure. It has worked until now, particularly with regard to housing, but its potential is limited. Building in new areas is in conflict with the protection of valuable outdoor or farming areas. Trondheim is located far north and much of the land is north facing. It affects the quality of the land used for residential purposes. A bridge to Fosen will provide available building sites relatively free of conflicts and situated 20 to 30 km from the city center. The available capacity is of the order of several ten-thousands of residents. And important, is largely south-facing with sea views.*

*Trondheim lacks industrial areas close to the fjord or sea. This has made industrial development here nearly impossible while industry is expanding rapidly in the other larger coastal cities in Norway like Bergen and Stavanger. With a short distance to Fosen, Trondheim will also have access to areas with similar qualities as the Bergen connected expanding industry at Sotra.*

*Today 1,200 workers commute from Fosen to Trondheim, and 300 in the opposite direction. These are low numbers compared to the distances and the opportunities within respective labor markets. Increased commuting will increase labor productivity in both areas, and will thus contribute to the local and regional economy. This will also be reflected in the increased value of the properties in Fosen, but will, perhaps, have the opposite effect in Trondheim, at least in the short run.*

## THE VALUE OF A BRIDGE

Many bridges have been built in Norway during the last fifty years, mostly in sparsely populated regions with relatively low traffic intensity. There is growing evidence that there is a strong correlation between replacing ferries with bridges or tunnels and economic and population growth. It is therefore worth noting the outcome of previous projects that have a comparable relation to that of Fosen and Trondheim. This may be illustrated by what happened to the population growth in municipalities close to Bergen, Norway's second largest city, when two islands were connected to the mainland by bridges. (Trondheim is Norway's third largest city).

Close to Bergen there are several large islands. Two of these have replaced ferries with bridges 20 and 40 years ago, respectively. Fosen has the same properties as an island regarding connections to Trondheim. Ferries have connected for more than 40 years. A bridge has not been a technical possibility until a few years ago.

The two islands close to Bergen are Sotra and Askøy. Askøy is 12-25 km and Sotra is 15-30 km from the center of Bergen, respectively. Both had about 12,000 inhabitants in 1964.

Fosen is 25-100 (70) km from center of Trondheim. The population was 27 000 in 1964. Neighboring areas with direct road connections, and of a similar distance south and east of the Trondheim, had about 50 000 inhabitants in 1964. Figure 1 shows population growth for the period after 1964 in the respective areas.

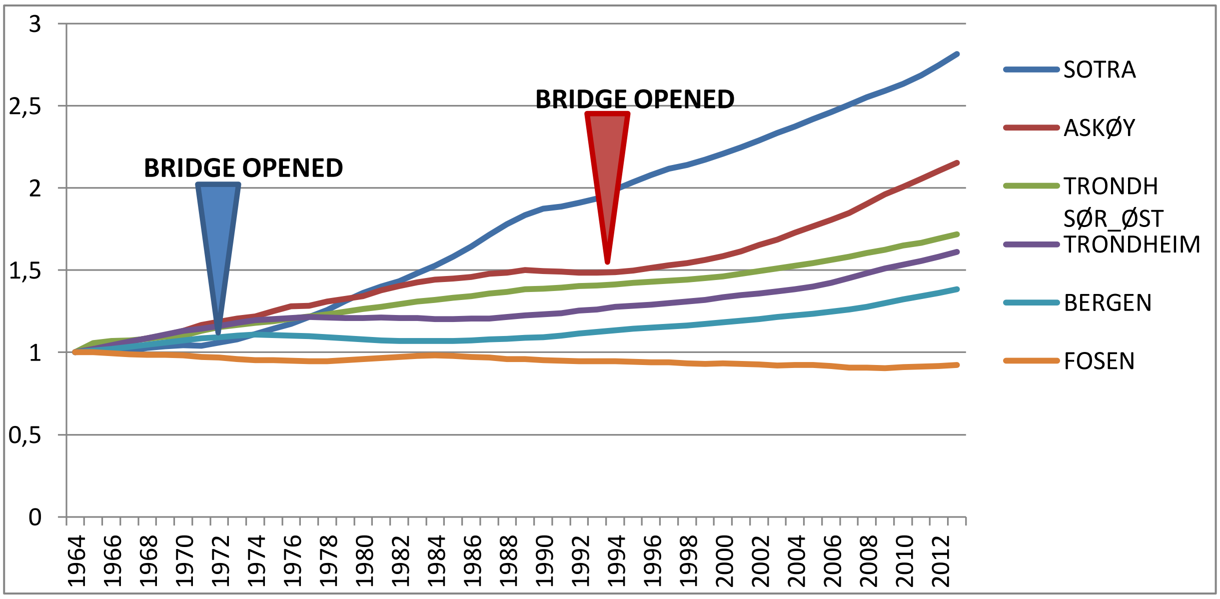


*Figure 6 Population development in Bergen, Trondheim and nearby regions 1964-2013. 1964=1*

Sotra was the first to get a bridge, in December 1971. Population growth increased shortly afterwards and has grown steadily thereafter. Today, it has increased by a factor of 2.8.

A connecting bridge to Askøy followed in December 1991. The island had experienced population growth in the 70's, but stagnated in the 80's. A few years after the bridge was established, it experienced strong growth at approximately the same annual rate as Sotra. Both these areas have grown at a substantially faster rate than Bergen.

The areas south and east of Trondheim did also experience stronger growth than Trondheim in the same period, but not as rapidly as Askøy and Sotra after the bridges had been built. The municipality of Trondheim has grown faster than Bergen. Fosen, however, has experienced a reduction in population over the same period.

The total population figures for Sotra and Askøy equaled the population of Fosen in 1964. Now these regions have more than twice the population of Fosen. There has been a strong population growth in all regions relatively close to big towns in Norway. Fosen is an exception. We believe that the explanation is closely linked to the lack of a road (and bridge) connection between Fosen and Trondheim..

There is strong evidence of a correlation between having a road, and if necessary bridge connection, and population growth in suburban areas. It does not prove a causal relationship, but few would deny this correlation. Similar experiences can be obtained from the corresponding project in Stavanger, Kristiansand and Ålesund. The knowledge of these experiences and further, that it is now technically possible, forms the basis for the commitment that Rissa Development, Rissa municipality and Fosen region have to secure a bridge across the fjord.

## SOME URBAN CHALLENGES IN TRONDHEIM

The town was founded more than 1,000 years ago. It was located centrally in the best agricultural areas north of Dovre. The city has been a trading center, like most ancient cities in Europe as well as a center for public administration. It was Norway's church capital until the Reformation. It has also been a cultural and industrial center. During the last 200 years, education, research and skilled jobs have gained a strong position. Urban growth was rapid until the early 1970’s, flattened for a while and then accelerated again in the late 1980's

Trondheim is topographically sandwiched between rich farmland and countryside hills or mountains higher than 250 meters above sea level. This level is considered as a practical limit for settlements with the expectation of a relatively pleasant summer and livable winter conditions.

Building density was relatively low in the period of rapid development of the 60’s and 70's, with most development happening on arable land. Only few houses in Trondheim are located at an altitude higher than 200 meters above sea level. These areas are now designated for recreation purposes. Previously exploited areas have subsequently been densified after 1990. Today it is a challenge to find relatively conflict-free development areas, particularly those suitable for land-intensive industrial and other activities.

Trondheim has at least two general topographical disadvantages. As mentioned earlier, Trondheim is on high latitude and much of it north facing. South facing residential areas is desirable at 64 degrees north. Summer is at least one month shorter on the north sloping areas. Short summers and long winters are a significant problem.

The above mentioned areas of Fosen on the Trondheimsfjord are mostly south facing. There is plenty of space below the contour line 250 that is not in conflict with agriculture or other interests. It has particularly good access to potential business areas associated with sea, which

Trondheim is completely lacking. It is important that sectors of industry showing rapid growth in Norway should get the opportunities to establish in or within easy commuting distance of Trondheim.

## NEW TOWN ON FOSEN (AS AN EXTENSION OF TRONDHEIM)
## (Proposal by Rambøll Architecture, Landscape & Spatial Planning)

The location of Stadsbygd, 7 kilometers to the west of the proposed arrival point of the bridge, could offer a solution to many of the problems and opportunities raised above and develop into a significant new primary growth point resulting from the construction of the bridge.

- It has numerous natural and man-made advantages and positive qualities which support the idea of an expanded settlement close to Trondheim.
- With the construction of the new bridge, the driving distance will only be 20 minutes to central Trondheim. (Comparable distance to that of Klæbu, Melhus and Malvik). (*Figure 7*)
- It already has an existing physical, social, economic and cultural infrastructure that one can build onto.


*Figure 7 Distance from center of Trondheim to different parts of Fosen*

Stadsbygd is unique in that it could support the development of a low carbon / high quality town which would look out over agricultural fields to the Fjord and, importantly, the south Sun without impacting on the thriving Agriculture (and fast growing organic sector). (*Figure 8*)

*Figure 8 Urban development with view to farmland and fjord.*

Further, the location is naturally protected from the cold north winds by forested mountains. The above are positive qualities not available in Trondheim. It further offers a rich cultural heritage:

- Viking history and Archaeology, the outdoor Amphitheatre on the fjord which host many people each year for the performance of 'the last Viking' based on the book by Johan Bojers, boat building and coastal heritage Museum, ancient rock carvings of reindeer at the old reindeer crossing spot near the edge of the Fjord and the cultivation of flowers which brings busloads of tourists annually.
- The Stadsbygd valley could support an addition of 15 000 to 20 000 people, with its own schools, health, sporting, business, industrial, cultural and recreational facilities without impacting on the productive and picturesque agricultural area, central to its current existence.
- Importantly, it could provide the opportunity for those with historical and cultural roots in Fosen to remain there while working in Trondheim, preventing the breakdown of social structures built over centuries.
- Its geographical location could also meet the needs of those families who have members working in both Trondheim and Fosen. – for example, one spouse working in Trondheim with the other working in Ørland, Bjugn or Åfjord. It could also provide for a high quality environment for those who would enjoy a micro-urban life in a pastoral setting in close proximity to city life with its greater cultural opportunities.

**GUIDING PRINCIPLES FOR DEVELOPMENT**

There are grounds to believe that a low carbon / high quality settlement of 15 000 – 20 000 people is achievable if the following principles are followed:

- The settlement must be economically generative, socially uplifting, culturally vibrant and ecologically restorative.
- The settlement pattern must be executed in a manner that protects and support the thriving agricultural and fast growing organic farming sector.
- It need be designed to protect and build onto the positive assets of Stadsbygd mentioned above.

At a more localized level a main principle is to wrap around the agricultural area so as not to impact on it. (*Figure 9*)

*Figure 9 Development wrap the agricultural area*

The development is to be connected as a series of micro-urban clusters so that each cluster remains strongly connected to nature and each other. (Fig E) The spaces between the settlements serve to ensure that the agricultural land remains connected to the natural landscape at regular points in order to sustain the patterns of pollination and biodiversity.

*Figure 10 Series of micro-urban clusters*

The main transport route between Trondheim and Rissa (and eventually Ørland) is to run through the agricultural area to provide for a direct vehicular route with low gradients. A secondary route which connects the clusters is to have an emphasis on ease and rationality of cycling and public transport so as to reduce car usage within the development. (*Figure 11*, right)

Mobility within the urban clusters is to be designed around the principle of the five minute city – the idea that most basic cultural, educational, recreational and retail activities are reachable within a 5 minute walking distance. (research showing that to be the rough threshold beyond which people consider using their cars).


*Figure 11 Transport routes and internal mobility*

The major commercial activities are to be located in the larger clusters at the two entry points to Stadsbygd and thus facilitating maximum client opportunity. (Figure 11, left) Together with allied industrial development, this will provide additional employment and self-sustainability for the town.

Only limited and selective recreational, historical and cultural activities will be situated within the agricultural area. The clusters are to be located above and around the agricultural land on the slopes of the valley to maximize on the magnificent views across the agricultural landscape to the Fjord (sunny south-facing) – so as not to impact on the agricultural land. (Figure 12)

As far as possible, the main public spaces are to be located in a position which can benefit from the magnificent views and sun.

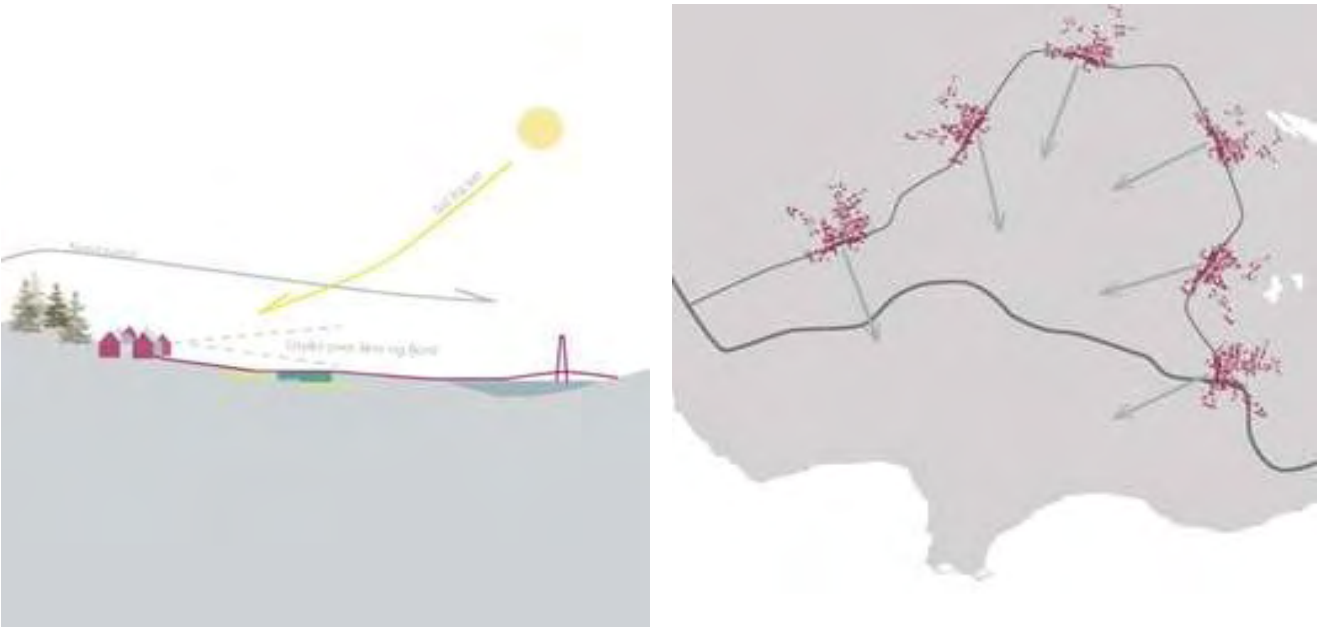

*Figure 12 Agriculture area and fjord view in south*

The settlement needs to provide for a range of different housing types and forms of ownership to suite different living requirements. It will also need to encourage opportunities for different scale players – large developers, local authorities, small developers and self-builders to facilitate a diversity of environment and income opportunity. (Figure 13)


*Figure 13 Different public service and building opportunities*

The sizes of the clusters are estimated to be between 2 000 and 5 000 people. The holding capacity of the natural systems, requirements for infrastructure, projected heavier rainfall, and the optimization of local energy provision and waste management need closer inspection in this regard. So is the question of the optimum required facilities (health, education, retail, etc.) and activities (cinema, sport, library, restaurants, coffee shops, etc.) to provide for a sustainable, dynamic cultural life. (Fig L

# REVIEW OF RECENT LONG SPAN CABLE SUPPORTED BRIDGES ACROSS ESTUARIES AND STRAITS AND PROPOSALS FOR APPLICATION TO FUTURE CROSSINGS

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## ABSTRACT

*Several long span cable supported bridges have been built across estuaries and straits in the last decade, some are under construction and some in the planning and design stage. All these crossings have in common the following requirements:*

- *Quality*
- *Cost*
- *Long Span*
- *Speed of Construction*
- *Durability*
- *Safety*
- *Design for Extreme Events*
- *Aesthetics*

*The authors have been involved with a number of these bridges and the paper will describe a number of construction methods used or proposed for achieving the above objectives, and how lessons learnt can be applied to future crossings, with reference to the following:*

- *Stonecutters Bridge Hong Kong*
- *Forth Replacement Crossing Scotland*
- *Proposals for Development*
  - *Partially Earth Anchored Cable Stay Bridges*
  - *Seismic Isolation of Suspension Cable Anchor Block*
  - *Foundations in Deep Water using Off-Shore Technology*

## 1. STONECUTTERS BRIDGE

Stonecutters Bridge carries a dual 3-lane expressway and spans the Rambler Channel at the entrance to Hong Kong container terminals, providing high level clearance of 73.5m for shipping.


*Figure 1: Location of Stonecutters Bridge*

### 1.1 Bridge Description

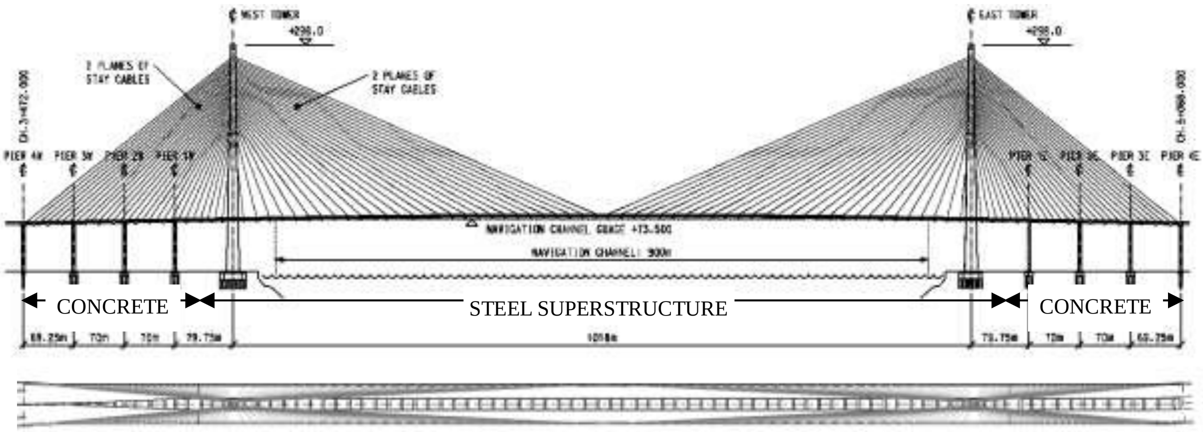


*Figure 2: Elevation and Plan*

Stonecutters Bridge is cable-stayed with an orthotropic steel main span of 1018m, and a total length of 1596m (Figure 2). There are four prestressed concrete back spans on each side. The tapered mono-towers are in concrete up to level +175m and steel-concrete composite from level +175m to level +293m with the outer steel skin being duplex stainless steel. The 2 planes of stay cables take a modified fan arrangement, anchored at the outer edges of the deck at 18m intervals in the main span and 10m intervals in the back spans.

The deck is a twin box-girder, with the two longitudinal girders connected by cross girders. The piers in the back spans are monolithically connected to the deck. Laterally the bridge deck is restrained by vertical bearings on the towers and by the back span piers. In the longitudinal direction dynamic movements are restrained by hydraulic buffers at the towers.

## 1.2 Detailed Design

The bridge was the first cable-stayed bridge in the world with a span over 1km for which detailed design was completed. The exposure of the site to typhoon winds created particular challenges.

**A Design for Extreme Events**

Wind Loading

The wind dominated the design. The bridge is a large highly flexible structure and required a complete wind model for dynamic calculations. Wind turbulence intensity measurements were made near the bridge site to measure the site specific wind conditions. This helped to calibrate and supplement the results from a 1:1500 scale wind tunnel model of the surrounding terrain. Together these studies provided an understanding of the turbulent wind climate resulting from the nearby hills.

Further wind tunnel studies included a deck section model at 1:80 scale and a high Reynolds Number deck section model at 1:20 scale to check for aerodynamic instability for wind speeds at deck level of up to 95m/sec. Also a 1:100 scale free-standing tower model was tested, and a 1:200 scale full bridge aeroelastic model to confirm the overall behaviour.

Wind buffeting calculations which allow the assessment of the actions on a flexible structure arising from the interaction between gusty winds and the dynamics of the structure were carried out in 2 separate pieces of software to ensure full confidence in the results from this complex analysis.

Ship Impact Simulations

*Figure 3: Ship Impact Model Test and Numerical Simulation*

The tower foundations are located approximately 10m behind the seawalls on both sides of the Rambler Channel. Given the close proximity, account was taken in the design for impact loading induced by a ship collision with the seawall. A series of centrifuge tests were carried out to model the effect of a 155,000 tonnes container ship impacting the seawall at a speed of 6 knots. The results of the test including pressure measurements aided calibration of a dynamic 3D finite element model, allowing the force exerted by the vessel impact at the front face of the tower foundations to be determined.

## B Towers

### Lower Towers

The concrete lower towers have a tapering shape reducing from an elongated circular section 24m by 18m at the base to 14m diameter at deck level and
10.9m diameter at +175m. The wall thickness is a constant 2m up to deck level, and then tapered to 1.4m at +175m.

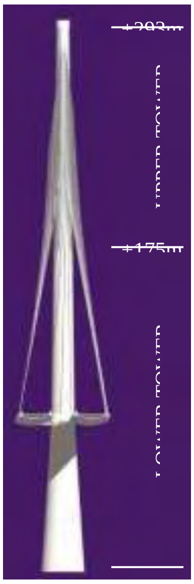


Figure 5: Tower Elevation

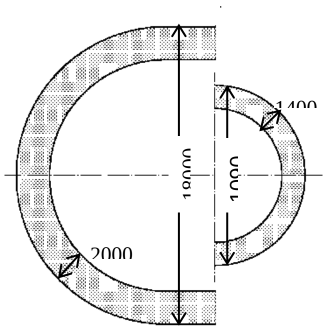


Figure 4: Lower Tower Section at base and at +175m

### Upper Towers

The structure of the composite upper towers is considerably more complex (Figure 7). The circular section has a constant taper from 10.9m diameter at +175m to 7.16m diameter at +293mPD. The outer skin is a 20mm thick structural stainless steel shell. This is composite with a concrete wall, which tapers from 1400mm to become a constant 820mm thick. The lowest 3 sets of stay cables anchor in corbels on the inside face of the concrete wall, whereas the remaining 25 sets anchor within a steel box section forming the core of the tower.

*Figure 6: Lower Tower Construction*

In each tower, 32 stainless steel skin sections make up the outer shell and 25 carbon steel anchor box sections stretch from +195m to +280m. The geometry of the steelwork was carefully controlled in the fabrication process by trial assembly to ensure that when placed on site it fitted into place.

*Figure 7: Upper Tower*

### Stainless Steel Reinforcement

Following durability analysis of the concrete in the towers for a 120 year design life, stainless steel reinforcement was used for all the outer layers of the tower reinforcement

**C Steel Deck**

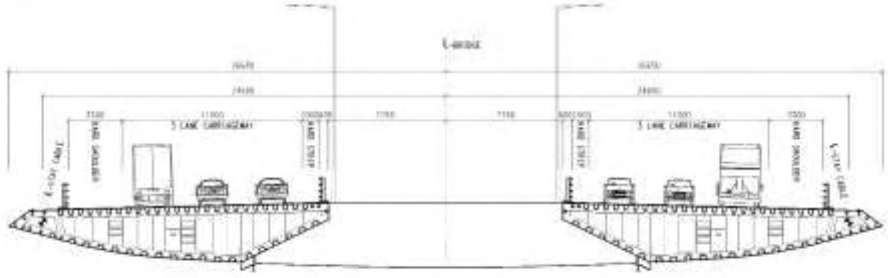

*Figure 8: Steel Deck*

*Figure 9: Two different methods were used in erecting the steel deck*

<u>Main Span Erection</u>

Main span deck segments were erected by cantilevering out from each tower (Figure 9). Each 18m long, 53m wide segment comprises the twin deck with connecting cross girder and weighed around 500T.

Due to the different support conditions there was a geometric mismatch between the lifted segment and the deck cantilever tip, which had considerable transverse sagging. A temporary bowstring prestress arrangement was installed on the lifted segment to manipulate the shape accordingly (Figure 9). Once in place, welding to the previous segment and installation of the stay cables followed.

### 1.3 Lessons that can be applied to future crossings

1.3.1 Twin deck configuration was first proposed by Dr Bill Brown for the 3.3km Messina Straits Crossing suspension bridge to cater for aerodynamic stability. The twin deck of Stonecutters Bridge enables the bridge to have flutter stability up to critical wind speed in excess of 180m/sec which is well above the design critical wind speed of 95m/sec under typhoon wind conditions. For narrow deck bridges across straits or estuaries, twin decks will be a suitable form of construction for both suspension and cable stay bridges. A twin deck has been used for the narrow 1495m Gwangyan Suspension Bridge in Korea which needs to cater for a critical wind speed of 72m/sec under typhoon conditions.

1.3.1 Using Centrifugal Model Testing method for determining the ship impact loads was used for the first time in the world on the Stonecutters Bridge project. This method is now becoming almost common and has been used for determining the ship impact forces on the Gwangyan Bridge tower foundations. This method should be used in evaluating the loads on bridge foundations in deep water to enable economic foundations to be realised.

## 2. FORTH REPLACEMENT CROSSING BRIDGE

The Forth Replacement Crossing, carrying a dual 3-lane motorway, will be built across the Firth of Forth in Scotland. The wide estuary will be crossed by a cable stayed bridge with 3 towers and a pair of 650 m main spans. In the centre of each main span the stay cables will overlap to stabilise the central tower, a unique design feature for a bridge of this scale.

The scheme design of the crossing has been carried out by Arup, in accordance with the Eurocodes and project specific design criteria. The structure will provide a fitting 21st century icon, to stand alongside the existing cantilever rail bridge from the 19th century and road suspension bridge from the 20th century (Figure 10).

*Figure 10: Visualisation – Three centuries of engineering in the Firth of Forth*

The Firth of Forth is a dramatic estuary. The downstream crossings of the Forth at Queensferry are a pair of historic bridges, the iconic cantilever rail bridge constructed in the 1880's and the Forth Road Bridge, Britain's first long span suspension bridge, which was opened in 1964. The replacement bridge will be slightly to the west of the existing bridges, making use of a natural granite outcrop in the middle of the Forth to allow the wide estuary with two navigation channels to be crossed by a cable stayed bridge with a pair of 650 m main spans, with an approach viaduct to the south.

### 2.1 Design Development

Three tower cable stay-bridges result in instability of the central tower when alternative spans are loaded with traffic. A number of solutions are possible to overcome this problem as shown in Figure 11. The solutions are:

- Introduce anchor piers
- Have horizontal stabilizing cables connecting the top of the towers
- Have stabilizing cables connecting the top of the towers to the deck/tower interface of the adjacent towers
- Have overlapping cables in the centre of the spans

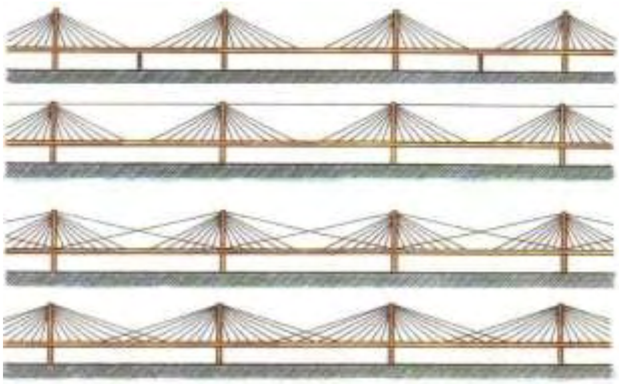

*Figure 11 – Methods for Stabilising Towers*

Another solution is to have stiff towers. The solution of over-lapping cables was first proposed by Prof. Niels Gimsing. This method allows slender towers to be used, and at the outset of the design it was proposed to use overlapping cables as shown in the general arrangement of the bridge in Figure 14.

A number of solutions were developed based on various types of towers and type of decks as shown in Figure 12. These designs were developed in sufficient detail and their costs evaluated. Based on cost and aesthetic considerations the mono-pole tower solution was chosen for further development as the Specimen Design.

Mono-Tower H-Shape Diamond A-Frame

*Figure 12: Tower Forms*

## 2.2 Specimen Design

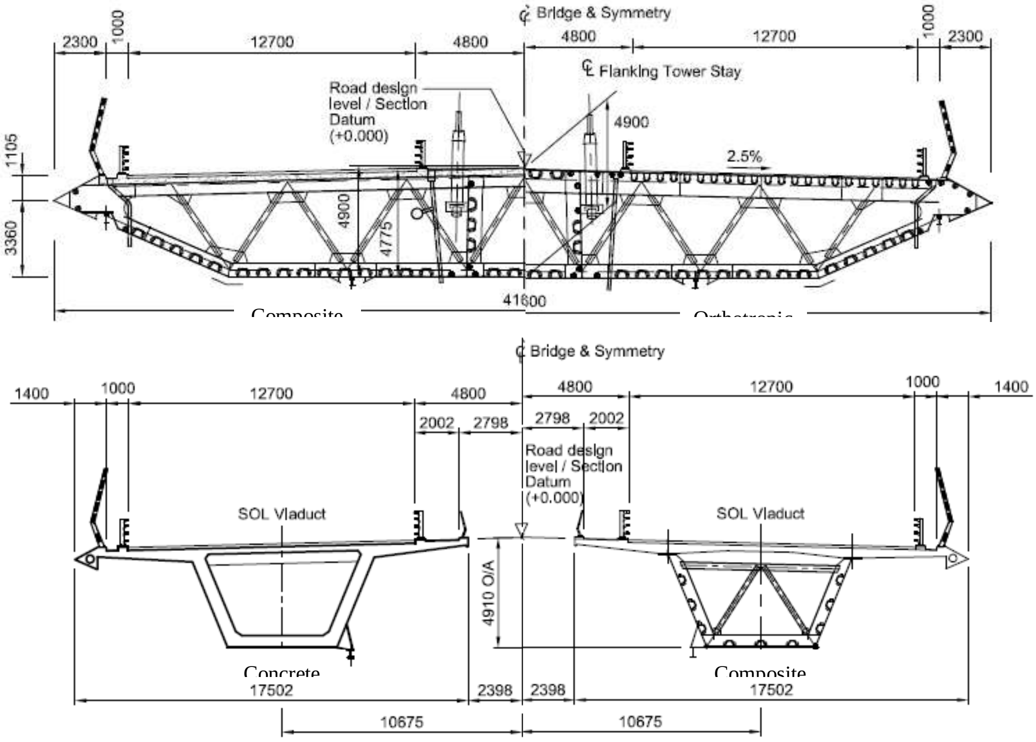


*Figure 13 - Deck sections, showing all variants*

The Specimen Design of the crossing is a scheme design incorporating a high level of detail. Transport Scotland, the client, wanted to have this specimen design as a starting point for the tendering contractors from which to prepare their design proposals.

The total length of the bridge is 2,638 m. Although the crossing is divided into a cable stayed bridge and a southern approach viaduct, the structure is continuous from abutment to abutment with no intermediate expansion joints. Longitudinal fixity is provided by a monolithic connection at the Central Tower located on Beamer Rock with transverse support provided at all towers and piers.

The towers are vertical reinforced concrete elements located in the centre of the deck with two planes of stay cables anchored centrally in the "shadow" of the tower between the carriageways. The stay cables overlap in the centre of the main spans. The deck itself is a streamlined box girder and stay cables are multi-strand type.

The key design requirements for the approach viaduct are long spans to minimise environmental impact, and visual continuity with the cable stayed bridge. The aesthetic requirements are achieved by a pair of constant depth box girders supported on V-shaped piers.

During preparation of the Specimen Design it became clear that there was no clear advantage to distinguish between all-steel orthotropic and steel-concrete composite construction for the cable stayed bridge deck box. Therefore both options were worked up as design solutions, and the contract permitted either to be adopted.

Similarly for the approach viaduct the choice between composite and prestressed concrete construction for the twin boxes was not driven by significant cost difference, so designs for both variants were completed and the option left open.

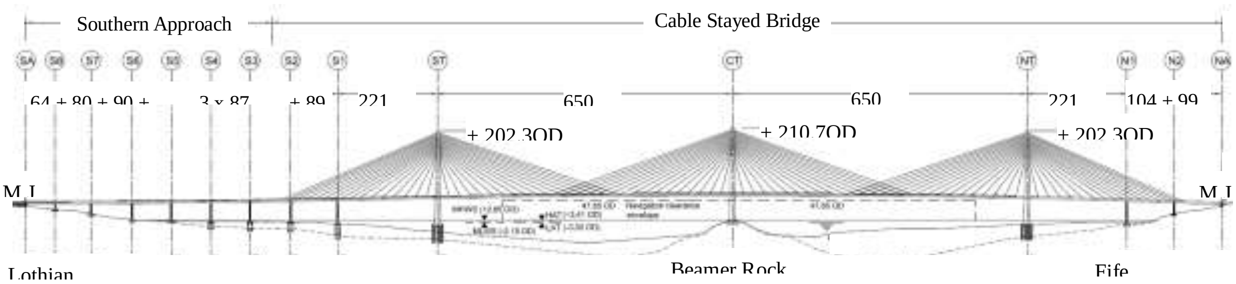


*Figure 14 – General Arrangement (orthotropic deck variant)*

## 2.3 Wind Tunnel Testing

### A Preliminary Wind Tunnel Studies

As part of the option selection process, different types of deck sections were tested at 1 to 50 scale to investigate the aerodynamic stability and force coefficients. At the early stages, ladder beam decks were included in the investigation as they may have provided a cost effective solution. Mitigation measures were required to improve stability including edge fairings and partially open central vents. The risk of aerodynamic problems for these decks was reduced, but not eliminated, and they were not progressed beyond the preliminary stage once the box option had been selected.

**B Wind Shielding Study**

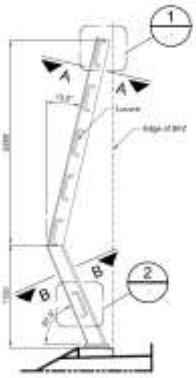

*Figure 15: Wind Shield*

Part of the design criteria is for enhanced reliability of the crossing remaining open in strong winds compared with the existing bridge which is subject to frequent restrictions and occasional closures. Wind shielding along the edges of the deck will be provided, but a balance is required to determine the level of protection to vehicles without increasing the forces that the structure must carry beyond reasonable levels. Performance criteria were set to select wind shields which would achieve conditions on the bridge which are no worse than the conditions that would be expected on typical approach roads around the site. A wide range of wind shield geometries were tested on a 1:40 scale model of the deck section. The wind shields selected were 3.44 m high with 6 horizontal slats, each 300 mm high. The wind shields provide the required reduction in moment for a double-deck bus.

### 2.4 Lessons that can be applied to future crossings

- Multiple tower cable-stay bridges with spans in the region of 1000m+ are possible with slender towers and crossed cables in mid-span. This could be a viable and economical solution as an alternative to multi-tower suspension bridges such as being considered for the Chiloe Bridge in Chile.
- Tall wind shields can be used on long span bridges to allow traffic to use the bridges in high wind conditions.

## 3. PARTIALLY EARTH – ANCHORED ULTRA-LONG SPAN CABLE STAY BRIDGE

With the rapid increase in span length of cable stayed bridges over the last four decades the question now being investigated by a number of researchers is what would be the practical span limit for this type of bridge construction?

Because the suspension bridge form already offers a well established alternative to the cable stayed bridge for spans up to 2,000m or more, the challenge for an ultra-long span cable stayed bridge with span greater than 1,200 m is not only technical but also economical. Such a bridge will only be built if a cable stayed span of that size is competitive against a suspension bridge. Aerodynamic forces acting on an ultra-long span cable stayed bridge are decisive in determining not only technical feasibility but also the construction cost.

A partially earth anchored system has been investigated as a way of reducing the cost of ultra long span cable stayed bridges by significantly reducing deck quantities. However, a key unresolved issue is how such a bridge might be constructed.

### 3.1 Partially Earth Anchored Cable Stayed Bridge and Tie-Cable System for Construction

It is evident that the transmission of the horizontal components of the stay cable forces will be one of the decisive factors for the competitiveness of the cable-stayed bridges when considering spans beyond the present limits. With a traditional cable stayed bridge and existing economically viable grades of steel, the plate thicknesses in the deck start to become impractical from a fabrication point of view beyond a span of around 1400m. It is, therefore, interesting to consider whether modifications of the structural system could reduce the critical compression in the deck of a cable-stayed bridge.

As one of the limiting factors will be the compression in the deck due to the horizontal components of the stay cable tension, a partial earth anchored system can be applied where the horizontal equilibrium of the deck is achieved not only by a compressive force at the towers but also by a tensile force at midspan which is introduced by anchoring some of the stay cables to the ground.

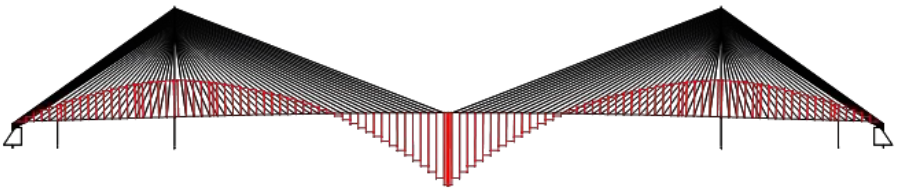

*Figure 16: Axial force in deck of a partially earth anchored cable stayed bridge*

The challenge with this form of bridge is how to construct it. An innovative tie cable construction method has been developed which has a number of significant system benefits:

- The axial compression in the deck during cantilevering is reduced
- The bending moments and deflections of the deck due to wind during cantilevering are reduced
- Apart from the tie cables the deck erection largely adopts tried and tested techniques

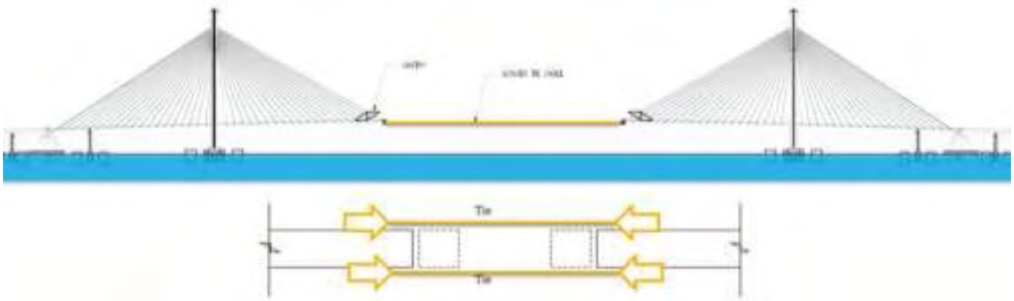

*Figure 17: Tie-cable method for construction of partially earth anchored cable stayed bridge*

### 3.2 Stay Cable Vibration

Even with a textured surface, a damping ratio of approximately $\delta > 4.0\%$ (logarithmic decrement) is still required. However, as the cable becomes longer it becomes harder to achieve the same level of damping. One solution is to introduce a tower side damper for the longer cables to achieve the required stay cable damping for vibration control.

*Figure 18: Additional Structure at Tower End to House Stay Cable Dampers*

The constructability of this method of construction is described in a paper by my colleague Steve Kite at this conference.

### 3.3 Lessons that can be applied to future crossings

Partially Earth anchored cable stay bridges could be a viable and economic alternative to suspension bridges in the range of 1400m span.

## 4. SEISMIC ISOLATION OF A SUSPENSION BRIDGE OR PARTIALLY EARTH ANCHORED CABLE STAY BRIDGE ANCHOR BLOCK

Traditionally, a suspension bridge adopts a very heavy anchorage to transfer the force of the main cable to the ground. This counteracts the vertical component of the cable force and gives sufficient pressure at the foundation level to assure the transfer of the horizontal component. Such a very heavy anchorage is unfavourable during a major earthquake since the large mass will attract very large inertial forces. Therefore the efficiency of a gravity anchorage is significantly reduced and the weight must be increased to prevent sliding of the anchorage under earthquakes. Although in some locations a rock anchored alternative may be viable, in many cases rock of sufficient quality may not be present close to the surface.

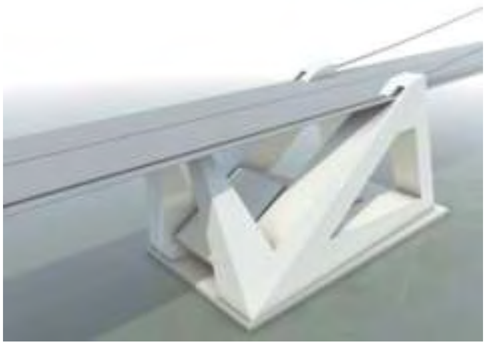

*Figure 19: Concept for a seismically isolated anchorage*

During an earthquake, the most significant forces on a gravity anchorage are due to the self-weight of the anchorage. The presence of the static force of the main cable is destabilising and means that any tendency for the anchorage to move during an earthquake is amplified and furthermore will be only in the direction towards the main span meaning that movements will be permanent and non-recoverable.

Seismic isolation is emerging as one of the most effective ways to protect a structure against earthquake forces. By isolating a structure and allowing it to move relative to the ground it can be protected from damage. However, the primary function of a bridge anchorage is to be an immoveable object which presents an apparent contradiction.

### 4.1 Seismic Isolation Concept

In order to overcome this inherent difficulty with seismic isolation of a structure subject to a permanent horizontal force the anchorage is separated into a structural part and an isolated counterweight as illustrated in Figure 20. The counterweight is articulated by an array of sliding pendulum bearings which are high quality seismic isolators. These introduce hysteretic damping into the system as well as shifting the period of vibration of the counterweight to a non-dangerous part of the seismic response spectrum. From this arrangement it is possible to gain the necessary advantage of the weight of the counterweight without feeling the full detrimental effects of its mass.

*Figure 20: The anchorage is divided into a structural part and a non-structural isolated counterweight*

This concept was developed during the preparation of a tender design for the Izmit Bay Bridge in Turkey (Figure 21).

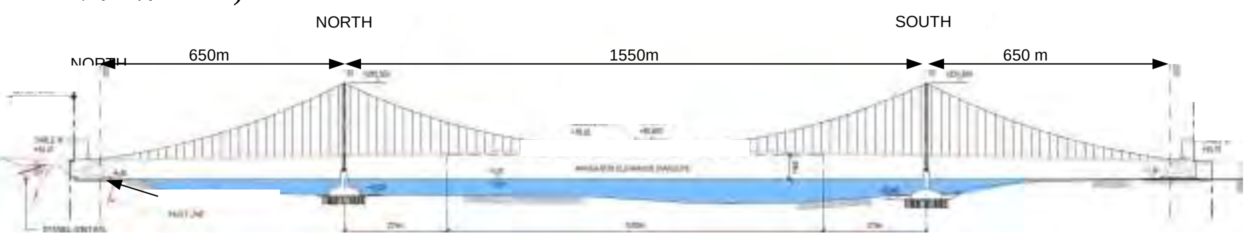


*Figure 21: General arrangement of Izmit Bay Bridge (BRDI-Arup tender design)*

Table 1 compare the anchorage displacement both with and without the seismic isolation system. For the analyses without seismic isolation it is considered that the counterweight is rigidly fixed to the structural part of the anchorage. The benefit of the isolation in reducing displacements is clear.

*Table 1: Anchorage translations with or without seismic isolation*

| Evaluation Event | North Anchorage Without Seismic Isolation (mm) | North Anchorage With Seismic Isolation | South Anchorage Without Seismic Isolation (mm) | South Anchorage With Seismic Isolation (mm) |
|---|---|---|---|---|
| Functional | < 2 | 0 | < 2 | < 1 |
| Safety | 132 | 54 | 317 | 41 |
| No-Collapse | 1,540 | 476 | 1,990 | 543 |

### 4.2 Lessons that can be applied to future crossings

Seismic isolation techniques could be applied to anchorages of suspension or partially earth anchored bridges in high seismic zones

## 5. FOUNDATIONS IN DEEP WATER USING OFF-SHORE TECHNOLOGY

Several bridge projects are in the planning and development stage that traverse across deep water such as the proposed Gibraltar Crossing and the crossing between Yemen and Djibouti across the Red Sea. (Figure 22)

Wandoo, Australia White Rose, Canada

*Figure 22: Off-shore Gravity Platforms*

The oil industry had been using off-shore gravity platforms for several years and many of these have been built in Norway. Such a solution has been proposed for the bridge linking Male and Hulhumale islands in Maldives. The spread foundations are proposed to be founded on coral at a depth of 60-70m. (Figures 23 and 24)

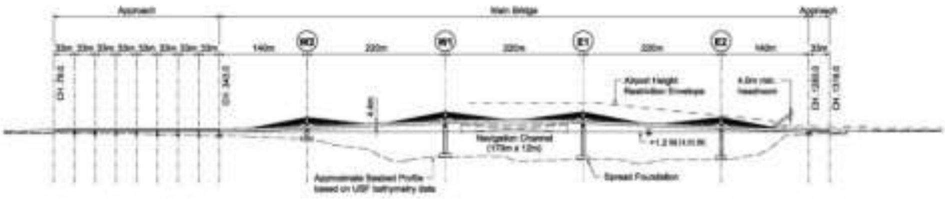

*Figure 23: Elevation*

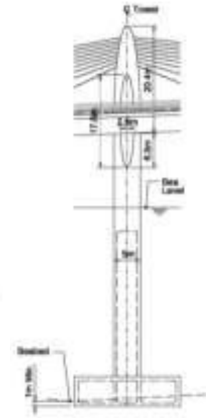

*Figure 24: High Tower Side Elevation*

### 5.1 Lessons that can be applied to future crossings

Gravity foundations using off-shore technology can be used to construct foundations for long span bridge projects crossing deep water.

# ACCESS SOLUTIONS FOR SUSPENSION BRIDGE CABLE MAINTENANCE

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## ABSTRACT

*Access provision to undertake construction and remedial works to structures is an increasingly demanding consideration for the execution of bridge projects worldwide. Both temporary and permanent access solutions provide engineers with opportunities to undertake projects in challenging locations whilst satisfying the concerns of multiple stakeholders, not least the bridge users. With many structures operating above capacity and subsequently demanding regular maintenance and enhancement, options to limit or mitigate disruption to the infrastructure whilst utilising cutting edge construction and maintenance techniques frequently demand extensive innovation in access provision alone, just to allow staff and equipment safe and adequate passage to the final work site.*

*Regardless of the structure type, age, usage and location, access provision to undertake construction, remedial, maintenance and enhancement projects is a constant consideration for Bridge Engineering globally. With ongoing changes to intended maintenance plans, design life and asset management technologies, few structures were designed and built with adequate access to facilitate the types of projects typically seen around the world today, as such facilitating access alone can occupy a disproportionate amount of the design and construction approach to any bridge project.*

*The paper will discuss current issues in today's industry and how new-build bridge projects can be designed with future-proof access in mind, whilst of course considering the capital expenditure implications of such provisions. The paper will focus upon the project-specific requirements for some typical and specialist remedial & maintenance projects and why access alone frequently becomes the pinnacle consideration in today's market. Finally the paper will consider variations in the working cultures and legislation of international markets and how this can impact upon the chosen access methodology for a given scheme.*

## 1. BACKGROUND

Existing and new build large bridges can be seen as a triumph of engineering. A result of significant capital expenditure, they provide essential function in connecting communities separated by environmental obstructions, whilst maintaining an elegance and beauty derived from their efficient structural origins.

Worldwide, our bridges are well used. They have become essential parts of natural infrastructure, with few alternatives available without causing significant disruption to local traffic flow, and therefore both local and wider economic performance. Structures of all ages are carrying significantly more traffic than was ever envisaged, rendering any disruptive traffic management extremely costly, whilst at the same time demanding a more intensive programme of desk study and invasive inspection of the structures to ensure that they continue to operate within acceptable design margins.

At the same time therefore the requirement for access solutions is driven by the above constraints, but also the required quality of these solutions, from both a safety and operability perspective has increased as regulation and user expectations have moved on. Bridge owners are therefore developing requirements for replacement systems for the under-deck solutions (see *Figure 1* below), together with more bespoke arrangements to cater for emerging works, as seen in *Figure 2*. This latter solution has to be designed in a bespoke manner to suit a particular structure, but also with an appreciation of future applications not yet specified, thus requiring a significant degree of flexibility in operation. Finally, fully bespoke solutions are occasionally required. *Figure 3* shows an access solution developed to support bearing replacement at the Humber Bridge where structural considerations prevent access from deck level.

*Figure 1 - Replacement Under Deck Gantry Solutions At Erskine Bridge*


*Figure 2 - Spencer Cable Crawler, Installed At Humber Bridge, UK*

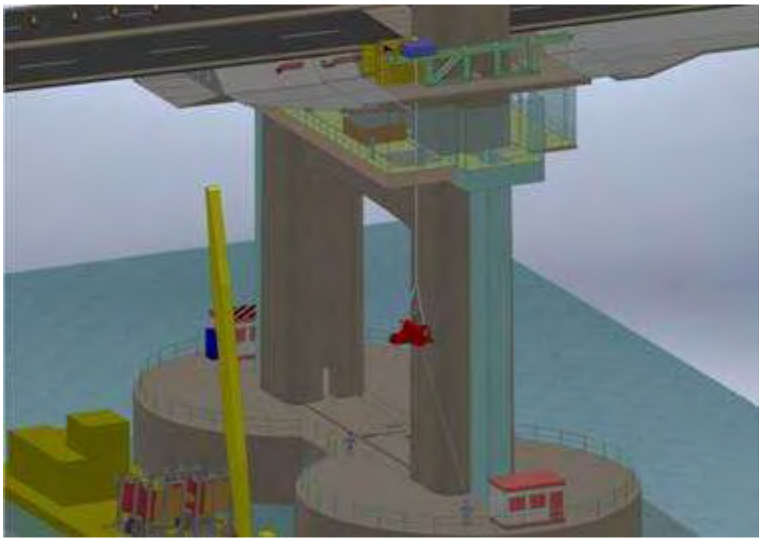

*Figure 3 - Spencer Proposal - Access To Under Deck Of Humber Bridge UK.*

All forms of access present complex and integrated challenges, as seen from the figures above. The remainder of this paper, however, concentrates on access for suspension bridge cables, in order to focus on the solution to a very particular problem.

## 2. REQUIREMENT

Suspension bridges are currently offering a series of challenges to engineers across the world. Some structures are approaching half of their design life, rendering the operators with little information about the absolute build conditions as would be expected in a modern construction. This, coupled with the age and use of the structure, means that significant inspection works, and remedial projects are needed to arrest premature deterioration. Whilst this is applicable to the full structure, the cables themselves are particularly vulnerable, as they represent a single point of failure and therefore demand special attention. Access to particular areas on the cables is also, by their very nature, challenging. At the Great Belt Bridge in Denmark for instance, the tower top cable shroud is 175m above the deck, with a total length of cable of over 3km.

Whilst the concept for the access solutions discussed in the remainder of this paper have typically been developed for operating campaigns across the entire structure, emergent work have tended to require access to particular areas of the cable. This requires a solution that can be rapidly deployed to any area of the cable, irrespective of the height of the area, or the associated slope of the cable at that point.

It is clear that works have to be ongoing with the bridge in its normal operation mode, and therefore access solutions are in operation with maximum traffic levels crossing the bridge. Any proposed solution therefore needs to be designed an operated whilst guaranteeing minimum impact on the operation of the structure. As requirements have developed over time, significant additional considerations have to be made. The approach, developed with the bridge operators, has been to completely isolate the access solution, and associated works, from the bridge.

This approach has delivered significant benefits over more conventional solutions, but has rapidly become the normal expectation when working on live structures. There is for instance no traffic management required when the gantry is in operation, a single night shift is sufficient to deploy the crawler.

When working on the cables, the crawler provides full containment for both personnel, and importantly, any dropped object. It minimises the need for rope access techniques and personnel. It also provides a fully weather proof working environment, that is suitable for containment of red lead, enables painting of the cable, space to wrap and compact cables after full inspection, and most importantly, maintains a safe and comfortable working environment for operators. This includes the provision for tool storage and onboard power. The gantries are designed to cover a full panel length when secured in the working state, thus enabling continuous, uninterrupted work and therefore increasing productivity.

Personnel access is also a key consideration for both bridge operators and maintenance contractors. The crawler solution enables access from both the main cable, following cable walking, or a deployable man basket that brings operators and equipment from road level. This works particularly well when there is a foot and cycle lane physically separated from the main roadway, as is the case with the UK’s large bridges. In order to help operators in the transfer of equipment, other lifting facilities such as Davit arms have also been deployed.

The solution needs to be designed to work independently of the bridge deck and cycle tracks. This is to remove the requirement for traffic management and negate the effect of maintenance work on the bridge operator and users. It also needs to produce a rapid transit between workstations, with experienced crews now moving between panels in less than thirty minutes.

Finally, it is critically important that the access solution doesn’t cause damage to the cables themselves. This has become even more critical over recent years as cables are wrapped in neoprene sheaths, much less robust than the steel wire wrapped cables, to facilitate dehumidification solutions. Support pads are carefully designed in order to minimise this loading. Furthermore, the operational mechanism does not rely on using the more delicate hand strands, again, ensuring protection for the bridge structure.

The design of the access solution must take into account the geometry of the bridge, with particular attention paid to being able to traverse across hand strand connections and cable bands without the need for special operations.

## 3. CONSTRAINTS

The key consideration when implementing any access solution is safety. Safety of the operators, and critically other bridge users, who will be travelling across the structure potentially unaware that any maintenance activity is taking place.

This consideration results in the adoption of several codes standards and regulations, in order to comply with the statutory requirements in the territory of operation. In Europe, this tends to consist of a structural code for the base structure [1], general gantry operation requirements [2] and the Supply of Machinery regulations [3], together with thorough risk assessments generated with a full understanding of the design.

Whilst the specific requirements will differ between territories around the world, the general principles of consideration and in depth requirements capture for design and operation will ensure a similar solution and a safe product. The gantries are included in Annex IV of the European Machinery Directive, which means that the design and safety systems have to be agreed with a notifiable body, thus giving an active third party input into the safe working of the system.

Further constraints are imposed as each and every structure is different, both in terms of geometry and also the environmental loads imposed on the gantry. This results in each solution being bespoke in nature, however also building on previous experience to create a flexible and functional solution. As span lengths and cable diameters increase, and payloads increase due to pushing for productivity, inevitably natural limits in terms of weight of structure and imposed loading onto the cable are reached. The challenge is to innovate to the next level of development that enables the design to be pushed further.

## 4. DESIGN PROCESS

The gantry access solution is a bespoke product to each application, designed to offer maximum flexibility in operation. The start point for design is therefore the structure geometry (obtained from existing documentation and also detailed site survey), coupled with the bridge owner requirements, both known and anticipated. Operational requirements will also be discussed and agreed, such as speed of operation, working areas, ease of access, storage etc.

At the start of each project, the regulatory regime is rechecked, ensuring that the correct design and operation standards are being used, and if third party consultation is required, this is started early in the process. The methodologies employed in Europe are viewed as consistent, and suited to operations worldwide. However, this check should be employed irrespective of the operating territory to ensure that the correct and most up to date standards are being referenced.

Further information should also be discussed with the bridge owner at this early stage. Power supply locations, together with the demand likely to be seen in operation are critical items, likewise the operation methodologies will further shape the design. All of these factors are likely to be different on each structure.

A full loading schedule is compiled to be used in calculation. This includes the gantry in the working condition, i.e. with operators accessing the cable. Critically however, the various phases of the gantry movement cycle should also be analysed in detail, as without this attention, risks may remain unchallenged.

Following the information gathering process, the design team use their experience of designing and operating earlier versions of the gantry to lay out the design. A full 3D model is constructed, which is initially used to check the gantry in operation, when climbing the cable, and also in work mode, ensuring sufficient clearances for operators and equipment to carry out the various tasks required of them. Members are sized during this process using a simple hand calculation technique.

When the design has been functionally checked, the geometry is exported to a structural analysis package to correctly size the members, before detailing directly from the 3D model.

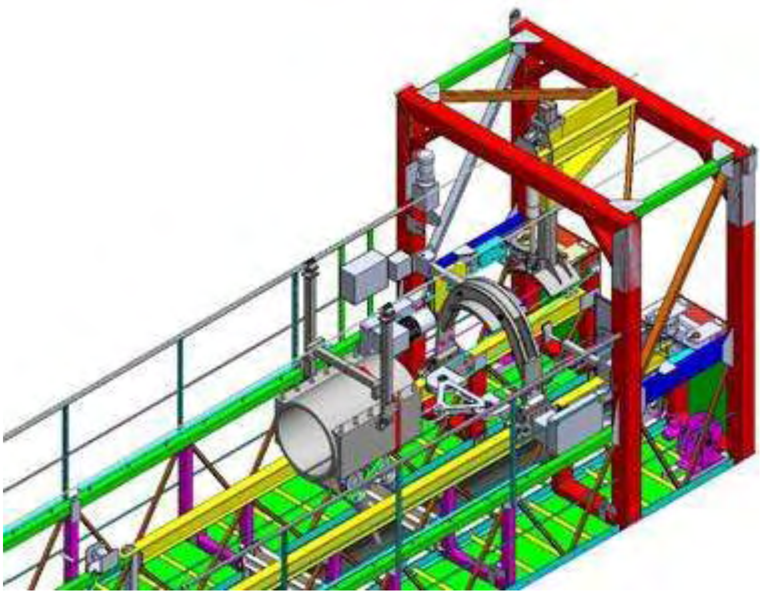

*Figure 4 - 3D Layout Of Gantry To Ensure Functionality*

The designs are put together to be fabricated in generic facilities. As such, all dimensional control, plus fabrication check regime ownership is retained by the design team, often with the support of a specialist fabrication inspection company. This retains flexibility for the units to be fabricated across a wide range of facilities around the world, but ensures that output quality is maintained.

As stated above, in Europe, where a CE mark[4] is required prior to putting the machine into service, constant dialogue is maintained with the third party notifiable body to ensure that all required safety features are designed in, rather than having to be retrofitted at a later stage which could adversely affect operability of the machine.

## 5. FABRICATION & SETTING TO WORK

Although the design and fabrication teams are entirely separate, enabling a generic facility method of procuring fabrication, in effect the two processes are intrinsically linked. This is achieved by involving the design team throughout the fabrication process, from procurement through to testing and training of operators. This continuity ensures that the vision of the eventual operation of the equipment is maintained throughout the process all the way to delivery.

In support of this, all of the processes are owned by the design team, from code and standard selection, to fabrication record keeping and final inspection. Fabrication is undertaken in accordance with the designer’s specification and checked by third party to ensure that all requirements are being met. Third party NDT is also undertaken to maintain the independent certification of the final structure, thus adding a level of reassurance to the infrastructure owners.

Due to the complex nature of the system, each unit undergoes a full works test before leaving fabrication, under the supervision of the designer. This includes load testing as required by regulations, but also operability tests, enabling the structure to be transported directly to the bridge and installed on the cable, without further intermediate steps.

The install process on the bridge is now well practiced, offering minimal disruption to the operation of the bridge, but also giving confidence that the access solution can be used immediately, therefore facilitating the procurement of specialist labour to perform the maintenance tasks required. Again, each gantry will undergo a fully planned commissioning process following installation, before being dispatched to the required work site. On full cable campaigns, this usually involves traversing to the very top of the cable, such is the confidence built in the system.

The operation and maintenance manual is seen as part of design, but it also forms an integral part of the training regime for those working on the structures. Training is carried out by the design team to again, maintain the continuity of information throughout the concept to commissioning processes.

Finally, whilst the general construction of the system is generic, and can be carried out in any location, a preferred local supplier is used for the mechanical and electrical fit out, where many of the systems ensuring that the gantry can be hoisted safely are located.

## 6. DESCRIPTION OF OPERATION

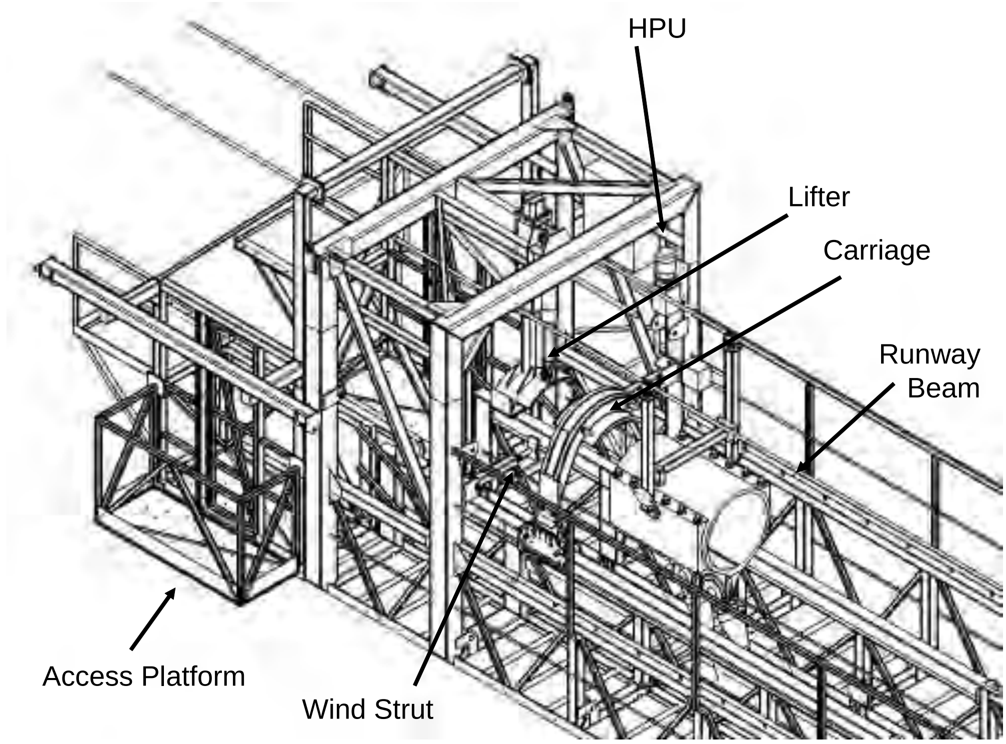


*Figure 5 - Isometric Layout Of Cable Crawler Head End*

The gantry operation is simple in nature, being hauled up the cable utilising specialist safety winches, backed up by a block stop system. The following general process is adopted:-

1. In the permanent condition, the gantry sits on the lifter system with the wind struts wound in to give support.
2. In order to prepare for a move, the safety winch system is set up, and the wind struts wound out.
3. The lifters retract, sitting the gantry on the carriage system.
4. The winches wind in or out, sliding the gantry on the runway beam
5. At the end of travel, the lifter deploys, taking the weight of the structure
6. The carriage is reset to enable a further movement
7. The process returns to step 3 until the correct position on the cable is reached.
8. When in position, the wind struts are wound in and the gantry lashed appropriately.
9. Operations continue on the gantry.

Careful configuration of the mechanical and electrical safety systems is required in order to ensure that the platform remains fit for occupation whilst moving. Likewise, a multi layer protection system is incorporated to ensure that no harm comes to operators or bridge users during operation.

## 7. POTENTIAL APPLICATIONS

The gantry was designed to support cable wrapping work, essential in retrofit dehumidification projects. However, as the concept has developed, the solution has transformed into a generic access solution for any part of the cable, which includes all of the required engineered safety systems to meet regulation and keep operators safe.

This expanded view of the machine has allowed enhanced services to be delivered. The units remain ideally suited for work on a full length of cable, offering full access to a full panel, plus the cable bands at each end. However, their ability to be rapidly deployed to any area of the bridge has seen them equally suited for undertaking maintenance on a specific component, such as replacement of cable band bolts. *Table 1* below shows the range of applications that Spencer Group have used the crawlers for.

*Table 1 - Cradle Crawler Applications Utilised By Spencer Group*

| | |
|---|---|
| Wrapping of full cable | Cable clamp resealing and painting |
| Hand strand post installation and replacement | Hand strand replacement |
| Inspection and replacement of cable band bolts | Tower top shroud inspection, removal and paint |
| Lighting installation and maintenance | Installation and repair of cable sensors |

Using the crawler has provided significant programme advantages. The ability to traverse the full cable quickly, and rapidly access a specific area has resulted in almost continuous productive working when on the cable. The fully protected environment has also enabled an increase in the acceptable weather conditions for working, with the enhanced working environment inside the gantries supporting an increase in productivity. Again, in pursuit of greater productivity, the gantry system can be locked down in extreme conditions, with the environmental protection systems removed, such that operations can start rapidly following return to more favourable weather.

*Figure 6 - Wrapping Main Cable Following Intrusive Inspection*

*Figure 7 – Neoprene Wrapped And Anti Skid Painted Cable*

*Figure 6* and *Figure 7* above show some of these applications. Even though the operations are very different, it can be seen that a clear and safe access is given around the work area, and the environment is as close as possible to shop conditions, even though the operations are taking place 100m above deck level.

## 8. WHAT ABOUT OTHER USES?

It is commonplace to install under deck access solutions on all new structures. This facilitates programmed paint maintenance activities as well as handling emergent requirements. Cable access has traditionally not enjoyed the same focus, either at the build stage in the lifecycle of the bridge, or later as a retrofit. This probably stems from the cables and their associated bands, hangers etc, being seen as fit and forget, a position that is being rapidly revaluated with the maintenance works being currently undertaken in Europe and America.

As can be seen from Table 1 above, having the access solution has already enabled more intense maintenance activity on the cables than would have been considered prior to the solution being available. In the case of bolt replacement, having clear access for proper inspections highlighted a potentially serious problem, before it occurred. It is cliché to state that an enhanced inspection and maintenance regime lowers overall lifetime costs. However, with structures as critical as the cables and their associated components, the maxim holds true. Enabling thorough inspection works will ensure that dangerous situations do not occur, and that the design lifetime of the cable is realised.

The crawlers have traditionally been used as a contractor's solution to undertaking work, with the programme and productivity benefits derived from the units enabling best value offers on tenders. However, the technology should be of interest to bridge operators, who if in possession of a flexible access solution, would be free to procure all works on the cable with these productivity benefits built in, enabling maintenance budgets to procure more wide ranging maintenance and repair activities. This gives the bridge operators the ability to plan projects with greater certainty, giving protection from having to rely on the contractor to provide methodology which may not fit client aspirations in terms of cost and programme.

## 9, HISTORY OF DEVELOPMENT

The development of the crawler system is an innovative activity driven by a clear and measured approach to identifying and solving of problems and obstructions that may not be obvious at the outset of a tender process, but will offer significant process advantages should these solutions be found. This innovation is driven by an understanding of both the global problem coupled with detailed engineering knowledge.

The initial focus is to identify productive work. In the case of wrapping a cable for instance, it is the wrapping process that is productive, with everything else, including getting to the worksite and making it safe, is seen as waste. Technology and multi-disciplinary engineering thinking are then applied to develop methods of eliminating the waste. In the case of the gantries, a consideration of the structural capacities of the permanent cable, the flexibility, strength and weight trade offs involved in the construction of the gantries, coupled with a full knowledge of mechanical processes, electrical control and safety considerations were necessary to develop the solution.

The crawler concept was developed in this manner, against the tender to dehumidify the cables on the Forth Road Bridge in the UK. The tender team identified the need for the following features to support efficient contract operation, and the design team was therefore tasked with achieving them. These were:-

- Minimise the overall programme of works
- Provide greater access to work areas with improved levels of safety
- Provide significantly shorter traversing durations between work stations than competing access methods
- Minimise, control and accurately communicate the loads being imposed on the main cables.
- Minimise interruptions to bridge users and operation

The continued success in this area of bridge maintenance with the crawler system has been enabled by adopting a continuous improvement philosophy to the solution. Later projects have therefore used the learning gained in the design and fabrication process to improve the product. The learn by doing approach to the development has been facilitated by using the same gang leaders on a number of early projects to provide constructive feedback on operability and ultimately develop the machine.

The focus on continuous improvement is driven through the full project. This leads to a greater understanding of new challenges. In the UK, the foot and cycle lanes are physically separated from the main channel, allowing the man access gantry to by guided by wires secured by kentledge at road level. In Europe, these additional lanes are not always available, with instead an emergency lane adjacent to the main roadway. The H&S risks associated with guide wires so close to operational traffic are imminent, and therefore a self contained man access solution is implemented , with no guide wires to be designed into the local solution for these bridges.

Other improvements have been implemented throughout the development process, some of which appear obvious in hindsight, others less so. All moving equipment is now colour coded, to aid operation and anti slip coating on walkways has been improved. The IP rating of electrical equipment has been taken to the highest level due to the extreme conditions seen on some bridges. Finally, additional lifting equipment has been designed into the structure, enabling it to operate at its most efficient.

In general terms, the full, integrated crawler solution is mature. However, in order to ensure that the full benefits of the concept are realised on each project, the full design process is followed from beginning to end, albeit building on the increasing database of knowledge gained through operation over a number of years.

When addressing the initial requirements for inspection repair and maintenance of suspension bridges, often companies limit themselves by using current available access. The standard access contractor can therefore limit themselves to the systems and tools they adopt or supply. The development of the cable crawler, has been driven by a highly in depth technical and innovative design team, free from these constraints, allowing the correct tool for each job to be produced.

## 10. NEW BUILD – WHAT WOULD WE DO DIFFERENTLY?

Nearly 20 crawler units have been produced to date, all looking at performing maintenance campaigns on existing structures. The design has therefore been pushed to a relatively manual operations philosophy. Furthermore, compromises have to be made in order to ensure that the gantry can operate around the constraints of the existing cable furniture. Innovative solutions also have to be found for securing the gantries in work locations, for instance to protect from wind induced skew, as no fixings are provided from the structure.

For a new structure therefore, numerous benefits could be realised, both in the design of the gantry and its interaction with the bridge can be optimised by consideration at the early stages of bridge planning.

It is believed that future suspension bridge structures will view cable access solutions in the same manner as under deck solutions are in the existing market. This can lead to permanent or semi-permanent (i.e. the gantry is designed specifically for the structure, with full integration, but can be deployed and removed simply) solutions.

These would be fully integrated with the bridge structure design, ensuring that maximum access can be given to operators working on the access platforms, rather than compromising to get around existing obstructions. For a longer term solution, additional automation can be brought to play, enabling operation by less skilled workers, and ensuring that the system availability is at its highest, and not dependent on availability of specialist resource. This will also reduce training requirements on the bridge owner’s operators. Furthermore, the control system of the gantries, together with any safety rated electrical systems could be integrated into the overall bridge monitoring system.

These developments do not by implication increase complexity, but undertaken correctly, improve controlled simplicity.

## 11. CONCLUSIONS

Large bridges are a triumph of engineering. By their very nature, all access solutions will therefore be both important – allowing proper maintenance of the structure and therefore enabling the design life to be realised – and complex, working at extreme height, with minimal structure available for protection, whilst balancing the needs for rapid operation and a clear working environment.

Cable access solutions on suspension bridges have generally not been considered at the design phase. This provides challenges in retrofitting the solutions, but also opportunities to innovate clever solutions.

Relatively recent identified requirements, such as cable dehumidification, have made the provision of access essential. With the solutions developed however, other emergent areas of work have materialised which utilise the technology for further investigation and repair. These emergent areas of work have prevented potentially dangerous situations occurring.

Access systems are at their most effective when a complete set of information is available at the outset of the design. This covers both bridge geometry, but also requirements, both known and anticipated such that an integrated and efficient solution can be designed.

The use of efficient access solutions, such as cable crawler systems, can significantly decrease the time taken to complete complex projects, and therefore enable a wider distribution of maintenance funds.

Should access requirements be considered at the outset of a large project build, then features can be designed in to improve cable access further, thus facilitating the protection of the cable and associated structures for the design life of the structure.

Bridge operators owning this technology represents an interesting mechanism for cost savings, as all of their contractors could be offered enhanced access provision, aiding price, programme and quality of delivered work.

## 12. REFERENCES


[1] - EN 1090 - Execution of steel structures and aluminium structures

[2] - Institute of Structural Engineer's Purple Book 2nd Edition; "The operation and maintenance of bridge access gantries and runways".

[3] - The Supply of Machinery (Safety) Regulations 2008

[4] – UK Department For Business, Innovation and Skills - How a product complies with EU safety, health and environmental requirements, and how to place a CE marking on your product

# TLP TECHNOLOGY EXPERIENCES IN THE NORTH SEA USED AS FOUNDATIONS FOR A BRIDGE TOWER.

*Presented by: Inge-Bertin Almeland, Aker Solutions*

## ABSTRACT

*This presentation describes a tension leg platform, designed and located in the North Sea. The platform is moored to the bottom by vertical tethers. The tethers are connected to a foundation at the seabed and moored to the lower part of the platform.*
*This mooring system prevents the platform to pitch and roll, but allows it to move horizontal.*
*A platform like this can be used for supporting a bridge tower where the tower is located on top of the platform and connected to the platform legs.*
*The tethers will keep the platform stabile. See the paper "TLP Technology, the Tether Systems" in ref. /1/, for a presentation of tether systems as used on TLP's. The paper on Tether Systems should be regarded in conjunction with this paper.*

*By modifying the offshore platform and use the experiences achieved, the platform can be designed to suite the requirements for supporting a bridge tower. The platform and the tower can be fabricated in a shipyard, transported to the bridge locations and connected to the tethers. The platform must be designed for ship impact and other accidental events.*

## INTRODUCTION

The first TLP installed in the Norwegian Continental Shelf was Snorre A in 1993. The design and engineering was performed by Aker Engineering for Saga Petroleum. To strengthen the Saga Petroleum Organization there was made a "Help of a Cooperating Agreement" with Esso. The project started in 1984 in the Saga Petroleum organization.
Snorre A was a new type of design for floating offshore structures. A smaller TLP had been done once before, the Hutten TLP in British sector. Lots of new technology had to be developed, documented, manufactured and tested before the platform was fabricated.

The fabrication of the hull was done by Rosenberg Yard in Stavanger. To be able to fabricate the hull in time, the hull was subdivided into sections. The Rosenberg Yard sub-contracted these parts to Belleli Yard in Italy and to Egersund Yard. The sections were transported to Stavanger and fabricated to one unit above the drydock.
The main contract was for the deck, fabricated by Aker Stord. The deck was an integrated truss-deck, specially designed to be mated on the hull.
After mating and testing the Snorre A was towed out to the field and was connected to the bottom foundations by 16 tethers.

An overall view of Snorre A is shown in fig. 1.

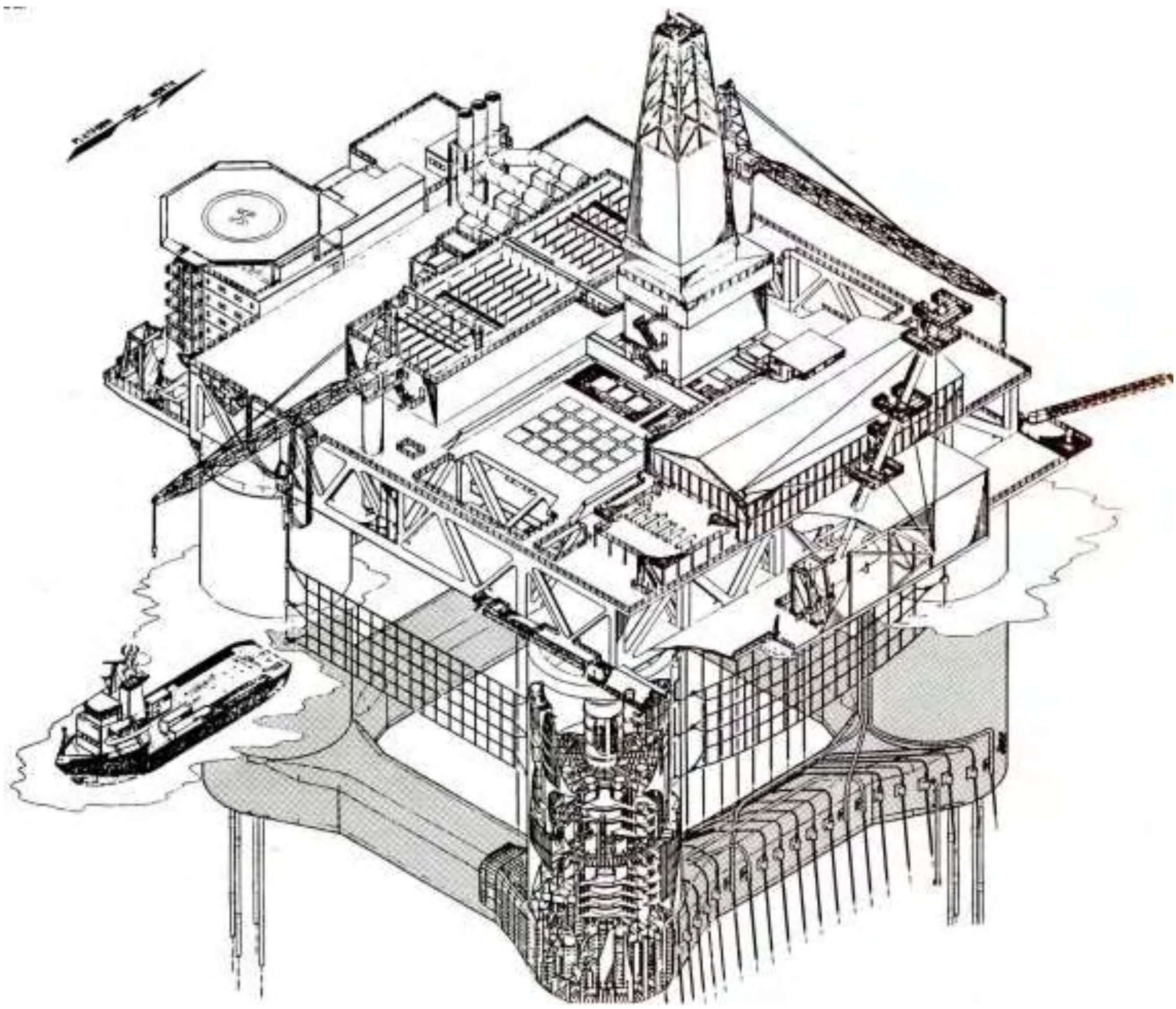

*Fig 1. Snorre A. As installed in the Field.*

**HULL FABRICATION**

A hull for a TLP is different compared to an ordinary floater. The roll, pitch and heave are eliminated by the tethers. A TLP moves different and accordingly need to be designed for this. The pretension in the tethers is approximately 20 % of the displacement. Accordingly the tether connections must be designed for this.

Snorre A had four internal tethers in each column. This made the internal column design quit difficult. Four tether conduits were located in the columns to lead the tethers from the node bottom to the mooring flat. See fig. 2.

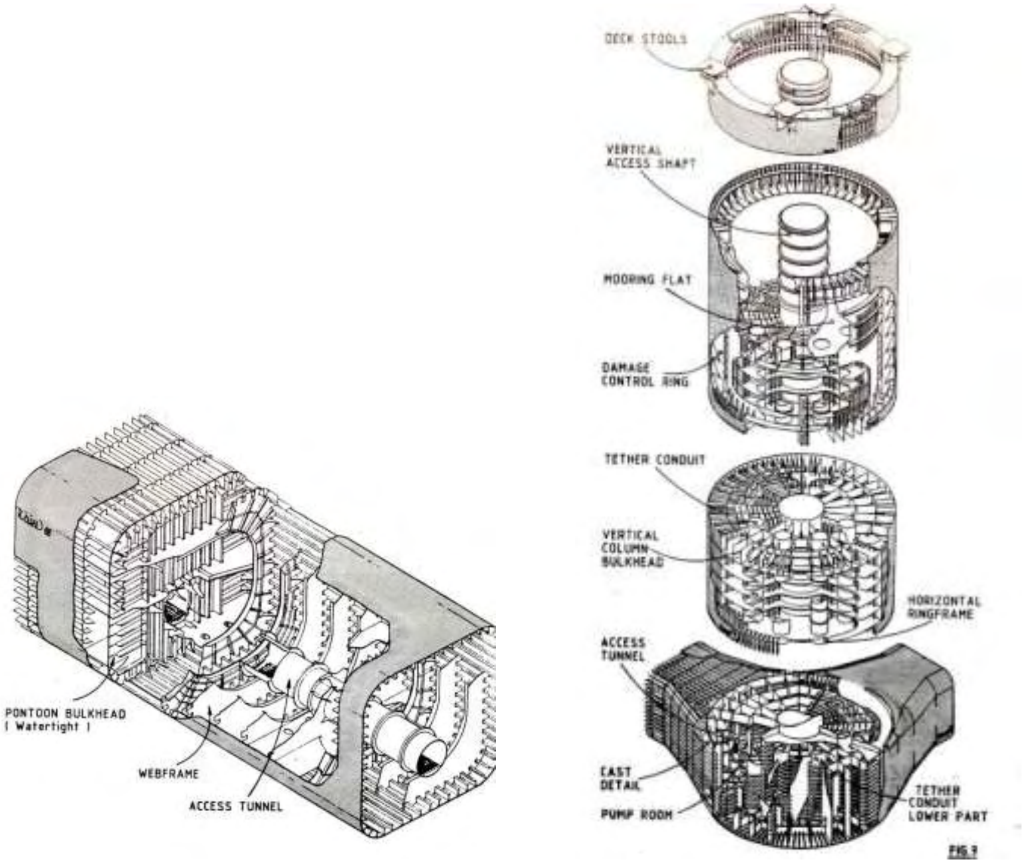


*Fig. 2 Snorre A. Column subdivided into fabrication sections*

Fabrication of the hull was divided into five main elements as shown in fig 2.
All four pontoon sections were fabricated by the Rosenberg Egersund yard. Three nodes and columns were fabricated by Belleli in Italy, and one node and column was fabricated by the Rosenberg Yard in Stavanger.

All sections were transported to Stavanger where they were mounted together to one unit above the drydock. Figure 3 and 4 shows some picture of the fabrications and lifting of modules. After launching, the hull was tested before being towed to Aker Stord to be mated with the deck-structure.

*Fig. 3. Nodes transported from Italy by a transport vessel. Three columns are shown during fabrication at Belleli Yard.*

*Fig. 4. Columns are lifted into position in Rosenberg Yard Stavanger.*

## SNORRE A TETHER DESIGN

Snorre A has four tethers per corner, which are mounted inside in each column. The tether parts were lifted into the column before deck-mating and installed by an internal crane arrangement during installation at the field. The elements were 20 meter long and screwed together. This was a costly design and complicated details.
See fig 5.

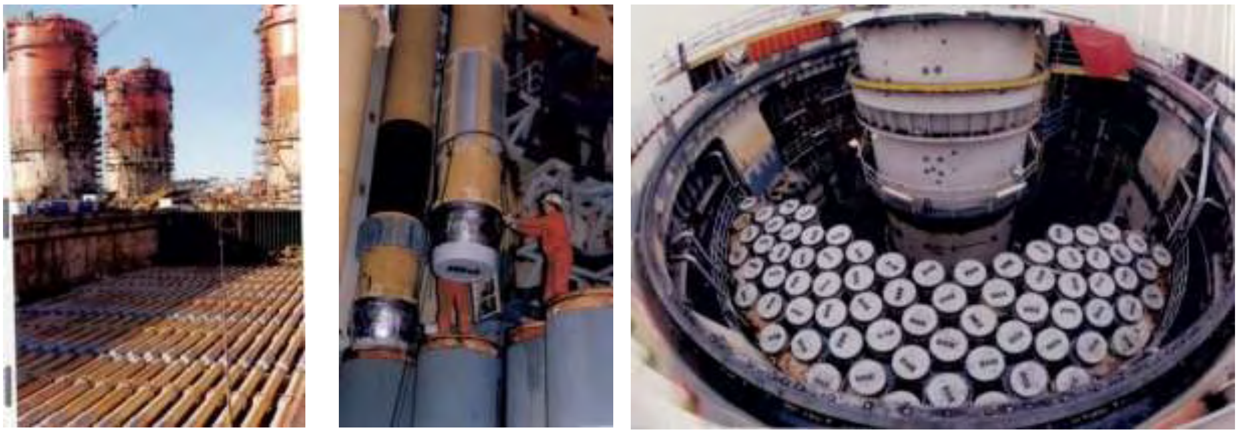

*Fig. 5. Tether elements before, during and after mounted into column tether-compartment.*

## COMPLEXITY OF A TLP

The deck of a production platform is the most costly and complex part. Here all the process equipment is stored, like the machinery, piping, pumps, riser-handling equipment, drilling-facility, production system, Living Quarter, etc.

From the main deck many pipes and risers are rooted outside the columns and down to the pontoons. From the pontoons they are connected to subsea pipelines. A riser-protection net is spanned between the columns to protect these pipes and prevent vessels accidentally moving or drifting in between the columns. All these items complicate the hull design and increase the cost for for an oil platform. Fig 6 shows an example of piping routed down onto the pontoon-top as arranged on Snorre A.

Fig. 6. Piping routed on pontoon top.

Fig 7 shows Snorre A installed in location in the North Sea. Lot of risers can be seen leading down from underneath the deck. The riser protection net can be seen spanned between the columns.

*Fig.7. Snorre A installed on the Field and operating.*

**OTHER TLP DESIGNS**

The Hutton was a TLP with a smaller hull, with 6 columns. Four tethers were placed in the outer four columns.
Heidrun was designed with four columns and the size of this platform is considerably larger than Snorre A. The hull was made out of concrete. Heidrun is a "Deep-Draught TLP and is connected with welded tethers connected on the outside each column. This made a cheaper tether design and a simpler installation.
Other examples of TLP platforms built after Snorre A are: Auger, Ursa, March, Magnolia. Lots of these platforms have been fabricated at the Belleli yard in Italy.

*Fig. 8 Other TLP concepts*

**EXPERIENCES FROM A TLP AS FOUNDATION FOR A BRIDGE DESIGN**

Experiences from the TLP design show a complicated deck and hull structure to support risers from the subsea wells.
To design a TLP foundation-type bridge tower the design can be much simpler. The only purpose of the structure of a TLP foundation-type would be to act as foundation for the bridge tower.
Less pretension is expected for an inshore “TLP” which is not exposed for the same wave spectrum as a TLP in the North Sea. Moreover the bridge tower will be stabilized by the top wire and bridge deck. Tethers can be supported outside the columns and installed by a smaller crane ship. For the design the experiences gained from fabrication, installation and use of offshore structures must be utilized.

In the following, some sketches are presented to show how a “TLP foundation” may be utilized as foundation for a bridge tower. See Fig. 9.

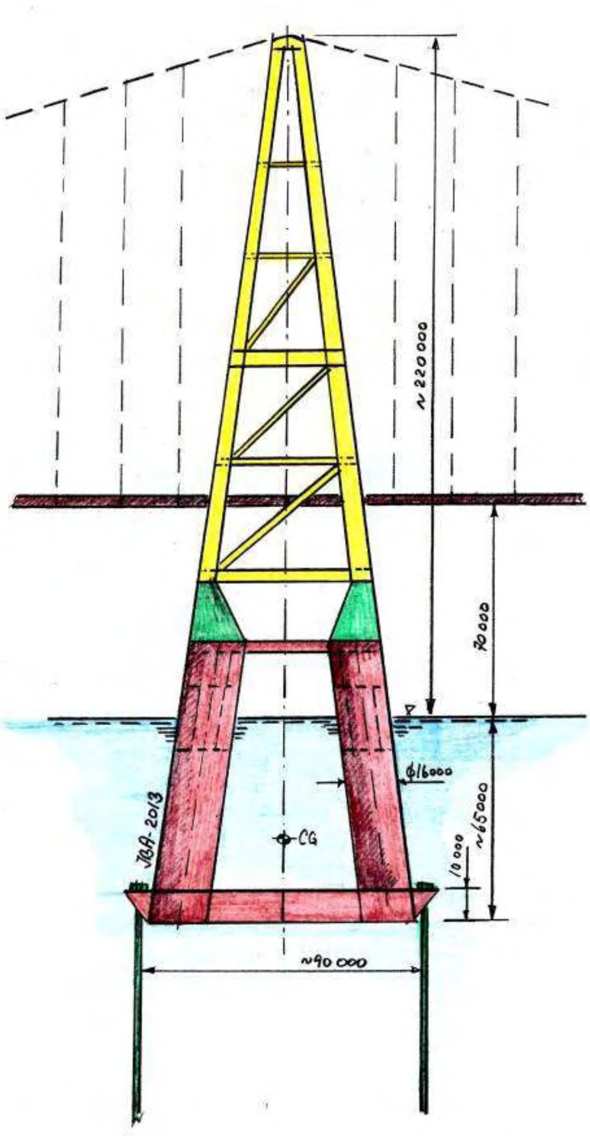


*Fig. 9. Proposal of a "TLP foundation" used as foundation for bridge tower.*

**DESIGN AND FABRICATION OF A "TLP FOUNDATION"**

Below some thoughts around bridge tower design and fabrication are given

1. Fabrication must be prepared for a shipyard or an offshore yard.
2. Same method as used for jacket fabrication must be evaluated. See fig 10.
3. Roll-up of the towers. Eliminating the vertical height during fabrication. See fig. 11.
4. The structure is skidded out on a heavy lift vessel, prepared for sea-transport.
5. Transported oversea and launched for free floating. See fig. 12.
6. The sea transport may be avoided by fabricating the "TLP foundation" at a local yard.
7. Up-ending by use of ballast water flooded into the lower tanks.
8. Smaller tugs and installation vessels will be used to move the righted bridge tower into final location. See fig. 13.
9. Use a smaller crane barge for installation of the tethers. See fig. 13.

*Fig. 10. Fabrication of a jacket on a yard.*

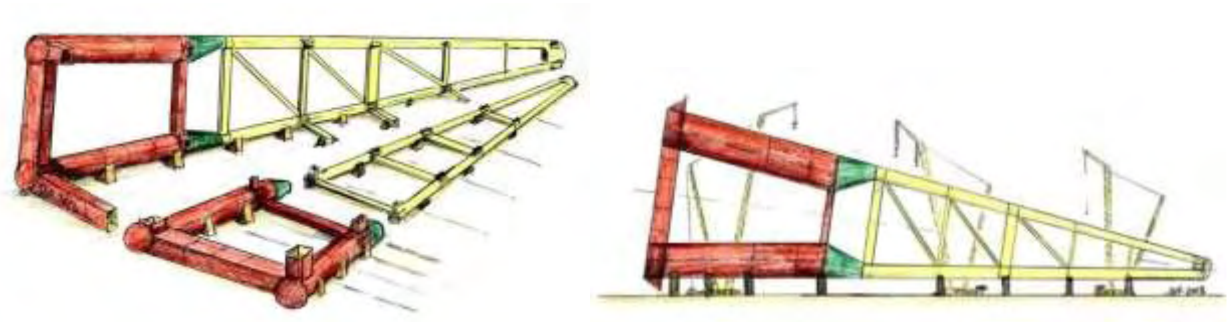

*Fig.11. Fabrication method with "Roll-up"*

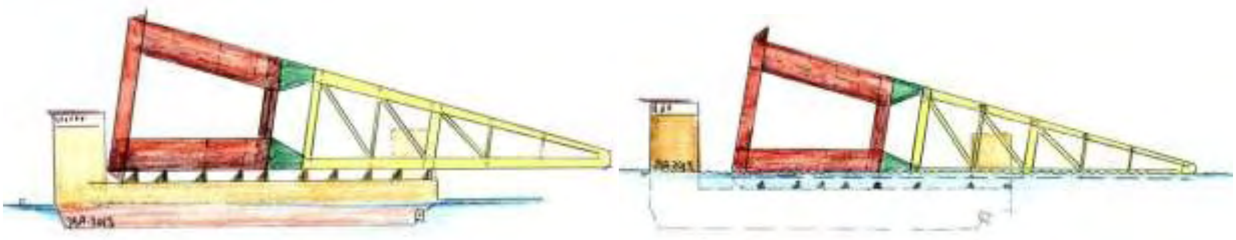

*Fig.12 Transportation of bridge tower from a yardinstallation site. Launched for free floating.*

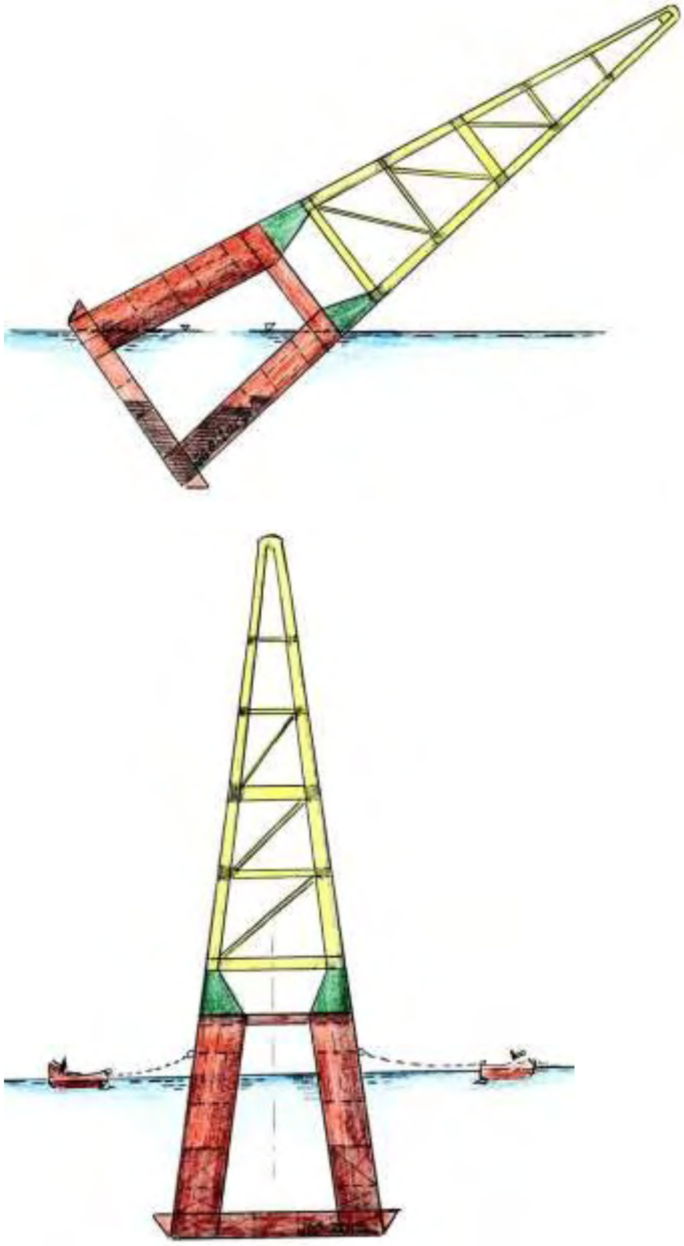

*Fig. 13. Ballasting lower tanks for upending. Then towed into final position.*

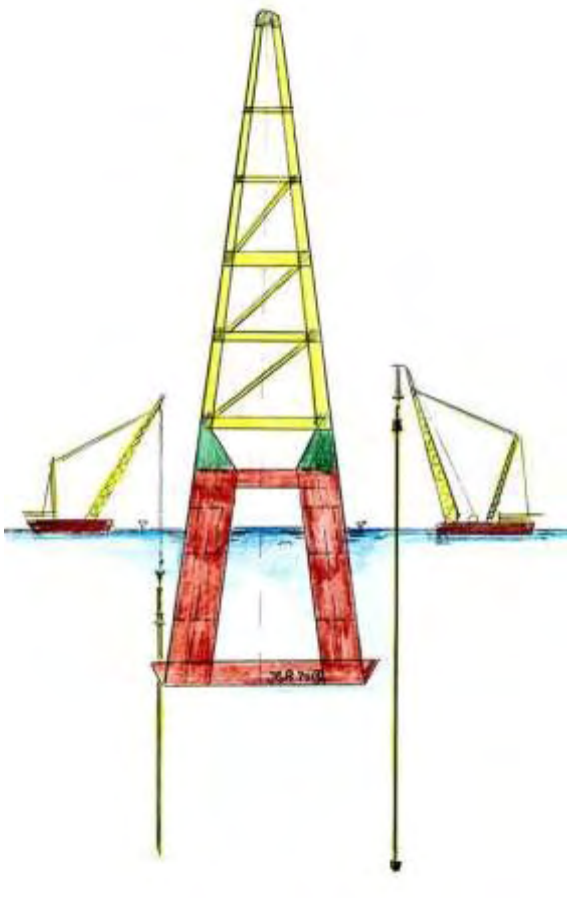

*Fig. 14. Tether installation.*

**SIMPLIFICATIONS**

Several details will be easier for a "TLP foundation" supporting a bridge tower compared to a TLP used as an offshore installation:

- The wave effect is less in a fjord, compared to North Sea.
- Reduced steel-weight.
- Reduced tether pretension and tether dimensions.
- Less number of tethers required.
- No outside risers to complicate the structure.
- Simple temporary ballast system for upending, tether installation and replacement.
- Simple tether installation by use of smaller installation vessel.
- Fabrication may be done horizontally and rolled-up as proposed.
- Fabrication might be done on a local yard.
- Barge transportation from a yard to the fjord (or free-floating tow).
- Simple ballasting for upending.
- Smaller vessel for handling and towing the TLP-floater required.
- Simple ventilation system for the hull.
- Simple instrumentation for controlling the hull.

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# TLP TECHNOLOGY: THE TETHER SYSTEM.

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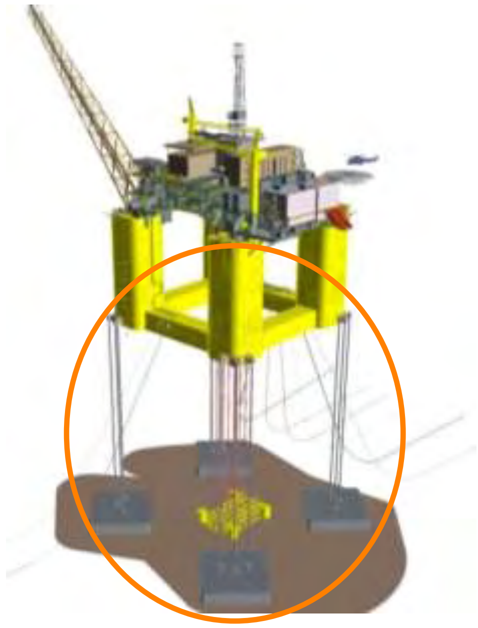
*Part of the paper" **TLP technology Experiences in the North Sea used as Foundations for a Bridge Tower**"*

*By Inge-Bertin Almeland*
*Specialist Engineer in Aker Solutions, Norway*




## ABSTRACT

*This paper is written to give an overview of tether systems which are used to connect offshore platforms to the seabed, as an alternative to conventional anchors with anchor lines or -chains. The features of tether systems might be of interest for use in the fixation of floating bridges to the seabed. Refer to the "TLP Technology Experiences" paper in ref. /1/.*
*In the paper a description is given of the components that form a tether system. Based on currently operational Tension leg Platforms (TLP's) various alternative solutions for the different elements of a tether system are shown.*
*The design methodology for a tether system is described, highlighting the design requirements and conditions. The design process results in an engineering basis for the detailing of the tether system components and the definition of manufacturing requirements for a tether system. Subjects that receive high focus during the design process are highlighted.*
*Based on design requirements general manufacturing specifications and fabrication requirements are presented. Depending on the set-up of a tether system, typical installation methods are shown. Possibilities and limitations are discussed. Repair scenarios for some typical damages are presented. Possible problems and high-risk operations in each of these stages are discussed.*

*Each of the above discussed topics will be visualized with numerous practical examples. Aker Solutions experience with the Snorre A platform is presented from idea to reality –20 years of service to-date.*
*A preliminary design for the tether system of the "Boknafjorden" bridge is presented and the chosen solutions discussed.*

## INTRODUCTION

The experience with anchoring of offshore platforms is of particular interest when considering floating bridge concepts. In this paper tether systems as used on today's offshore platforms are presented, to give an impression of the main principles of such systems. The advantages and disadvantages are discussed and with these in mind, a proposal for a tether system for the Boknafjorden Bridge is discussed.

Since the tether systems on both Snorre and Heidrun were developed in an early phase of the development of tension leg systems, today these systems are -though functioning as supposed- outdated and major improvements have been achieved. Therefore a modern tether system is presented.

## GENERAL

### Definitions

Prior to describing tether systems as in use on today's Tension Leg Platforms (TLPs), a brief description of frequently used terms is given.

- The terms *tether* or *tension leg*, are different terms describing the same; the terms *tether string* or *tether* describe the same.
- A tether is (free to Wikipedia) is a tough band that connects two bodies and is capable of with-standing tension.
- A tether system consists of a certain number of tether strings or tethers, which connect the floating platform to the seabed.
- One tether string composes of various main elements:
  - Foundation and bottom connection assembly
  - Tether main body
  - Top connector assembly and tether porch

  Each of these elements will be further described in section 3.
- *Seabed* is the bottom of the sea

**Tether String**

In the adjacent figure a typical tendon string is given, showing the different elements:

- Top connector assembly with tendon porch and tendon string top.
- The tether string main body.
- The tether bottom connector assembly, with tendon string bottom connector and receptacle, which is connected to the foundation at the seabed.

In the following the design and engineering of a tether system is described. Also the various components in use on modern tether systems are presented.

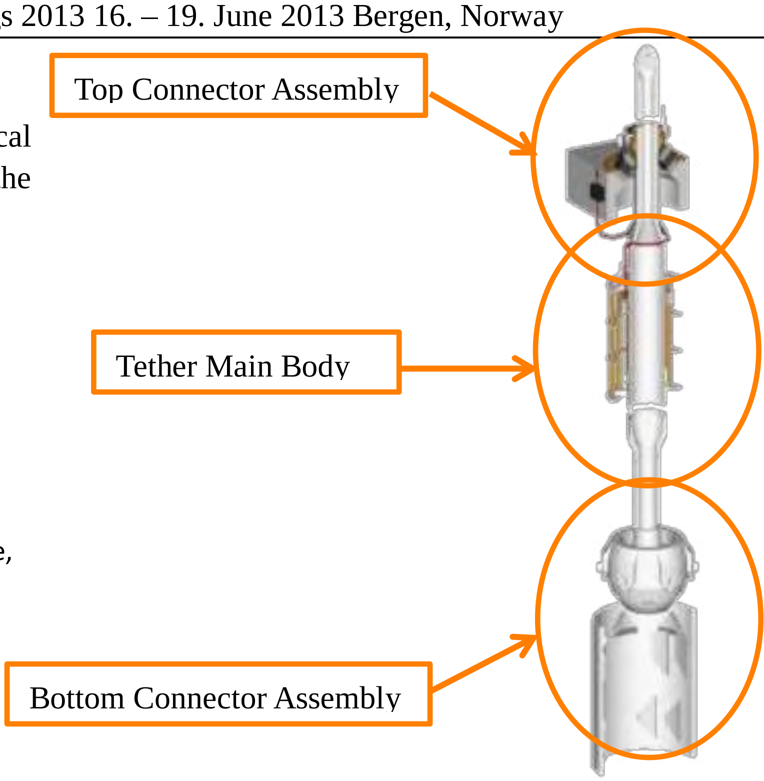



*Figure 8 Typical Tether String*

## DESIGN AND ENGINEERING

### General

The design of a tether system is determined by the following parameters:
Environmental conditions
Water depth
Platform geometry and dimensions
Platform allowable offset
Practical issues regarding installation maintenance and repair of the tether system components

The first two items are determined by the location of platform. Wind, waves and current forces acting on platform and tether system induce loads on the platform and tethers.

Platform geometry and dimensions are determined by hull weight, riser loads, anchoring loads, weight of hydrocarbon-processing equipment and other equipment, accommodation, etc.

Platform offset is limited by the type of risers used.

All of these factors are influencing the design of the tether system.

**Platform Design**

The Platform geometry is first of all determined by the amount of buoyancy needed to equalize the total sum of weights of the platform. This weight is found in a repeated process of estimating and detailing of hull and equipment onboard.

Once the platform weight is known, the hull, which is the buoyancy-generating part of the platform, needs to be shaped. The design of the hull determines the platform reactions to wind, waves and current. High, slender structures generally experience less wave induced forces then low, wide structures. Also the layout of the hull, like the shape, number and position of columns, the size and shape of pontoons, the presence of bracings etc. influences the motional behaviour of a platform in waves.

When a hull shape has been chosen, the performance of a hull in the offshore environment needs to be analysed. An important factor for the motional behaviour of the platform is the kind of anchoring system, and the number and type of risers used. The main advantage of a tension leg system to anchor is the near elimination of vertical motions. The only vertical motions that might occur are due to elasticity in the tethers and set-down following platform offset.

**Tether System Design - Dimensioning**

The purpose of the analyses is to quantify the loads that will be acting on the tether system. The three main load-groups are:

1. The (maximum) static and dynamic loads
2. The minimum loads: when a tether would experience loss of tension, there might be a danger of unhooking the (bottom) connection, losing the tether completely.
3. The fatigue loading during the lifetime of the tether system.

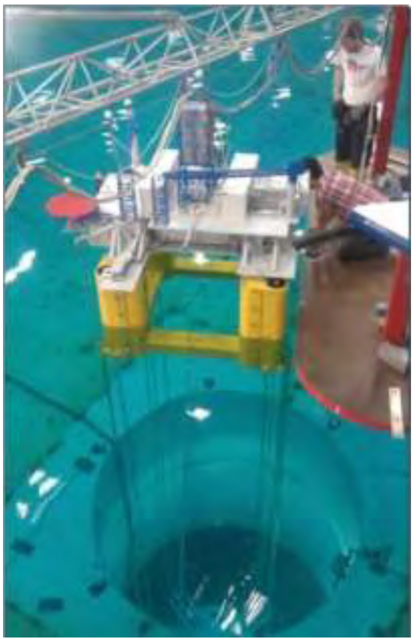


*Figure 9 Model of Platform in Test Tank*

**Tether System Design - Layout**

Once the main requirements for the tether system are set, the system can be detailed: Sizing of the components, connectors and pipes, determination of lengths of the various segments, choice of the type of top and bottom connectors, and so on. National authorities or classification societies like DNV have issued requirements for tether systems, ref. /1/. Also oil companies have developed standards, often exceeding the international requirements. Specifically Shell has a much used set of standards.

When detailing the tether system layout, the installation method is the most important factor steering the layout of the system. The way a tether string is installed, and the kind and size of the installation vessel set the limitations for the system during installation. The lifting capacity and lifting height determine how long and how heavy the separate parts of a tether string can be. The tether system connector design currently available, meets a wide variety of conditions and requirements covering dynamic, environmental and safety concerns:

- All static and dynamic tensile, shear and torsional loads from the platform into the tether string are transferred to the foundations base
- Capability to withstand considerable fatigue loading
- Top and bottom connector assemblies allow for angular variation in the tether string, inducing limited bending moment in the platform tether porches and the foundation (normally limited to approximately 10°)
- Repeated latching and unlatching of the various connectors in the tether, for the purpose of repair or replacement, is possible
- Hook-up of the connectors is rotationally independent

In the next section a short description of the various components of the tether system is given.

## TETHER SYSTEM COMPONENTS

### General

The paper on TLP Technology Experiences in the North Sea, as presented by Inge-Bertin Almeland presents the features of the Snorre-A platform. The Snorre-platform was one of the first platforms moored with a tension leg system and development of tether systems had not started yet. Therefore most of the components are developed and designed in-house by Aker and Kværner. Since then Aker Solutions has been involved in the development of various tether systems.

Presently there are around 25 TLPs installed and active globally, of which only 3 in the North Sea. The development of tether connection systems is done by only very few companies. Most of the platforms today use tether components from a single source supplier. This is an unfortunate situation from a commercial point of view, but technically the experience and quality of the products is well-proven. The components presented in the following are from the single source.

### The Foundation

The way a tether system is connected to the seabed, is dependent on the geological structure and composition of the seabed. Prior to determining which type of foundation is suitable, a thorough investigation of the bottom structure needs to be made, using a combination of the available methods for bottom research, for example:

- Seismic profiling of the bottom at the location of the foundations of each bridge tower
- Test drilling to achieve bottom samples
- Cone Penetration Tests (CPTs)

Dependant on the bottom structure currently different types of foundations might be applied. Each type of foundation has specific features and is suitable for a type of geological composition. In the following some types of foundation are shown, but foundations might be designed in other ways, suitable for the local purpose.

Subsea Piles: steel pipes, possibly with a piling support structure, that are hammered into the bottom

*Figure 10 Pile Guiding Support Structure*

- Gravity based structures: Large hollow structures that are filled with a heavy mass after being installed on the seabed.

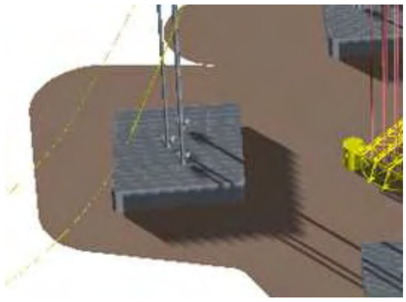


*Figure 11 Gravity Based Structure*

Partly penetrated gravity based structures: structures that are penetrating the bottom with the lower end. Afterwards the top part is filled and/or covered with a mass. This type of foundation was used on the Snorre-A platform.

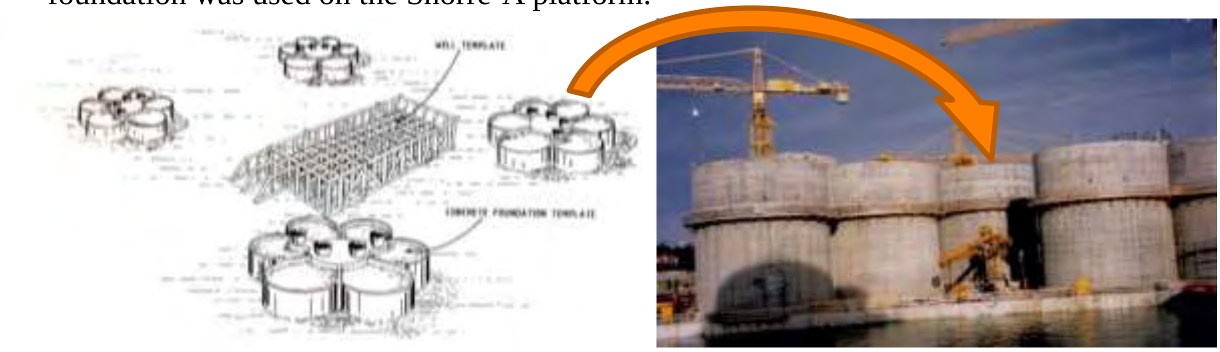


*Figure 12 Partly Penetrated Gravity Based Foundation*

**Tether Bottom Connector Assembly**

Apart from the general requirements mentioned in section, there some specific issues and requirements to be considered for the design of the bottom connector:

- Possible bending moments between the tether string and the foundation must be limited during angular displacement of the string
- Entry of the bottom connector into the pre-installed receptacle on the seabed must be self-guided, without the necessity of a specific radial orientation to mate the two elements
- Latching of the mechanism must be passive and continuous, i.e. the bottom connector must stay fully engaged unless clear and deliberate action is taken to release the latch
- It must be possible to rapidly and remotely latch or unlatch the tether string from the base
- Temporary loss of tension in the tether string must not impose compressive forces on the foundation or tether string structure
- The system design must be capable of meeting a wide variety of conditions and requirements as designated by dynamic, environmental and safety concerns

Though there are two or three different systems, there is only one reliable type of bottom connection. This patented roto-latch system is applied on most of the TLPs currently in operation. The tether part of the bottom connector assembly consists of a forged steel shaft, connected via flexible connection to the bottom connector head. The casting forming the head, has lugs on the outside hooking to the receptacle, which is welded onto the foundation. The lugs also guide the head correctly into the receptacle. The latching and unlatching process is shown in the process below.

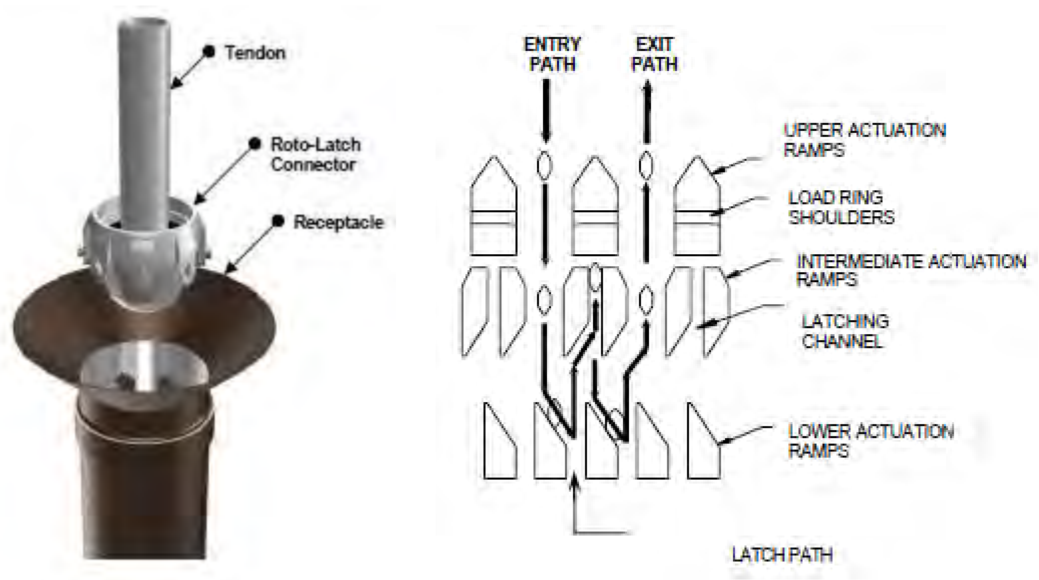



*Figure 13 Bottom Connector Assembly and Functioning*

The receptacle is a made of a steel forging and is welded to the foundation. Inside the receptacle slots are machined which guide the lugs to latch or unlatch the connector to the foundation. After latching locking pins are to be installed to prevent the connector from unlatching in case of accidental loss of tension.

**Tether String Main Body**

The tether structure connecting the platform (via the top connector) to the seabed (via the bottom connector) is called the main body of the tether. The structure normally consists of steel pipe, possibly split in limited lengths and connected with special tether connectors, depending on water depth and the method of installation.

*Figure 14 Tether main Body Segments Prior Installation*

For simpler floating structures, steel wires might be used, dependant on requirements and loads. For platforms, the loads are generally too high for steel wire to be applicable. Smaller floating objects might be moored by use of steel wire tethers.

When a tether main body segment is constructed out of several elements, these need to be connected. For this purpose a patented connector type is used, based on a circular, threaded pin- and box principle. Connection is achieved by using a special tool, forcing the pin and box together. Using hydraulic pressure the box is expanded, allowing the pin and the treading to slide into place, thus creating the connection.

**Tether Top Connector Assembly**

The top connector assembly design is determined by various critical factors, besides the requirements in available regulations like.    The top connector should be able to maintain tension, but during the installation process no upward load shall be absorbed in the assembly. Connection to the threaded top    of the tether, the length adjustment joint (LAJ) is achieved via threaded wedges or slips, which sit in a seat that is located on the porch structure of the hull. The load bearing ring (the seat) is resting on the porch on a flexible ring, the flex-element. The figure below shows details of a typical top connector assembly.

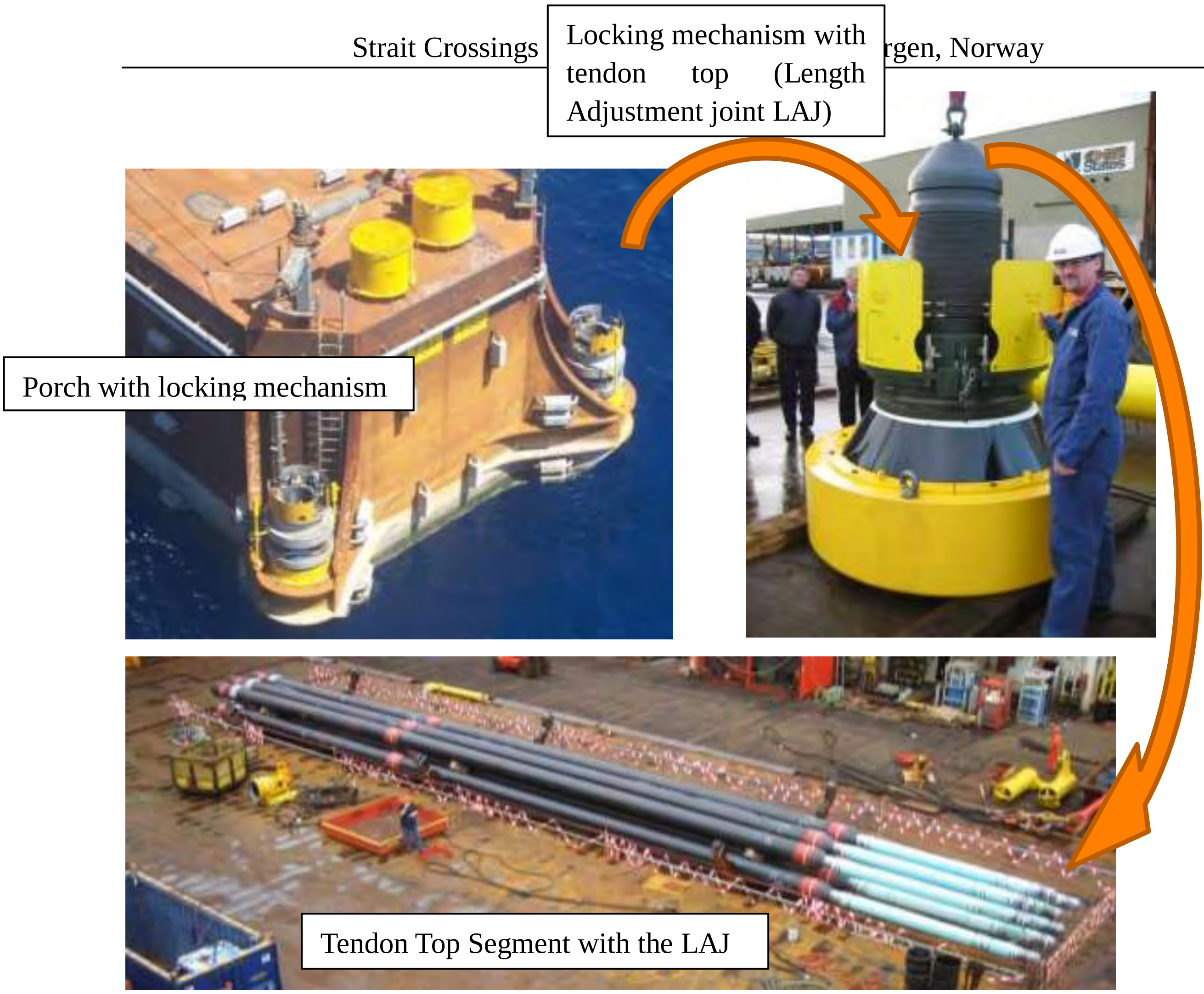



*Figure 15 Typical Top Connection Assembly*

**Other Vital Components**

Without going into the details of these components following components are necessary, and will require careful design and analysis prior installation:

1. A tether tension monitoring system: other than visual inspection, during the lifetime of a tether system, there are very few ways to thoroughly inspect tethers. Therefore the continuous monitoring of the tension on the tethers is an important tool. In case of damage to one of the tethers, the change in tension will indicate possible problems.
2. A corrosion protection system. For obvious reasons the steel material of the tether system needs protection against corrosion. Based on corrosion analyses, a system of anodes will need to be mounted along the tether system.
3. Corrosion covers over the top connector assemblies.

## FABRICATION, INSTALLATION AND REPLACEMENT

### Fabrication

Due to the extreme loads and fatigue acting on the tether string, the materials and fabrication of the tether system are subject to very strict requirements. In a cyclic loaded cross section, the loads will peak at the locations where the material cross section is minimal. Since a tether is subject to fatigue over long periods, often at relatively high loads, tight control of the materials, welds and welding geometry and tolerances during manufacturing are of highest importance. Regulations define fabrication tolerances for fatigue exposed structures based on S-N curves.

### Tether Installation

Installation of a tether system is generally done in steps:

1. The tether segments need to be transported from the manufacturing site to the installation location.
2. After installation of the foundations, the tethers will be installed. The tether segments need to be rotated from the horizontal transport position to the vertical position for installation. When several connections need to be made up for connection, the lower section will be hung of on the installation vessel side, in a special spider. Then the next segment can be liftin on top and the connection made up, lowered down, and then hung-off again. Until the tether string is finished. With the tether string hanging in the crane, the bottom connector need sto be inserted in the receptacle, where it is latched as shown previously.

*Figure 16 Hang-off of Tether Segment*

3. After installation a Temporary Buoyancy Module will be mounted at the top of the tether string, in order to keep the tether connected to the foundation with some slight tension, prior to arrival of the platform.

*Figure 17 Tether Buoyancy Module*

4. When the platform arrives on location, the tether tops will be attached to temporary winches, after the platform has been towed into position.
5. During the ballasting of the platform the winches will pull and guide the tether top into the tether porches.
6. After ballasting to the final draught, the locking mechanism is engaged, and de-ballasting is started immediately after locking of the top connectors is conformed.
7. When the correct amount of ballast water is pumped out and the tension is verified, the temporary winch wires can be released, and the Temporary Buoyancy Modules Removed

**Tether Replacement**

During the lifetime of a tether, inspections are required, but only visual inspections are possible by use of divers and remotely operated underwater vehicles (ROV's). Maintenance should not be necessary or be minimal. Due to accidents or failures in the tether structure, it might be necessary to replace one or more tethers. The tension on the tether string needs to be diminished by ballasting the platform. The tension should be low enough to release the top connector, but tension should be maintained in the other tethers.

When the top connector has been released, the reverse procedure of installation shall be used to remove the tether string and install a replacement.

**TETHER SYSTEM PROPOSAL FOR BOKNAFJORDEN BRIDGE**

Referring to the presentation of the Boknafjorden bridge concept in the TLP Technology paper (ref./1/), a tether system consisting of eight tethers per tower is proposed. Two tethers will be placed in each corner of the tower.

Each tether will be welded together of steel pipes, into one single string, with top and bottom connector assembly as described in section. The tethers will be welded horizontally onshore and towed out into the water. Reference is made to the paper in ref. /3/, where the principle of towing and installing a riser is verified (a riser is very similar to a tether, both in geometry and construction).

The tether string can be manufactured onshore in a controlled atmosphere. After finishing it can be drawn into the water while sliding on a roller track. A second tug then connects to the free and, and by paying out a controlled length of towing chain, the draught of the tether can be controlled.

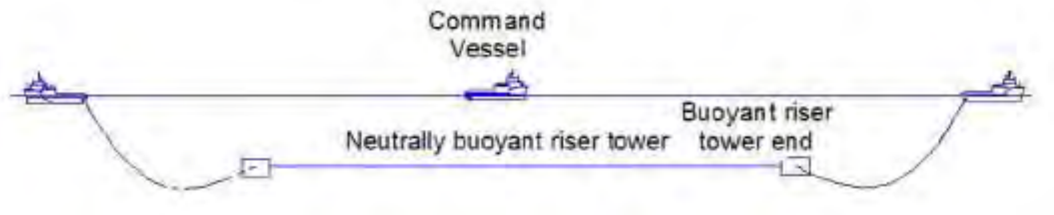



*Figure 18 Typical Tow of a Tether String or Riser*

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**Abbreviations**

DNV Det Norske Veritas
LAJ Length Adjustment Joint
ROV Remotely Operated Vehicle
S-N curve Graphical presentation of the dependence of fatigue life (N) on fatigue strength (S)
TLP Tension Leg Platform

# CHALLENGES AND SOLUTIONS OF FUTURE CHANNELS CONSTRUCTION OVER STRAITS OR BAYS IN CHINA COASTAL AREA

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## ABSTRACT

*This paper briefly introduces the main achievements of large infrastructure construction sea-crossing engineering and the advances on technology of the bridge and tunnel construction in coastal areas of China in recent 30 years. At the same time, aiming at the climate, geology, ecological environment condition of these planning and constructing Channels cross Bohai Strait, Taiwan Strait, Qiongzhou Strait in China, discusses and analyses the faced technical difficulties and key issues in three strait channels construction. For the probable planning schemes to be taken, such as the cross-sea bridge, crossing-sea tunnel, combination scheme of bridge and tunnel, their comparison is completed. It is pointed out the rationality of the considering submerged floating tunnel (SFT) scheme, the technical difficulties to be studied and solved and application prospect. The current study should focus on the design theory, analysis and construction method, disaster prevention and emergency rescue, prevention terror attack, the risk analysis evaluation and control system, and relative new materials development and application of submerged floating tunnel and bridge of the super long span sea-crossing channel.*

**Keywords**: *cross-strait channel, construction, sub-sea tunnel, bridges, submerged floating tunnel*

## 1.INTRODUCTION

With the rapid development of China's economy, especially the rapid development and concentration of population density in the coastal areas, the traffic demand is always increasing. To the more than 18000km coastline in eastern China, Mr. Tang Huancheng had put forward an magnificent project in 1989, that coastal channel (highway or railway lines) starts from Shenyang, take ways in Dalian and Lushun, cross Bohai Strait to Penglai, Yantai, Lianyungang, cross the Yangtze River estuary to Shanghai, cross Hangzhou Bay to Ningbo, then to Wenzhou, Fuzhou, Xiamen (side across Taiwan Strait, connecting Taiwan), Guangzhou (from Shenzhen to Hong Kong), Zhanjiang, finally cross Qiongzhou Strait to Haikou, ends in the Sanya. The construction of this line has a great role in promoting economic development in China's coastal areas. But because there are some natural long water areas, deep water straits and bays, those become the bottlenecks in constructing fast traffic connection channel. Conquering them become

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dreams of the people on both sides of the straits and workers engaged in transportation infrastructure construction for decades. Figure 1 gives the schematic diagram of the coastal channel and several typical straits or bays.

China started putting forward to developing economy, strengthening infrastructure construction, and gradually improving the living standard of the people since 1978, after 30 years of construction and development, it has made remarkable progress and achievements. In civil engineering field, the development of science and technology of China, the progress of material and the connection with the economy and trade of the world, all provide the conditions and platforms for Chinese engineers to design and construct these large civil engineerings. China has been able to build span 1600m-long suspension bridge, super cable-stayed bridges with main span more than 1000 m, 300m-long prestressed concrete bridges, 550m-long concrete filled steel tube arch bridge, the Olympic stadium - the bird's nest has completed in 2008 and the domestic highest, the world's fourth highest tower—600m high Guangzhou TV tower, which has completed in September 2010. A large number of bridges or tunnels over coastal bays or straits channel have been built or under construction and planning, such as the already built Qingdao bay bridges, Sutong Bridge connecting Nantong and Zhangjiagang across Yangtze River(main span of 1088m, the world record of cable-stayed bridge, 2008), the Chongming Island channel across the Yangtze River estuary, Xiamen-Xiang'an undersea tunnel (the first sub-sea tunnel in mainland, 2009), 36 kilometers long channel bridges across the Hangzhou bay (2008), Zhoushan Island-Land Project connecting Ningbo and Zhoushan crossing four islands, whose total length is 28 km, all use bridge connection, including a main span of 1650m suspension bridge-Xihoumen Bridge, and the construction of the Hong Kong, Zhuhai and Macau channel engineering，which adopts the scheme of bridge and tunnel, etc. These channel bridges are generally set their foundations at the position where water depth is about 50m or more to cross the water areas.


*Fig.1 Diagram of coastal channels and typical large channels or bays plan of China*

Some of bridge and tunnel engineerings over straits are being planed, such as: Bohai Strait channel, Taiwan Strait channel, Qiongzhou Strait channel. The characteristics of these channels are that lines across the straits are long, water is very deep in some areas, the span is beyond the largest spanning capacity of the existing bridges, the marine geology and climate are complex, and deep foundation construction and tunnel excavation are very difficult. How to use the fruits of the development of modern science and technology is a challenge faced by civil engineers for planning, choosing, designing and constructing these channels reasonably. It is also the dream of workers engaged in the traffic infrastructure construction for years and the necessary request of national economic development.

## 2.TO BE PLANED CHANNELS OVER LONG-LARGE-DEEP WATER AREAS

### 2.1 Bohai Strait channel

The planning Bohai Strait channel[1] starts from the southwest corner of Laotie mountain in Liaodong peninsula in the north, to the Penglai Dengzhou corner in Jiaodong peninsula in the south, the shortest distance is about 57 miles (105. 56 km) between two sides, the average depth is 25m. The northern Laotie mountain waterway is about 42 km wide, the average water depth is more than 40m, the largest depth is about 86m. In the southern Miaodao archipelago, there are many rock outcrop islands can be used. There are a total of 32 islands, 66 bare reefs, 16 submerged reefs, 2 long beaches (water depth is 1.6~4.0m) from north to south, the maximum depth is about 20m.

For a long time, because of the cutting off by the wide channel, a large number of passengers and cargoes in the northeast and east China, south China coastal areas are transported in "C" type around Bohai Bay, or by short shipping. This kind of transportation pattern greatly limits the development of circum-Bohai and the coastal area’s economy.

**The geology conditions of Bohai Strait**: downside of the exposed stratum is phyllite clamped with quartzite, upside of it is slate clamped with quartzite, rock is relatively hard. Over the stratum is eluvium, diluvial slope-diluvium and marine deposition. Eluvium is the composition of gravel and red brown loam of 1~3m thick, diluvial slope-diluvium’s thickness changes largely, general thickness is 20~40m. It can be more than 60m at some local positions. The diluvial slope-diluvium mainly consists of gravel, sand containing gravel and clay, which distribute on both sides of the valleys, slope zones and gentle places. Some clip with gravel layers and are rich in calcium. The marine deposition is mainly composed of gravel, clay, silt, marine biological remains and shell, which distribute in gentle places of each big island coast.

Bohai Strait’s geological structure is relatively stable., There are 21 earthquakes more than level 4 in the 421 years’ record between 1548~1969, of which 4 times is more than level 6, 2 times is more than level 7, the others are earthquakes below level 3~4. These earthquakes below level 3~4 do not cause destructive threat to bridges and tunnels, but for the important engineering, the seismic safety evaluation is required[2].

**Climate and ecological environment**: Bohai Strait is influenced by typhoon in summer, the wind effect strength is more than 12 grade. The district is cold in winter, and has ice, snow and severe frost. Bohai Strait is abnormal tidal and current sea area. The biggest range of tide is more than 4m, the sea current in summer is generally 0.6~1.03 m/s for the southern sea area, and it is commonly 1.2 m/s for northern sea area.

Miaodao archipelago consists of 32 islands of different sizes, being groups in south and column in north, the forest coverage rate is as high as 53%, warm in winter and cool in summer. It is the only place which must be passed by birds migration. The marine organisms in the areas around the islands are rich, the ecological environment is good, it is a national nature reserve.

### 2.2 Taiwan Strait channel

Taiwan Strait channel is the planning main channel from Mainland China to Taiwan, across Taiwan Strait. Taiwan Strait is in the northeast, southwest direction, with a length of 375 km, a width of about 120~350 km, in which the northeast section is narrow, southwest section is wider. Terrain of it is generally like this: southwest (Taiwan bank) is high, the depth is between 10 to 30 m; Northeast (Taiwan Strait basin) is low, the depth is between 40~60m. Southeast section is lower, the maximum depth in Penghu waterway is 185 m, the depth at the submarine canyon in southwest island is above 200~1000 m [3].

The geological conditions of Taiwan Strait, Rock stratum is mainly sandstone, siltstone, basalt, hard shale clamped with powder sandstone the south. rock stratum is marine clastic rock, shale and sandstone alternating layers in the north and east. Partly covered by a thick layer of 50m[4].

Taiwan Strait is located in the edge of Eurasian plate, in the circum-pacific seismic belt. There are a lot of fault zones, it is an earthquake-prone area. According to preliminary statistics and historical records, the earthquakes whose Ms≥5.0 have taken place for 85 times in the Taiwan Strait, of which Ms = 5.0~5.9 for 55 times, Ms = 6.0~6.9 for 23 times, Ms≥7.0 for 7 times, the biggest magnitude is 7.5 [5].

Meteorological and hydrological conditions of the strait: in the winter from October to march of next year, the wind above level 6 is 37%~53%, waves on the sea is often 2~3m high. In summer from May to September, typhoons at level 7~9 are common, with an annual average of 3~4times, wind effect is up to level 12. Spring and fall are often influenced by wind above level 6, and the surge of level 5 at the same time[6]. There are often heavy fogs in winter and spring, especially in March to April.

### 2.3 Qiongzhou Strait channel

The planning Qiongzhou Strait channel, which connects Zhanjiang and Hainan. The strait is about 80 km long in east-west direction, and about 30 km wide in north-south. The narrowest position is in the middle, with a width about 18.6 km. It is shallow in the west and deep in the east, water depth in the middle is more than 50m (part of the water depth is 80~114 m), width is 10 km. On both north and south sides of the strait is scarp, the maximum altitude difference of scarp is up to 70 m, the largest slope can be up to 22°~24°. The eastern channel is a series of shallow notching seated alternately, at some places the depth is only 20~30 m, west narrows is a huge underwater delta, it is shallow, with a depth of only 40~50 m[7].

The geological conditions of Qiongzhou Strait: cover layer over the submarine stratum mainly consists of sand clamped with clay, mix of clay and sand or floury soil interbed, thick clay and floury soil interbed below them, The bed rock is deep [8].

From 1400 to 1995, a total number of recorded earthquakes whose Ms≥4.75 is 31 in this region, of which more than level 6 is 9 times, the biggest magnitude is 7.5, which appeared in 1605 in Qiongshan. Seismologists predict that the probability for earthquakes at level 7 and above is very small in the coming 100 years.

Meteorological and hydrological conditions of Qiongzhou Strait: It is located in the tropics, which has a tropical monsoon climate. Every year from May to November is the typhoon season, especially in September, the average continuous days of wind at level 8 is more than 5. It is foggy in the Channel, foggy weather is more than 24 days for a year.

Qiongzhou Strait has a rare typical diurnal tide in the world. Maximum tidal difference is about 5 m, the increase or decrease of surge caused by wind and storm is needed to be considered. There are tidal current, radiation flow and density flow, wind flow and runoff, etc. in the strait. The hundred year tidal difference is about 6.5 m in Qiongzhou Strait.

## 3 THE CHALLENGES AND TECHNICAL DIFFICULTIES OF CHANNELS ACROSS LONG-LARGE-DEEP WATER AREAS

At present, the ferry is the main way to cross the long-large-deep water areas. It can be found from these channels' position, geology, marine conditions, climate, environment, because of the long distance, deep water in some place, limit spanning capacity of the existing bridges, the complexity of marine geology, current, the influence of fog, monsoon, typhoon and tide, all of these bring great challenges to build channel engineering across the these straits, construct deep foundation, excavate under seabed and operate in all-weather. The technical difficulties of fixed ways-underwater tunnel and cross-sea bridge are emphatically discussed as follows.

### 3.1 Technology and difficulties of underwater tunnel

To ensure the all-weather operation, avoid bridge construction in complex marine and deep water environment, underwater rock tunnel or immersed tube tunnel could be used in some deep water or foggy areas. There mainly are drilling and blasting method, immersed tube method, shield tunneling method and TBM method in the tunnel construction [9].

If use the drilling and blasting method for tunnel construction, the main technical difficulties are as follows [9] :

- Geological survey in deep water is more difficult and expansive than it on land, and the accuracy is relatively low, the risk is high.
- The high pore water pressure will reduce the effective stress in the surrounding rock, influencing the stability of the strata. Fault fracture zones are vulnerable to sudden flood water. Underwater tunnel can't drain naturally, how to prevent water infiltration is the

key technology of construction.

- Underwater tunnels have a long excavation length. Requirement of ventilation is very high during construction. The discharge of waste muck is also need special consideration.

Although using the shield tunneling method similar to the English Channel tunnel has many advantages, such as high degree of automation, safe construction, work environment is relatively good, when it applies to the channel tunnel construction, the main technical difficulties are:

- Shield tunneling method is extremely sensitive to the geological and hydrological conditions. Construction risk is high when the unverified geological conditions and obstacles exist. Excavating in saturated loose sand, it needs to keep stability of the excavation operation face under higher water pressure and earth pressure.
- Changing cutting tools and repairing cutter in the process is complicated.
- Factors such as anti-floating of the tunnel tube, underwater shield machine maintenance and submarine connection should be considered during long distance driving under water.

The immersed tube tunnel belongs to shallow buried tunnel. At present, although using immersed tube method to build underwater tunnel has been relatively mature, but when it comes to crossing the strait, deep water, all kinds of complex undercurrent, and big changes of submarine foundation bring difficulties in the foundation treatment, design, construction and stable operation, the risk and cost is very high.

### 3.2 Submerged floating tunnel

Submerged Floating Tunnel, referred to as "SFT", is also called "Archimedes bridge" in Italy, referred to as "PDA" bridge. It generally consists of tubular structure floating at a certain depth in the water(the structure has very large space, enough to adapt to the requirements of road and railway transportation), anchor cable and device fixed in underwater rock (or tank floating on surface of water, this device can prevent big displacement) and the connecting structure to the shore. It is a kind of new transportation structure for crossing straits, long lake and waterway[10]. SFT can be prefabricated on shore, floated to a certain place, sunk and installed. Fig. 2 shows the schematic diagram of typical SFT.

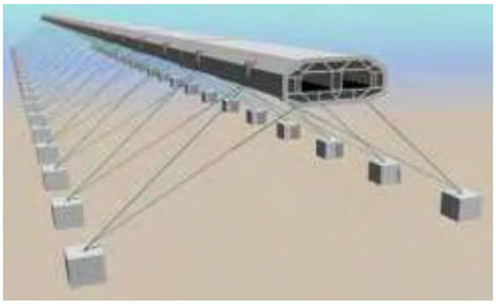

*Fig. 2 Typical Submerged Floating Tunnel*

Compared with the traditional bridges and tunnels, SFT's length is shorter than immersed tube tunnel and underground tunnel.. It is particularly suitable for constructing in the areas where water depth is more than 50~200m. The unit length cost of SFT almost remains unchanged. It is also convenient to protect environment ion and lay various pipelines [11].

As a new way of traffic crossing, SFT is still under research, there are still many key technologies to solve before the official building.

1) In addition to directly affected by the moving vehicle loads and the seismic actions, SFT is also affected by the incentive effects of wave, current, tide and other environmental load, anchor cable and floating pipeline are vibrating. So, it is necessary to study the mechanical behavior of the floating tunnel tube and anchor cable, especially fluid-solid coupling vibration under all kinds of dynamic loads.

2) SFT is in the marine environment for long term, the corrosion of seawater on the tube material and anchor cable may be very serious. Under the effect of dynamic loads, the anchor cable vibration can easily lead to fatigue failure. It would influence the security of the whole structure. Therefore, the durability of the structure and resistance to fatigue failure should be paid attention. Corresponding solutions should be put forward.

3) Due to the tunnel is at least 30m underwater, once collision disasters or terrorist attack accident take place in the tunnel, the consequences is more serious. How to improve the design safety reasonably, take corresponding measures when the accident event happens, evacuate people as soon as possible, organize rescue on the water and assure maximize safety of SFT are very important problems.

### 3.3 Technical problems of bridge

Although China has accumulated valuable experience in the construction of the East China Sea (Donghai) bridge, Hangzhou Bay sea-crossing bridge and Zhoushan Island-Land project, the complicated geological conditions, weather conditions in strait areas and the higher requirements of the span, all make the bridge engineering face with many technical difficulties.

**1) Problem of wind**

China's coastal areas are influenced by typhoon and monsoon seriously, the biggest wind level is above 12, this is a major test to bridge's wind resistance. Although pneumatic performance of large span bridge structures can be represented and solved partly through the site measurement, wide tunnel test, theoretical analysis and numerical simulation[12]. But solving the wind problems of bridges under complicated climate conditions thoroughly is still need further research and engineering test, at the same time, traffic safety in the strong monsoon also cause the attention of the engineers.

**2) Problem of deep foundation**

At present, China's engineers have mastered the capacity to build deep foundation with 50~80m and pile foundations with more than 100m deep. There are examples like setting 2.13m diameter steel piles into the 130m dust under 300m deep water and directly setting floating open caissons or bell foundations at the depth of 80~100m abroad[13], but building bridge foundations in 100m deep ocean is still no precedent. Building deep foundations in the deep water areas with thick soft sediment is still confronted with difficulties and challenges.

**3) Problem of spanning capacity**

Bridge's spanning capacity is restricted by many factors, such as materials (inchuding strength and mass), structure type and construction technique. The largest span built is 1991 m for suspension bridge (Akashi-Kaikyo bridge, Japan), 1088m for cable-stayed bridge(Sutong Yangtze River bridge, China). The proportion of bridge's self-weight in its bearing capacity increases with the span. Therefore, at present the span of bridges built by traditional materials and forms is close to its limit. The development and application of new materials have important significance to the improve bridge span.

The 5000m bridge scheme has been put forward in the competition project over Gibraltar Strait. Ideas of building 3000-3500m span bridge in the sea-crossing channel over Italian Messina Strait and channel over Chinese Qiongzhou Strait have also been proposed. On the basis of material and engineering technical progress, the largest span of bridge is expected to obtain a breakthrough by continuous exploration innovation.

**4) Problem of durability**

The cross-sea bridge is in the marine corrosive environment, chloride ion is the main factor which cause steel, concrete corrode. In the long span concrete cable-stayed bridges, about 20% of the concrete main girders are cracked. This is the one of most serious durability problems which were met in concrete cable-stayed bridges. Once the cracks of concrete appear, chloride ion penetration intensifies. It will seriously damage the bearing capacity of the bridge and reduce its service life. Other types of long-span bridges are also facing with the cracking problem. For the cross-sea bridge whose investment and influence are huge, its design life should be longer than a common bridge, 120 to 150 years , even 200 years may be considered. During this period, whatever is the structure itself and its external environment will constantly change, engineers should consider the influence of all kinds of changes during design, construction and operation. Therefore, the designed structure must be capable of inspection, repair, change, maintainance, control and sustainability. Engineers have to know the life of the whole structure is not completely equal to the life of components. For the components whose own life are less than the whole life of structure, it is ensure that they must be checkable, convertible, repairable, and can be strengthen to meet the need of durability of the whole structure [14].

## THECHNICAL PLANNING IDEAS AND SOLUTIONS

Table 1 gives the planning China's coastal channel ideas and solutions.

*Table 1 Planning China's coastal channel ideas and solutions.*

| Name | Scheme | Characteristics | Merit and demerit | Preliminary Conclusions |
|---|---|---|---|---|
| Bohai Strait channel[1] (Fig. 3) | ferry for train in recent year | Adopt train ferry directly | Greatly affected by weather, not convenient for transferring persons and material, low transport efficiency. It is a temporary measure of connection. | "Bridge in South and Tunnel in North" is reasonable. |
| | "Bridge in South and Tunnel in North " for atuomobile [15] | 1)The distance between islands in Miaodao archipelagic area is about 2~5km, the average depth is 25m. Hence, bridges in a line shape in the south can be built. 2)In the 42.2km wide Laotieshan waterway, the average water depth is more than 40m, the maximum depth is 86m. Tunnel can be adopted for crossing. | 1)The islands distribution pattern in the south provides a rare work platform for the bridge construction, and greatly reduces the difficulty and cost of the project. 2)In the Laotieshan waterway, due to the complex marine environment, geological conditions and deep water, building tunnel is relatively easy. | |
| | All tunnel scheme[15] | Including deep buried tunnel and SFT, especially SFT can be considered for crossing the Laotieshan waterway. | 1)The weather and the environmental impact is small, all-weather operation, economic. 2)Using SFT can reduce the total length, driving longitudinal slope compared with deep buried tunnel, and it be helpful to traffic safety and energy and cost saving, etc. | |
| Taiwan Strait Channel (Fig.4) | Bridge scheme | Lin Yuanpei proposed the bridge scheme at the north line. Arrange several 3500m span suspension bridges in the place where the depth is less than 80m, total length is more than 100 km. | 1)North line's water is shallow, it is more convenient to build bridges, but it is in the circum-Pacific seismic belt and typhoon zone, and vulnerable to environmental impact, so it needs to consider wind and seismic problems, etc. 2)International waterway, the probability of ship hitting is high. 3)Strategic location is very important. 4)Atrocious weather and some particular time would interrupt the traffic. | It can build a bridge on the north line, but bridge is vulnerable to influence of earthquake and typhoon, tunnel length is the shortest, geological conditions is more stable than the other two lines. Considering the technical difficulties and economic factors, the north line is the best line scheme. |
| | Tunnel scheme | Proposed the north, middle and south schemes, namely: 1)the north line is from Pingtan, Fujian to Xinzhu, Taiwan. 2)the center line is from Putian, Fujian to central Taiwan. 3)the south line is from Xiamen, Jinmen, the Penghu islands to Jiayi, Taiwan. | North line, about 122km long, no earthquakes more than level 7 happened in the history, only has medium earthquake of level 5 with a low frequency. Center line, 128km long, the central terrain has the characteristics of "two trough and an upheaval", the largest water depth is 76~78m at west trough, and 62m at east trough, the depth at central upheaval is 45m. Near the west bank is very steep, east coast is relatively slow. Although no earthquake more than level 7 took placed in the history, there are strong earthquakes of level 5~6 | |

| | | | | |
|---|---|---|---|---|
| | | 4)Compared with bridge, tunnel has more advantages in the Taiwan Strait. | with a slightly high frequency.<br>South line, about 174 km long. The maximum depth of Penghu waterway is more than 100 m. Penghu waterway is in the fault zone, its seismic frequency is high [16]. The south line is beneficial to the connection between the islands across the Taiwan Strait. | |
| Qiongzhou Strait Channel (Fig.5) | Bridge scheme[18] | The width of the relatively shallow silt and water is about 14 km in the Qiongzhou Strait, the relative economic plan of bridge is about 2000~5000m. (1200+4×3000+1200) m and (1200+ 3×4000+1200) m bridge scheme is reasonable[19]. | Vulnerable to the typhoon and heavy fog; Deep foundation is complex, Under the present technological conditions, the 3000m bridge's spanning capacity is limited, the condition is not mature. | Considering weather, geology, hydrology, economy, etc., using bridge scheme is unfavorable. It is appropriate to use tunnel scheme, such as shield tunnel or SFT, especially SFT scheme is optimal. At present, it is the train ferry crossing the channel. |
| | Tunnel scheme | According to the characteristics of deep water, fast current and erosion and deposition on the seabed, suggest that the shield tunnel scheme is adopted.<br>When cross at the narrowest position (18.6 km), the length of sub-sea tunnel is more than 40km. | 1) The Buried depth decides tunnel's length and cost.<br>2) Set service hole between the two main tunnels. During construction period, the service hole can probe geological conditions, seek for the shield construction parameters and improve bad geological conditions in front of the main tunnels.<br>3) During the operation, the service hole can be used as maintenance channel, safe escape way and drainage channel [17]. | |
| | SFT | The length is about 20 km. | 1) Longitudinal slope is small, the length is short, all-weather operations.<br>2) The increase of the distance has no effect on its unit cost, the total cost is less than under sea tunnel, so it has obvious competitive advantages[20]. | |


*Fig. 3 Schematic diagram of Bohai Strait Channel Project*


*Fig. 4 North, south, middle three lines over Taiwan Strait*


*Fig.5 Qiongzhou Strait channel*

## 5. CONCLUSIONS AND OUTLOOKS

The construction of the three strait channels will play a great role in promoting the further development of China's coastal economy. Of course, it also faces with great challenges. Considering weather, geology, hydrology, economy, analysis, from the comparison of schemes, we can draw the following conclusions:

(1) "Bridge in South and Tunnel in North" scheme is advisable for the Bohai Strait. The tunnel in the north line is the best for Taiwan Strait. It is appropriate to use shield tunnel or SFT scheme to cross the Qiongzhou Strait.

(2) For the coastal sea-crossing engineering, bridge is not always reasonable choice. Sometime, tunnel has its unique advantages to a certain extent, and especial SFT as a new type of traffic structure across the deep water, it has the advantages of all-weather operation, little environmental impact, energy conservation and environmental protection, relatively reasonable cost, so it has a potential application prospect and is worth further studying and developing.

(3) At present, further systematic study on aspects of design theory, construction methods, disaster prevention, emergency rescue and evacuation system, risk analysis and control of SFT are needed. The corresponding design and construction standards or guidelines are to be proposed, so as to speed up realize SFT.

(4)Through the use of new materials and new construction technology, as well as continuously exploration and innovations to relative complex problems, the largest span and deep foundations are expected to get another breakthrough for the super long span bridges (3000-5000 m).

The large crossing-sea channel engineering should be built according to the actual needs and local conditions in site, long-term planning, staged investment and construction, by making full use of all resources and techniques. The largest characteristic of this kind project is that the early investigation, research and demonstration work are great and take very long time. It is believable that the ideas of super long-span bridges and SFT must be realized by the continuous effect of researchers and engineers, and natural moat must be changed to thoroughfare.

**ACKNOWLEDGEMENT**

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# FEASIBLE CONTROL STRATEGIES IN THE PROTECTION OF LONG SPAN BRIDGES AGAINST EXTERNAL DYNAMIC LOADS

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## ABSTRACT

*Two numerical models of long-span bridges, namely a suspension and a cable-stayed one, are herein developed in a commercial finite element code, starting from original data, and used to simulate the structural response under wind excitation and seismic excitation. Passive and semi-active control strategies have been proposed and implemented on the bridge structural models for mitigating the induced dynamic effects. Such control schemes have been designed and proven effective on the suspension bridge for wind action, on the cable-stayed bridge for the seismic action.*

*In light of this introduction, this paper is intended to collect the research group recent advances in the mitigation of unwanted vibrations on different long-span bridges typologies aiming to underlining general observations and strategies useful for their protection.*

*The attention has been initially given to seismic and wind excitation, characterized to high intensities and low probability of occurrence. The control performance in such conditions is typically related on the reduction of instantaneous structural variables, as accelerations, displacements, internal forces. However, by changing the input characteristics in terms of intensity and statistic, wind actions described by low intensity and high probability of occurrence has been addressed also on the suspension bridge model, by evaluating the effectiveness of the proposed control strategies in the mitigation of indirect effects such as fatigue damage in the steel frame deck.*

## INTRODUCTION

The contemporary world is faced with the great, and still increasing, number of large structures as long span bridges owing to the role they play for the social and economic transformations of the regions where they are built. A growing interest in control of such type of structures against strong external dynamic loads, as wind and seismic actions are, is constantly spreading through the designers for both existing and new types, driven by the existing standards and the struggle to expand the requirements of performance and safety to a wider range of different conditions. An example is the trend to the spreading of *Performance Based Design*, that include additional requirements to be comprised when special structures, as long span bridges, are considered. Il light of these considerations, structural control solutions can give a contribution so as to satisfy such requirements.

When civil structures, such as long-span bridges, are addressed a considerable uncertainty, including loading conditions, nonlinearity associated with physical properties, description of the excitations, such as earthquakes and winds, occurs. In this work, a model of an existing suspension bridge is firstly developed at the numerical level starting from original data, and it is used to simulate the structural response under strong wind excitation. Optimal passive and semi-active control strategies are implemented for the bridge protection, with due attention on their feasibility and reliability. Their efficacy is shown and the factors contributing to their positive performance are highlighted. Moreover, the proposed control strategies show interesting positive effects against fatigue damage in the bridge steel frame deck when lower wind intensities, with higher probability of occurrence, are also considered.

Subsequently, the attention has been focused on cable-stayed bridges, due to their natural frequencies characteristic, more prone to suffer earthquakes motions. A refined version of the international ASCE control benchmark model for an existing cable-stayed bridge is considered as a case study in order to evaluate the seismic performance of passive and semi-active control schemes.

Finally, a reversing of the type of the excitation has been considered. The control strategy, designed and proven effective for the wind action on the suspension bridge is tested for seismic excitation, while the one designed for the seismic action on the cable-stayed bridge is evaluated when the bridge is subjected to wind loading.

Assessing principles of the mitigating devices, in their both passive and semi-active configurations, one can recognize that they result able to act as an internal fuse for the forces applied by the deck to the supports, to dissipate part of the energy introduced into the structural system, to shift the main bridge natural frequencies far from the most dangerous input frequencies, to supply additional damping to the structure. Their technology spans from electro-magneto-rheological-fluid dampers, to electro-inductive ones [1].

## STRUCTURES

Two different bridges model have been developed inside a commercial finite element (FE) framework: a model of a suspension bridge and another one of a cable-stayed one. In the following a brief resume of these structures is exposed, as well as references to more exhaustive descriptions.

### Suspension Bridge

The suspension bridge is the Shimotsui-Seto one, located in Japan, spanning from the side of Mt. Washu to the Hitsuishijima Island. The main dimensions of the bridge are shown in Figure 1a. For the sake of brevity the interested reader is pointed to [2] for details on the geometry and the FE numerical model (Figure 1b).

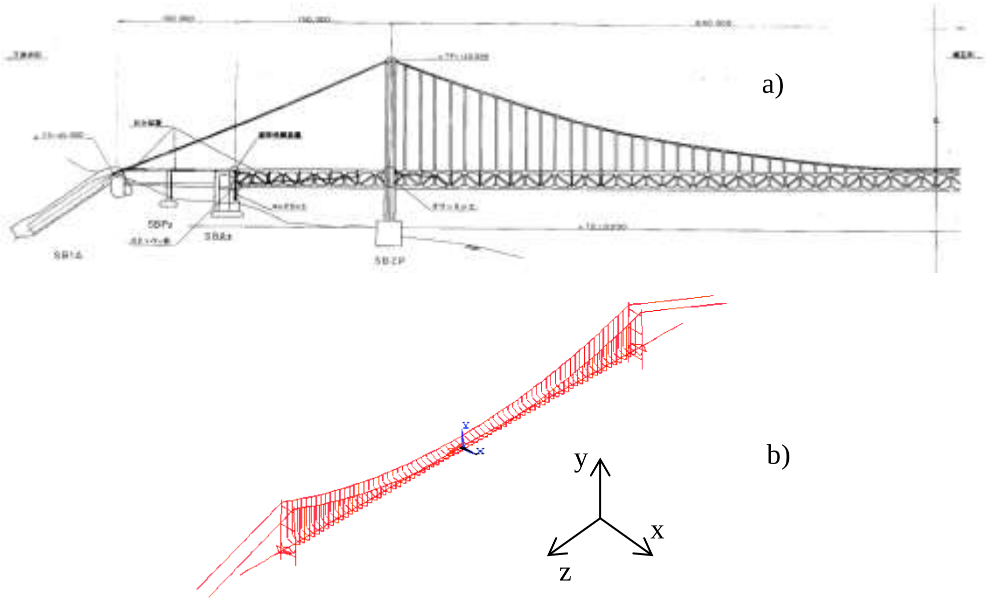


*Figure 1. Bridge geometry (a) and FE model (b). Courtesy of Mr. M. Nishitani HBSE-JP.*

The model is fully restrained to the ground at the towers' foundations and at the ends of the main cables. At the bents, dynamic translations and twist rotation are restrained while the remaining flexural deformations of the main girder are free. The deck is fixed to the towers, by connecting rods, for the vertical and twist movements. Transversally it is partially restrained.

Nonlinear geometric transient dynamic analyses are carried out on a simplified model derived from a more detailed one; this last having almost a one-to-one correspondence between structural and finite elements. Preliminary static and modal analysis, with the application of the MAC (Modal Assurance Criterion) method, validated the more manageable FE model. It is worth underling that the FE numerical model match well the real, experimentally measured, frequencies of the bridge.

## Cable Stayed Bridge

The control benchmark [3] is focused on a fan-type cable stayed bridge: the Bill Emerson Memorial Bridge, located near Cape Girardeau (USA), spanning the Mississippi River (Figure 2).

The bridge design is determined to be controlled by seismic effects. Due to temperature effects, in the longitudinal direction force transfer devices are present between the towers and the deck in the form of sixteen 6.67 MN shock transmission devices; under dynamic loads these behave as rigid links. In the transverse direction earthquake restrainers are employed at the connection between the tower and the deck, while the deck is constrained in the vertical direction. The bearings at bent 1 and pier 4 (Fig. 2) are designed to permit rotations about the transverse and vertical axis and thermal longitudinal displacements.

With respect to the original benchmark statement [3], a more general and refined numerical model in a multipurpose FE code has been developed for geometric nonlinear transient dynamic analyses, which considers not only the horizontal seismic components but the vertical component of the earthquake as well. Furthermore, the seismic input is not the same on all the supports but a coherence function taken from the literature is introduced to have at the supports different ground motions satisfying a fixed correlation function. The soil type regulates the correlation degree (lagged coherency) [4,5].

The new benchmark model comprises soil-structure interaction through the use of impedance functions [6,7], the piers and bents foundations are simulated by lumped masses with soil-equivalent springs and dampers.

The original benchmark statement does not consider degrees of freedom for the stay cables beside those of the extreme nodes, neglecting their modal and dynamic description. Focusing the attention on the simulation of the structural dynamics, the cable model is refined moving from the single rod type representation to a description with six rope elements for each cable.

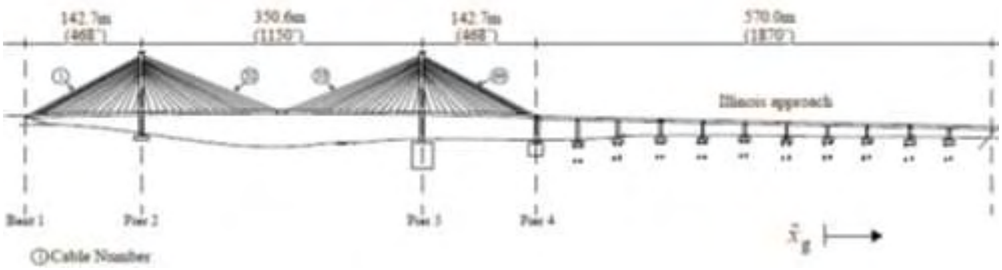


*Figure 2. Bill Emerson Memorial bridge geometry (from [3])*

## CONTROL STRATEGIES

Passive and semi-active strategies have been designed mitigating external dynamic loadings on the bridge structures. Specifically, for the suspension bridge the control system is aimed at mitigating the wind buffeting effects while for the cable-stayed one it is aimed at the seismic ones. The control devices are simulated and implemented into the structural systems through the passive Bouc-Wen model and the semi-active Domaneschi one [8,9]. It is worth underlining that, looking at the deck as an independent structure, for the suspension bridge the control is aimed at constraining the deck degrees of freedom by the application of additional boundary conditions, represented by control devices. On the contrary, for the cable stayed model, the control system releases the deck restraints in the horizontal plane.

Passive control strategies have been firstly considered on both bridges. As the control theory suggests, when the uncontrolled structure configuration originally employs rigid links, as the herein evaluated cable-stayed bridge, the implementation of passive strategies allow mitigating internal forces in structural elements paying the bill of larger deformations. On the contrary, when the control devices increase the restraints condition for the controlled object, force and displacements can be mitigated together.

Following this approach, the optimal passive solution has been detected, under fixed design excitation intensity, as the best compromise between internal forces mitigation and attainment of suitable effects in terms of displacements. Since the main disadvantage of passive strategies consists in their lower efficiency when the external excitation has a different intensity with respect to the design one, semi-active control systems have been adopted also.

Such choices reflect the control issues which indicate the passive and semi-active systems, or alternatively their hybridization, as the most promising for efficient applications in structural engineering, rather than active systems with their well known negative aspects (e.g. related to energy supply during extreme events) [10]. More details on the implemented control arrangements can be also found in [2,4,5,8,9].

## WIND DESIGNED CONTROL STRATEGIES FOR THE SUSPENSION BRIDGE

The system design for the buffeting low frequency vibrations control on the Shimotsui-Seto bridge model is performed by the application of a methodology inspired to the *Sequential Placement Algorithm* [2]; consequently the devices are set at points where the deck is next to the towers. The devices optimal parameters for the passive control strategy are obtained from an optimization procedure carried out by considering a reference design mean wind speed $v_m$ = 45.8 m/s. The objective functions are defined as the standard deviation of the internal actions at the base of the towers (shear in horizontal transversal direction $T_x$, bending moment about the deck longitudinal direction $M_z$) and the horizontal transverse mid-deck displacement $U_x$ (considering a Cartesian reference system in Figure 1b).

The semi-active control for the Shimotsui-Seto Bridge is evaluated with collocated and non-collocated settings and adopting a specially developed on/off Skyhook control law. A decentralized scheme with a low order control implementation is here adopted, following [8,9]. The semi-active devices are characterized by two configurations named respectively: SH150-300, which implements the parameters for the optimal passive case as the higher working state; SH300-750, in which the parameters of the optimal passive case define the lower working state. The “SH” in the name denotes the use of a Skyhook type control law.

Table 1 reports the main results at the design wind velocity where the positive performance of the controlled bridge configurations can be observed. If the mean values of the structural variables remain constant, the standard deviations (objective functions) are reduced, if compared to the uncontrolled bridge configuration. The semi-active control schemes perform better than the passive one when the SH150-300 configuration is employed, independently from the system collocation.

The positive performance of the semi-active solutions is highlighted when the mean wind level increases or decreases. In other words, the semi-active control schemes turn to be able to adapt themselves to different wind intensities, with respect to the design one, as the control theory suggests [8,9]. Additional details one these issues when higher and lower wind intensities are accounted for can be found in [2].

*Table 1. Mean and standard deviation for $T_x$, $M_z$ and $U_x$ ($v_m$=45.8 m/s)*

| | St.Dev. *Tx* (kN) | St.Dev. *Mz* (kNm) | St.Dev. *Ux* (m) | Mean *Tx* (kN) | Mean *Mz* (kNm) | Mean *Ux* (m) |
|---|---|---|---|---|---|---|
| Uncontrolled | 695 | 8452 | 0.26 | 3709 | 29054 | 0.56 |
| Optimal passive | 531 | 6399 | 0.13 | 3873 | 30960 | 0.55 |
| Collocated Skyhook 150/300 | 496 | 5993 | 0.14 | 3866 | 30877 | 0.55 |
| Collocated Skyhook 300/750 | 556 | 6699 | 0.13 | 3875 | 30987 | 0.55 |
| Non colloc. Skyhook 150/300 | 501 | 6060 | 0.14 | 3866 | 30879 | 0.55 |
| Non colloc. Skyhook 300/750 | 549 | 6618 | 0.13 | 3877 | 31000 | 0.55 |

Figure 3 depicts the power spectral density (PSD) response in the frequency domain at the bridge mid-span in terms of lateral bending moment. It is worth noting the positive effect of the control schemes implemented in terms of amplitude reduction. Moreover, the control devices, as expected, supply additional constraints for the bridge superstructure, shifting the deck main lateral frequencies toward higher values. Even if small, the better performance of the semi-active solution can be recognized.

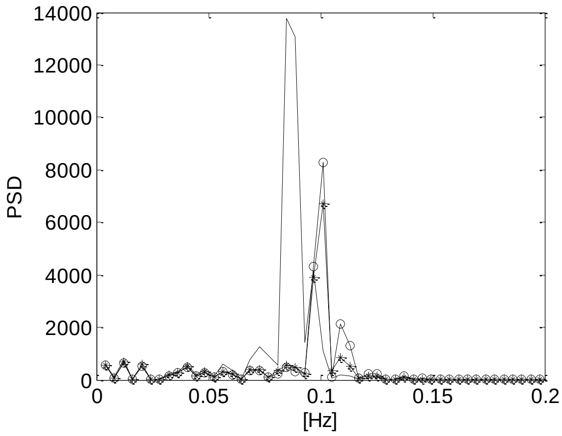


*Figure 3. PSD of the lateral moment $M_y$ in the bridge deck at the mid-span ($v_m$= 30 m/s). Continuous line – uncontrolled, circles – passive controlled, stars – semi-active version*

The optimization process for the passive control system herein developed has been preformed by a simplified model of the buffeting excitation. Wind actions are applied on the towers, the cables and the deck considering drag forces only. A re-evaluation of the control optimal parameters into a refined indicial formulation, to reproduce bridge deck motion induced wind forces, is under development by this research group. The first results show that there is no variation of the previously identified optimal passive configuration [11].

**Seismic performance of wind designed control strategies**

Such wind designed control schemes have been evaluated under the Kobe seismic record. Table 2 reports the extreme values of the internal actions at the towers base while Table 3 lists the results relative to the deck response. While in terms of tower internal action there is little or no difference between the control schemes (only the more dissipative semi-active configuration leads to a slight improvement), for the deck response all the control systems reduce the selected internal forces response parameters (up to 15-30%) with a displacement range very close to that of the uncontrolled structure. The best performer is the most dissipative collocated semi-active one (CSH300-750).

*Table 2. Base tower internal actions ([kN, kNm])*

| Bridge Configurations | $T_x$ | | $M_z$ | |
|---|---|---|---|---|
| | Max | min | max | min |
| Uncontrolled | 44680 | -35920 | 392340 | -501100 |
| Optimal Pass. | 45410 | -36750 | 401650 | -509830 |
| CSH150-300 | 45360 | -36780 | 401980 | -509310 |
| CSH00-750 | 44530 | -35360 | 386370 | -499650 |

*Table 3. Deck lateral bending moments $M_y$ and displacements $U_x$ ([kNm, m])*

| Bridge Configurations | Deck $M_y$ @mid-span | | Deck $M_y$ @tower | | Deck $U_x$ displacements | |
|---|---|---|---|---|---|---|
| | max | Min | max | min | max | min |
| Uncontrolled | 617040 | -619190 | 287760 | -388020 | 0.2273 | -0.2391 |
| Optimal Pass. | 539260 | -429800 | 230920 | -269210 | 0.2698 | -0.1876 |
| CSH150-300 | 538280 | -432990 | 233090 | -266280 | 0.2729 | -0.1926 |
| CSH300-750 | 462570 | -388350 | 222500 | -274360 | 0.2176 | -0.1620 |

The seismic performance has been evaluated for different levels (50% and a 200% scaled) of the seismic input. Under such conditions the semi-active control system should adapt itself and perform better than the passive scheme. In both cases the internal forces in the tower do not vary much between the uncontrolled and the controlled case, with CSH300-750 showing a slight reduction. The control schemes are in general effective both in reducing the deck displacements and internal forces. The best performer is the CSH300-700 scheme.

It is worth underlining that such different configurations of the control parameters in the devices can be implemented by the technology presented in [1]. It allows switching from one set (e.g. semi-active 150/300) to another one (e.g. passive) by an external command, operating on the device controller.

**Fatigue damage mitigation**

Moving from strong wind excitation to lower intensity and higher probability of occurrence loadings, in this subsection the effectiveness of the proposed control strategies mitigating the fatigue damage in steel members of the Shimotsui-Seto bridge frame deck has been evaluated. Cyclic loading from interaction with the wind is regarded as an important source of fatigue damage for structural steel members in decks of long span bridges. The standard deviation of the internal forces in the bridge deck (lateral bending moment) is a valuable parameter for assessing possible fatigue effects suffered by suspension bridges as a result of vibrations induced by buffeting [12,13]. The proposed control strategies are able to reduce significantly the value of such parameter for both gale and strong type winds [14] which results in a valuable decrement of the fatigue damage [15,16].

The efficacy of the proposed control strategies in reducing the potential damage for the bridge deck is investigated by looking at the internal moment in the horizontal plane around *y* axis. Figure 4 depicts the mean and the standard deviation of the lateral moment for 30 m/s mean wind velocities. The horizontal axis is the position along the bridge deck from the bent to the middle.

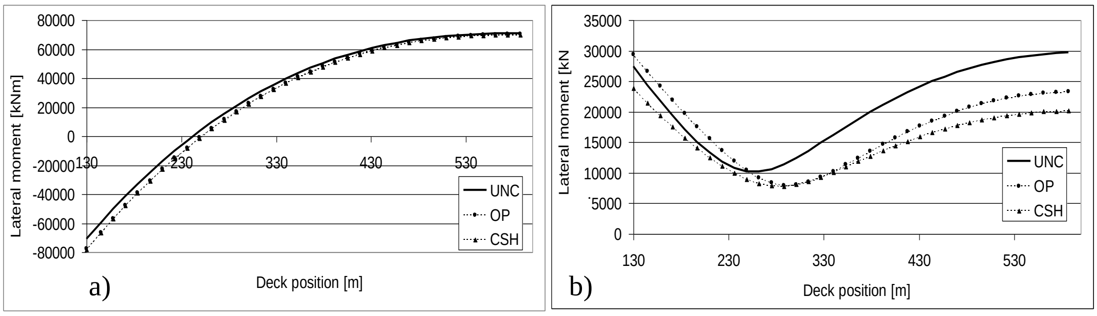


*Figure 4. Mean (a) and standard deviation (b) of the lateral moment at deck positions for $v_m$ = 30 m/s. UNC: uncontrolled. OP: optimal passive. CSH: collocated Skyhook 150-300*

The mean value of the lateral bending moment is quite insensitive to the control strategy since it is related to the mean wind velocity. On the contrary the standard deviation is sensible to the control configurations. For the CSH case, is always reduced at deck mid-span and at the towers also. More details on the control of fatigue damage can be found in [14].

Further investigations on this interesting aspect, related to the mitigation of indirect effects of loading, such as fatigue damage, by structural control systems is now under development by this research group. The first outcomes are promising and highlight how the statistical description of the average wind velocity, for the wind loading, and the fatigue strength characteristics of the deck members are crucial.

## EARTHQUAKE DESIGNED CONTROL STRATEGIES FOR THE CABLE-STAYED BRIDGE

A large number of control systems have been studied for the Bill Emerson Memorial Bridge, which is the object of the international control benchmark [3]. Among the others, hysteretic passive and semi-active dampers are proposed in [8] in the original statement, subsequently in [4]

the passive ones have been re-evaluated comparing two different device technologies in a refined numerical model of the bridge. In [5] a new isolator device, the Roll-N-Cage one, has been proposed and studied on the Bill Emerson Memorial Bridge refined model.
In this section some results from previous investigations under earthquake motions are summarized and new developments of the controlled bridge performance under strong wind excitations are presented. A passive control system, consisting in hysteretic devices, has been firstly adopted as seismic retrofit strategy for the Bill Emerson cable-stayed bridge thanks to its ability to decouple the deck motion from the excitation, to dissipate the seismic energy by supplying additional damping, to shift the main bridge natural frequencies far from the most dangerous input seismic frequencies.

Different settings have been considered for the devices connecting the deck with the piers: 16 control devices are implemented below the deck in the horizontal plane. At each support point along the deck (Bent 1, Piers 2-3-4 in Figure 2) a device is positioned in the longitudinal and the transversal direction. The devices are characterized by the hysteretic behaviour simulated by the Bouc-Wen model into a commercial FE code through the implementation of an external user-element, as detailed in [4,8].

A parametric analysis has been performed in order to obtain, within a passive scheme [8], the values of the yielding forces and the post-yield stiffness so as to optimize the response reduction in terms of the shear force and moments of the piers, and of the displacement of the deck. When such passive scheme is adopted, the devices take-on the following characteristics: 1000kN yielding limit, 80000kN/m stiffness, 2% post yielding stiffness.

A semi-active decentralized scheme has been then introduced by the Domaneschi model [9] with the aim of ameliorating the control performance in different conditions. As in the case presented in the previous section for the Shimotsui-Seto Bridge, the whole control arrangement is partitioned in sub-systems, collocated and low-order for each subsystem associated to each device. The devices are semi-active in the sense that they are able to change their hysteretic dissipative cycles by increasing or decreasing the elastic limit force. When the semi-active properties are implemented the elastic limit is fixed to 1000 kN for the high state of the Sky-Hook algorithm and to 250 kN for the low state [4,8].

Table 4 reports the mean results on five seismic sets [4] for the uncontrolled and the passive controlled bridge configurations. It is worth noting as the internal forces are generally mitigated by the control system, in particular the standard deviation is strongly reduced. Also the extreme values of the shear force and of the bending moment at the tower base show a useful decrement. Accordingly to the control theory of seismic effects, these positive results come at the cost of a small increment of the structural displacements.

Table 4. Mean on five seismic sets ([m, kN] x longitudinal, y transversal, z vertical axis)

| | Uncontrolled | | | | Passive Controlled | | | |
|---|---|---|---|---|---|---|---|---|
| | Mean | Std | Max | Min | Mean | Std | Max | Min |
| Deck mid span displacement (X axis) | -0.0031 | 0.0311 | 0.0790 | -0.0801 | -0.0108 | 0.0458 | 0.0957 | -0.1275 |
| Tower base shear (X axis) | 3667 | 6837 | 21346 | -13242 | 398 | 1592 | 5595 | -6154 |
| Tower base shear (Y axis) | 11440 | 3635 | 25658 | -1209 | 11359 | 2222 | 20252 | 2879 |
| Tower base: bending moment (Y axis) | 42966 | 74447 | 286795 | -225504 | 43147 | 47776 | 202749 | -119488 |
| Tower base bending moment (X axis) | 75427 | 53738 | 471563 | -306210 | 5613 | 41641 | 119763 | -135949 |

The semi-active solution shows its powerful performance when the input intensity is different from the design one. In particular, it is able to adapt itself to the specific demand, increasing or decreasing the dissipation level, being also capable to regulate the motion decoupling between the superstructure, the deck body, and the bridge supports, piers and bents [8,9].

## WIND PERFORMANCE OF SEISMIC DESIGNED CONTROL STRATEGIES

Wind loading derives from generated 3D turbulent wind fields, non-homogeneous in space to consider the atmospheric boundary layer, it is applied on the deck and the cables nodes of the updated bridge model in the horizontal direction, transversal to bridge span. Only the drag contribution is accounted for and the wind action is simulated as a spatial correlated process. More details on the wind simulation procedure can be found in [2].

Figures 5a illustrates the transversal base shears at the windward side of Pier 2 for the uncontrolled and controlled bridge options. They are quite similar even if the positive effects of the semi-active choice are reasonable (Figure 5a). These positive outcomes, in terms of internal forces at the pier base, are associated with larger transversal deck displacements (Figure 5b) with respect to the uncontrolled configuration of the bridge.

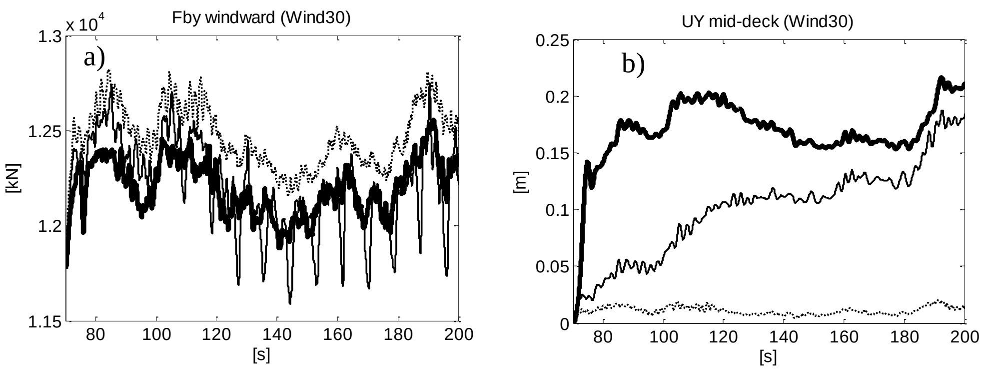


*Figure 5. Windward, transversal base shear force at Pier 2 (a). Transversal mid-deck displacement (b). Lines: dotted-uncontrolled, thin-passive, thick-semi-active CSH150-300*

## CONCLUSIONS

This work is focused on gathering the recent advances in the mitigation of unwanted dynamic vibrations in different long-span bridges typologies and on collecting general observations and strategies helpful for their protection. Passive and semi-active solutions are considered and their effectiveness is demonstrated.

Examples of control solutions, designed for one type of excitation and tested under a different one, are given. In spite of having designed control strategies for one type of excitation (wind for the suspension bridge and earthquake for the cable-stayed one) and tested for a different one (earthquake and wind respectively), they result effective in both conditions. Thanks to the characteristics of the available control devices technology, a specific tuning of the control parameters, online with the external loading as well, can be exploited, adapting the system performance to several conditions.

## ACKNOWLEDGEMENTS

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# DELIVERING THE FOREVER OPEN ROAD PROGRAMME

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**ABSTRACT**

*FEHRL's flag ship, the Forever Open Road (FOR) programme (http://www.foreveropenroad.eu/) is based on a clear vision for roads in the 21st century. It aims to enable, by 2025, a suite of innovations to allow road operators to improve their cost-performance of road operations by 50% based on a 2010 baseline.*

*The vision that FOR is based on, includes a number of key features that comprise not only roads, interchanges and the physical structures that support them, but also communications, IT, the control of information systems and the financial methods and systems for raising the investment for construction, operation and maintenance of road infrastructure.*

*This paper explains the development of the activities behind FOR including the creation of the first European-US cooperation in the competitive programming of research project. Finally the links with activities relevance to the Straits Crossing activities are considered including the relevance for other transport modes.*

## 1. THE DRIVERS BEHIND THE FOREVER OPEN ROAD PROGRAMME

In order for Europe to have a sustainable and inclusive economy, it should stay in the centre of the global trade routes. Fact is that Europe's global competitiveness is strongly linked to its success in maintaining a world class transport infrastructure. In this context the main ingredients of its 2025 road system are likely to be the major links and corridors needed for transport within Europe and beyond. These corridors, whilst largely road-based, in the period to 2025 would increasingly become integrated across the modes for passenger transport and provide logistical connections for freight, including routine, scheduling and real-time decisions in the movement of freight and passengers; a vision of seamless transport from origin to destination.

Another key element is the integration between road infrastructure and the users, vehicles and services. Already the overwhelming majority of European population is connected to online services. This will progressively expand to levels that might be beyond current comprehension. On a slightly slower but still progressive pace, the telemetric integration between vehicles and road infrastructure will increasingly influence driving patterns. The challenge is not only achieving the right mix between in-car and road side systems but also ensuring (cross border) interoperability of the information systems, not in the least where it is concerned with information exchange between road operators. A similar trend is occurring for the integration with logistical and mobility services.

From a broad perspective, a far-reaching and yet realistic vision is required for a number of reasons. Transport policies that are aimed at short term, and fact free decisions simply will not deliver the aspired global role and position of Europe's economy. Instead, evidence-based decision-making within a clearer overall picture of where to go and be over the years to 2025 is vital.
It will enable the enormous technical potential being offered by new computing and communications systems to be maximised and the growing importance of the road and, in general, transport user to be much more fully recognised. It will enable sustainable and cost-efficient solutions, sometimes quite novel ones, to be developed.

Obviously, the development of this vision requires a joint effort in organizing and coordinating over the relevant European, national and industrial programmes. This involves removing the many current barriers for innovation, for example through establishing common standards, specifications and guide lines. In addition, a well-conceived and executed knowledge transfer programme, aimed at capacity building across Europe and where needed addressing the work force in the local language, could reinforce the pace in which road operators could upgrade their infrastructures to the demands of the 21$^{st}$ century.

Finally, the road network is founded mainly on infrastructure that is intended to last for many decades. As a result, future implications for, in some cases, one hundred or more years, should be considered and evaluated carefully.

FEHRL examined many of these elements within its comprehensive vision of roads in 2025 that was published in 2004 [1]. This Vision document established a firm conviction that a new way of looking at road infrastructure was required and perhaps a reconsideration of how we define what is infrastructure. In 2011, FEHRL carried a comprehensive assessment of the requirements of the two main stakeholder organisations for Europe roads, the European Commission and the Conference of European Directors of Roads (CEDR). The result was FEHRL's fifth Strategic European Research Programme (SERRP V) [2]. This analysis of the EC's White Paper on Transport [3] and CEDR's Strategic Plan [4].

The analysis of the CEDR and EC strategies lead to the following key issues that can be attributed to road operations:

- Energy-efficiency. Through intelligent traffic management and advanced material properties (rolling resistance), road operations can contribute considerably to the required increase of energy-efficiency of the (road) transport system.
- Safety and Security. Next to intelligent traffic management and proper maintenance, the design of roads has a major influence on the casualties in road transport.
- Reliability and Availability. The disturbance/disruption of the traffic flow due to failures, incidents and climate effects can be largely overcome by innovative measures on the durability of materials and components, the speed and planning of maintenance and repair activities and the adaptation to extreme weather events.
- Liveability impacts. The liveability features of road operations need to be improved considerably. In particular through better planning, design and construction as well as through intelligent traffic/demand management.

- Cost and Financing. In developing and maintaining infrastructure, the financial perspective will always be important. New financial instruments will be developed and the whole-life models on which they are based will need to be reviewed.

## 2. THE FOREVER OPEN ROAD PROGRAMME

Following this view of the challenges for roads in the 21$^{st}$ century, FEHRL has established the Forever Open Road (FOR) programme as its flagship. FOR will enable the next generation of roads by 2025; a generation that is capable of meeting with society's future demands for reliability, availability, maintainability, safety, environment, health and cost of road operations.

The concept of FOR has been developed with the acknowledgement that our current road transport network already struggles to meet society's demands and expectations. Indeed, under the pressure of rapidly-developing societal challenges and fiscal constraints, it has already become difficult, if not impossible, for road operators to ensure the 24/7 operation for 365 days of the year. In the 21$^{st}$ century, society expects better and needs better.

The FOR programme will enable a new concept for intelligent roads that can be adopted both for maintaining the existing network and building new roads. The concept is an intelligent road that will enable future road operators to adopt emerging innovations, such as electric vehicles and automated driving, whilst overcoming the increasing constraints on capacity, transfer, reliability and integration.

To ensure constant availability, FOR will involve a new system of travel by road; whereby the vehicle, its driver and the road operator are integrated through a common communications and power system; where the operator automatically provides inbuilt vehicle guidance, as well as travel information and performance measurement.

Power systems will match the needs of emerging electronic vehicles, as well as harvest solar energy to service road systems requirements. The road will be built from sustainable material, will cope with excess water and temperature change, and will be able to clean and repair itself. It will also be built to adapt to future maintenance needs, changing capacity demand and evolving vehicle concepts.

### 2.1 Benefit and result driven

The FOR concept is conceived in line with the ambitions set out by the national road authorities and the EU to enable their road operations to provide the world's highest levels of reliability, availability, maintainability, safety, security, health, liveability at the lowest cost possible.

This is to be achieved in the recognition that the effective utilisation of the available road network should increase by at least 50% to accommodate the expected growth in traffic between 2010 and 2050, whilst decongesting our roads and reducing fatalities and severe injuries towards zero in the long term.

In addition, the resulting road operations should comply with the policies set on noise, air quality and natural habitat. They should also seek decarbonisation by aiding the energy efficiency of the freight and passenger transport process, as well as reducing its net energy consumption during operation, construction and maintenance.

This vision is translated in ambitious guiding objectives concerning Decarbonisation, Reliability, Safety, Liveability and Cost. Taken together, the indicators and their respective guiding objectives represent a considerable improvement in the road operations services to the user of the road network, which may be reflected in an improved appreciation. User appreciation will be an important factor in the cost-benefit evaluations that lead to a decision on the actual implementation of the individual solutions in the toolbox.

## 2.2 Roadmaps towards implementation

Full-scale deployment of the FOR concept is envisaged for 2025, and a roadmap has been developed that will set out the tasks to make it reality. The required technology will be developed and demonstrated in three distinctive stages. The transition between the stages is marked by concrete milestones (Figure 1).

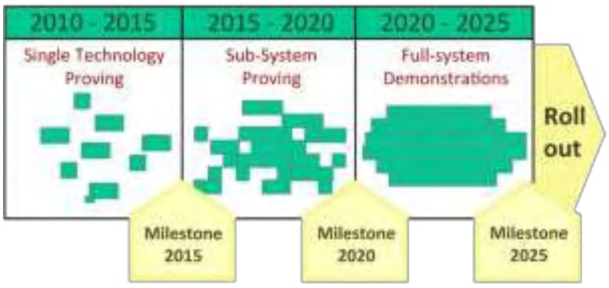


*Figure 1: Three stages of FOR development*

**The first stage** is concerned with proving single technologies, such as sensors or prefabricated components. In this stage, all preparations and preliminary actions will also take place that are essential for the successful field tests results in the two later stages. They are concerned with actions such as agreeing on key European routes/corridors to serve as test beds on which the field tests will be focused, aligning the current and future programmes from road authorities, industry and services as well as the collation and harmonisation of data, models, methods and regulation.

The guiding objectives are attributable to road operations only and are consistent with those recently stated by the ERTRAC platform [5] and the e-Safety forum [6]. Research and innovation on the other road transport system components (vehicles, services and energy and resources) may yield additional societal benefits.

Once a variety of single technologies is tested and all preliminary actions and preparations are done, **the second stage** is entered in which the successful single technologies are integrated to be proven in sub-systems (e.g. comprehensive and integrated traffic management systems and strategies to keep city rings in permanent flow).

This stage is planned to commence as from 2015. As a general principle, the scale of the respective field test will also increase to correspond with the increase in complexity the sub-system will have to address.

Following the proving of the viability of on a sub-system level, **the third stage** of field test is entered. In this stage, the technologies are integrated to the full-scale systems level. This stage is planned to commence as from 2020. Again as a general principle, the scale of the field test increases to involve the entire pilot corridors/routes.

By 2025, it is expected that the full-scale systems field tests will have yielded first conclusive results, yielding three corridors/routes that are significant to the European economy and that are effectively upgraded in line with the FOR concept.

By then, a fully-developed toolbox is available to road operators which holds many proven viable solutions on all integration levels, and which are provided with corresponding common standards, guidelines and specifications.
Table 2 on the last page presents the highlights in enabled technologies for the three consecutive milestones (2015, 2020 and 2025). In the description of the milestones, the scale of the field tests is distinguished between tests on local, sections, regional or national scale and tests on the scale of the entire corridor/route.

It should be noted that the milestone planning applies in general to the technology that is either already available but holds considerable untapped potential, or that is near market and can easily be adapted to road operations testing. In practice, it will be seen that for emerging technologies it will take longer to reach the stage of field test. Therefore, the milestone description shows that at the indicated dates (2015, 2020 and 2025) some of the technologies are still in the process of being field tested in a ‘single’ mode (i.e. not yet integrated). As a consequence, the toolbox is expected to evolve even after 2025 with new proven solutions added.

## 2.3 Delivery strategy

The vision behind the FOR programme is that many of the required solutions exist already from previous research, but are not (yet) implemented to their full potential. Investigation on the untapped potential and the eventual barriers to their implementation will undoubtedly offer quick wins to the road operators.

Therefore, the FOR delivery strategy is aimed at delivering early results from bringing together existing knowledge and best practices and systematically building that up in the following years with new research. Thus, the realisation of the FOR concept will be achieved through a combination of:

Current best practice in construction and technology,

Transfer of technologies and methods from other sectors,

Use of early stage and emerging techniques and products,

New (grassroots) research into technologies and products,

Demonstration of all the technologies and systems developed.

The programme actions are set out to provide the stakeholders, in 2013, with a significant portfolio of demonstrated, viable solutions. In the years following on from then, and following a consistent plan towards 2025, the results from current field test projects will be added to this solutions base making it more comprehensive and versatile.

At the same time, in preparing yearly work packages for funding, systematic analysis will focus on identifying the remaining technology gaps to be followed on by new research. The first focus for this new research will be to investigate if the adoption of near market technologies offers the opportunity to short track the development cycle by avoiding the basic research stage with the promise of early field tests. Only when this investigation offers no opportunities will new research be initiated from a grassroots level with the premise that field tests be scheduled on a significantly longer term basis. As these technologies require considerable research to bring them to results, the first scan will start in 2011.

During the entire term, all demonstrated results will be actively disseminated to the relevant stakeholders. This will be done through a variety of instruments ranging from the public availability of a well-managed database through to publications, interactive seminars and workshops.

The FOR toolbox will cover the entire range of proven solutions: from single technologies to integrated solutions on the full-system level. It will concern physical technology as well as data, models and methods.

## 2.4 The FOR Systems' Approach

To focus and prioritise its research and innovation efforts in line with the guiding objectives set out, the FOR programme follows a comprehensive and consistent systems approach (see Figure 2). This approach gives focus to the key elements of the FOR concept (Adaptable, Automated, Resilient).


*Figure 2: FOR key elements*

The enabling research and innovation is structured and categorized along the three key elements of FOR: the Adaptable Road, the Automated Road and the Climate Resilient Road.

***The adaptable road:*** The adaptable road focuses on ways to allow the road operator to respond in a flexible manner to changes in the road users' demands and constraints as they are defined in the service level agreements with the road owner. Within this element, innovation themes will address quick and cost-effective road design, construction and (year-round) maintenance as well as minimising the local and global environmental impact of road operations and management. As such, it holds innovation topics such as modular design and prefabrication concepts; self-repairing abilities; rejuvenation methods and robotized construction, inspection and maintenance. [7]

***The automated road:*** The automated road focuses on the full integration of roadside intelligence with information and Communication Technologies (ICT) applications on the user, and in the vehicle, the services and the road operations and management itself. Within this element, innovation themes will address intelligent systems and strategies to maximise the utilization of available infrastructure and monitor asset condition. As such, it holds innovation topics such as cooperative systems; in-built and wireless sensors; advanced utility, sensory and communication systems for intelligent traffic and incident management. [8]

***The climate change resilient road:*** The climate change resilient road focuses on ensuring adequate service levels of the road network under extreme weather conditions. Within this element, innovation themes will address adaptation of road operations and management to the effects of extreme weather (flooding, snow, ice, storm, drought, heat) to such extent that adequate service levels are ensured. As such, it holds innovation topics such as soil strengthening and rock stabilisation, early warning systems based on local weather forecasts, and dedicated weather proofing systems.[9]

## 2.5 Programme Deliverables

The FOR programme will deliver a toolbox of solutions for building, maintaining and operating roads that can be applied whether they are motorway, rural or urban, and regardless of region or country. This toolbox is built up from system trials and full-scale field tests that relate to the key challenges to road operations in view of the societal needs for efficiency improvement. These field tests will provide road, industry and services with the evidence that the FOR concept will produce the stream of benefits that it is seeking (see figure 3 below).

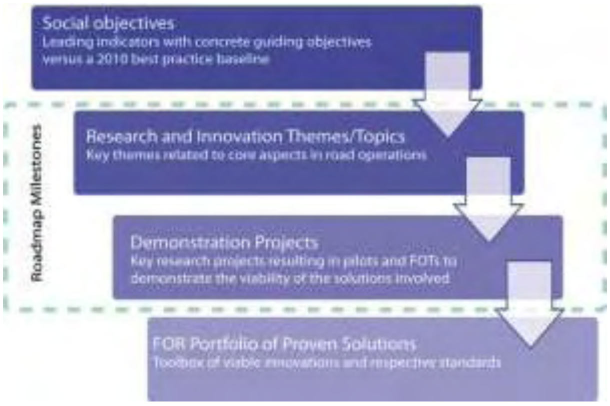


Figure 3: FOR roadmap

The solutions will contain future innovations as well as current best practices to allow the road operator to upgrade its operations to different degrees depending on local or regional requirements.

For example, in key economic centres and urban areas, a maximum upgrade could be in order, involving all applicable solutions, whereas in remote rural regions it could be the case that only a basic upgrade, involving a selection from the current best practices, is supported.

To enhance easy implementation of the toolbox, the solutions will be accompanied by (proposals for) common standards, specifications and guidelines that are easy to adapt to specific national conditions and requirements.

A sound knowledge transfer and dissemination process will be executed to ensure awareness in the sector and monitor implementation experiences. Throughout the deployment of the programme, stakeholders will be engaged to cooperate and collaborate in consolidating the roadmap milestones, executing the projects and disseminating the results.

## 3. BUILDING A NETWORK OF THE NATIONAL PROGRAMMES

In delivering the Forever Open Road Programme co-operation is linked to a number of ‘sister’ National Programmes with shared aims and goals. The sister programmes that are already under development and with which co-operation will be encouraged are

- Die Straße des 21. Jahrhunderts (Road of the 21st Century, R21C) – Germany
- Ferry Free E39 – Norway
- Corporate Innovation programme (CIP) - Netherlands
- Exploratory Advanced Research Programme (EAR) – USA
- Route 5ième Génération – R5G (The 5th Generation of Roads) - France

FOR will build on the existing knowledge and means of cooperation developed between these related programmes.

## 4. COOPERATION WITH THE USA AND THE BUILDING OF INFRAVATION

In October 2009, USA's Federal Highway Administration (FHWA) became an Associate of FEHRL. FHWA's objectives of becoming part of FEHRL were to formalise the cooperation and to enable the organisation to more actively engage in FEHRL's research prioritisation and project proposal activities. One of the objectives was for FEHRL members and FHWA to jointly define and conduct research projects including the co-funding of such work.

To further stimulate this development, in 2011, FHWA and FEHRL signed a Memorandum of Cooperation (MOC) in order to establish a common and transparent coordination and communication platform to leverage experience and expertise to address current and future highway transportation system research and technology needs. The MOC's purpose is to more formally establish cooperative partnerships to leverage resources between the EU and US research, development and technology programmes.

The scope of the cooperation includes research, development or deployment of technology and innovation activities in the following highway transport areas: planning, environment, right of way, asset management, materials, structures, hydraulics, traffic operations, traffic impacts, road user behaviour, economics, safety, and policy.

In 2012, a Cooperative Agreement provided a further stage for the partners by creating the mechanisms for joint funding decisions.

### 4.1 Infravation

Infravation - standing for Infrastructure Innovation - has been initiated in a way that will enable national and regional bodies to support research and demonstration of innovative products, systems and services collectively with the knowledge that the funded projects will be addressing pan-national requirements and the contractors involved will be encouraged by the opportunity to present their results with limited national boundaries. The partners of Infravation have determined that their support to the projects is not limited to financial compensation but includes partnering with the contractors in dissemination, knowledge transfer and ultimately to implementation.

The topic of the first call will be 'Advanced Systems, Materials and Techniques' for road infrastructure. The detailing of call topics has been based on the workshops between the funding agencies (Infravation partners) which were in turn based on their programmatic priorities. For design, inspection and monitoring, advanced systems could include breakthrough sensing and analysis technologies, including 'manu-services'. The call will include the development of advanced and novel materials based on nano-technology, bio-mimicry etc. In the case of techniques for construction and maintenance, aspects such as advanced robotics could be developed.

The Infravation consortium is composed of 12 participants being owners and managers of funding programmes related to road infrastructure, which got together to initiate a transnational call for R&D projects on infrastructure innovation.

They represent national programmes from 11 countries in Europe and the USA that can be distinguished in

- EU member states: Sweden, Denmark, The Netherlands, Germany, France, Italy and Spain
- FP7 associated countries: Norway and Iceland
- Third countries: USA

All in all, the consortium involves major participants from large EU member states complemented by highly committed participants from the Nordic countries and the Netherlands as well as the USA (represented by their programme manager FEHRL) that contribute financially to the trans-national call.

The FHWA Cooperative Agreement is used in Infravation to create a link to FHWA's Exploratory Advanced Research (EAR) Program which was established to address the need to conduct research on longer term and higher risk breakthrough research with the potential for transformational improvements to plan, build, renew, and operate safe, congestion free, and environmentally sound transportation systems. The Federal Highway Administration (FHWA) engages stakeholders in the EAR Program--from evaluating potential research topics through to communicating research results. FHWA identifies and scopes topics through extensive initial-stage investigation. The programme has provided 14 Million USD per year in support.

Infravation is a challenge driven programme. Whilst it aims to support the development of advanced systems, materials and techniques, it remains an applied research programme and all projects will be expected to deliver tangible, demonstrable, benefits. The focus is on R&D projects with the objective to develop and demonstrate advanced systems, materials and techniques for green, cost-effective, reliable next generation road infrastructure.

Projects will be expected to address one or more of the following challenges:

- Advanced predictive infrastructure performance processes
- Enhanced durability and life-time extension
- Rapid and non-destructive methods for routine quality and performance checks of materials and construction.
- Keeping freight routes open through zero-intrusive maintenance
- Ensuring infrastructure performance under all weather conditions
- Resource and energy efficiency in road construction and maintenance (Eco-design)
- Virgin material reduction by substitution or recycling.

Infravation will adopt a coordinated approach to knowledge transfer to ensure that all projects provide consistently high-quality information to relevant stakeholders. All projects will be required to contribute to a regular Infravation newsletter and provide material and participation for national workshops. These workshops will be arranged in several countries and will, where necessary, involve, translation for local practitioners. The workshops will focus on the proven benefits of the solutions developed.

## 5. CONCLUSIONS

For Europe's economy to stay competitive in a global arena, it needs support from road infrastructure that provides 24/7 high level 'Forever Open' service to industry and public. Hence a coherent innovation programme is required to achieve this next generation of roads.

In view of the urgency of this issue, the approach of choice is to start with bringing together the best of research that we have already and the best that is to come and bring it to deployment,.

The Forever Open Road programme provides the concept and the plan. It targets the key elements of road operations in the 21st century: adaptability, automation and climate resilience. For each of these elements it has drafted a consistent roadmap that holds detailed research milestones for 2015, 2020, and 2025.

The programme is now in the stage that research is programmed for funding from European research frameworks as well as from national and industrial programmes, such as Routes 5me Generation from France, Roads of the 21$^{st}$ century from Germany, Ferry Free E39 from Norway and The exploratory Advanced Research programme from the USA.

With the first projects of the FOR programme underway, FEHRL is developing the 'sister' programmes **Forever Open Railway, Forever Open River** and **Forever Open Runway** for railway, river and runway infrastructure, respectively.

Together these four concepts constitute the **FOR x 4 initiative** on transport infrastructure. Due to the strong technical overlaps in the infrastructure requirements, there will be many complementary research actions in these other modes - e.g. bridges, earthworks, materials.

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# BATTERY POWERED SHIPS - ECONOMIC AND GREENER

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## ABSTRACT

*Electrification with Li-ion batteries has been a global trend for years across different sectors, driven by decreasing battery prices and increasing energy density; now it is the maritime sector's turn.*

*Increasingly, fuel costs are a large part of ships' operating expenses and thereby competitiveness. Studies show that all-electric ships and hybrid ships with energy storage in large batteries and optimised power control can achieve significant reductions in fuel costs, maintenance, and environmental pollution.*

*Figure 1. Eidesvik's Viking Lady is ready to install its 0.5 MWH battery system in March 2013.*

## THE OPPORTUNITY

Several projects have investigated the electrification of ships showing that there is considerable potential to reduce both energy consumption and emissions of $CO_2$, NOx and particles. In addition to all-electric ferries for shorter straits and fjord crossings, ideal ship types for battery hybridisation typically have large variations in power demands and/or low utilisation of the engine for longer periods of time. (Diesel engines have low fuel efficiency in the low power area and in addition transients create engine inefficiency.) Combined with a battery package, the power system onboard may become more responsive and flexible with increased robustness towards peak loads and fast transient load changes. Furthermore, a battery hybrid solution of the auxiliary engines can provide fuel efficient and low emission energy in port and when sailing, and in particular to achieve environmentally friendly sailing in regulated and environmental sensitive areas. Use of LNG, renewables and waste heat recovery make batteries even more relevant.

DNV has performed hybridisation analyses of selected ship types showing interesting business cases. Some examples are shown below.

*Table 1. Cost comparison of hybridisation of selected ship t ypes*

| | Offshore Supply Vessel (DP) | Tug |
|---|---|---|
| Hybrid system cost | $ 2 000 000 | $ 300 000 |
| Annual fuel costs | $ 5 000 000 | $ 250 000 |
| Savings potential | 20% | 30% |
| Annual savings | $ 1 000 000 | $ 75 000 |
| ROI | < 2 years | < 4 years |

**SHIP TYPES**

Ship types of particular interest: Ferries, offshore vessels, wind farm vessels, “speedboats”, fishing boats, tugs and other workboats and special ships with large load variations of the machinery.

*Figure 2. Electric NORLED ferry concept*

NORLED will produce two hybrid and one 100 % electric ferry (120 cars/360 persons) the next two years

## THE CHALLENGE

If the battery-based solutions can be validated to be economic, safe and reliable, it could result in a significant market penetration and environmental savings.

However, the maritime battery might be 10 - 100 times or more, greater than a traditional electric vehicle battery. The large energy content, combined with "extreme" charging and operational patterns, present new challenges in relation to safety, reliability and service life.

To avoid accidents and unwanted incidents that may have significant safety and cost implications, and that can put development back a long time, it is important that the battery related systems are validated in an appropriate way.

DNV, jointly with Zero Emission Mobility (ZEM) and Grenland Energy, has initiated a project to perform a type-qualification of maritime battery systems in accordance with "best practice". In addition to addressing safety risks the project is addressing economic risks such as failure of the business case due to wrong dimensioning of the batteries, (resulting in too big and expensive batteries, or too small batteries with too short service life.).

## THE TYPE-QUALIFICATION PROJECT

*The main objective of the project is to improve the* ***systematics, tools and criteria*** *for safe and efficient introduction of Li-ion battery technology for use in hybrid offshore supply vessels and all-electric ferries and speedboats*.

DNV’s Technology Qualification process has proven to be a highly effective methodology to identify and address challenges and weaknesses at an early stage in the realisation of new technology. The methodology has a risk based approach, focusing on the vital few when uncertainty is removed in a systematic way.

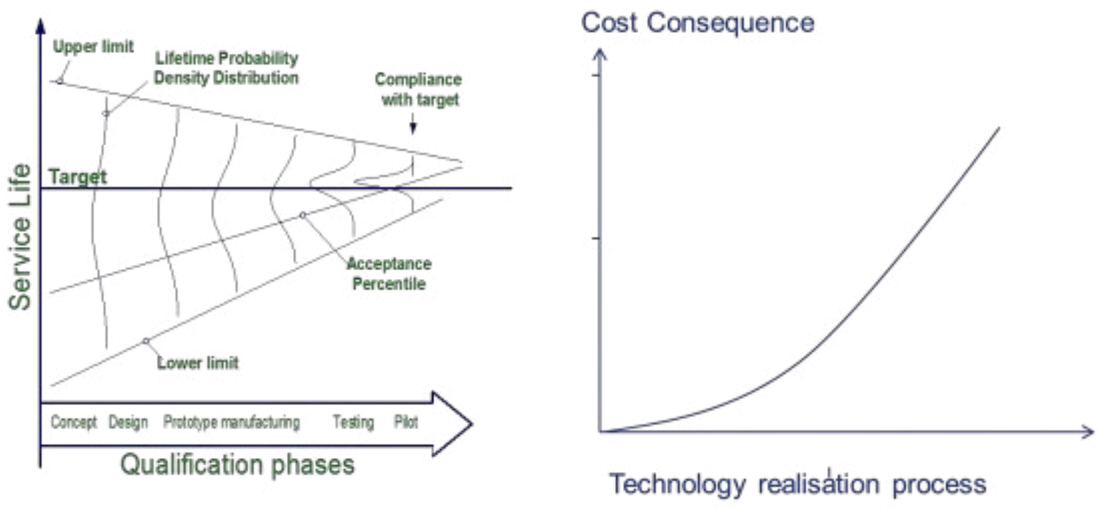


*Figure 3. Schematic illustration of DNV's technology Qualification Process*

The project will reduce barriers and contribute to faster electrification of the ferry sector. Deliverables are a public guideline, presentation of results and a public seminar. Furthermore, a platform is established for the project participants to perform Failure Mode Effect and Criticality Analyses (FMECA), which can be utilized as a starting point to perform FMECAs for specific designs of maritime battery based systems. This has already been done in a commercial job with the objective to describe and analyse different offshore supply vessel operations to assess the suitability of various battery based solutions.

## *PROJECT RESULTS*

- "Best practice" for the development and qualification of hybrid and all-electric ferry battery systems
  - Based on a generic design and a unique combination of risk management methodology, technical and operational expertise and independence
- A platform for quality assurance
  - A solid starting point for effective quality assurance of specific Li-ion battery system designs
  - Safety and economic risk model
  - Battery degradation model for battery size optimization
- Input for updating DNV's battery class rules
- Project Deliverables
  - Public Guideline
  - Presentation of preliminary and final results

## WHY WE SHOULD EXPLORE THE HYBRIDISATION OPPORTUNITY

- Good investment
- Improved ship responsiveness, functionality and reliability
- Improved environmental profile and reputation with customers, employees, investors and authorities
- Increased robustness
    - Increases in fuel prices
    - Changes to stricter environmental regulations

# CROSSING THE STRAIT OF GIBRALTAR

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Peter Jackson, COWI, Denmark

## ABSTRACT

*In 2009, AMBERG Engineering together with COWI and SYSTRA was charged by SNED/SECEG to perform a global project review of the preliminary technical studies (Avant Projet Primaire 2007, APP07) and of all the social, economic, exploitation and safety studies that had been prepared for the sub-sea tunnel solution of the Strait of Gibraltar Fixed Link (Projet de Liaison Fixe à travers le Detroit de Gibraltar). The aim was to give clear guidelines about the conditions and future actions in order to conclude about the feasibility of the project.*
*This paper, in addition to giving a short overview of the project, describes two of the main challenges encountered during the development of the tunnel project, which are the geological conditions and the challenges involved in the investigation of the geotechnical conditions. At the beginning of the planning process, little information was available regarding the geological conditions for the zone of the crossing. The paper describes the investigations and the evolution of the understanding of the geological conditions.*

## PROJECT DESCRIPTION

The project consists of a fixed link between Spain and Morocco across the Strait of Gibraltar. A joint Spanish Moroccan inter-governmental commission was constituted in 1980 by the two countries to promote the Fixed Link and to lead the technical studies for the project. The studies were led by two organisations, SNED in Morocco (Société Nationale d'Etudes du Détroit de Gibraltar) and SECEG in Spain (Sociedad de Estudios para la Comunicación Fija a través del Estrecho de Gibraltar, S.A.). During the early studies and colloquiums, a comparison between a bridge and a tunnel solution was made and decision reached to proceed with the tunnel solution after the 4th Colloquium on the Gibraltar Strait Fixed Link, in 1995 [1].

The last phase of the studies carried out in the period 2006-2009, took into account the most recent data from the investigations and surveys. The goal of this phase of study was to reach a better knowledge of the costs, risks, the environmental impact and socio-economic effects of the project. The studies were also to determine the legal steps and regulatory requirements for the construction and the operation of the fixed link and to propose an outline of the financial arrangement.

The global assessment carried out by COWI, SYSTRA and AMBERG was to independently review the 2006 to 2009 studies, including the preliminary technical studies (APP07 developed by Geodata , Lombardi, Typsa and Ingema), in view of preparing a Final Stage Report (rapport de Fin d'Etape) for submission to the Governments of the two countries and the authorities of the European Union. The scope of the mentioned report was to obtain the necessary institutional support to proceed with a next and justified phase of the studies to finally state on the project feasibility.

## History

In May 2007, the Gibraltar tunnel project as well as the following trans-Maghreb railway connections were included in the larger regional action plan for transportation from the EuroMed transport programme. The project has also almost from the beginning (1981) been followed by means of biennial reports by the Economic and Social Council (ECOSOC) of the UN. Obviously, the EU und the Union of the Arab Maghreb are also following the development of the project.

## Scope

As a strategic link between the continents, the Strait of Gibraltar presents a unique opportunity to link the European and the African transportation systems. The objective of the fix link between Spain and Morocco is to create and improve the economic exchange between not only both directly connected countries, but also both larger regions, in this case the western European Mediterranean zone and the Maghreb. The railway tunnel would link the railway systems of Spain and Morocco and connect to the rest of Europe and the Maghreb, see figure 19, allowing for both passengers and freight transport. The new tunnel will link Madrid to Casablanca in about 5 hours, independent of the weather situation in the strait, which is a large problem of the existing ferry link.

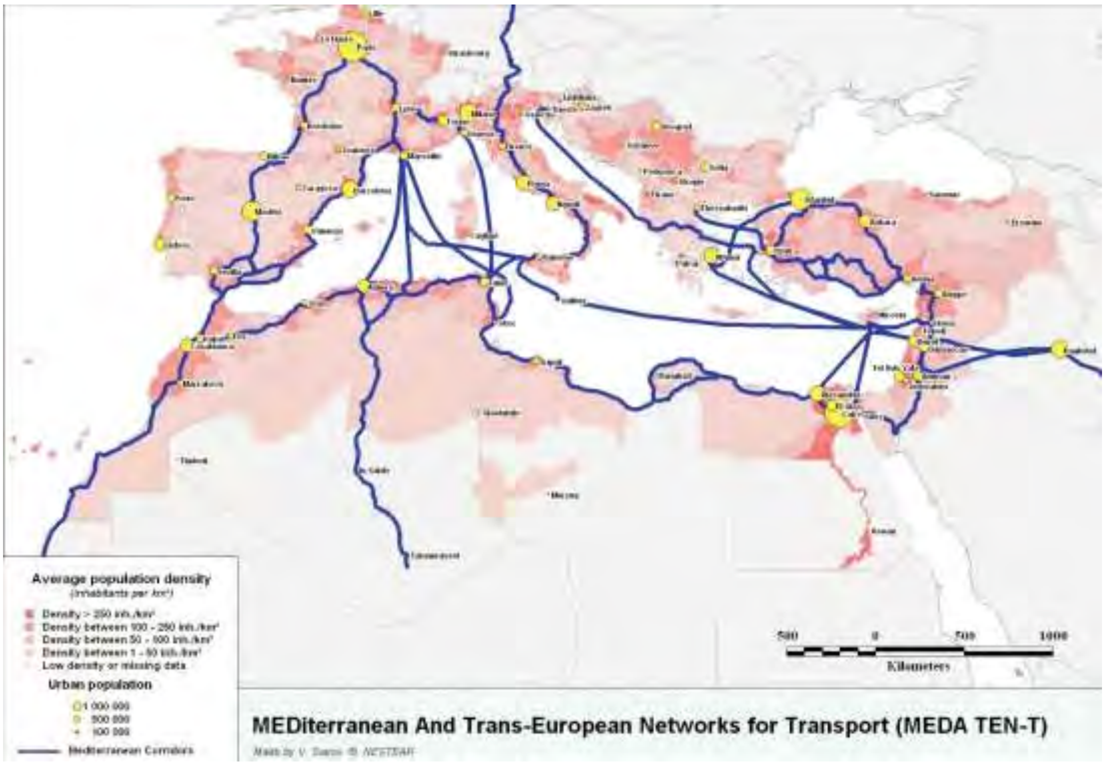



*Figure 19: The Mediterranean and Trans-European networks for transport*

**General Tunnel Layout**

The distance between the terminals on the two continents is 42.8 km. This stretch is divided into three sections. One section of about 4 km is above ground, a second section of about 11 km is located underground, however not below the sea, and 27.8 km of tunnel are below the sea. Figure 20 gives the plan view of the tunnel alignment.

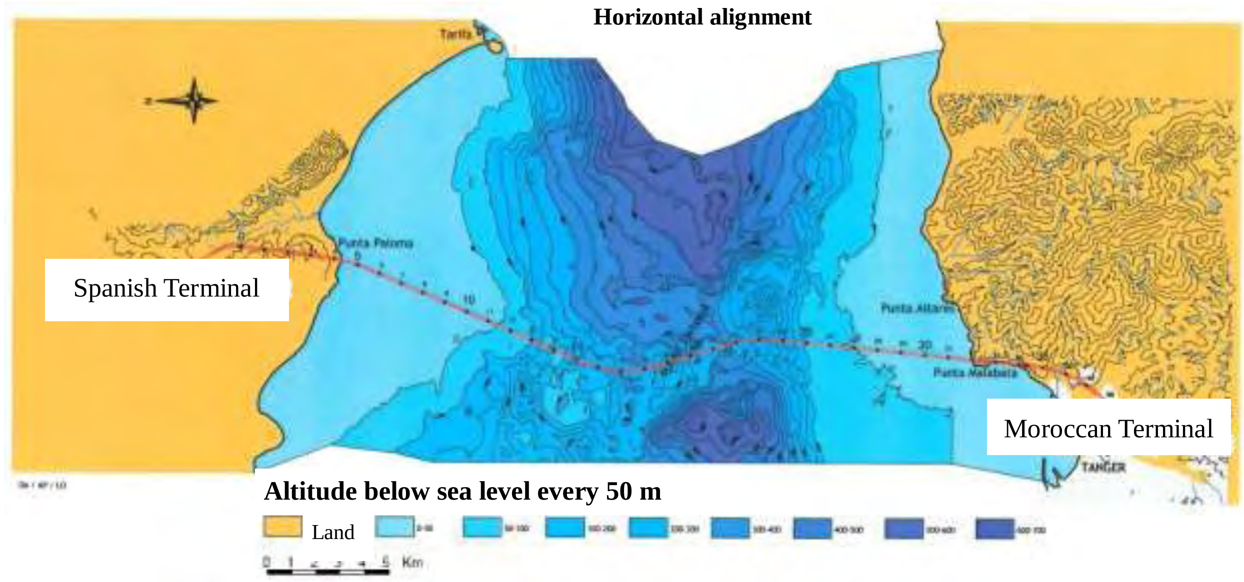



*Figure 20: Alignment of the tunnel from the Spanish to the Moroccan side through the Strait of Gibraltar*

As can be seen in figure 20, the alignment of the tunnel follows the sea bed trying to stay in the zones with the highest possible ground level. This is not only to avoid the higher water pressure in deeper zones, but also to permit a vertical alignment adapted to standard traction equipment for trains. As shown in figure 21, the tunnel has a constant slope of 3% running from both terminals to a low mark in the middle of the tunnel. This slope had been justified in the technical studies as being the maximum that could be run by standard railway traction equipment, without having to reduce the maximum charging of the train compositions or to operate with additional traction devices. It was also considered that a higher slope would have had a negative influence on safety issues (risk of fire due to overheating of the train brake system) and operating costs (electricity and maintenance of the water pumping system).

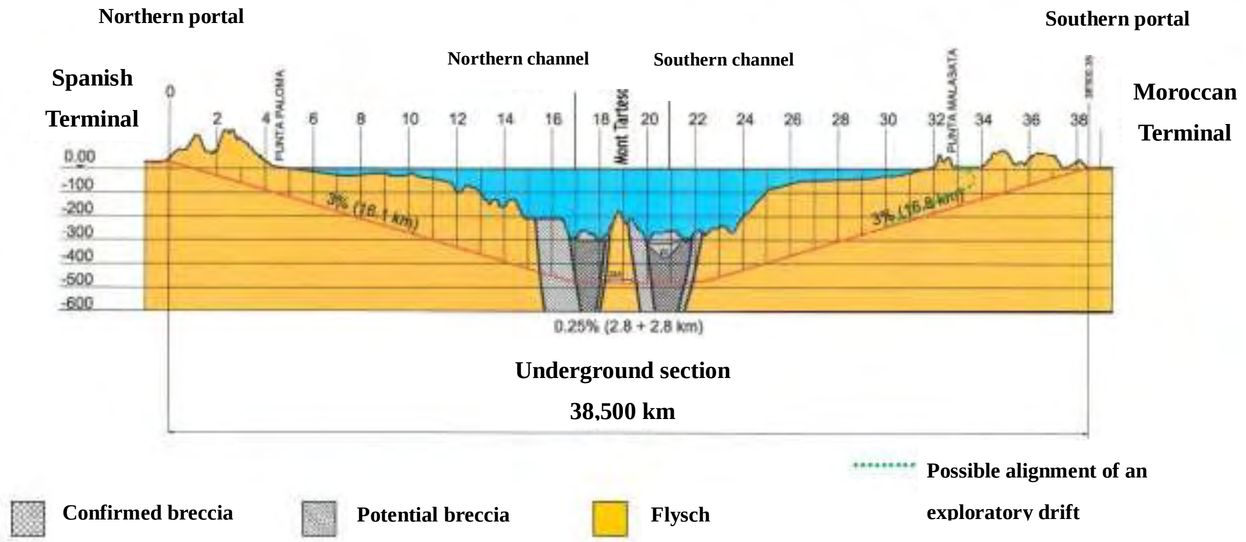



*Figure 21: Vertical alignment of the tunnel from the Spanish to the Moroccan side through the Strait of Gibraltar*

The tunnel link will consist of two single-track tunnels with a safety tunnel running between them, as can be seen in figure 22. The traffic tunnels run at a distance of about 70 m from each other and are connected by cross-passages every 340 m. A safety station for stopping the trains and evacuate passengers in case of emergency is foreseen in the central section of the alignment.

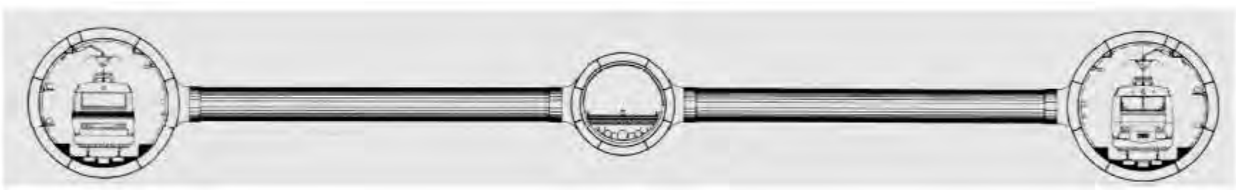


*Figure 22: Section of the tunnel layout*

## MAJOR PROJECT CHALLENGES

The global project review highlighted the major project challenges and uncertainties as well as the need for additional integrative investigations and studies, and gave recommendation on the type and sequence of additional future actions to advance with the project development.

The challenges and uncertainties related to the location and geological environment include:

- the depth conditions, with the tunnels being located at -475 m from the sea level, with a rock/soil overburden of about 200 m and a potential water pressure of 50 bars at the tunnel level;
- the depth of the seabed which offers limited alternatives to the location of the railway tunnel corridor;
- the complex geotechnical conditions that will be encountered along the railway tunnels, including flysch of variable quality and – according to the discovery made in 2005 by the

first deep borehole campaign – two breccia paleochannels which cannot be easily avoided;

- the slope of the alignment, which is based on a compromise between avoiding the most permeable geological formations and limiting the total tunnel length;
- the marine currents and the tide cycle which impose difficult conditions and limited time to deal with the off-shore geological and geotechnical investigations, and which demand for innovation in the field of investigation;
- a lack of real geotechnical investigations, since to date mainly the morphological, topographical and geological conditions have been addressed;
- doubts on the consolidation state of the breccia and technical studies based on hypotheses founded on limited and not confirmed geotechnical data;
- challenging tunnelling construction methods to be foreseen requiring relevant and non-negligible technological developments to cope with the geological, geotechnical and hydrogeological conditions.

In the following sections the geological and geotechnical challenges are addressed in particular.

## GEOLOGICAL ASPECTS

The geology of the Strait of Gibraltar in the corridor of the potential alignment shows two main characteristics.

The primordial characteristic is the convergence of the African, the European and the Alboran plate, which can be seen in figure 23. In the project area, the rock mass is formed by a thick layer of flysch. Flysch typically consists of a sequence of shales, interbedded with thin, hard, graywacke-like sandstones. Due to the tectonic movements in the project zone, the flysch is highly fractioned in several directions, which does not make it an easy material to excavate a tunnel. Experimental tunnels excavated in the 90s from both the Spanish and Moroccan sides in such a geological formation have shown, however, that tunnelling difficulties can be overcome in a reasonable way.

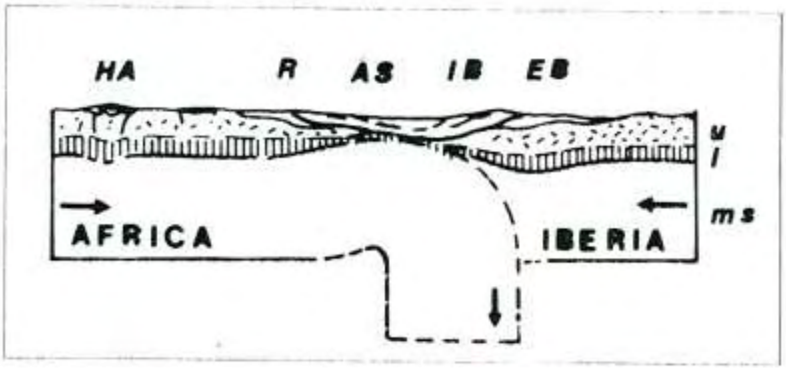



*Figure 23: Plate tectonics in the Strait of Gibraltar [2] – u: upper crust; l: lower crust; ms: upper mantle; HA: High Atlas; R: Rift; AS: Alboran Sea; IB: Internal Betics; EB External Betics.*

The second characteristic is the formation process of the strait itself. The strait is, in geological terms, young and the investigations performed have helped in deepening the understanding of its formation mechanism.

Approximately 7 million years ago, the convergence of the three plates completely blocked the connection between the Mediterranean Sea and the Atlantic, so that the Mediterranean Sea remained totally isolated from other seas or oceans. As evaporation was higher than the water inflow from rivers and rain, the level of the Mediterranean Sea decreased. This period culminated in the Messinian Salinity Crisis.

At the beginning of the Pliocene, approximately 5 million years ago, small quantities of water entered from the Atlantic to the Mediterranean Sea due to an eustatic sea level variation or from a tectonic abatement of the plates in the zone of the strait, or both together. Over the following ten thousand years, the out flowing water eroded the nowadays 300 m deep Strait of Gibraltar. In the middle of the strait, the water flow eroded the flysch bedrock already weakened by the action of the tectonic plates, creating two deep channels which can be seen in figure 24 and which are located now more than 600 m below sea level. Subsequently, the two channels were in-filled, and, due to tectonic movement, the filling became a clayish breccia enclosing fragments of flysch.

The exact position and extension of the two former channels – called paleochannels – was initially unknown; it was revealed by the investigations carried out for the project, once the technique to drive deep investigation boreholes in a difficult marine environment was fully developed and implemented. The paleochannels can be seen in figure 24, where the extension of the channels filled with breccia is shown in green.

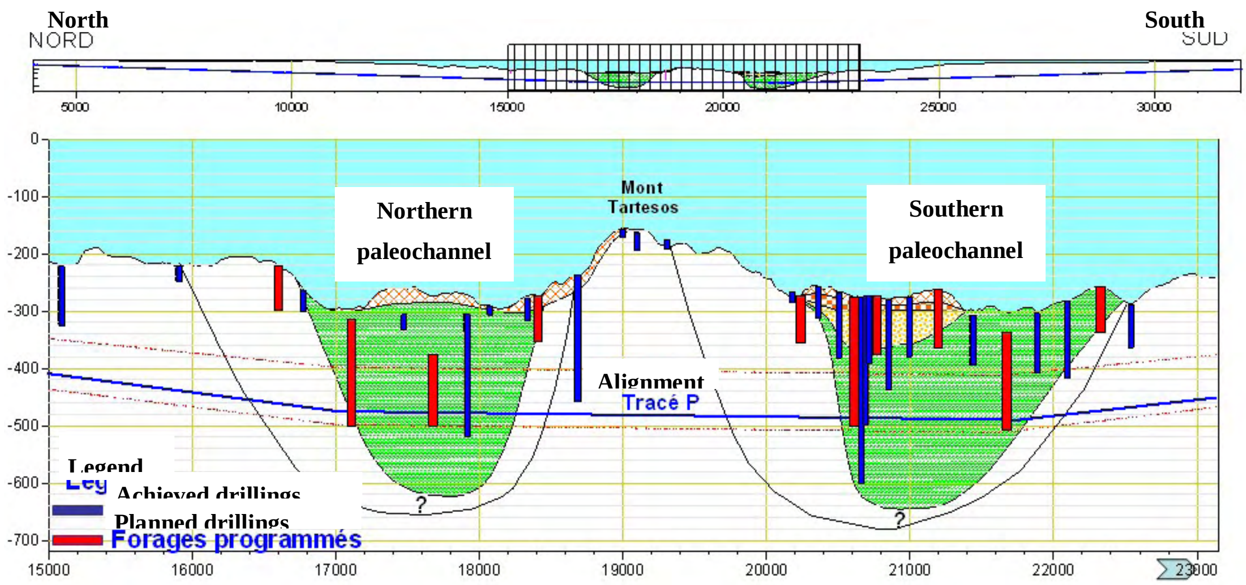



*Figure 24: Localisation of the paleochannels defined by the exploration campaign of Kingfisher in 2005. In blue the executed drillings, in red the planned drillings, "Tracé P" indicates the tunnel*

## INVESTIGATION CHALLENGES

### Preliminary investigations

To gain an overview over the complex morphologic and geological situation in the project zone, several exploration campaigns were conducted from the beginning of the project development. The first investigation phase included cartography/bathometry, geophysics, meteorology, oceanographic, and the geological analysis of the seabed at shallow depth, which were all extended at a large scale. The second phase was more focused on the area of the potential alignment corridor identified based on the first investigation phase. It included two experimental tunnels, Malabata and Tarifa, to study the behaviour of flysch, which were considered the most representative geological/geotechnical conditions for the long tunnels to be excavated under the Strait, and deep off-shore boreholes.

As mentioned, the first explorations aimed at defining the undersea topography of the Strait of Gibraltar to allow identifying potential locations for the underground fixed link. From 1980 to 1984, a bathymetric sounding of the sea bed in the project zone was carried out to define the topography. In 2001, another bathometric sounding was conducted to improve the resulting maps.

From the beginning (from 1980 to 2004), also several seismic investigations with different energy sources and systems were conducted. The seismic investigation only defined the upper levels of the sands overlying the bedrock. None of the seismic soundings showed the breccias-filled paleochannels in the middle of the strait that are known today.

Together with the seismic investigations, also sonar investigations were carried out. The results of the sonar investigations permitted the definition of the continental plates and the saliency of the flysch bedrock from the sand covering. Two gravity investigations were also conducted.
The result of all these non-invasive investigations was the definition of the undersea topography, and no evidence of the rock type and quality below the sea bed was obtained at this stage.
Between 1981 and 2004, several campaigns obtained samples from the surface of the sea bed by gravity coring (see figure 25). The results of these samples helped to define the surface geology of the sea bed, but still did not give insight into what lies below.

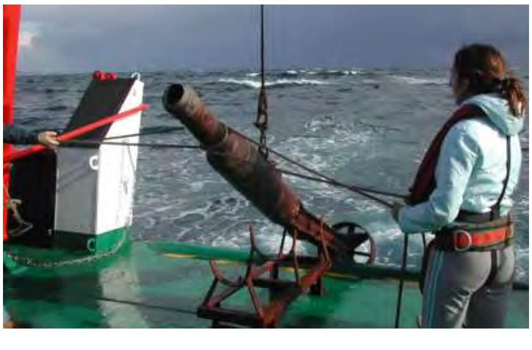



*Figure 25: Gravity coring applied in the Strait of Gibraltar*

The sea bed was also investigated by the Russian submarine Argus, which provided nearly 40 hours of films of the sea bed and some more samples collected from the bottom of the strait. Neither the gravity core sampling, nor the submarine sampling could furnish useful data for the elaboration of the geological and geotechnical model to be used for the tunnelling project.

**Deep boreholes**

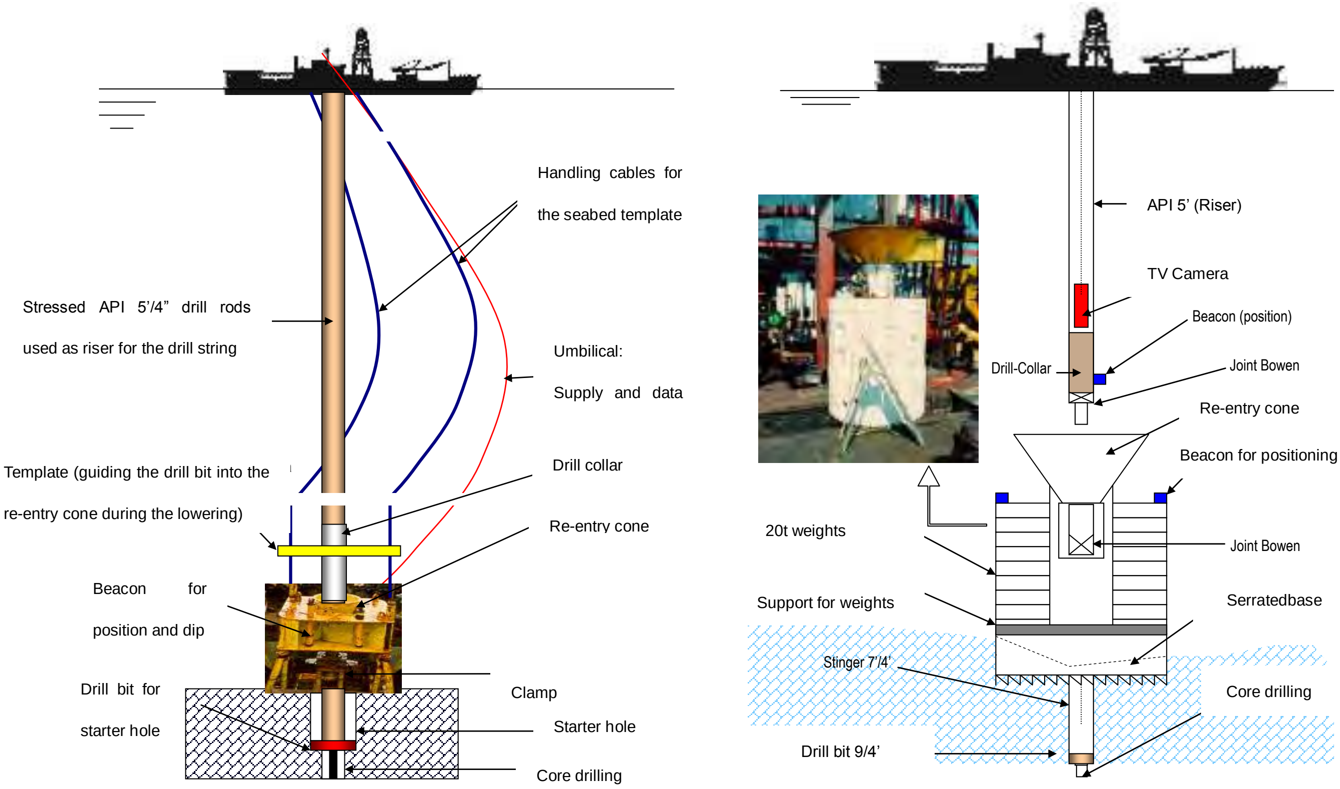
To gain the data needed for the tunnel planning, core drills had to be carried out in the Strait of Gibraltar in the area defined the most probable for the future tunnel. However the first drilling campaigns from 1990 to 1995 could not provide satisfying results, because the drilling techniques were not sufficiently effective to deal with the strong currents in the strait and the relative short time slots available to perform the investigations due to tides.

The first technique used was the autonomic "Rockdrill" sonde from the British Geological Survey. The sonde, permitting an extraction of 5 m-long cores, was guided by a remote control from a vessel with a dynamic positioning system. The system brought up several cores, but of a maximum bore depth of 5 m.

In 1992 the first experience entailing a vessel with a dynamic positioning system and a string of drill rods connecting the vessel to a heavy template on the seabed ended in a loss of equipment due to the strong currents, causing a large load on the string of rods and resulting in insupportable vibrations in the string of rods. After a second test with a similar system, studies of the marine currents were conducted in order to develop an adapted rod system, and find a way to reduce the influence of the currents on the rod system as well as to predict the marine currents and their forces.

The result of the studies on marine currents showed that the currents move at up to 6 knots (1 knot = 0.5 m/s) at the surface and up to 4 knots at the sea bed of the Strait of Gibraltar. The drilling equipment for further exploration campaigns was specially developed and dimensioned for such a marine current regime. The system designed is of the piggy-back type with a re-enterable sea bed template and a hollow riser with special aerofoil type fairings (see figure 26). The fairings freely rotate around the riser in the direction of the prevailing current, eliminating vortex induced vibration and minimising drag, which caused problems and/or the loss of equipment during the first campaigns.
The system had evolved over several investigation campaigns from a single-use template at the top of the borehole to a multiuse template at the top of the borehole. The positioning of the template on the sea bed evolved, as well. At the last drilling campaign, the positioning was not only done based on the bathymetric information and camera information from within the rod, but with a small ROV (Remote Operated Vehicle) with a camera on board guided by remote control was used to choose a suitable spot on the sea bed for placing the template and drilling afterwards. The development of the GPS (Global Positioning System) and the resulting dynamic positioning systems for vessels also had an important influence on the result of the last campaigns.

With the specially developed system, more than 100 m deep drillings in the sea bed of the strait could finally be carried out, which can be seen in figure 24, and led to discovery the breccia paleochannels.




(a) (b)

*Figure 26: Drilling equipment for marine drilling explorations as used during the different exploration campaigns; (a) single-use marine ground frame type Fugro used in 1998; (b) multiuse marine ground frame type Seacore (Dart) used in 2005 during Kingfisher campaign*

## TUNNELLING THROUGH BRECCIA PALEOCHANNELS

As shown in figure 24, the tunnel alignment indicated in blue and named “Tracé P” passes through both of the paleochannels. Due to the estimated depth of the channels, it is impossible to lead the tunnel below them through the flysch bedrock without heavy impact on alignment, safety and cost. The nature and characteristics of the breccia of the paleochannel needs to be properly understood in order to design the tunnel and ensure that it can be constructed safely.
Some disturbed breccia samples collected during the geological investigations were used to perform some preliminary geotechnical analyses and testing to obtain preliminary indications to be further confirmed, the quality of the cores not being fully adequate for the scope.

With drainage ahead of the tunnel face, the increased strength of the breccia results in less convergence during tunnel construction. As can also be observed, the values for the convergence, even with drainage, are still very high and could cause problems for tunnelling, such as the risk of clogging of a TBM during the excavation and the pressure created by the convergence process on the lining of the tunnel.

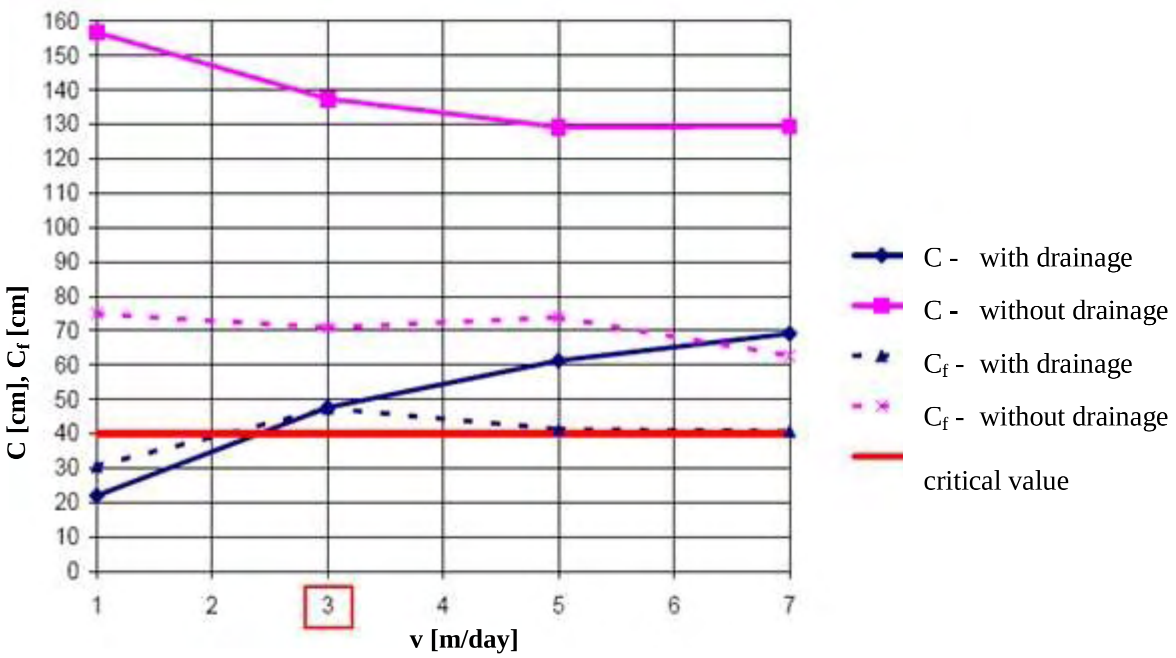



*Figure 27: Convergence behaviour in the breccia as a function of the tunnel advance speed, as calculated in geotechnical simulations with and without a drainage system ahead of the tunnel face.*

The overall conclusion of all exploration campaigns and tests is that further exploration and testing are required, both to broaden the knowledge about the geology to be encountered along the future tunnel and to obtain reliable data concerning the geotechnical characteristics and behaviour of the materials in the paleochannels which are filled with weak and disturbed breccia.

## FUTURE GEOTECHNICAL CHALLENGES

As already stated, the challenge arising from tunnelling the Strait of Gibraltar is how to obtain reliable geotechnical data and further confirm the geological conditions. According to the result of the project review, the future investigation campaigns should include about ten boreholes and be based on the following:

- use both destructive and core recovering borehole techniques ;
- sampling, using a combination of rotary coring possibly in combination with a rigid core liner (reduction of the samples' disturbance) in the zones rich in clasts and using pressure coring in the more plastic zones of the breccia;
- carry out in-situ tests: CPT tests in breccia; seismic tests with shear wave velocity measurement in the breccia in order to determine the deformation properties; measurements of the pore water pressure, by the use of CPT tests or the installation of piezometers in boreholes; permeability tests by a combination of dissipation tests during

CPTs and packer tests in boreholes; geophysical surveys by including calliper, gamma-gamma and porosity/density.
- use techniques to improve coring and sample quality and perform proper geotechnical in situ and laboratory test;
- perform identification tests on samples directly after extraction, on the vessel which should have an adequate laboratory equipment;

Primarily, two approaches are possible for these investigations:
- to continue the use of the 5" API riser pipe (casing) used at the time of the Kingfisher campaign in 2005,
- to develop a system based on a larger riser pipe system (API 8").

## CONCLUSIONS

Gaining geological data for designing a tunnel in complex geological areas under a relatively deep sea subject to strong marine currents is extremely difficult and requires the development of specialised equipment and exploration systems. The first core drilling campaigns in the strait demonstrated that the drilling equipment, generally coming from offshore gas and petrol explorations state-of-the-art at the moment of the prospection, was not adequate to the conditions encountered in the Strait of Gibraltar. The development of specific drilling equipment brought up valuable information.

For a further exploration of the project area, the drilling equipment will have to be further improved to permit the extraction of undisturbed cores, which are very important for a better understanding of the behaviour of the rock. Moreover, the manner in which the extracted cores are tested and handled to gain the most possible valuable results has to be improved for the next investigation campaign in the Strait of Gibraltar.

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# NEW TEST METHODS FOR CABLE SYSTEMS

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## ABSTRACT

*In cable bridges the cables are subjected to different kind of loading scenarios during their lifetime. Before the installation of stay cables the systems have to proof their sustainability in tests. The different types of tests are mentioned in several national and international accepted recommendations [1 – 5].*

*Up to now the most common loading tests for stay cable systems are the axial fatigue loading test and subsequently a static loading test to determine the ultimate breaking load (UBL).*

*Existing experience showed that bending effects due to installation tolerances of the anchorages and other sources of bending like sag, wind-rain induced vibration or flexural deformation of the bridge deck are fatigue relevant in practice too. For the investigation of these effects in tests - predominantly in fatigue tests – the anchorages are usually installed with a specific angle in the axial test set-up. Recent research in bending fatigue tests underlined the sensitivity of the bending effects on the fatigue resistance of stay cable systems. In this context also modified test set-ups for bending fatigue tests were presented.*

*The mentioned bending fatigue effects can also occur in vertical hangers of suspension bridges and arch bridges due to wind and rain effects or flexural deformation of the bridge deck. This is also focused in research at the moment.*

*The paper will present the different test set-up for bending fatigue tests with constant axial force and with synchronized axial dynamic force.*

## INTRODUCTION

### Stay cable systems

For stay cable bridges different kind of stay cable systems are used. On one hand there are the rope systems e.g. locked coil cable with z-wire in the outer layers and helical or spiral wire inside. They are fabricated from drawn wires. Each wire is hot galvanized with a zinc layer for corrosion protection. On the free length the wires are close spiral coiled; at their ends they are fanned out for anchoring. Anchoring of the wires is made by casting the conical hole in the anchor socket with a zinc-aluminum alloy. These cables are pre-fabricated in a plant without any influences from out-site fabrication. This also means that the length of the cable is fixed. Adjustment in length on site is only possible with adjusting plates or screws anymore. Due to that the installation and the structural requirements have to be planned carefully and in advance. The longitudinal stiffness of the spiral strand and locked coiled cable usually varies between 155,000 and 165,000 MPa due to the spiral effect. To eliminate the structural stretch of the cable they are usually pre-stretched before installation.

On the other hand stay cable systems with tensile elements consisting of prestressing steel are used. The prestressing steel can be wire, strand or threadbar. These systems are mainly covered in [2]. Stay cable systems with prestressing steel can be compared to the corresponding post-tension systems. But due to the different service stress and fatigue performance in stay cable bridges and in post-tensioned bridges the systems are not equal in all details.

*Table 1: Comparison of service stress and stress range in dynamic fatigue tests for stay cable systems and post-tension systems*

| | Stay cable systems, cp. [2] | Post-Tension system, cp. [7] |
|---|---|---|
| Service stress | 0.45 GUTS | 0.65 GUTS |
| Stress range in fatigue test $\Delta\sigma$ | Wire 200 MPa<br>Strand 200 MPa<br>bar 110 MPa | 80 (up to 100) MPa |

GUTS: Guaranteed Ulitmate Tensile Strength of steel

For stay cable systems with prestressing steel the components (prestressing steel, anchorages, deviator devices etc.) are usually delivered separately and finally assembled on-site. The installation on site is combined with adversities like weather etc. The advantage of these kind of systems is that the length can be adjusted to the actual situation. The longitudinal stiffness of these types of stay cable is with app. 195,000 MPa usually higher than that of spiral cable systems.

For long span stay cable bridges nowadays very often parallel strand cable systems are used. The strands are zinc galvanized and coated with a thin wax layer and a HDPE sheath. The strands are arranged parallel between both anchorages at the ends and fixed with wedges. In the area of the anchorages the sheath of the strands is demounted for fixing the strand with the wedge in the anchorage. This area is finally filled with a corrosion protecting material like wax, grease soft resins or cement grout. In a fixed distance to the anchorages the strands are usually compressed by a deviation device. For more details regarding these stay cable systems see [2].

## Requirements

The requirements for stay cable systems are close combined with the different kind of loading scenarios during the life time of stay cable bridges. Here only the effects due to static and dynamic performance of the cable will be focused on. Prior to the installation the cable systems have to proof their sustainability in tests.

The different types of tests are mentioned in several national and international accepted recommendations, see [1, 2, 3, 4, 5]. In most of these recommendations for stay cable systems an axial fatigue loading test and subsequently a static loading test to determine the ultimate breaking load (UBL) is mentioned. Testing parameter, e.g. according [2]:

| | |
|---|---|
| Upper stress: | 0.45 GUTS |
| Fatigue stress range: | Strands, wires: 200 MPa, bars: 110 MPA |
| Acceptance criteria | 0.92 AUTS or 0.95 GUTS |
| | AUTS (Actual Ultimate Tensile Strength of steel) |
| | GUTS (Guaranteed Ultimate Tensile Strength of steel) |

Under service conditions stay cables are firstly subjected to an axial force due to the load bearing (dead weight, traffic etc.) of the construction. Secondly also flexural effects due to installation tolerances, bridge deck deflection, cable sag and vibration effects like rain-wind induced vibration or periodic displacement caused by wind or traffic actions can occur.

In the past these bending effect had been covered in tests usually by placing wedge-shaped shim plates beneath the anchorages which impose a fixed rotation, $\alpha = 10$ mrad ($\approx 0.6°$) are recommended in [2]. This test set-up allows testing in most of the existing laboratories without any major modification of the testing equipment.

In [3] for stay cable systems it is mentioned that the axial dynamic fatigue test shall be combined with simultaneous dynamic transverse deviation of the cable. The angle between the anchorage centerline shall be between $\alpha_{min} = 0$ and $\alpha_{max} = 10$ mrad with amplitude $\Delta\alpha/2$. The maximum axial stresses should be synchronized with the maximum angle deviation. Alternatively the frequency of the angular frequency should be a few percent higher than the frequency of the axial stress which yields to random combinations of axial and angular loading.

In a recent research work [6] fatigue bending tests were performed with single strand systems. In the bending fatigue tests the single strand was stressed with a constant axial stress. Then bending fatigue tests with different deviation angle were performed. It was found that the deviation angle has a significant effect on the fatigue resistance of the strands. Maybe the breaking of the strands was also caused by friction effects of the wires of the strands. As the tests were performed without the usual deviation device in stay cable systems which are installed in a fixed distance to the anchorages the deviation angle occurred directly at the wedges. This fact can have had a negative influence on the bending fatigue resistance of the specimen. It seems necessary to perform the bending fatigue tests with stay cable systems according to practice.

## MODIFIED TEST-SETUP FOR BENDING FATIGUE TEST ON CABLE

In the Civil Engineering Materials Testing Institute, Braunschweig, Germany, the existing testing machines for testing post-tensioning systems, stay cable and ropes were modified recently with the aim to perform bending fatigue test with constant axial force and in combination with the axial fatigue test, Figure 1 - 4. The two test facilities have a maximum load of 10,000 kN and 30,000 kN respectively which covers tests on systems up to e.g. 109 strands. The test machines allow dynamic fatigue tests up to a frequency of 4 Hz and a specimen length up to 10 m. The frequency of 4 Hz makes fatigue tests possible to perform efficiently in a short time, e.g. 2.0 Mio cycles in approximately one week time.

Testing facilities:

| | |
|---|---|
| a) 10 MN testing machine: | constant axial force, bending fatigue, 4.4 Hz, see Figure 1. |
| b) 30 MN testing machine: | axial fatigue force, synchronized bending fatigue, up to 4 Hz, specimen length max. 10 m, see Figure 2. |

## TEST RESULTS

In the two test machines bending fatigue tests were performed on different stay cable systems e.g. systems with parallel strands and also fully locked cable. The systems with parallel strands were tested with the common sockets and deviation devices which are connected with anchorage in the transition to the free length of the cable. Bending fatigue tests were performed with deviation angle of 10 up to 20 mrad on the free length for 2.0 Mio. cycles with a constant axial load and with synchronized dynamic axial load as well. Subsequent to the bending fatigue tests axial static load tests were executed to determine the ultimate breaking load.

As all tests were commissioned tests for companies and no research tests detailed results cannot be presented here. Only one result of a bending fatigue test on a cable with 12 strands can be presented here in parts. Details of the axial load and the bending deviation amplitude in the test are mentioned in Table 2. The result of the axial tensile test subsequent to the bending fatigue test with 2.0 Mio. bending cycles is shown in Figure 6. The cable fulfilled the requirements of $F_u > 0.92$ AUTS or 0.95 GUTS and a deformation of $\varepsilon > 2.0$ %.

From recent test results it can be concluded that bending fatigue and the deviation angle seems to have an influence on the fatigue resistance of cables comparable to the maximum axial load and the load amplitude in axial dynamic fatigue tests. Furthermore it seems that the negative influence of dynamic bending fatigue can be reduced by a common and partly stiff socket and the deviation kit connected with the anchorage. These deviation kit (or deviation element) is usually a part of the cable system. It seems that it has a damping effect and prevents the full dynamic angle deviation at the anchorage and the outlet of the prestressing steel at the anchorage and the wedges.

*Table 2: Parameter of a bending fatigue test on a cable system with 12 strands.*

| Initial axial load | Deviation angle | Jack opening | Frequency | Load cycles | Wire breaks |
|---|---|---|---|---|---|
| [kN] | [mrad] | [mm] | [Hz] | [Mio.] | [piece] |
| 1507 | -10 / +10 | -27 / +27 | 4.4 | 1.5 | |
| 1507 | -15 / +15 | -40 / +40 | 4.4 | 0.2 | none |
| 1507 | -10 / +10 | -27 / +27 | 4.4 | 0.3 | |
| | | | Total | 2.0 | none |

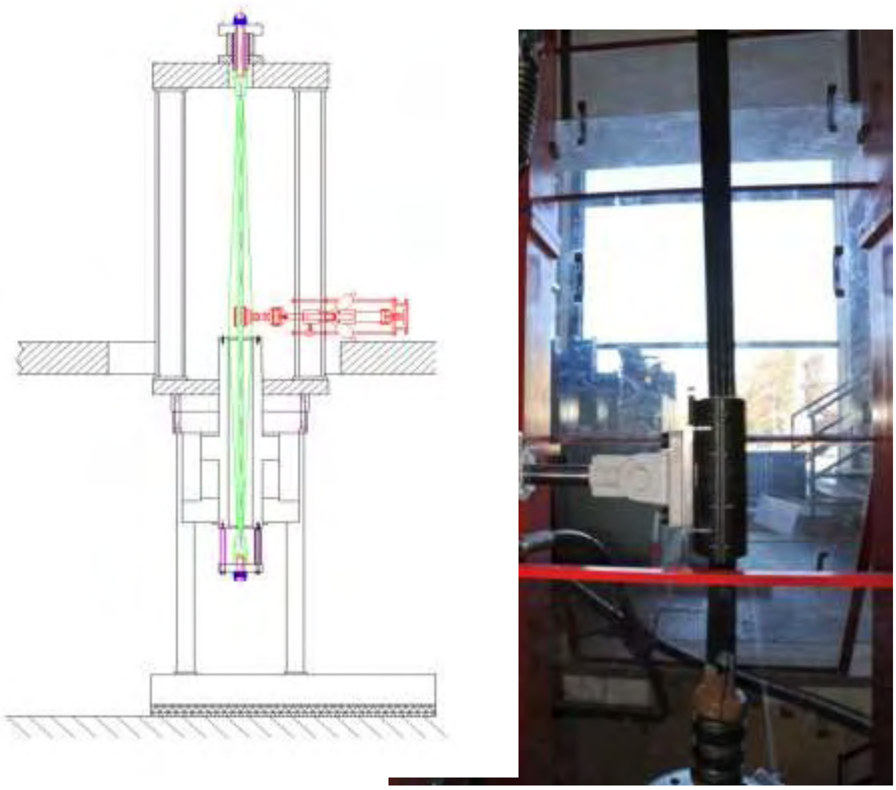

*Figure 1: Left: 10,000 kN testing machine with device to set up deviation angle in the bending fatigue test, axial prestressing force as constant static load; right: bending deviation of the cable in a dynamic bending fatigue test.*

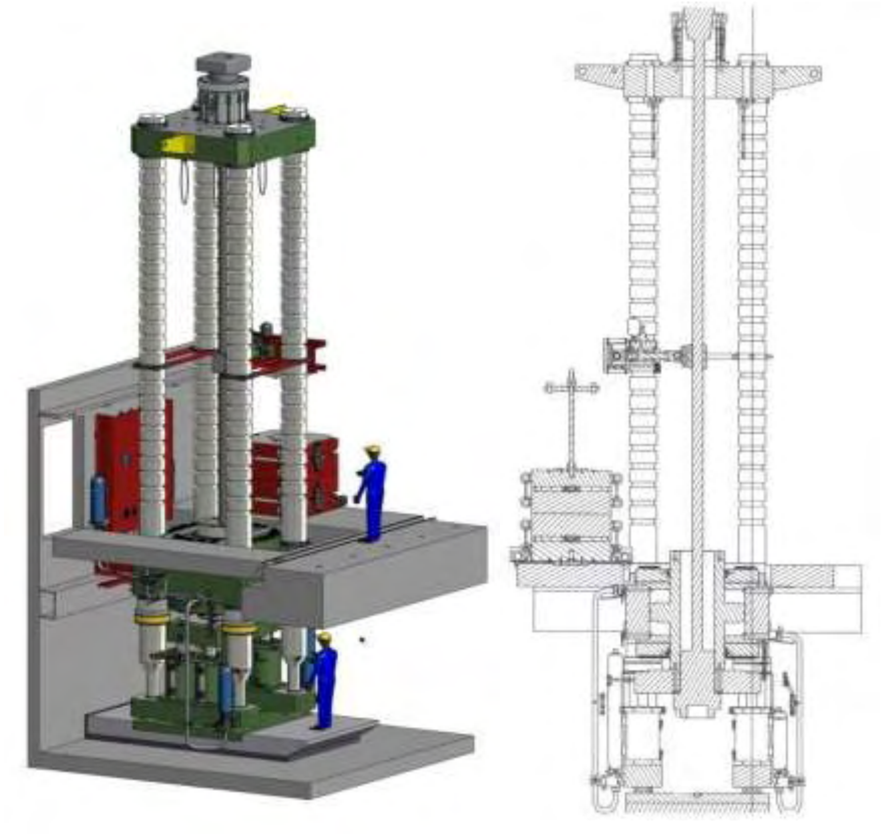

*Figure 2: 30,000 kN testing machine with device to set up deviation angle in the bending fatigue test synchronized to the axial fatigue force*

*Figure 3: Left side: 30,000 kN testing machine; right side: 10,000 kN testing machine*

*Figure 4: 30,000 kN testing machine with deviation device for dynamic bending deviation*

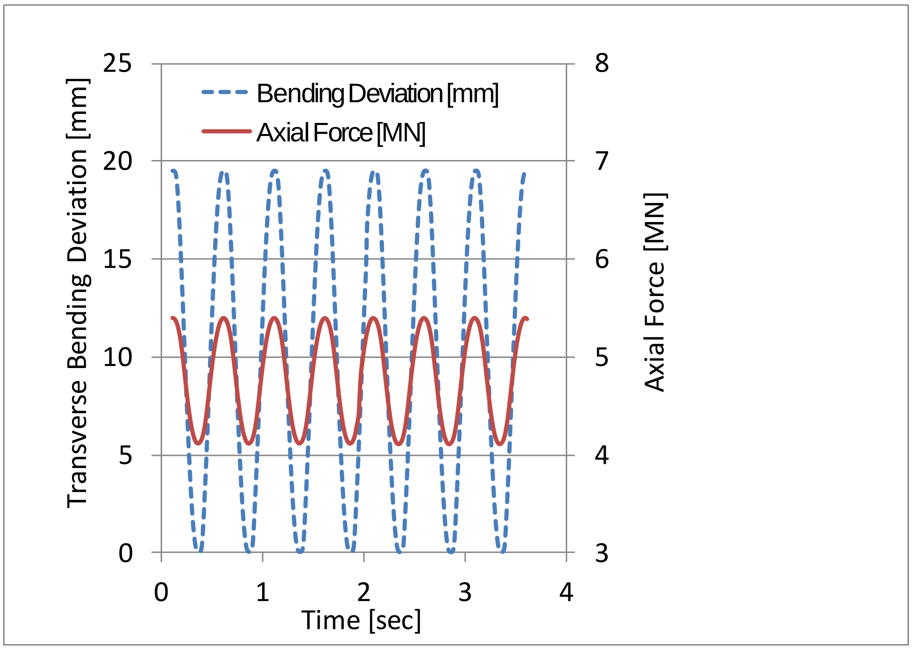


*Figure 5: Synchronization of transverse deviation in bending fatigue test and axial force due to bending effect.*

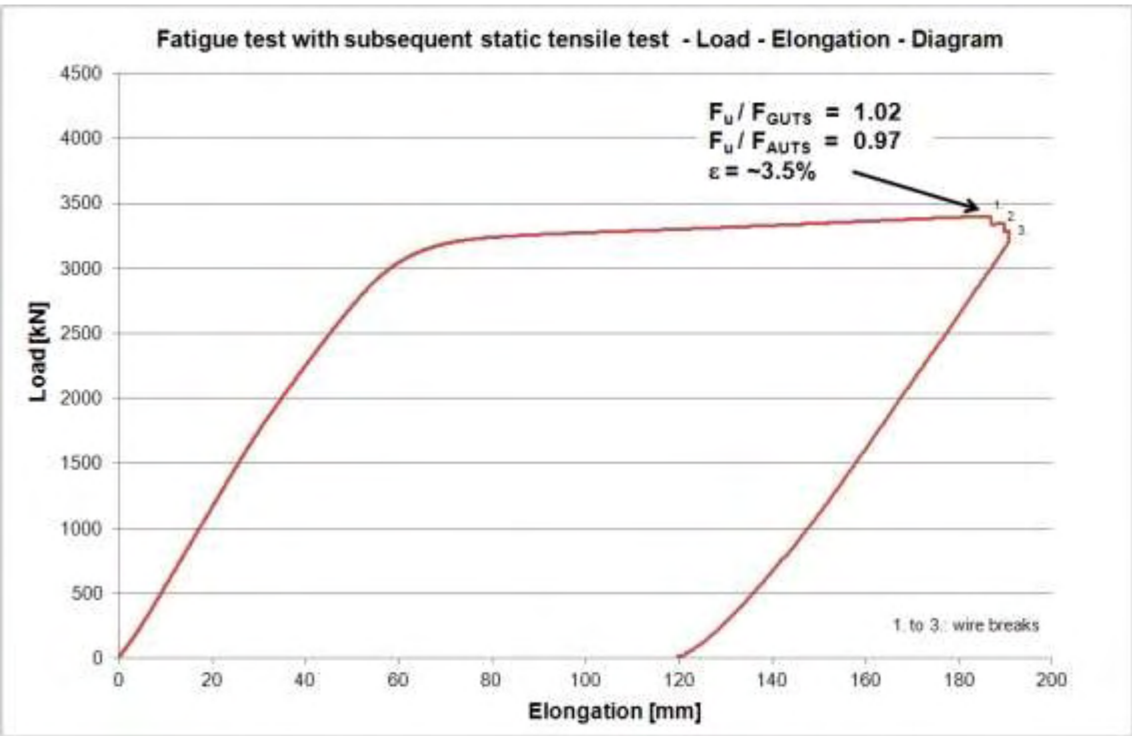


*Figure 6:* *Result of the axial tensile test subsequent to the bending fatigue test on a cable system with 12 strands; axial force vs. total deformation.*

## CONCLUSIONS

Existing testing machines with maximum load of 10,000 and 30,000°kN for testing post-tension-systems and stay cable systems were modified for performing bending fatigue test. In the 10,000 kN-machine the bending fatigue test is performed with a transverse deviation on the free length of the specimen and with a constant axial force, e.g. usual prestressing force of 0.45 GUTS. The 30,000 kN testing machine enables axial fatigue tests synchronized with the transverse deviation in the bending fatigue test. Both machines run with up to 4 Hz what make fatigue tests economical.

Recent bending fatigue tests on cable systems showed that dynamic bending has an influence on the fatigue resistance of the cable. With common sockets and deviation elements connected with the anchorages in the transition to the free length of the cable the bending effects on the anchorage and the prestressing steel can be reduced due to their damping effect.

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# CONSISTENT STRUCTURAL ANALYSIS AND DESIGN OF LONG-SPAN BRIDGES

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## ABSTRACT

*As the bridge engineering community sets sails to using longer and longer spans, more and more sophisticated analysis models have to be used in the design process. For large pre-stressed concrete and composite bridges built using the incremental launching or free cantilevering methods, this applies mainly to accurately modeling the erection process in time, with considering the different construction stages, the time dependent behavior (creep and shrinkage), and the required pre-cambering to achieve the design shape after the construction process has been finished.*

*The erection geometry can only be seriously controlled using a precise structural analysis that considers all the structural stages occurring during the construction of the bridge as well as the effects from the pre-fabricated shapes (where applicable) of the elements.*
*The basic principle of classic structural analyses is that elements are connected to the common nodes, representing the degrees of freedoms in terms of displacements and rotations. Difficulties arise if the "face to face" connection is somehow modified in the construction process, or if the segment geometry is deviating from the design state.*

*However, the challenges for ultra long-span bridges such as stay cable or suspended bridges with high pylons and slender steel or concrete decks are mainly related to optimizing the stressing sequence of the cables, to the geometrically non-linear behavior of the structure, and to dynamic problems such as wind-induced vibrations. Some projects examples are highlighted in the paper, where importance of above mentioned topics have been considered within professional bridge analysis and design software solution RM Bridge.*

**Key words:** creep/shrinkage, time effects, pre-camber, erection control, CFD analysis, wind buffeting, aero elastic damping

## INTRODUCTION

Bridge design and analysis is an iterative process. During this process the engineer is looking for the best solution for given criteria by changing specific system parameters. Engineering experience helps to reduce the time required, but there will still be a need for many iteration steps until the design criteria are met. Computer programs nowadays should provide the best possible support for this design process. The construction sequence combined with long-term effects has an influence on the target engineering design. Within structural analyses it is necessary to account for long-term effects in the calculation and to minimize undesired influences [4].

Continuous change of structural systems is a major reason for getting non-linearity in structural analysis. For cable stayed bridge design process special optimization procedures are necessary.
Special attention must in this context be drawn to pre-cast segmental erection techniques, where it is essential that the individual segments have an appropriate fabrication shape for getting the intended design shape of the bridge after assembly. This requires a very accurate deformation analysis early in the design phase with accurately taking into account further governing parameters in addition to the general structural stiffness. The detailed construction sequence should be fully included in the analysis to obtain insight into the real segment positions at each stage of construction.

For long cable stayed and suspension bridges, bridge designers must consider also dynamic wind effect. The extraordinary, ultrathin design of these structures yields significant susceptibility for wind-induced vibrations.

Various bridges have already been designed and analyzed considering all aspects above and using the described solution below within commercial computer solution. The bridges presented in this paper are provided to give insight into some specific problems that were encountered and solved in the design process.

## TIME EFFECTS

The occurrence of time dependent plastic strain is a material property of concrete. Total plastic strain consists of creep plastic strain and shrinkage plastic strain.

$$\varepsilon_p = \varepsilon_c + \varepsilon_s \qquad 1)$$

The effect of the loading history on the strains in concrete is much more pronounced compared to the other materials. Fortunately the laws governing the behaviors of such creep are taken as linear in stress so that quasi superposition principles are valid. National design codes for reinforced concrete, pre-stressed concrete or concrete in composite with steel take effects of the past loading history into account. Concrete creep models are generally defined by separated creep factors for each stress increment in the loading history. These creep factors depend of the loading time and of other factors like concrete quality, environment (humidity, temperature, …), section properties etc.

Rather complicated calculation of creep factors for each stress increment is impossible for hand calculation but presents no difficulty for computer implementation. The storage of each stress increments creates a huge amount of data which must be handled properly.
The numerical solution is accomplished step by stepping in the time domain. It can be assumed that complete load/response history is known (including all stress increments) from time equal to

zero up to the start (time $t_1$) of the time step. Basic unknowns are stresses and strains (or corresponding integral forces and displacement) at the end of time step (time $t_2$) – equation 3.

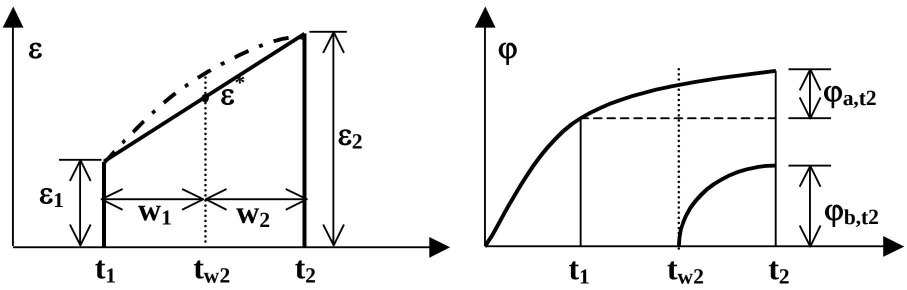


*Figure 1: Elastic strain increment and creep coefficient over time*

Instead of determining the continuous change between time $t_1$ and time $t_2$ it is advantageous to look only for the final solution at the end of the time step (at time $t_2$). Linear change of elastic strain within time step according to the "finite differences" theory will be assumed.

$$\varepsilon = w_1\varepsilon_e^1 + w_2\varepsilon_e^2 \,, \quad w_1 + w_2 = 1 \tag{2}$$

$$\varepsilon^2 = \varepsilon^1 + \Delta\varepsilon \,, \quad \Delta\varepsilon = \int_{t_1}^{t_2} \frac{\partial \varepsilon}{\partial t} \cdot dt \tag{3}$$

The total strain increment Δε can be divided into elastic and plastic part (equation 3). It can be assumed that elastic strain depends linearly on the stress state.

$$\Delta\varepsilon = \Delta\varepsilon_e + \Delta\varepsilon_p \tag{4}$$

The total plastic strain increment consists of three parts:

$$\Delta\varepsilon = \sum_{i=1}^{n} \int_{t1}^{t2} \frac{\sigma_i}{E} \cdot \frac{\partial \varphi_i(t)}{\partial t} \cdot dt + \int_{t_1}^{t_2} \frac{\partial \varepsilon_s}{\partial t} \cdot dt + \int_{t1}^{t2} \frac{1}{E} \frac{\partial \sigma}{\partial \tau} \cdot \left(1 + \varphi^*(\tau, t)\right) \cdot d\tau \tag{5}$$

The first part is a plastic strain increment due to the creep of all stress increments occurring before time t1.

The second part is a plastic strain increment due to shrinkage of concrete. Those two terms are basically known and can be evaluated in closed form at any time t.

The third part of the plastic strain is produced by creep of additional elastic strain increments which continuously occur within the time step (between time t1 and time t2). This part together with corresponding elastic strain increment is unknown and needs to be determined. It is obvious that the third integral term produces major difficulties.

The final expression is then given by:

$$\Delta\varepsilon = \sum_{i=1}^{n} \frac{\sigma_i}{E} \cdot \left(\varphi_i(t_2) - \varphi_i(t_1)\right) + \left(\varepsilon_s(t_2) - \varepsilon_s(t_1)\right) + \left(1 + \varphi_{w2}\right) \cdot \frac{\Delta\sigma}{E} \tag{6}$$

The solution is straightforward. There are two basic possibilities. The first option is to establish the new stress/strain relation using an equivalent E-modulus and solving by stress increment. This solution is very elegant but has the disadvantage that it cannot be combined with further non-linear effects like p-delta effects, large displacements etc. A more appropriate solution is to iterate the corresponding plastic strain due to the additional elastic strain increment within a general Newton/Raphson procedure. This solution is implemented in bridge analysis and design software RM Bridge in a closed and consistent algorithm.

The use of equal time steps is inappropriate because creep functions have an exponential characteristic. Therefore it is useful to make equal time steps in the logarithmic time domain. Experience has shown that the calculation of end creep (up to time infinity) can be done using 3-4 time steps in the logarithmic time domain. Smaller creep intervals within the construction schedule only need (on average) one time step. Bending moment due to the final creep can have differences of about 12% to 18% between solution with 1 time step and 3 logarithmic sub steps (Figure 2: Rach-Mieu bridge, first CSB in Vietnam with main span 270 m). [5]

The solver within RM Bridge considers geometrically non-linear, time-dependent structural analyses. Newton-Raphson algorithm is used for the geometric non-linearities and extended Newmark time integration for the time dependent portion of the analysis. If less sophisticated types of analyses are sufficient for a specific problem, then this can be achieved simply by deactivating the appropriate portions of the analysis process.

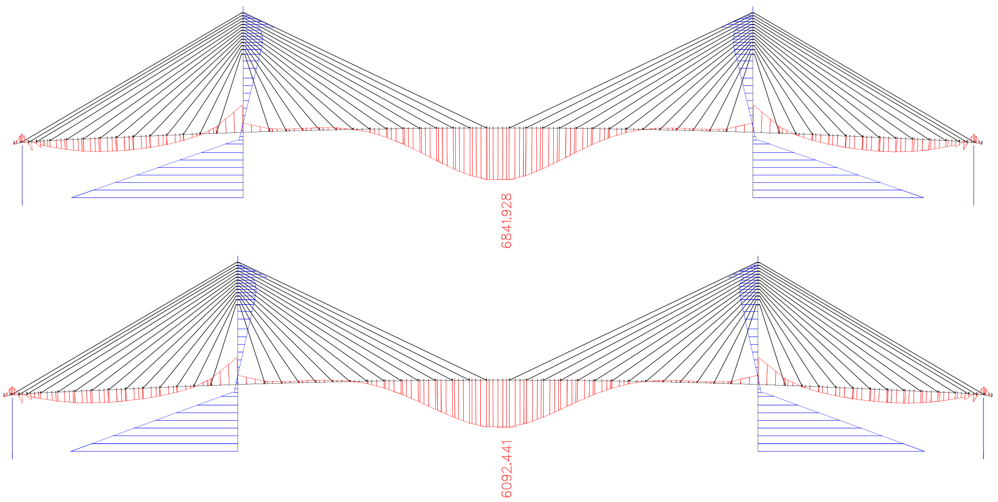


*Figure 2: Rach Mieu Bridge (project engineers TEDI, Vietnam) – bending moments for final creep*

Using a finite difference in the time domain the presented algorithm includes coupled effects of the creep, shrinkage and steel relaxation within overall structural non-linear analysis. Cable sagging, p-delta effects, large displacements or even contact problems can be combined with long term effects within consistent analysis. The proposed method for the numerical analysis can satisfactorily predict long term effects up to the time infinity; it is generally suitable for the investigation of all kinds of bridges with reinforced concrete, pre-stressed concrete or composite sections.

Figure *3* shows the target final state for the Stonecutter's Bridge (SCB), Hong Kong (Project engineers: Ove Arup, Hong Kong). The main span of 1018m is a steel girder while the back span is done in concrete and the towers which are almost 300m high are composite. In Figure 4 and Figure 5 the construction stage analysis with and without long-term effects are compared.

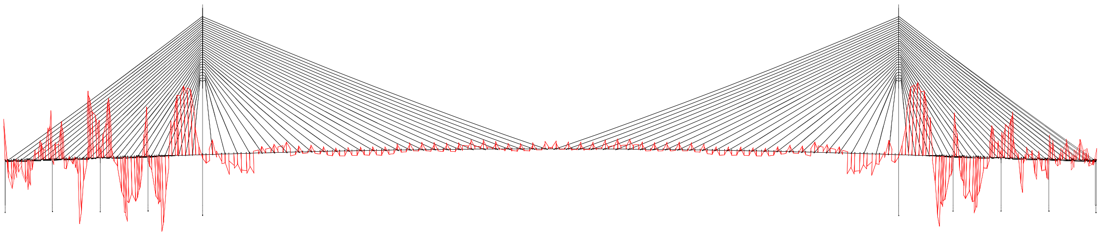

*Figure 3: SCB - Final state analysis.*

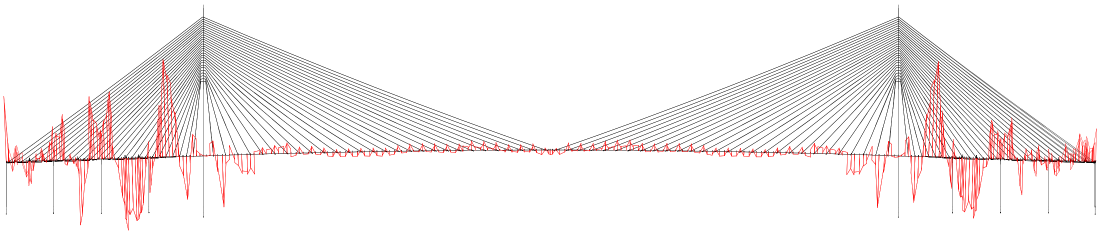

*Figure 4: SCB - Construction stages analysis (without creep and shrinkage).*

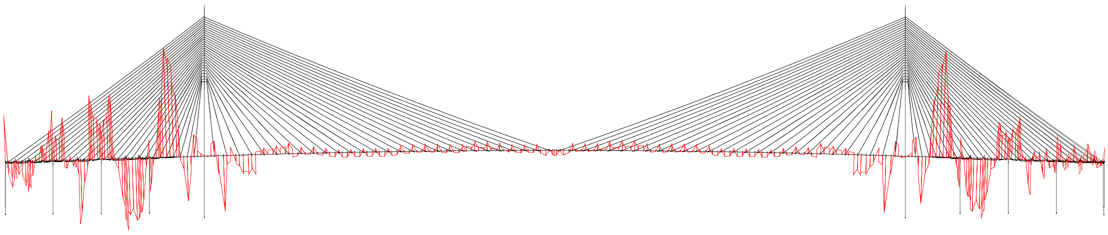

*Figure 5: SCB - Construction stages analysis (including creep and shrinkage).*

The bending moment redistribution as well its intensity is significant changed.

**PRE-CAMBER, ERECTION CONTROL**

Depending on the erection sequence significant displacements occur during the construction of bridges. These displacements are compensated with pre-camber and specific fabrication shapes of girder components. The deformed structure is the start position for the calculation; therefore, full account of the current location in space is taken for the large deflection analysis. A precise structural analysis in construction stages is required to seriously control the erection geometry. The analysis is performed with a given starting geometry - usually the structure in Pre-camber is the geometry of the bridge required for assembly to reach the final geometry.

The basic principle of classic structural analyses is that elements are connected to the common nodes, representing the degrees of freedoms in terms of displacements and rotations. In reality, the structural segments are connected "face to face" and common nodes are only used to discretise the structural system. This concept works excellent if no changes occur in the "face to face" connection between the segments (Figure 6).

However, difficulties arise if the "face to face" connection is somehow modified in the construction process, or if the segment geometry is deviating from the design state. On the one hand, the unassembled segments possibly do not fit into the displaced geometry of the structure and on the other hand, the engineer has to find and optimize necessary future correction steps in the erection procedure in order to come close to the designed set of forces and displacements.[2]

$$\left\{\delta^{I}_{Elem}\right\} = \left\{\delta^{I-1}_{Elem}\right\} + \left\{\delta^{I-1}_{Node} - \delta^{I-1}_{Elem}\right\} + \left\{\delta^{Kink\_Correction}_{Elem}\right\} \quad 7)$$

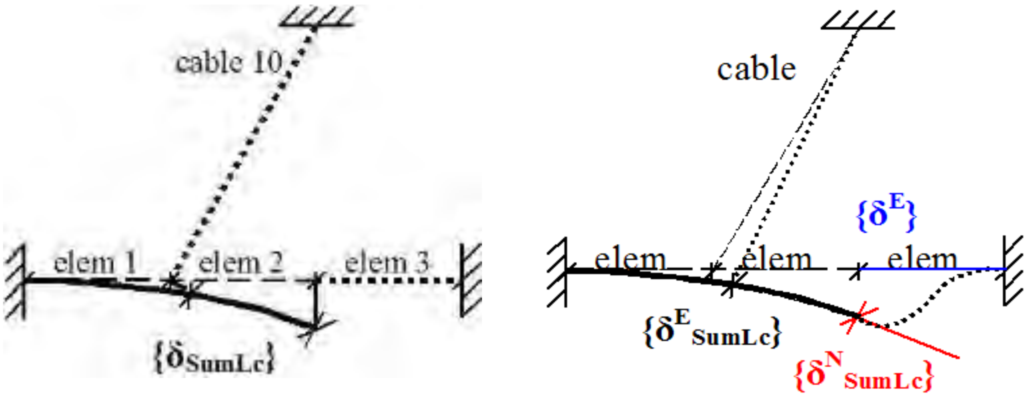


*Figure 6: Geometrical information while assembling new element 3 and cable 10*

Standard structural analysis codes can obviously not be used anymore after inserting displacement constraints at element faces. Element stiffness properties, including both linear and non-linear geometric terms, have to be transformed accordingly, and the local coordinate system of structural elements (defining local forces) is changed.

Structural assembly in RM Bridge erection control mode can be used to fully simulate certain construction conditions. With an option to automatically correct the kink at the segment face each newly-active element is fully constrained with a face-to-face connection to the currently active structure in its displaced position. As additional result, Erection control gives information if any force action is necessary to assemble the new segments. This allows determining any necessary equipment and possible construction problems already in the early design stage. It is important to note that both, linear and non-linear analyses are performed on the displaced structure in erection control mode, taking into account the exact geometrical lengths and rotations during the construction stages. [1]

Main application field of the erection control functionality is in *construction engineering*. When different parties using different analysis programs perform design and construction engineering as it is mostly always the case, required pre-camber values are given to the construction engineer in form of Excel tables or construction drawings. The construction engineer has to make sure, that those specifications are meaningful and fit into the intended design shape.

The proposed procedure is to establish an appropriate mathematical model and to enter the prescribed camber values as actions on this structure. This is performed with using the same load types as automatically created with the certain schedule action.

*Application example – Stonecutters Bridge*

The Stonecutters Bridge is a high level cable-stayed bridge which spans the Rambler Channel in Hong Kong, connecting Nam Wan Kok, Tsing Yi Island and Stonecutters Island. With a main span of 1018 m it has the third longest cable-stayed span in the world (Figure 7).

*Figure 7 Artists view of the Stonecutters Bridge in Hong Kong*

Consultant Ove Arup was responsible for detailed design and construction supervision over the whole Route 8 project, working with Cowi on the detailed design of the Stonecutters Bridge. Arup used program RM Bridge including the wind dynamics module for the design analysis of the structure.

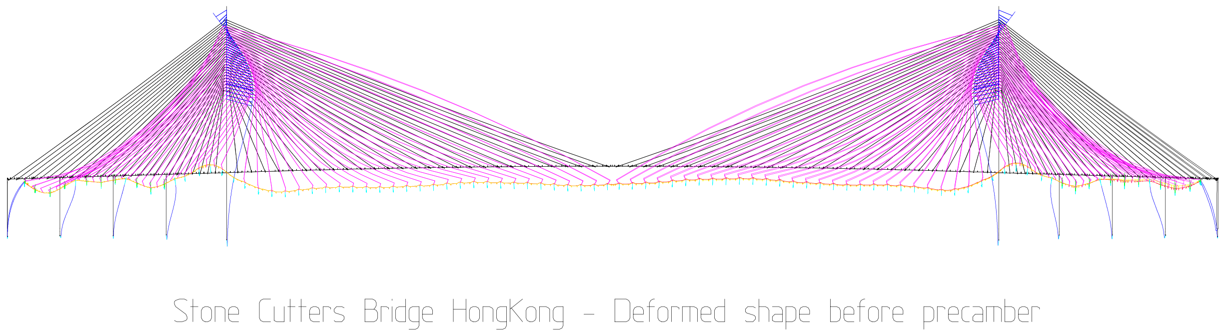


*Figure 8 Deflection shape of the Stonecutters Bridge to be compensated by pre-cambering*

Once the mathematical model had been established, the program has also been used for detailed pre-camber analyses for defining the exact shape of the individual segments (Figure 8). The erection monitoring module is now regularly used in the construction phase for checking any irregularities occurring during construction. This direct link between design engineering and construction engineering is applied with great success, allowing for designing effective compensation measures as soon as deviations from the nominal condition are detected.

**WIND-INDUCED VIBRATION OF THE WHOLE BRIDGE**

It is natural that wind load effects are getting bigger on the design of a long span cable supported bridges with its longer span lengths. The wind vibrations are conventionally classified into buffeting (gust response), vortex excitations, galloping and torsional flutter of the whole bridge.

Wind induced vibrations are to be verified also for various stages during the erection as well as, for such structural elements as pylon and stay cables, after completion.
The buffeting of a structure always takes place in the direct effects of turbulent natural wind. In the case of a long span bridge exposed to wind, there will be random vibrations of lateral bending by drag, vertical bending by lift and torsion around the bridge axis by pitching moment.

Referring to wind impact, long-span bridges require sophisticated wind buffeting analyses with considering both, the aero-elastic behavior of the structure and the wind loading correlation.

The solution for structural wind buffeting calculation within RM Bridge is performed in the modal space and in the frequency domain. It includes aerodynamic damping and stiffness effects due to structural movement caused by the wind flow. All computations are based on the tangential stiffness of the structure at a given point in time – the structure under permanent loading and mean wind – allowing to include all prior non-linear effects. [3]

This calculation is commonly performed in the modal space, presuming linear behaviour based on the tangential stiffness matrix attained in a previous fully non-linear analysis for the dead loads. It includes aerodynamic damping and stiffness effects due to the structural movement caused by the wind flow. The analysis is based on the wind profile and on the aero-elastic parameters of the cross-sections (drag, lift, moment coefficients and derivatives). The wind profile is characterised by the mean wind velocity and the fluctuation (turbulence) velocity, both being a function of the height above terrain level. The stochastic is accounted for by using power spectra in the frequency domain. [6]

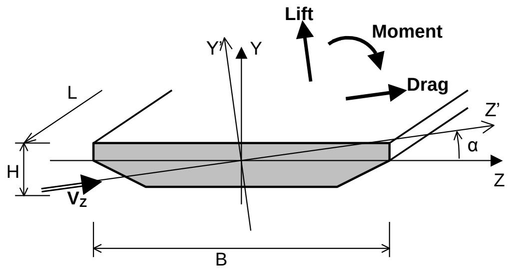


*Figure 9: Wind forces acting on the bridge section*

<u>*Application example – Vam Cong Bridge*</u>

The Vam Cong Cable Stayed Bridge will cross Hau River in Vietnam and will be the first steel composite cable stayed bridge in Vietnam with cross-section height between 1.75m and 2.75m (Figure 10).

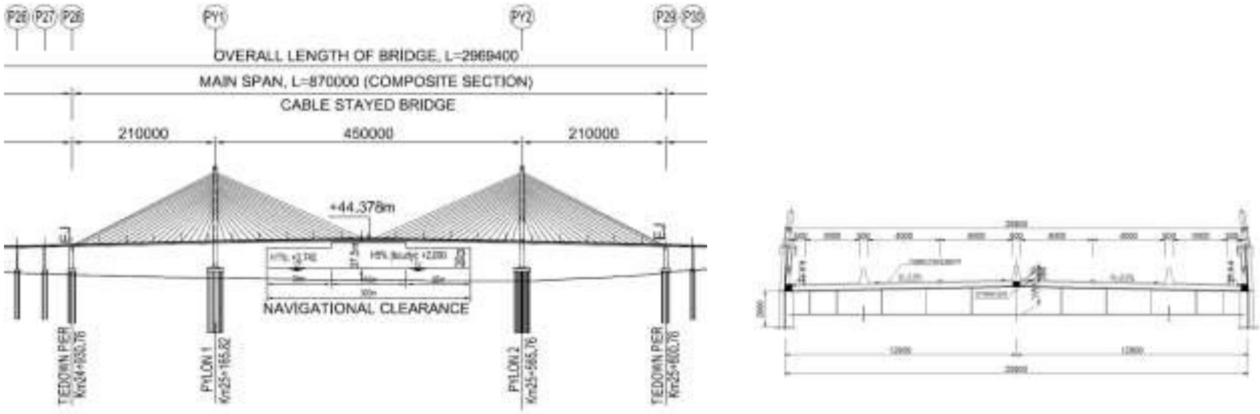


*Figure 10: Side view and typical cross-section of bridge deck – Vam Cong Bridge*

Design of the bridge as well as dynamic wind calculation was carried out by Joint Venture Group: DASAN Consultants Co., Ltd, Kunhwa Engineering & Consulting Co., Ltd and Pyunghwa Engineering Consultants Ltd, Korea.

RM Bridge Dynamic Wind module was used for dynamic wind check by company CTDCC, University of Transport and Communications, Vietnam.

I-Shaped steel composite girder was chosen due to light weight, simple shape and easy construction. However, this is meaningful only in case of satisfied aerodynamic stability. The wind tunnel test and aero elastic analysis have been carried out in order to show the girder cross section aerodynamic capacity (Figure 11).

*Figure 11: Global model and typical cross-section in wind tunnel test in Korea*

Aerodynamic coefficients have been checked by CFD analysis and wind tunnel test. The results were showing stability for different wind angles (Figure 12).

During design check the wind buffeting analysis, which considered static and dynamic wind effects for lateral wind was performed for the Vam Cong Bridge in construction state with free cantilevers and in final state.

Structural response by wind load was calculated in frequency domain. Aerodynamic damping and stiffness effect by wind vibration of the bridge and aerodynamic coefficients of the girder were considered. The Karman spectrum was used to simulate the frequency characteristic of wind load of the main flow and vertical direction and stochastic characteristic of the spatial distribution of the wind load is applied as define the decaying factor of the spatial correlation function (Figure 13). The peak value of the wind response is the function of the structural frequency and has values between 3 and 4.

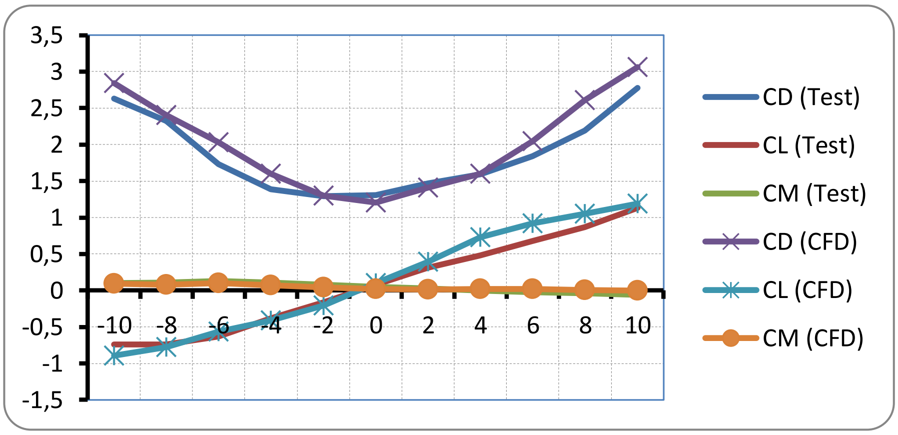


*Figure 12: Static aerodynamic coefficients by Wind tunnel test and CFD analysis at construction stage*

For construction stage a wind cable was necessary to add in order to secure dynamic stability of the bridge before closure (Figure 14).

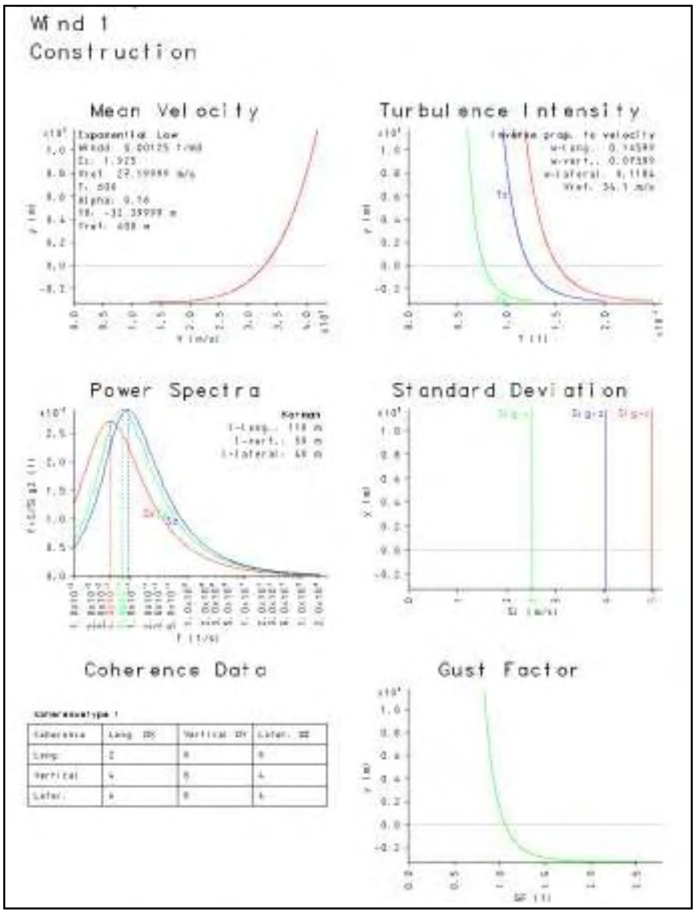


*Figure 13: Wind specifications for construction state conditions*

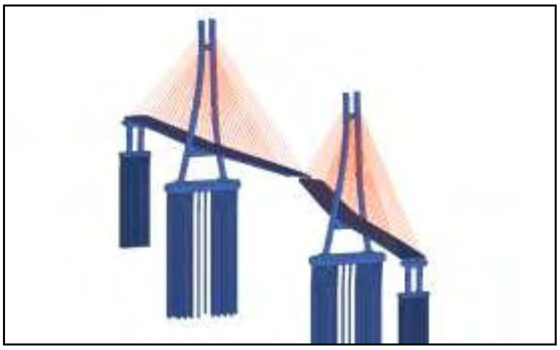

Figure 14: Maximum lateral displacement due to lateral wind direction

## CONCLUSION

Using a finite difference in the time domain the presented algorithm includes coupled effects of the creep, shrinkage and steel relaxation within overall structural non-linear analysis. Cable sagging, p-delta effects, large displacements or even contact problems can be combined with long term effects within consistent analysis. The proposed method for the numerical analysis can handle satisfactorily static and dynamic bridge behavior up to the time infinity; it is generally suitable for the investigation of cable supported bridges, but also all kinds of bridges with reinforced concrete, pre-stressed concrete or composite sections.

In the case of stage-wise erection, an exact anticipate calculation of the bridge deformations arising throughout the construction schedule – and therefore the definition of the required pre-camber values and fabrication shapes – was for long time an almost insoluble problem.
The respective erection control facility accurately controls the position and the forces in the segments in stage-wise built structures. The presented erection control tool also supports the definition of required compensation measures due to deviations from the scheduled position.

Dynamic wind analyses are increasingly becoming important in bridge engineering. These phenomena include vortex shedding and the lock-in phenomenon, across-wind galloping and wake galloping, torsional divergence, flutter phenomena and wind buffeting.

Several successfully realized long span cable supported bridges worldwide have proven, that wind related functions as well as all other high end numerical methods in RM Bridge solver match nearly all needs for the design of long-span bridges. Arbitrary complicated wind profiles with varying wind speed and turbulence intensity are easily defined. Together with the cross-section related shape factor diagrams defining the dependency of the drag- lift- and moment coefficients on the attack angle of the wind impact, these wind profiles allow a comprehensive wind buffeting analysis taking into account the varying along-wind and lateral forces of gusty wind events.

## AKNOWLEDGEMENT

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# MORE EFFICIENT TRANSPORT ACROSS THE OSLOFJORD
## - a feasibility study at an early stage

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## ABSTRACT

*The Norwegian Public Roads Administration in cooperation with The National Rail Administration and The Norwegian Coastal Administration are studying different concepts to improve communications across the Oslofjord. As a background this paper starts out with an overview of the purpose and methodology of the Norwegian scheme for analysing concepts before entering the pre-project phase. It also describes the present situation around the Oslofjord, objectives and the strategic approach for developing concepts and presents some indications from ongoing transport analysis. Finally the paper discusses how to balance the expectation that early studies of concept choice should require relatively modest use of resources against the need for necessary certainty that concepts could be implemented with acceptable costs. Especially this is a challenge when considering concepts that require unproven technology which is the case for this choice of concept study.*

## INTRODUCTION

This paper is about an ongoing strategic analysis of possible concepts to improve communication across the Oslofjord. The background is political initiatives at community and county level to replace the present ferry service with a new fixed link between the cities of Moss and Horten. The Ministry of Transport and Communications has commissioned The Norwegian Public Roads Administration to develop and analyse effects of different concepts in cooperation with The National Rail Administration and The Norwegian Coastal Administration. This work is carried out according to a governmental scheme for front-end governance of larger projects.

In my presentation I will address the following topics:

1. A brief overview of the Norwegian scheme for front-end governance of major public projects, i.e. projects with anticipated costs above 750 million NOK (approx. 100 million EURO)
2. Objectives and major topics in our project
3. Challenges regarding investigation of technological solutions and costs in a front-end analysis. How much resources should be used as basis for a governmental decision on whether to start “real” planning?

## FRONT-END GOVERNANCE OF MAJOR PUBLIC PROJECTS

Since 2006 major public transport projects in Norway have been subject to a formal procedure for front-end governance. In Norwegian these analyses are called “konsept-valg-utredning” (KVU) which directly translated “for concept-choice-study”. In this presentation I will mostly use the phrase “front-end analysis”.

Before resource demanding planning of a major project is started, the responsible governmental agency has to produce a report evaluating different concepts to meet identified needs for society.

The agency recommends which concept should be the starting point for further planning. After quality assurance[5] by external consultants and consultation among stakeholders the report is presented to the government. The question is whether planning of the project should continue into a pre-project phase. If the answer is yes, the government specifies premises for further process.

Choosing the right projects and concepts is considered to be more vital for society than implementing a given project within the adopted framework in terms of costs and time. The overall purpose of the new scheme is to increase the possibility that the most beneficial projects and concepts are planned and thus prepared for decision in government and the Parliament which formally adopts major projects. This early stage scheme is a response to a situation where major public projects to a large extent were chosen and planned at regional level without being considered in government. Politicians at national level were invited to consider financing of major projects after years of planning at local and regional level.

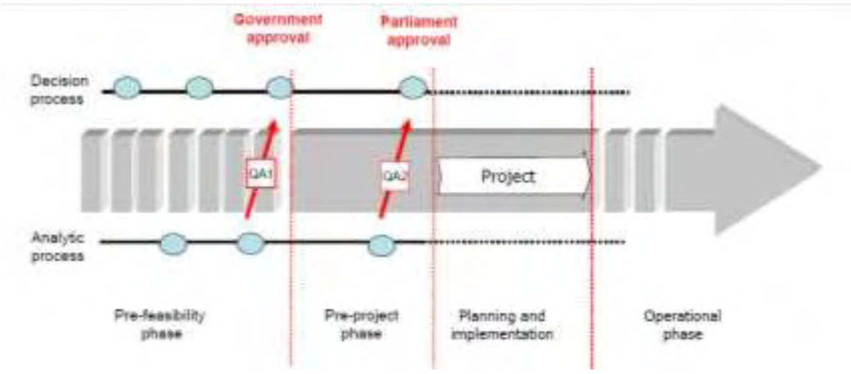


*Figure 1. The Norwegian quality-at entry-regime (QA1 and QA2) for major public investments. (From 2006 Samset, Berg & Klakegg)*

The early stage analysis carried out by the responsible state transport agency should explore at least two alternative concepts compared to effects of the zero-alternative without new investments. The basis for developing concepts is a thorough analysis of relevant stakeholders and their needs – also needs that may be in conflict with the project. The report from the front-end analysis of different concepts should comprise:

- Needs analyses with a description of present situation and expected development, mapping of stakeholders at national, regional and local level and their needs.
- Objectives and overall strategy based on the need considered to be most important for giving priority to the project. An overall objective for society should be identified and approved by the Ministry of Transport and Communications. In addition objectives for improvement in transport services and environmental quality are identified. This part of the report should also comprise a broad investigation of the range of possibilities to meet the set of objectives.
- Analysis of alternative concepts describing different concepts for the future transport system. The concepts are compared with a reference concept regarding economic costs and benefits, non-monetary effects and to which extent they meet objectives on efficient transport and environment.
- Recommendations

[555] So called QA1. Before a major transport project is presented in Parliament there is also a second round of quality assurance (QA2) on the calculation of costs based on a detailed plan for the project.

**THE FRONT-END ANALYSIS OF THE TRANSPORT SYSTEM ACROSS THE OSLOFJORD**

The region around the Oslofjord has a population of nearly 2 million, i.e. around 40 per cent of the total population of Norway (Figure 2). The present growth rate is among the highest in Europe and is expected to continue (expected growth 20 – 30 per cent within 2030). The Oslofjord region, especially Oslo, is a hub in the national transport system - both for passengers and goods. This means that the Oslofjord is a barrier to national, regional and local transports, and there is limited interaction between the southern “corridor-cities” situated on either sides of the fjord.


*Figure 2. Population density in the Oslofjord area (population figures in red)*

Today there are two possibilities for transport across the Oslofjord serving separate transport markets. A northern market uses the tunnel at the city of Drøbak (average annual daily traffic AADT 7 400 vehicles), while the ferry Moss – Horten is the preferred route for transport flows further south (AADT4 500). There seems to be little interaction between these crossings. Going through the centre of Oslo is also an option for some transports between the two sides of the Oslofjord, but this route is in periods congested with considerable delays.

Regional and local authorities in the two counties of Østfold and Vestfold find the present ferry service insufficient and have asked the government to carry out an analysis of a fixed link. The Ministry of Transport formulated the terms of reference for the front-end analysis in October 2011 and a report should be submitted by summer 2014.

The overall objective for society, approved by the ministry, is:

*To develop an environmentally friendly, efficient and predictable transport system to meet the needs of business and industri and to facilitate development of a more integrated market for housing and jobs across the Oslofjord.*

So far the project has described a set of concepts for an improved ferry service and for new fixed road crossings and has started analysing possible effects. The terms of reference also comprise considering new rail crossings. At present the project is investigating if the potential market for rail travel is sufficient compared to the costs of building a rail crossing and running a train service across the Oslofjord.

Broadly speaking the fixed road crossings considered are situated in a northern, a middle and a southern corridor (Figure 3). The first and latter corridors are expected to serve respectively the northern and southern markets, while the middle corridor to a larger extent is expected to meet the transport needs in both these markets. The first round of transport analysis seems to support this hypothesis.


*Figure 3. Fixed road crossings in three different corridors*

The transport model indicates that new road crossings in the southern corridor in a 2030-situation may attract a considerable increase in transport flows compared to the present ferry service. New travels on southern fixed links seem to be a combination of motorists choosing an alternative route east of the Oslofjord (mainly for longer journeys) and a change in destinations for shorter trips. Present policy on financing major road projects in densely populated regions of Norway means that fixed links probably will be partly financed by user payment. User payment will reduce traffic considerably, especially the amount of short travels. Nevertheless the transport model indicates that traffic on southern fixed links in 2030 might be more than twice as much as in a reference situation with a ferry service.

Obviously there are considerable conflicts between the building of new transport infrastructure in densely populated coastal areas and land use interests such as nature conservation, agriculture and recreation. Possible effects for other land use which cannot be measured in monetary terms, will of course be an important part in evaluating different concepts.
Many stakeholders expect that a fixed link, especially in the southern corridor, will stimulate increased growth in a new "corridor city" across the Oslofjord. The front-end analysis is told to look closely into possible regional impacts of a new transport system. Regional impacts might include different effects such as increased growth in population and employment in the Oslofjord region, relocation of expected growth within the region and economic benefits in addition to those covered by traditional calculations in transport models followed by cost-benefit analysis. The latter so called "wider economic effects" are partly due to increased population in housing and job markets within accepted use of time for daily commuting. There is no established method for calculating "wider economic effects", but that does not mean that their value should be zero.

Regional effects to a large extent depend on future competition between the Oslo-area with a present population of more than 900 000 and a future "corridor-city" 60 – 90 km further south with 230 000 inhabitants today. In this context we must bear in mind that investments in improved infrastructure for Inter City rail travel within 10 – 15 years will probably reduce travel times between Oslo and the cities in Østfold and Vestfold considerably.

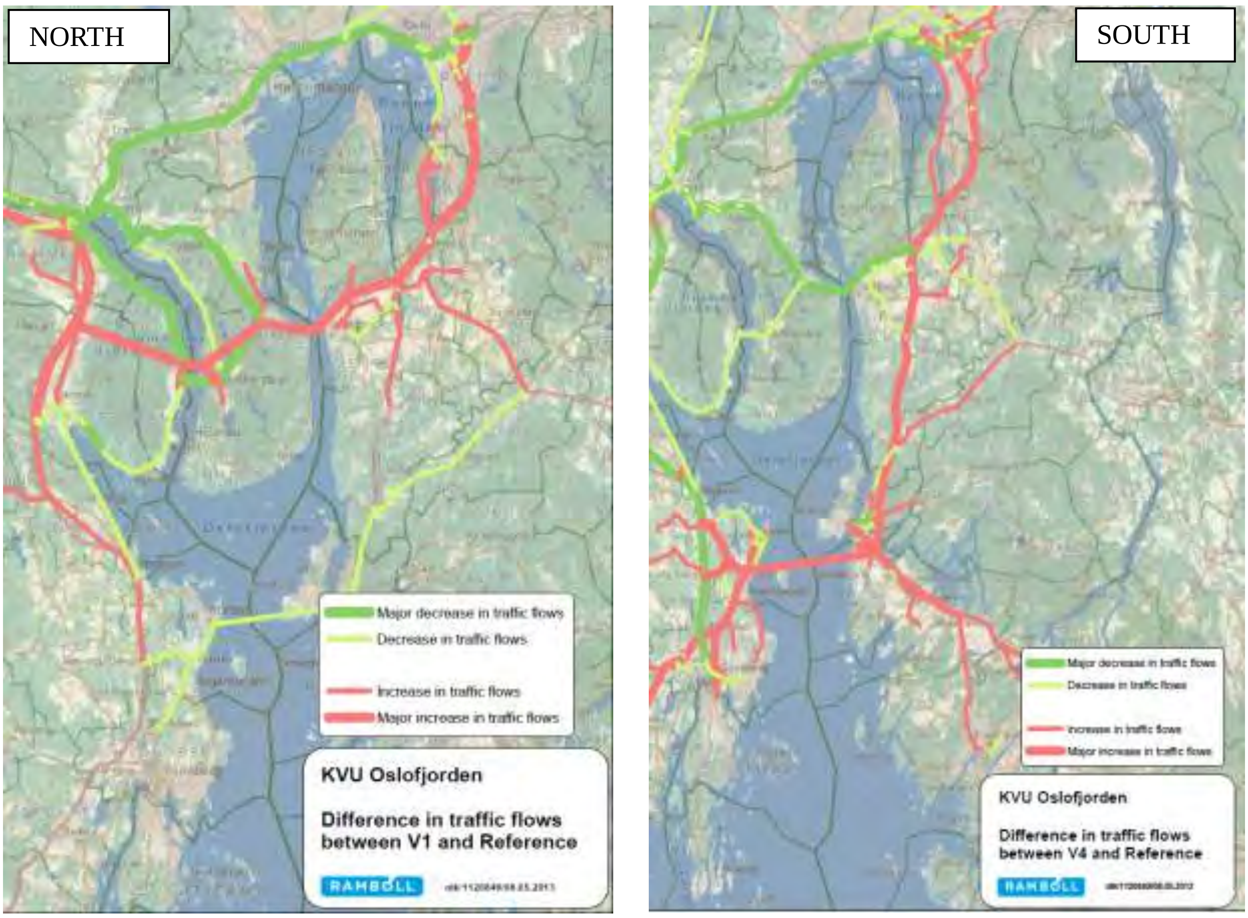


*Figure 4. Transport model results for traffic on new links in the northern and southern corridor.*

## TECHNOLOGICAL SOLUTIONS AND COSTS IN A FRONT-END ANALYSIS

In general a front-end analysis (followed by QA1 and a governmental decision on concept) should consider broad corridors, be based on existing knowledge and require relatively modest use of resources – both in terms of budget and time spent.

A recent evaluation of QA1-processes for major transport projects shows that the state agencies typically need 1 ½ - 2 years to present a front-end strategic analysis as basis for QA1.

Many would conclude that the result is that planning of important projects is delayed by two years. On the other hand one may argue that the formal procedure before starting ordinary planning facilitates and accelerates more detailed planning. Obviously nobody would claim that efficient planning of the wrong projects and concepts is beneficial for society.

According to the method of front-end analysis concepts should be rejected if costs are considered to be unrealistically high. Concepts should also be discarded if they require unproven technology with great uncertainty when it comes to implementation or costs.

When it comes to development and evaluation of technological concepts, front-end analyses should in principle use experience from similar projects. In the front-end analysis of an Oslofjord crossing, especially for the long crossings in the southern corridor, there is no or very little relevant experience regarding technological solutions and costs. The shorter crossings in the northern corridor could be built based on proven technology, even though they have bridge spans somewhat longer than existing Norwegian bridges.

With little experience based knowledge it is challenging to study proposed concepts and estimate costs with sufficient certainty for a large number of alternatives. At the same time uncertainty related to technological feasibility and unrealistically high costs may be important arguments for rejecting concepts before more detailed analysis. In an early stage of a front-end analysis the need for considering technology and costs has to be balanced against budget and time restrictions.

Lack of information on subsea topography and depths to bedrock in the Oslofjord is a challenge. Thus it is decided to carry out new (and some might say too costly) seismic surveys. Is this overuse of resources in a very early stage evaluation of a large number of concepts for a project with large uncertainties as to whether and when it will be realised?

The reason for relatively detailed geological surveys is to safeguard with necessary certainty that it will be possible to build a bridge or a tunnel in the different corridors. The aim is to reduce the risk for having to stop planning a concept already adopted by government. This may be necessary if new data reveal that realisation is very difficult and represents great technological uncertainties.

Similar challenges are faced related to lack of experience when studying necessary technology and estimating costs. Especially this is the case for very long bridges that might require completely new technical solutions, for example adapted from the offshore sector. There is very little empiric information on technology and prices per linear meter to estimate costs for such unconventional bridges. The implication is that evaluation of proposed concepts and costs in the early stage analysis becomes more resource consuming and costly than for projects with proven technology.

## CONCLUDING REMARK

An important challenge in implementing the Norwegian system of front-end analysis is to secure sufficient knowledge for choosing among competing projects and alternative concepts with efficient use of resources. This is especially difficult when there is little experience from similar projects. It is important to succeed in this effort if front-end analysis shall contribute to more efficient planning and increased benefit for society.

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# NORWEGIAN COASTAL HIGHWAY ROUTE E39
# PROJECT CONTENT AND OVERVIEW

by
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## ABSTRACT

*Since April 2011 the Norwegian Coastal Highway Route E39 Feasibility Study project, commissioned by the Ministry of Transport and Communications, has been investigating the potential for trade and industry, regional employment and settlement patterns of eliminating all ferries along the western corridor (E39) between Kristiansand and Trondheim. Further, this project is exploring the technology required for the remaining fjord crossings, and will also consider how the road and bridge infrastructure can be utilised to generate power from solar energy, currents, waves and wind. Implementation strategies and suitable types of contracts are also included in the studies. The very deep, wide and long Sognefjord has been used as a pilot site for developing and customising crossing technologies, and studies have confirmed the feasibility of crossing the fjord with either a suspension bridge with a main span of 3700 m, a floating bridge or a submerged floating tunnel. As a "game changing" project it will significantly change traffic patterns in Western Norway, making the E39 the shortest and most attractive western north-south route. A reduction of travel time in the corridor from Kristiansand to Trondheim from the current 20 hours to some 11-12 hours will result in direct transport cost savings of more than USD 1 billion per year. This will also mean a dramatic reduction in the number of man-years spent by drivers along the route and in the size of the transport fleet servicing the corridor. But the most interesting effects seem to be the long term structural and productivity increases coming from new and enlarged residential and employment areas. The Government's National Transport Plan for the next period (2014-2023), which was launched in April 2013, proposed the project implemented with investments of USD 25 billion over 20 years to completely upgrade the corridor to a modern standard without any ferry connections.*

## BACKGROUND

Norway's coastal highway E39 is part of the European road system. The route runs along the western coast of Norway from Kristiansand in the south to Trondheim in central Norway, a distance of almost 1100 km. By the end of 2013 there will be seven ferry connections remaining along this route; most of them are wide and deep fjord crossings that will require massive investments and longer spanning structures than have previously been installed in Norway.

The feasibility study for Coastal Highway Route E39 was commissioned by the Norwegian Ministry of Transport and Communications to clarify the technological challenges and possibilities and to explore the benefits for industry and for society at large of making this a more efficient corridor with no ferry connections. This project may reduce the travel time along the coast from Kristiansand to Trondheim by 8-9 hours, to a total of about 11-12 hours. The current travel time of some 20 hours between Kristiansand and Trondheim is also influenced by the overall road standard on this route.

The feasibility study contains four components: Society, Fjord Crossings, Energy, and Implementation strategies and types of contracts.
The feasibility study was primarily designed to create cutting-edge knowledge through studies and research. This project has already generated strong interest at home and abroad, and its character requires access to professional expertise domestically and internationally within a number of fields and professions.

The project is administered by the Norwegian Public Roads Administration (NPRA), with responsibility for the components distributed among different organisational units. The project presented its status report in December 2012, with final reports due in 2013. In the Government's National Transport Plan 2014-23 it is intended that the corridor should be completed within 20 years from 2014.

## SOCIETY

About half of Norway's traditional export is generated by industries and companies from the six counties route E39 passes through. Upgrading this route would provide the shortest possible travel time for north-south bound traffic in western Norway. Likewise, this route is crucial to regional development in western Norway, as many synergy effects depend on how effectively the corridor interconnects areas with large populations and substantial trade and industry.

The basic socio-economic methodology for quantifying impacts is well established in Norway and provides the basis for the analyses by calculating benefits from reduced transport costs, road accidents, and noise pollution. Parameters such as discount rate and investment depreciation period have recently been changed to 4% and 40 years by the Ministry of Finance. New calculations may also accept residual values beyond 40 years for elements like large bridges with long life spans. These changes mean that major infrastructure investments will become more economically viable than before.

Model analyses show that there is much suppressed north-south bound traffic along the corridor, resulting in many vehicles using substantially longer routes with lower direct costs in order to avoid the ferry links and narrow roads along the E39 itself. Model simulations for a corridor with typical speeds of 80 and 90 km/h, no road tolls, and all ferries replaced with fixed connections indicate traffic increases of between 250% and 500% at ferry crossings, as traffic will also be transferred to the E39 from alternative routes and east-west mountain passes. Person-related transport costs are projected to decrease by some USD 1 billion per year in 2020, figures that will increase when the benefits particularly related to the reduction of road accidents and transport costs of goods are added.

However, there are many other impacts that current methodology has not been able to substantiate, including long-term changes in general productivity, productivities in industry and trade, long-term impacts on the national and regional economy and on export values due to enlargement of residential and employment markets, and more efficient and less costly transport of goods. Approximately 15 000 man-years are currently spent each year by drivers driving along the corridor, which is about the same as if all employees of the Norwegian coastal town of Haugesund were to do nothing but drive along the E39 corridor. Calculations indicate that the proposed level of investments and improved road standards may reduce this figure by about 6000 man-years a year, and transporters may reduce transport fleets by up to 40 % for transporting the same volume of goods.

Among the important effects being studied are the wider economic impacts of investing in infrastructure. It is considered important to take into account the ripple effects that are not reflected in traffic or in direct economic growth, such as the conglomeration effects of human resources and the broader regional "knowledge hub" available to businesses along the western coast of Norway. In almost every business sector along or in close connection to the E39, companies consider better infrastructure a key element for continued existence and growth in the future. This is primarily because of the competition for human resources.

One study carried out by the Institute for Research in Economics and Business Administration (SNF) in Bergen indicates a long term annual general productivity increase of nearly USD 2 billion realised simply by joining the employment areas of about 300 000 employees between Bergen and Stavanger. Other studies project substantially lower figures, but mainly due to different assumptions as to whether levelling incomes in the longer run will influence the entire labour market or the commuting labour market only. Considering the differences between the respective labour markets involved, another study by the BI Norwegian Business School in Oslo reveals similar levels of increased productivity for the labour markets between Molde and Ålesund further north. Such studies are currently being rolled out for the rest of the corridor as well.

Another possible impact not previously studied in Norway derives from the fact that the level of unemployment due to health reasons is highly influenced by the size of labour market available. This has been substantiated by Agder Research Institute in Kristiansand, with the lowest levels being found in Oslo. There is substantial unemployment for health reasons along the southern coast and some places along the coast further north. Agder Research Institute is currently assessing how removing the ferry connections may influence the health-induced unemployment pattern along the E39 corridor, and its economic potential for society.

Further studies are about to be commissioned for impacts on the national and regional economy, and for the value of traditional exports.

**FJORD CROSSINGS**

This component has explored technological alternatives for the seven fjord crossings still being operated by ferries, and the general conclusion is that a fixed crossing can be established at any desired location. Further development of traditional long-span bridge technologies has enabled suspension bridges span longer, and floating structures are developed further with acquired experiences from oil and gas installations in the North Sea.

The Sognefjord, which is about 3.7 km wide at the existing E39 ferry crossing, has been used as a pilot site for new concepts for extreme bridges. With its vast depths of up to 1300 m and 2-300 m of bottom deposits above the rock, the Sognefjord is considered the most difficult and challenging fjord to cross. While the depth of the Sognefjord is extreme, the other fjords along the route are more typically some 500-600 m deep. Different depths and lengths require different concepts and solutions, and the technological implications and costs related to anchoring systems for floating structures are of particular interest.

Consultancy groups have been engaged through competitive dialogue procurement processes to explore the floating bridge and submerged floating tunnel options. Three main alternatives are being considered:

- Suspension bridge with a main span of 3700 metres: very steep slopes into the fjord require that the main span have the same length as the width of the fjord. Preliminary design involves two separate carriageways with a total width of 33 metres and 455-metre-high towers. The suspension bridge studies have been undertaken by the Bridge Section of the NPRA Directorate of Roads in Oslo.

- Floating bridge concept: a three-span suspension bridge, each span measuring 1234 metres. Two of four towers are placed on floating pontoons. The pontoons are anchored to the seabed at a depth of 1250 metres. The floating bridge studies have been undertaken by the consulting groups Dr.ing Aas-Jacobsen AS, Johs. Holt AS, Cowi, NGI, Skanska, Tor Vinje, and Rolf Øyvind Johnsen.

- Submerged floating tunnel concept: end-anchored designs with two separate and interconnected curved circular concrete tubes that are anchored to floating pontoons. The floating tunnel, with a length of 4083 metres, will enter traditional rock tunnels at both sides of the fjord for connection to the main corridor. The submerged floating tunnel studies have been undertaken by the consulting groups Reinertsen/Olav Olsen, Snøhetta Architects, Rambøll, Faltinsen, Johansson, Arup and Berger ABAM.

*Figure 1* *Suspension bridge with a main span of 3700 m (Ill: Norwegian Public Roads Administration)*

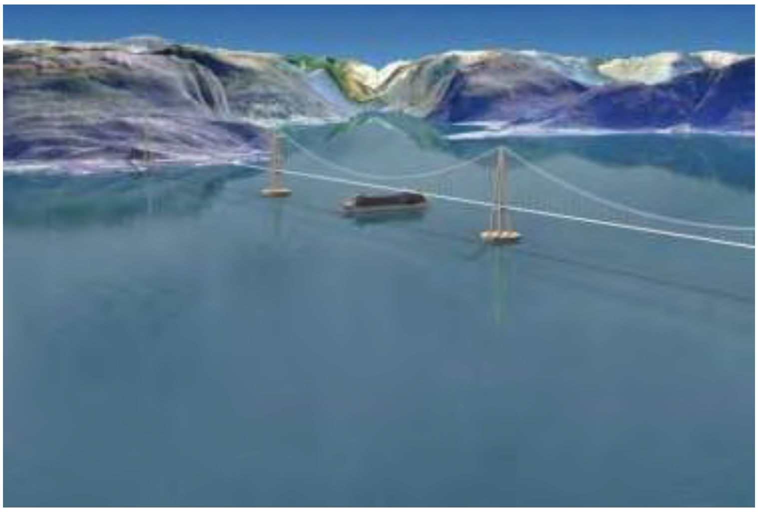

*Figure 2Floating bridge with suspension bridge on floating pontoons (Ill: Aas-Jakobsen)*

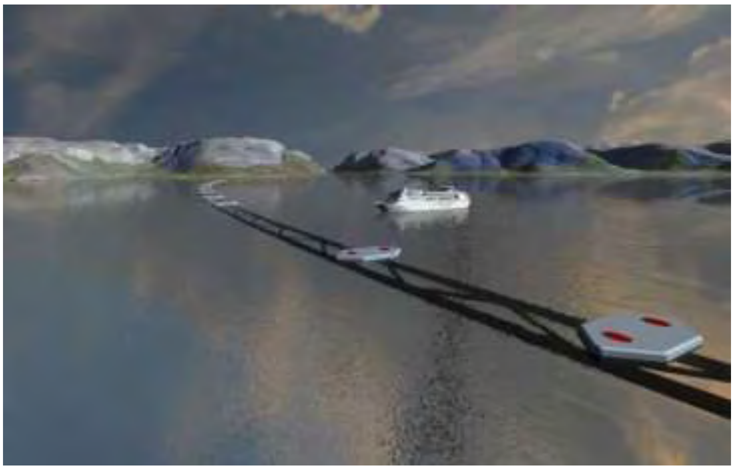

*Figure 3* *Submerged floating tunnel held in position by pontoons (Ill: Reinertsen/Olav Olsen)*

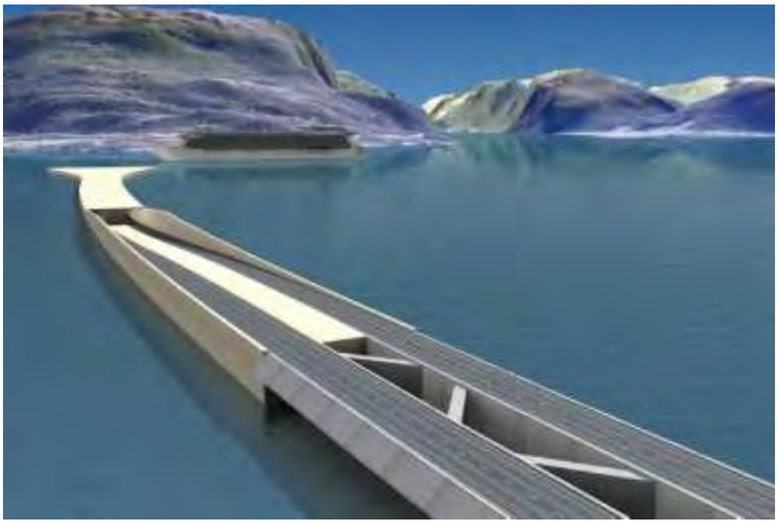

*Figure 4 Combination of floating bridge and two submerged floating tunnels connected to rock tunnels in a Y-shaped alignment (Ill: Norwegian Public Roads Administration)*

All concepts satisfy width, depth and height sailing clearances of 400 metres, 20 metres and 70 metres, respectively, which are currently the overall design requirements in this area. If calculations show the risk of ship collisions to be higher than the acceptable level, the concepts are designed to absorb collision energies without the risk of fatal accidents for road users as well as for passengers or crew on board the ship.

The conclusion is that crossing the 3.7 km wide and 1250-metre-deep Sognefjord is feasible with any of the three concepts: single-span suspension bridge, floating bridge or submerged floating tunnel. Even though the technical feasibility study involves only one design within each main concept, other designs (such as combinations of two) are likely to be viable. As yet the technical feasibility study has not completed the considerations of construction cost or technical optimisation of the three designs.

In order to reduce the costs of high depth anchoring systems, the Reinertsen AS consulting company is undertaking studies of an artificial seabed providing a cable and steel members grid into which such anchoring systems could be connected. Reference is made to the parallel sessions.

We have also made separate concept studies for the approximately 8 km long multi-span floating suspension bridge crossing of the Boknafjord north of Stavanger, and the approximately 6 km long crossing of the Bjørnafjord south of Bergen. The latter involves multi-span floating suspension bridges, possibly in combination with cable stayed or cantilever bridges. Those structures will be presented in the parallel sessions.

A few private companies are developing optional floating designs to the ones described in the project, and the respective companies are estimating substantially lower construction costs compared with the ones developed by the project. We have not yet been able to fully identify the reasons for that, but we are currently working on that with some of those companies. Provided that there is no major difference in terms of estimated Life Cycle Costs (LCC) or other negative aspects, such structures will certainly be interesting options to consider. It is very positive that private companies involve themselves in such development processes, thus contributing to the innovation processes in the sector. Reference is again made to the parallel sessions.

All the concepts mentioned above require further studies and research to verify alternative technical solutions and to establish design standards, analytic tools and technical approval regimes for such extreme structures. However, taking technology further can progress in parallel with formal planning processes and need not delay start of construction. Further technical research and investigations being planned for the four-year period 2014-17 will involve research in design, materials, instrumentation and monitoring systems, operation and maintenance, and testing regimes. Systematic collection of more detailed data related to the seabed, wind, tidal currents, and waves is in the programme for the immediate future

The objective is to provide access to applied technology within the coming 4-year period that will enable construction of any of the fjord crossings along the E39.

## ENERGY

The energy component has investigated how bridge infrastructures can be utilised to produce energy from renewable sources: solar power, tidal currents, waves and winds.

The production potential of such installations is of interest, particularly for power from waves in combination with floating pontoons, and it seems possible to produce levels of up to 2-300 GWh/km if conditions are favourable for the characteristics of the installed power generating equipment. The total consumption of electric power in Norway is about 115 TWh/year; NPRA consumption for street lights, subsea tunnels, water pumps, and tunnel ventilation is about 1% of the total amount. In that perspective it may be of great interest if such power generation should prove economically feasible. However, with the price levels experienced lately in Norway, such power generation will currently not be economically viable. But if such installations are built, they may contribute to meeting environmental goals related to $CO_2$-emissions.

Some locations are also of interest for producing energy from tidal currents, while wind and particularly solar power will not be given priority at this stage. While renewable sources are often found to be uneconomical due to high infrastructure costs, such costs may change considerably if shared with other investments.

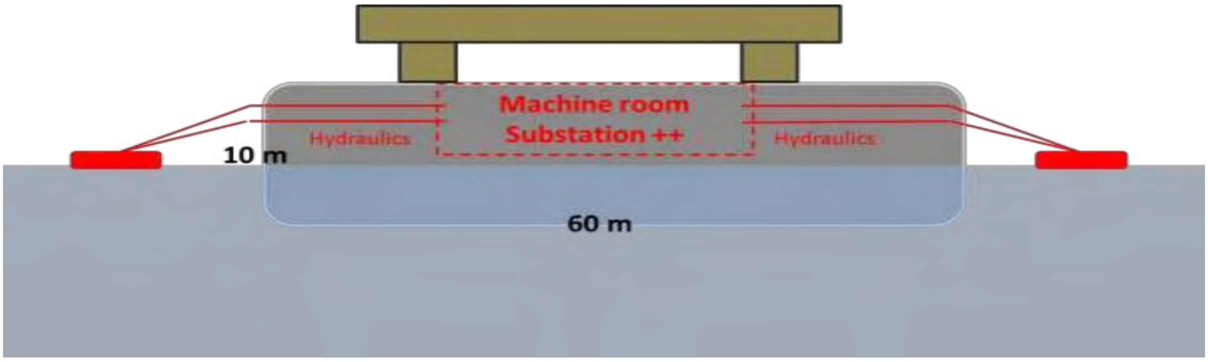


*Figure 5* *Principal design for wave power generation with installations in floating bridge pantoons*

There are several other innovations in this field that may be worth considering, and we will be starting to look for sites that have interesting renewable energy potential if combined with generator installations with the production characteristics corresponding well with characteristics of the energy source.

Other sources for producing power from renewable energy may be found underneath the approximately 160 00 river bridges along roads currently under NPRA management.

## IMPLEMENTATION STRATEGIES AND TYPES OF CONTRACTS

This component has considered which strategies and types of contracts are most appropriate and efficient for a project of a magnitude of about USD 25 billion. An implementation period of some 20 years is considered possible, including planning.

In terms of investments, the fjord crossings and the road sections in between may each require about half of the total investment of USD 25 billion. It is expected that the main risk of not completing the project within 20 years is not the time required for the construction contracts, but rather the vast planning tasks of the almost 1100 km long corridor. It is important to make planning and preparations for contracting as efficient as possible, and to leave room for innovation and technical development with the contracting consortium. In the same way, the distribution of risks between the contracting consortium and the client should be given thorough consideration and distributed in such a way that risks are managed by the party that can best control and manage the respective risks.

The contracts foreseen are long corridor contracts over long contract periods, mainly of contract types with considerable negotiations, such as competitive dialogue or alliance contracts. There is no doubt that the implementation of this and other large projects in the years to come in Norway will require substantial increases in the consulting and construction capacity. A condition for success will be to improve long-term predictability for the companies involved. We invite the construction sector associations to discuss how this best can be done.

A key recommendation is thus to engage a contracting consortium before detailed formal land use plans have been drafted, in order to align formal planning processes with contractors' preparations for construction. That approach requires acceptance by the Government for the established system of external quality control (KS2) to be undertaken based on less detailed plans compared with current practice. The fully detailed land use plans may then be developed in parallel with the contracting consortium's detailed technical design and operational preparations. This will also make it possible to use the innovative potential of the contracting industry in technical solutions, design and planning.

There is considerable political resistance in Norway towards PPP-contracts, is mainly due to the contractors' financing of the construction period. A new Norwegian version could be PPP-regimes where the public sector will provide the construction period financing, while other advantages of the PPP-types of contracts would be retained.

## CONCLUDING REMARKS

Access to necessary professional skills, and sufficient capacity within civil and maritime engineering, are paramount for the success of a project of such magnitude and complexity. The same applies to models for implementation and financing, types, size and length of contracts, and possible patent rights and conflicts of interest in the innovation and feasibility phases.

The scope and design of the project may also spur advances in civil and maritime engineering both domestically and abroad. It is thus imperative to involve firms and institutions and other stakeholders from the construction and maritime sectors at an early stage in the process.

The following link provides access to a video presenting the main concepts considered in the Sognefjord technical feasibility study:

http://www.youtube.com/watch?feature=player_embedded&v=L7en6etg2Mk

# REGIONAL IMPACTS OF TRANSPORT INVESTMENTS

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## ABSTRACT

*The paper reports the findings of a comprehensive global literature review and associated meta-analyses undertaken of relevant studies on the impact and benefits of regional transport investments. The report identifies the various macroeconomic and local areas of benefit and provides some evidence to give an introduction to the links between transport investment and outcomes. Some of the benefits identified include, most prominently, the improvement of trade efficiency, including inter-industry trade resulting from reduced travel time and lower costs. These and other outcomes, attributed to the transport investments, are derived from analysis of data collected and examples of regional benefits of transport projects. These outcomes also include those foreseen and unforeseen both positive and negative. The paper also looks at any evidence of societal impact, who benefitted and how.*

*The paper also discusses the potential of attracting inward investment and the opening of entirely new lines of business, agricultural and technical and how to identify these pre-project. There is also some analysis on the benefit to third parties such as electricity and gas transmission networks and seemingly unrelated businesses.*

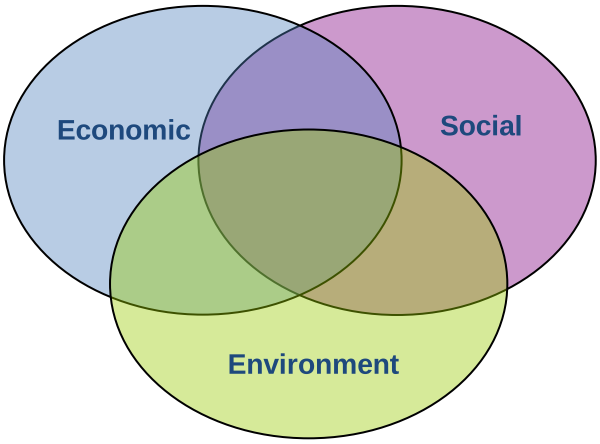
*Also examined is the evidence of how and where transport infrastructure investments have delivered benefits to regional development and what those benefits may be beyond the simple objectives of the provision of additional infrastructure. It examines some key conceptual and analytical issues surrounding the provision of benefits, with the help of some empirical evidence, draws policy conclusions.*

## 1 INTRODUCTION

There is a commonality between titanium veins in rock and empirical evidence-based impact assessments of regional benefits from transport investments, in that they are both rare finds. The reasons for this are mainly caused by the difficulty in securing funding ex-post analysis from reluctant budget-holders focused on project delivery targets, and the difficulties in attributing cause and effects to the investment. Impact studies also vary on the definitions of benefit as there is inevitably, to varying degrees, also trade-offs between the definitions of benefits in social, economic and environmental terms that studies find difficult to assess. Furthermore maximising benefit usually requires other investment to compliment the primary project as transport investment alone is usually insufficient to provide the greatest social, economic and environmental gain and positive impacts are realised if investments are made in tandem with support at local, national, or global levels. The allocation and type of project can cause disparities between different groups and how much they benefit from the investment. Global evidence of impacts of project investments can be found in many different locations and literature does exist in many languages and varying formats.

Perceived wisdom, from planners and transport project providers, is that transport development is a major contribution to increasing regional economic growth. Its major contribution is in lowering production and distribution costs, improving labour productivity, stimulating private investments and facilitating technological innovations. This paper looks at the evidence behind this perceived wisdom.

Not all benefits are economic, many can be social, many can be environmental, but other benefits may be less tangible or even less obvious. Certainly there will always be the issue of attributing benefits directly to the transport investment, but in the lack of any other significant change, we can presume that the investment was the primary change agent.




More importantly, the lack of transport infrastructure can often be a binding restraint on the regional economy or potential of the regional economy and also stifle possible known and unknown benefits more broadly. As important, is that for a region to remain competitive or become competitive, transport infrastructure needs to be present. Maintaining advantages and improvement on the long-term is a more complex argument where institutional and tax frameworks need to be considered alongside the employment demography of the region as it fits in a national demographic.

All this should be taken in the context in that development of transport infrastructure is a necessary but not sufficient condition for national and regional economic development and growth.

## 2 THE FUNDAMENTAL ASSUMPTION AND ROLE OF TRANSPORT INVESTMENT IN PROMOTING REGIONAL ECONOMIC DEVELOPMENT AND SOCIAL BENEFITS

- Upgrading of transport links within or to/from a particular ('disadvantaged') region are likely to be an effective means of enhancing the economic and social development of that region, and in what circumstances?

Many benefits may not be immediate and are more likely to evolve over time, and need to be linked, sometimes with difficulty to the original investment. Some investments are translational or even transformative producing a step change from low productivity even subsistence farming to commercial crop production.

Many benefits are responsive to the infrastructure provision, such as farmers starting businesses in highly perishable produce attracted to the comfort and being able to get their produce to consumers reliably and quickly (not restricted by ferry crossings and bad weather cancellations for instance). Although highly catalyzed, some benefits will be only partly attributable such as new start-up businesses.

Some transport investments can be transformational, such as an airport opening up an area to tourism, a rail connection opening up opportunity to exploit natural resources of a bulk nature, road expressways allowing efficient access to urban markets and job opportunities

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**Causality - Direct and indirect effects**

Transport investment and benefits including social and economic development suggests that transport investments generate two principle effects:

**Direct**

Accessibility improvement and time saving impacts: The multiplier effect results from the public-work nature of the investment as it generates employment and income in the local area, and these benefits last throughout the project's implementation service and maintenance period.

**Indirect**

Mainly economic multiplier and environmental impacts: The outcome of the economic multiplier effects where the price of commodities, goods or services drop and/or their variety increases. Indirect value-added and jobs are the result of local purchases by companies directly dependent upon transport activity. Transport activities are responsible for a wide range of indirect value-added and employment effects, through the linkages of transport with other economic sectors (e.g. office supply firms, equipment and parts suppliers, maintenance and repair services, insurance companies, consulting and other business services). Indirect effect includes "indirect third party", separate business activity that responds to the opportunity of regional economic growth such as an entertainment complex construction attracted by higher spending power of the region's population with further impacts on indirect investment related activities and the wider economy (induced employment).

The second category, 'indirect benefits", depends on the specific transport facility (e.g., port, rail, or highway), and benefits depend on size and location of the network and various regional features. These benefits, in turn, are assumed to generate long-term development effects as they improve the economic performance of individuals and firms and generate more efficient location patterns.

**Long-Term benefits**

In their paper 'The value of transport research, (2007) O'Neill, Greening and Salter maintain that in some cases benefits from projects can not be fully seen until many years into the future, in some cases ten years or more after completion of a project For instance, in some cases, deregulation may be required to encourage or allow competition in the provision of new transport services. In these circumstances, the initial benefits from the project may be delayed but benefits may then continue to accrue over many years.

Cost Benefit Analysis (CBA) is the normal method used for economic evaluation of projects and the results are usually quoted as the Benefit/Cost ratio (B/C), Net Present Value (NPV) or Internal Rate of Return (IRR). The NPV can be regarded as profit from the research. The ratio of benefits to cost (NPV/RPV) provides evidence of the profit in relation to the cost of the research. The IRR is the discount rate that reduces the NPV to zero. This measure is sometimes favoured by aid agencies because it does not require disclosure of a country's discount rate. The '1st year rate of return' is a measure which is sometimes used for timing the start of projects.

In order to compare the costs and time dependent benefits, costs that are incurred over the project period and the benefits that subsequently occur over a given time period are discounted (or compounded) to a fixed point in time usually referred to as the base year. The rate at which costs and benefits are discounted over time is referred to as the discount rate. This rate has a considerable influence on the value of benefits that accrue over the analysis period with long-term benefits being valued less than those that occur in the short-term. In general, the higher the discount rate then the lower will be the total benefits.

## 3 IMPACTS AND BENEFITS

### Assessment Methodology

Methods used for assessing economic impacts of proposed transportation projects have continually evolved over time. Whereas they once focused largely on the economic benefit of time and cost savings for travelers, they may now encompass broader factors such as accessibility roles in supply chains, labour market expansion, global trade growth, and their economic development implications. This broader view can be particularly important when considering transportation projects affecting network connectivity and activities of logistics centers, inter-modal terminals, and international gateway facilities.

Using examples throughout history, a generalized description is developed of the range of access, reliability, quality and cost factors that can affect the nature of economic growth impacts of transportation projects. While the set of factors is consistent with both theory and research findings, there has been a significant shortfall in their coverage by applied computer analysis models used for transportation decision-making.

### Macro-economic impacts

The spatial economic impacts of transport infrastructure investment can vary significantly depending on the status of existing infrastructure service as well as the type of infrastructure investment itself. To estimate the level macro-economic impacts or the impacts for a whole economy, it can be derived from changes in level of output, employment and income. There are also various types of impacts to consider whether the impacts are direct or indirect, temporary or permanent, and market and non-market or external[6]. The type of impact can also be positive and negative.

The most direct impacts would be generated from the construction investment itself which creates employment and income as well as capital investment. Another direct impact which is usually the main reason for such transport infrastructure investment is the benefit that passengers and freight service gained from the reduction in transport cost and time. This can, in turn, generate the indirect impacts on how people and firms make decisions on their production and residential locations which will activate further impact on income and employment.

[6] Oosterhaven, J., and Knaap, T., "Spatial Economic Impacts of Transport Infrastructure Investments" in Pearman, A., Mackie, P., and Nellthorp, J., 2003, "Transport Projects, Programmes and Policies: valuation Needs and Capabilities", Ashgate, Aldershot, pp. 87-1, http://www.rug.nl/staff/j.oosterhaven/transtalk03_raem_zzl.pdf (May 21, 2013)

The level of this benefit also depends largely on the existing status of infrastructure. Additional transport infrastructure service to the city or area where transport infrastructure services are already abundant will create relatively smaller marginal impacts. However, if the infrastructure is expanded to overcome a certain constraint or limited capacity, marginal impacts will be certainly higher.

Transport infrastructure policies can be tailored to be passive or active. A passive one indicating that the investment is planned in response to the emerging demand for such transport services. The impact from such policy would primarily be the total user benefits on top of the value of investment and other negative externalities. The active infrastructure policy, in contrast, tailors the investment toward the indirect impacts or the decisions of firms and people in their location choices. This type of policy would have additional impacts in term of the size and pattern of the geographical distribution of economic activities.

In addition to the positive impact on economic growth and productivity, there are other benefits from infrastructure investments. Available evidence suggests that infrastructure investment can raise property values, which reflects an improvement in living standards. For example, research suggests that proximity to public transit raises the value of residential and commercial real estate. Bernard Weinstein studied the effect of the Dallas light rail system on property values, and found that a jump in total valuations around light rail stations was about 25 percent greater than in similar neighborhoods not served by the system. This is consistent with studies conducted in St. Louis, Chicago, Sacramento, and San Diego, all of which find that property values experience a premium effect when located near public transit systems. Research has also shown that broadening the definition of housing affordability to include transportation costs reduces the number of effectively affordable neighborhoods in the United States; thus, infrastructure investment which lowers transportation costs should help increase access to homeownership.

A study by Climent Quintana-Domeque and Marco Gonzalez-Navarro[7] makes progress on estimating the causal effect of infrastructure investment on property values, using an experimental design. Specifically, the study randomly assigned some roads to be paved and others to be in a control group in the Mexican city of Acayucan. Their analysis suggests that such infrastructure investment substantially raised housing values on the newly paved roads, as well as provided benefits for home values on nearby streets. The rise in housing values on affected streets significantly exceeded the cost of paving the roads.

The benefits from transportation infrastructure extend beyond its effects on property values and housing affordability.

A well-maintained and robust network of transportation infrastructure, which allows individuals to access multiple modes of transportation, results in significant efficiency benefits. One study found

[7] Department of the Treasury, 2012, "A New Economic Analysis of Infrastructure Investment", *A Report Prepared by the Department of the Treasury with the Council of Economic Advisers*, pp. 9-10, http://www.treasury.gov/resource-center/economic-policy/Documents/20120323InfrastructureReport.pdf (May 21, 2013)

that in 2009, households at the national median level of income residing in "location efficient" neighbourhoods with diverse transportation choices realized over $600 in transportation cost savings, compared to similar households living in less efficient areas. In addition, well-maintained roads with adequate capacity, coupled with access to public transit and other driving alternatives, can lower traffic congestion and accident rates which not only saves time and money but also saves lives. Congestion is not limited only to roads but also to railways. Freight rail systems can play a vital role in relieving road traffic and in moving goods in a more fuel efficient manner. One study estimated that on average, rail freight are four times more fuel efficient than trucks. These benefits can also reduce dependence on foreign oil, improve energy efficiency, and reduce air pollution. For example, one study in the Los Angeles area found that traffic congestion has a significant effect on $CO_2$ emissions, and that reducing stop-and-go traffic conditions could potentially reduce emissions by up to 12 percent. Another study estimates that America's public transportation system reduces gasoline consumption by 17 billion litres annually.

## Economic Opportunities and Employment

Transport contributes to economic development through job creation and its derived economic activities. Accordingly, a large number of direct (freighters, managers, shippers) and indirect (insurance, finance, packaging, handling, travel agencies, transit operators) employment are associated with transport. From a business development viewpoint, transportation improvements can affect economic growth and development in at least four ways: (1) by enabling new forms of trade among industries and locations, (2) by reducing cargo loss and enhancing reliability of existing trade movements, (3) by expanding the size of markets and enabling "economies of scale" in production and distribution, and (4) by increasing productivity through access to more diverse and specialized labour, supply and buyer markets. Sound transport systems can help facilitating development of inter-industry trade. Inter-industry trade accounting was formalized in input-output analysis (Leontief, 1951) [8] and later supply chain management (e.g., Bowersox and Closs, 1996) [9].

Over time, continued infrastructure investments result in improved travel times, reliability and capacity. Not only does reliability in time and certainty increase from transport investments but reductions in loss rates were predicted using mathematical formulations of uncertainty risk analysis (e.g., see Bedford and Cooke, 2001) [10].

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[8] Weisbrod, G., 2007, "Models to Predict the Economic Development Impact of Transportation Projects: Historical Experience and New Applications", *The Economic Development Research working paper for the Annals of Regional Science*, pp. 3, http://www.edrg.info/images/stories/Transportation/models-to-predict-the-eco.pdf (May 21, 2013)

[9] Ibid

[10] Ibid

**Reducing Isolation and Enhancing Access to More Diverse Markets**

The theme of market access enhancement was already identified into the 1960's, as highway investment was seen by US government officials as a means for facilitating income growth through enhancement of access for labor, materials and customer markets. An early federal report focused on the benefit of the interstate highway system as increasing access by appearing to reduce effective distances between areas (FHWA, 1970) [11]. In 1964, a Presidential Commission reported that "economic growth in Appalachia would not be possible until the Region's isolation had been overcome" and Congress reacted the next year by funding the Appalachian Development Highway System "to generate economic development in previously isolated areas." (ARC, 1964) [12]. Three decades later, the economic efficiency benefits of greater access to diverse inputs was formalized in work by Krugman (1991)[13] and Fujita et al (2001)[14]. Lower entry barriers with reduced trade costs can result in higher competition as more businesses compete towards price convergence and lower volatility.

**Efficiency and associated time savings**

For industry in a given region, time and cost savings as well as gains in accessibility and reliability, arising from the improvements in transport infrastructure allow productivity gains to be achieved by improving their production and distribution. Wider access to the market creates both new business opportunities and increased competition, leading to further increases in profitability. The market is redistributed to the advantage of those companies which are able to adapt to the new market. The same process could occur for the labour market. Thus, a transport infrastructure project could be said to have an impact on private capital and labour productivity, and hence on overall economic growth. Access to a more diverse base of production factors such as skilled labour and capital can also contribute to economic growth.

The benefits associated with logistical impacts and assessment of the reduction in transport unit costs have been documented in various papers. For example, Austroads unit values of time (also adopted by the NSW Road and Traffic Authority's (RTA's) Economic Analysis Manual (EAM)) allow for the value of time of freight contents (from the point of view of freight consignees) (Austroads 2004; RTA 2004) [15] and represents a value for logistical efficiencies resulting from speedier freight deliveries to business.

[11] Ibid

[12] Ibid

[13] Ibid

[14] Ibid

[15] A Guide to the Structural Design of Road Pavements (AUSTROADS 2004), version 2, 2007, http://www.nzta.govt.nz/resources/nz-supplement-2004-austroads-pavement-design/docs/supplement.pdf (May 21, 2013)

In Kazakhstan the Almaty-Khorgos Corridor road had a significant impact on the regional economy with vehicle operating costs 13 percent lower and significant improvements in road safety.

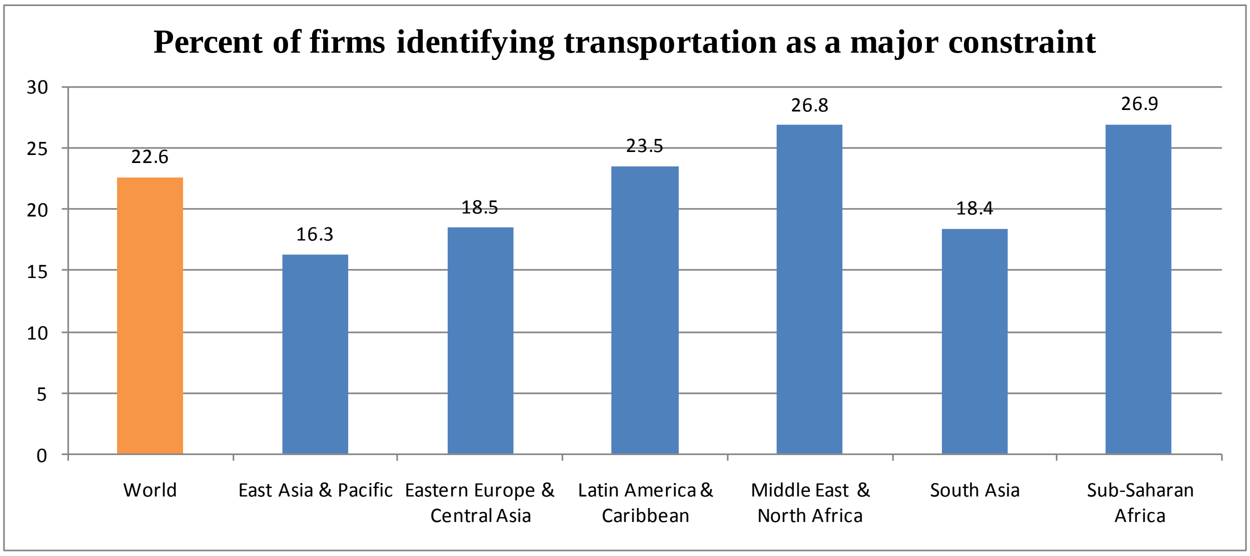


*Figure 1: Percent of firms identifying transport as a major constraint*

Conversely, inefficient transport can act as a substantial constraint to the development of private enterprises. As illustrated above (Figure 1), more than 22% of firms surveyed worldwide through the World Bank Enterprise survey indicate transport as a major obstacle to the growth of their business.

## Agglomeration benefits

Venables and Gasiorek argue that transport improvements will result in a reduction in costs as firms have agglomerated within a region. This agglomeration and associated external scale economies resulting from companies co-locating within a single region can result in regional centres of excellence and specialism. 'Knowledge spillovers' may emerge as firms and individuals are better able to observe new methods and techniques and industry 'best practice' the closer they are to the event (BTE 1999[16], SACTRA 1999[17]). These impacts will ultimately foster greater productivity and increased inter-firm competition, resulting in cheaper products. Such additional pro-competitive impacts are said to be ignored by conventional SCBA (BTE 1999) [18].

[16] Bureau of Transport Economics (BTE), 1999, "Facts and Furphies in Benefit-Cost Analysis", *Transport Report 100*, Commonwealth of Australia, pp.95, 178-179

[17] Transport and the economy: full report (SACTRA), 2012, http://webarchive.nationalarchives.gov.uk/20050301192906/http:/dft.gov.uk/stellent/groups/dft_econappr/documents/pdf/dft_econappr_pdf_022512.pdf, pp. 4.12 (May 21, 2013)

[18] Bureau of Transport Economics (BTE), 1999, "Facts and Furphies in Benefit-Cost Analysis", *Transport Report*

Similarly Rye, Ohr and Lyche (2001) [19] maintain that knowledge spillovers, resulting from agglomeration, generate external economies of scale, thereby increasing firm profits. This may also produce a higher willingness to pay for the marginal labour, especially within industrial clusters, than is the case for the average industrial wage. Therefore measures of the value of travel time savings, based on average wages will understate project benefits 10-fold.

This agglomeration can also take advantage of regional endowments and the relocation of resources to take advantage of comparative advantages and reliability enhancements that secure global supply chains integration.

For example, in Chicago, transportation agglomeration benefits have led to greater business clustering and economic growth associated with manufacturing, as businesses took advantage of Chicago's position in a national transportation network.

**Accessibility**

In many cases, the objective of transport infrastructure investment is to improve the accessibility of a given region by reducing travel time or increasing the potential to travel. The Severn Bridge, the bridge linking England to South Wales has its reputation in its travel time reliability. According to Cleary and Thomas report (p. 84, para. 8.16), the bridge reduced journeys between the two sides of the estuary by up to 50 miles (80 km) and by up to two hours travelling time (or possibly more, under congested conditions). The time and distance savings represent a major improvement in accessibility between the two banks of the Severn. In term of induced traffic, during 1967, the first full calendar year of operation, the flow of vehicles over the bridge averaged about 16 000 per day (Cleary and Thomas, Appendix 2). By 1991, the flow had reached 51 149 vehicles per day, significantly in excess of the design capacity of 46 575 (Miller, 1993, p. 7). The opening of the bridge clearly represented a major improvement in accessibility to the other side of the Severn for both private car and commercial truck users. Opportunities for leisure travel were significantly increased.

The Fixed Road Connection to Kristiansund, another example, is a road project consisting of three strait crossings. One 5.1 km undersea tunnel, one 1.3 km suspension bridge and one 0.9 km floating bridge connected with a new 18 km ordinary road. This gave Kristiansund a fixed road connection to the mainland and replaced three ferry connections. The largest contribution to reduced socio-economic costs was time savings. The weighted average time saved was 23 minutes. The quality of transport service has been improved e.g. less hazardous driving to reach the ferry and easier planning and time management when travelling. The main improvement in accessibility is reduced travel time between Kristiansund and Molde, with travel time between the two towns reduced from approximately 1 hour 35-45 minutes to 1 hour 15 minutes. This has led to an increase in commuting, with the number of commuters from Kristiansund to Molde

*100*, Commonwealth of Australia, pp. 179

[19] Rye, M., Ohr, F., and Lyche, L., 2001, "Cost-benefit analysis, regional impacts and external economies of scale", *7th International Conference on Competition and Ownership in Land Passenger Transport*

rising from 110 in 1990 to 300 in 1995 and total commuting-trips crossing the straits in both directions rising from 300 in 1990 to 540 in 1995. This indicates that 70% of the total growth in commuting which took place after the project concerned inhabitants of Kristiansund working in Molde. The project also gave Kristiansund a 24-hour open connection, which was not possible before.[20]

## Environment

In the design phase successful projects exhibit a detailed analysis on the proposed changes in noise; local air quality; greenhouse gases; landscape; townscape; heritage of historic resources; biodiversity; water environment; physical fitness; and journey ambience and design mitigation strategies accordingly.

Over the past three decades, transportation investment has resulted in a number of important environmental benefits. Most impressive are the improvements in air quality that have come even as the number of vehicle miles travelled has increased, thanks to cleaner vehicles, cleaner fuels, and a variety of transportation control measures including improved transit, bicycle and pedestrian programs, traffic flow improvements (often relying on Intelligent Transport System (ITS)), rideshare programs, and shared vehicle lanes. For instance, particles matters (PM10) emissions of the Road Transport sector has been significantly reduced between 1990 and 2010 in Europe with a drop in emission of 32.4% (Figure 2).

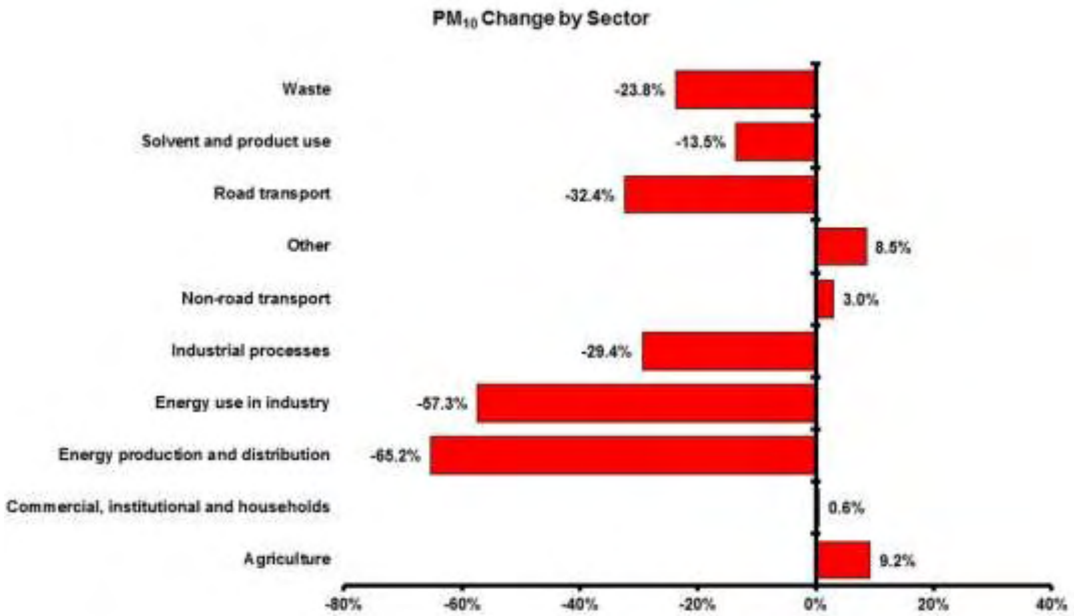


*Figure 2: Change in PM10 emissions for each sector and pollutant between 1990 and 2010 in the European Environment Agency (EEA) member countries*[21]

[20] Organization for Economic Co-operation and Development (OECD), 2002, “Impact of Transport Infrastructure Investment on Regional Development” pp. 76-79, http://www.internationaltransportforum.org/pub/pdf/02RTRinvestE.pdf (May 21, 2013)

[21] http://www.eea.europa.eu/data-and-maps/indicators/emissions-of-primary-particles-and-5/assessment-2

Advancements have also been made to reduce transportation-related noise, water, and light pollution. At the same time, rigorous preservation measures ensure that new transport projects tread lightly on the land; indeed, were it not for some new projects, many important archaeological and historical sites would go undiscovered and undocumented, and many contaminated industrial sites would remain vacant. In the future, the air we breathe will become even cleaner as vehicle emissions decline still further. The environment as a whole will benefit from an increasingly multimodal approach to transportation investment, which focuses not just on increasing vehicle movement, but on improving access for people and goods.

**Social inclusion**

Successful transport investments have appropriate and fair compensation procedures for those directly affected both individually and communities, but as important is to realise the benefits of infrastructure investment as a contribution to social welfare in a region.

Investments in transportation infrastructure can improve the quality of life in many ways, large and small. on broad range of social aspects – including access to education, health and other welfare facilities and yield important benefits to community including increased value of real estate. They can increase mobility and access, provide a greater choice of travel modes, improve safety, enhance the visual appearance of communities, cities, and natural landscapes e.g. eye-catching streetscapes, bridges, and transit stops, and increase community cohesion. While social benefits are more difficult to quantify than economic and environmental benefits, they are nonetheless every bit as important. For China, its 20-year development plan (1991-2010) for the National Trunk System (NTHS) calls for improving access for the less developed communities to economic opportunity. The Hebei Province is in the north region of China and its Expressway Project is part of the 35,000 km plan and is one of the priority sections of the NTHS from Beijing to Shanghai. The expressway is financed by Asian Development Bank (ADB) with social equity as one of the important component of the project. It was the first ADB-financed project in China that included a social equity specific component.

The upgrade and construction of the road network in Herbei Province, at county and village levels, has helped boost access for isolated rural areas and extend socio-economic benefits by providing more than 156,000 rural people from 168 villages direct access to main activity centres and overall road network Per capita income of sample village increased more than 10 per cent during 1999-2003. The overall expressway management offices employed some 1,400 permanent staff. In addition, more than 500 staff are employed to work at petrol stations, hotels, restaurants and shops in the roadside stations. 60 per cent of staff employed in the toll and roadside stations are woman.

A strong link between the improvement of quality of life resulting from the development of a sustainable road transport system were also shown. Hebei Academy of Social Science (HASS) assessed the project's financial impact and found the per capita income of sample villages increased on average more than 10 per cent, total rural population fell from 93 per cent in 1998 to 86.5 per cent in 2004. Per capital farmer income of the project area grew from CNY212 in 1998 to CNY958 in 2004.

**Removing a disadvantage**

In the Greater Mekong Subregion (GMS[22]), a study of the Southern Economic Corridor (SEC)'s impact on Cambodia conducted by the Mekong Institute found an increase in living standards of those along the corridor (Phyrum, Sothy, and Horn 2007) [23]. The study reported improved access to healthcare, education, and markets as well as the development of additional public service facilities. It also reported an improvement in trade routes and reduced trade costs at cross-border points. The tourism sector was said to have added more than 560,000 jobs in light of the SEC, representing over 8% of total employment in 2004. The authors of the study also estimated that this sector added almost 5 percentage points to GDP in direct economic activity and another 10.5% in indirect.

In the same GMS subregion, the immediate benefit of the East-West Economic Corridor (EWEC) was the improved connectivity and integration with the neighbouring countries—Thailand, Lao PDR, and Viet Nam—resulting in reduced travel time and transport costs. A 2006 Japan External Trade Organization (JETRO) study found that with improvements in the land transport network of Thailand, Lao PDR, and Viet Nam, including completion of the Second Mekong International Bridge linking Lao PDR and Thailand (part of the EWEC), transit times could be reduced by 25% (JETRO 2005) [24]. These findings were based on surveys of Japanese firms operating in the region.

Banomyong (2007) [25] analyzed the impact of the North-South Economic Corridor (NSEC) on logistics in the GMS region. He found major improvements in both time savings and shipping costs with the full implementation of the economic corridor.

Another example is the Jamuna Bridge which was opened in Bangladesh in June 1998 connecting Bhuapur on the Jamuna River's east bank to Sirajganj on its West Bank. The bridge has substantially reduced poverty, increased economic and social opportunity and is generating these benefits at an increasing rates nationally and especially within Rajshahi Division, a region in the northwest of Bangladesh.

At the national level, the Jamuna Bridge has led to a significant reduction in the transport margin for goods entering and leaving the northwest, which results in markedly improved trade

[22] The Greater Mekong Subregion (GMS) is a natural economic area bound together by the Mekong River, covering 2.6 million square kilometers and a combined population of around 326 million. The GMS countries are Cambodia, the People's Republic of China (PRC, specifically Yunnan Province and Guangxi Zhuang Autonomous Region), Lao People's Democratic Republic (Lao PDR), Myanmar, Thailand, and Viet Nam (source: ADB)

[23] Stone, S., and Strutt A., 2009, "Transport Infrastructure and Trade Facilitation in the Greater Mekong Subregion", *ADBI Working Paper Series*, pp. 9 http://www.adbi.org/files/2009.01.20.wp130.transport.infrastructure.trade.facilitation.mekong.pdf (May 21, 2013)

[24] Ibid

[25] Ibid, pp. 10

flows. The bridge lowered transport costs, causing regional exports of goods and migration of labor to increase. It also helped increasing the production of electricity, facilitated by the conversion to gas turbines, which is increasing the supply of electricity in the national grid line; and, increased attractiveness for locating processing plants.

At the Northwest regional level, there is at least 18% increase of passengers, a sharp increase in number of trucks and buses, and an increase of at least 5% of the volume of interregional trade involving Northwest. Transfer of gas by pipeline to the Northwest has already resulted in significant cost savings on existing power generators and enabled additional investment on power generation at Baghabari. Reduced transport cost and significant time savings have led to an increase in prices received by producers of farm products, thus, has encouraged increases in the production of vegetables, potatoes, tobacco and sugarcane in the Northwest. The opening of the bridge has also stabilized the regional supply and prices of items such as day-old chicks, which has boosted local production and consumption of poultry. All evidence suggests that the bridge has encouraged major investment in the cement industry in the Northwest, which reflects the confidence private investors have regarding the prospects for the region.

At a local level, construction of the bridge contributed substantially to increased employment opportunities, wages, and incomes. Quality of life indices in the local impact area have generally improved. Access to potable water and sanitary waste disposal facilities have increased dramatically due to construction of the Project. The Post-Project documentation indicates that the Project has enhanced the status of women and their access to resources in relation to land ownership.

For direct impacts to bridge/ferry users, overall, traffic has been considerably higher than previously projected. Passenger traffic in particular has exceeded expectations for 2001; the number of light vehicles and buses crossing the bridge was 110 percent and 168 percent greater than projected. As a result of the above, the Economic Internal Rate of Return (EIRR) of the Project is now estimated at 18.2 percent, compared to 16.8 percent in the ICR and 14.5 percent in the SAR. Once these benefits were distributed among income classes, it was found that a significant majority (72.5 percent) go to the non-poor, while the poor (91.6 percent) and very poor (1 7.9 percent) receive relatively few direct benefits.

**Tourism**

Facilitating movement of tourists between their place of origin and their destinations, transportation sits at the heart of tourism industry in many countries. Specifically in USA, the travel and tourism industry is one of its' largest industries generating $171 billion in payroll, $654 billion in direct travel expenditures and $105 billion in Federal, State and local taxes, providing 7.5 million jobs directly and another 10 million indirectly in 2005. In more than two-thirds of the states, the travel and tourism industry is among the top three sources of jobs.

Approximately 80% of all travel occurs on highways and driving is the most popular recreational activity for Americans[26].

Opening up a region to touristic opportunities can be transformatitive. In the Turks and Caicos Islands the an interested resort operator (Club Med) in the early eighties asked for an international airport to be built as part of the deal to invest in the hotel complex. The island of Providenciales gained an international airport that enabled expanded growth in tourism and associated services. This led to a significant economic growth from the mid-1980s onwards, with full employment and with an average annual GDP growth rate of 8% by the early 21st century [27].

**Safety**

New transport investments create the opportunity to design safe systems that can substantially reduce the number of deaths and casualties from road crashes.

The calculation[28] of the most recent data available in the Asian-Highway Database shows interesting facts about the average number of fatalities per billion vehicle kilometre for each of the Asian Highway classes[29], roads with the condition lower than that of class III standard show the worst safety record (Figure 3). The average number of fatalities per billion vehicle kilometre for such class is approximately 167. The average fatalities of class III and class II roads are significantly lower than that of the below class III roads at 68 and 96, respectively. The overall safety record is substantially enhanced further when the class II road is upgraded to class I. A sharp reduction to the average fatalities of 31 per billion vehicle kilometre is shown. When the road is being upgraded to the top class, the primary class, the safety record is reaching its top performance. The calculation shows that the fatalities rate drop sharply to less than 3 fatalities per billion vehicle kilometre.

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[26] King, Aubrey C and associates, 2007, "Relationships between transportation and Tourism: Interaction between state departments of transportation and state tourism offices", pp. 3, http://onlinepubs.trb.org/onlinepubs/trbnet/acl//NCHRP202423D_FinalReportContractor_20071126.pdf (May 21, 2013)

[27] Department of Economic Planning & Statistics Turks and Caicos Islands Government Ministry of Finance and National Insurance Assessment methods, 2004, "Turk and Caicos Islands/European Community Development Cooperation: Single Programming Document for EDF 9", pp. 7, http://ec.europa.eu/development/icenter/repository/print_turc_spd_en.pdf (May 21, 2013)

[28] The calculation is based on the most recent data available (2008 and 2010 data). Based on data availability, calculation is made from data of 23 countries for 485 AH sections which account for 32.5 per cent of total AH sections.

[29] AH Classification:

Primary Access-controlled highways with asphalt or cement concrete pavement

Class I 4 or more lanes with asphalt or cement concrete pavement

Class II 2 lanes with asphalt or cement concrete pavement

Class III 2 lanes with double bituminous treatment pavement

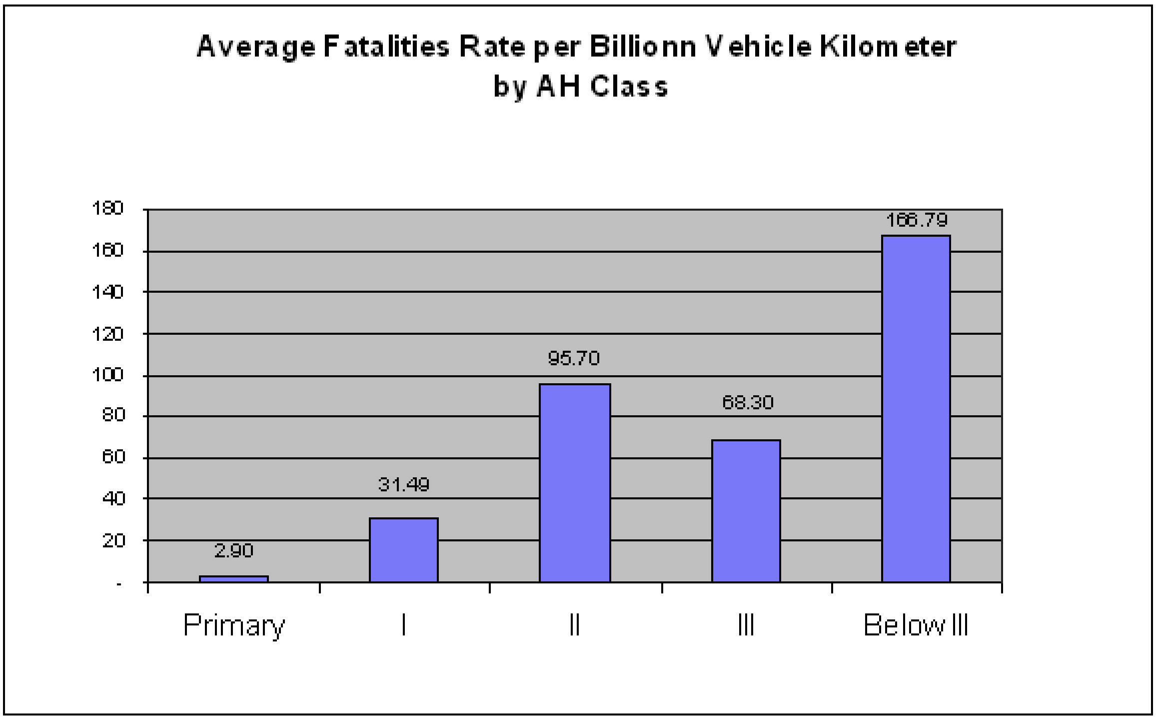


*Figure 3: Fatalities for billion vehicle kilometres of different classes of Asian Highway*

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## 4. EFFECTIVENESS OF TRANSPORT INVESTMENT IN PROMOTING REGIONAL BENEFITS

Regional benefits generated from the effectiveness of transport investment have been significantly recognized for larger nations such as USA, China and India. According to the paper on “Recent Examples of the Economic Benefits from Investing in Infrastructure” by the Executive Office of the President (2011), the US economy relies heavily on transport infrastructure. In order to meet the growing need of a growing economy with the recognition of the pressing need to revitalize USA’s infrastructure network, President Obama has proposed $50 billion in immediate investments in transport infrastructure, as part of the American Jobs Act which includes investments to make highways safer and more efficient, to repair and modernize public transit systems, to improve intercity passenger rail service and develop high-speed rail corridors, to improve airports and modernize the air traffic system, and to support innovative multi modal transport programmes. “Our generation inherited the world’s best transportation system made possible by the commitment of the past two generations to invest in the country’s future. We have spent that inheritance”.

For the case of China, a 41,000 km National Expressway Network (NEN) is a central plank of the government's "Go West" initiative, which is intended to help narrow regional inequalities and to connect all cities of more than 200,000 people. Over the last two decades, China has invested heavily in the upgrading of its road network, constructing a National Expressway Network which is second in its size only to the US Interstate Highway System. The construction of this network has formed an important part of China's national development strategy, which has been increasingly focused on promoting the catch-up of China's lagging inland regions with its prosperous metropolitan and coastal regions. The NEN has brought sizeable aggregate benefits to the Chinese economy, increasing overall real income by roughly 6 percent in 2007 [30], over the level that would have prevailed in the absence of the network, it has, as of yet, however, had little impact on regional disparities. The dispersion of the log of real income per worker levels across prefectures is essentially the same as would have existed in the absence of the network. And the largest aggregate gains have tended to be concentrated in what, in any case, would have been the most productive prefectures.

Similar to China, India also intensely focuses on its 5,846 km network of 4-6 lane expressways connecting India's four largest metropolises: Delhi, Mumbai, Kolkata and Chennai, thus forming a quadrilateral of sorts. Four other cities among the top ten metropolises: Bangalore, Pune, Ahmedabad, and Surat, are also served by the network, which connects many of the major industrial, agricultural and cultural centres of India. The expressways help establishing faster transport networks between major cities and ports, providing an impetus to smoother movement of products and people within India, enabling industrial and job development in smaller towns through access to markets, providing opportunities for farmers, through better transportation of produce from the agricultural hinterland to major cities and ports for export, through lesser wastage and spoils, driving economic growth directly, through construction as well as through indirect demand for cement, steel and other construction materials, and giving an impetus to Truck transport throughout India.

The Golden Quadrilateral highway project of India has improved the connectivity and market accessibility of districts lying close to the highway compared to those more removed. Non-nodal districts located within 0-10 km from the GQ network experienced substantial increases in entry levels and higher productivity. Dynamic specifications and comparisons to the NS-EW highway system mostly confirm these conclusions, with the most substantial caveat being that the productivity gains may be upwardly biased by a pre-period dip. The GQ upgrades also appear to have facilitated a more natural sorting of industries that are land and building intensive from the nodal districts into the periphery locations; the upgrades also appear to be encouraging decentralization by making intermediate cities more attractive for manufacturing entrants.

For assessing the benefit of regional infrastructure investments in Asia, some studies have used global computable general equilibrium model where infrastructure investments were considered as lowering trade costs. One of such model has been developed by the Asian Development Bank Institute (ADBI 2010)[31] which concluded that developing Asian countries will overall gain 6% of real income by 2020 if the estimated transport investment needs in Asia are met. In money terms, it is close to $ 1 trillion that could be generated through investment in regional transport infrastructure. The table below shows the distribution of benefit per country (Table 1).

[30] Roberts, M., Deichmann, U., Fingleton, B., and Shi, T., 2010, "On the road to prosperity? the economic geography of China's national expressway network", pp. 29, http://elibrary.worldbank.org/content/workingpaper/10.1596/1813-9450-5479, pp. 30-31 (May 21, 2013)

[31] http://www.adbi.org/files/2010.06.30.wp223.regional.infrastructure.investment.asia.pdf

*Table 1: Simulated Gains in Real Income from Investment in Regional Transport Infrastructure in Asia, by Type of Infrastructure, 2020*

| **Country/Region** | **(% of Baseline GDP)** | **(2008 US$ billion)** |
|---|---|---|
| PRC | 4.7 % | 345.8 |
| Indonesia | 11.6 % | 92.7 |
| Malaysia | 22.5 % | 75.3 |
| Philippines | 8.9 % | 24.4 |
| Thailand | 16.2 % | 80.5 |
| Viet Nam | 19.2 % | 32.4 |
| Bangladesh | 6.2 % | 11.2 |
| India | 6 % | 161 |
| Pakistan | 3.8 % | 12.5 |
| Sri Lanka | 6.8 % | 4.5 |
| NIEs [a] | 2.8 % | 84.2 |
| Central Asia [b] | 5.9 % | 19.6 |
| Rest of Asia [c] | 12.5 % | 23.5 |
| Developing Asia | 6 % | 967.7 |

a) Hong Kong, China; Republic of Korea; Singapore; and Taipei,China.
b) Armenia, Azerbaijan, Georgia, Kazakhstan, Kyrgyz Republic, Tajikistan, Turkmenistan, and Uzbekistan.
c) Afghanistan, Bhutan, Brunei Darussalam, Cambodia, Lao People's Democratic Republic, Maldives, Mongolia, Myanmar, Nepal, and Timor-Leste.

## 5. THE CRUCIAL ROLE OF POLICY DESIGN

At least three types of policies are vital for the attainment of potential development benefit. The first is investment policy, which determines attributes such as mode type, investment's scale, facility location and function in the larger network. Second, regional economic policy, which influences firms' location, labour market conditions and land market economies. Lastly, general public policy making whose essential task is to resolve conflicts among stakeholders. In Western democracies, the ability of governments to coordinate their activities and design complementary policies to gain maximum economic development effects from their capital investments, by and large, is fairly limited. Paradoxically, however, it is only in democratic societies that economic development reaches its maximum potential.

## 6. CONCLUSIONS

Transport projects do affect the economy of a region. They can in particular affect the location and pattern of economic activity, and be used to reduce regional disparities.

Successful transport projects general comply with a sustainable development framework setting out their environmental, economic, safety, accessibility and integration effects. It is evident that transport investments that take into account wider benefits can be improved from appraisals that go beyond the more conventional 'time-savings' assessments.

This paper has focused on the wider view of benefits including economic and macro-economic forecasting with the presence and scale of "wider economic benefits". These wider benefits contribute to the impact of transport on productivity and GDP. Evidence from studies and appraisals has been included where appropriate.

Social gains from regional transport investments can contribute to increases in GDP and the social rate of return from their primary transport benefits should be the driver of transport investment decisions. The predicted economic development outcomes should be second to transport output evaluation.

Transport investments serve as supporters of regional economic development and act as a catalyst non-transport factors. Transport development can generate significant economic development when market externalities are met by transport opportunities.

An enabling environment with complimentary policies should be in place in order for development benefits to materialize. Impacts from indirect third party activity is separate business activity that responds to the opportunity of regional economic growth such as an entertainment complex construction attracted by higher spending power of the region's population and should be taken into account.

This paper lays a foundation to commission further work to investigate impacts from transport investments and determine a more robust planning tool for regional development that clearly links transport investments with expected outcomes.

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