Ferry free E39 –Fjord crossings Bjørnafjorden | 304624

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CONCEPT DEVELOPMENT FLOATING BRIDGE E39 BJØRNAFJORDEN

Structural response analyses

Norconsult

DR. TECHN. OLAV OLSEN

Prodtex Production / Technology / Excellence IFE Pure Logic – The science of probabilistic reasoning HEYERDAHL ARKITEKTER AS H&BB MIKO MARINE AS BUKSÉR OG BERGING FORCE TECHNOLOGY SWERIM

REPORT

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CONCEPT DEVELOPMENT FLOATING BRIDGE E39 BJØRNAFJORDEN

Document name:

STRUCTURAL RESPONSE ANALYSES

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Summary

In this reports a description of the applied loads are presented along with the limit states and requirement that have been applied in design.

The limit state response is also presented together with the calculated capacity with regards to the requirements given in design basis.

In general the structure shows good capacity with regards to;

- Comfort requirement
- Global stability
- ULS/ALS response
- Deformation requirements (rotation/vertical)

Some optimization should be done with regards to rotation from traffic. Besides this, all requirements presented in this report are fulfilled.

Table of Content

1 INTRODUCTION

1.1 Current report

In this report the global structural response of the bridge is presented.

Chapter 2 give an introduction to the loads applied in the analyses, both static analyses and coupled analyses.

Chapter 3 introduces the load combinations applied in order to calculate response values for various limit states.

Chapter X Charactersitic response

Chapter Y Structural capacity requirements

Appendix A Takes on the geometry input to the global analyses

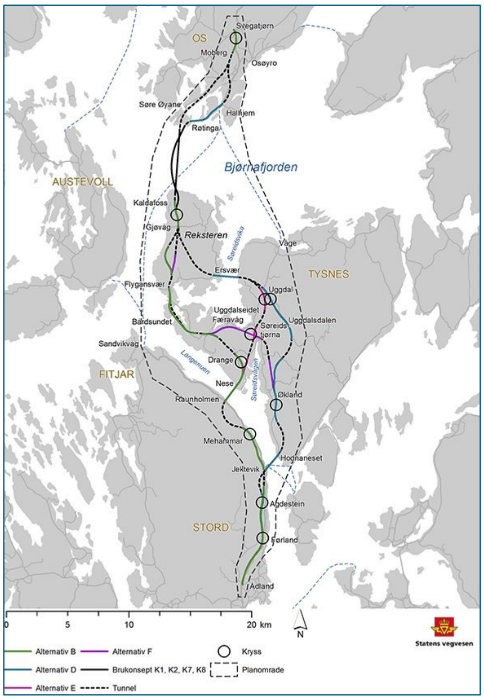
Appendix B presents the results from the modal Eigen value analysis

Appendix C presents results from the coupled analyses

Appendix D presented results from the global stability evaluation

1.2 Project context

Statens vegvesen (SVV) has been commissioned by the Norwegian Ministry of Transport and Communications to develop plans for a ferry free coastal highway E39 between Kristiansand and Trondheim. The 1100 km long coastal corridor comprise today 8 ferry connections, most of them wide and deep fjord crossings that will require massive investments and longer spanning structures than previously installed in Norway. Based on the choice of concept evaluation (KVU) E39 Aksdal Bergen, the Ministry of Transport and Communications has decided that E39 shall cross Bjørnafjorden between Reksteren and Os.



SVV is finalizing the work on a governmental regional plan with consequence assessment for E39 Stord-Os. This plan recommends a route from Stord to Os, including crossing solution for Bjørnafjorden, and shall be approved by the ministry of Local Government and Modernisation. In this fifth phase of the concept development, only floating bridge alternatives remain under consideration.

1.3 Project team

Norconsult AS and Dr.techn.Olav Olsen AS have a joint work collaboration for execution of this project. Norconsult is the largest multidiscipline consultant in Norway, and is a leading player within engineering for transportation and communication. Dr.techn.Olav Olsen is an independent structural engineering and marine technology consultant firm, who has a specialty in design of large floating structures. The team has been strengthened with selected subcontractors who are all highly qualified within their respective areas of expertise:

- Prodtex AS is a consultancy company specializing in the development of modern production and design processes. Prodtex sits on a highly qualified staff who have experience from design and operation of automated factories, where robots are used to handle materials and to carry out welding processes.
- Pure Logic AS is a consultancy firm specializing in cost- and uncertainty analyses for prediction of design effects to optimize large-scale constructs, ensuring optimal feedback for a multidisciplinary project team.
- Institute for Energy Technology (IFE) is an independent nonprofit foundation with 600 employees dedicated to research on energy technologies. IFE has been working on high-performance computing software based on the Finite-Element-Method for the industry, wind, wind loads and aero-elasticity for more than 40 years.
- Buksér og Berging AS (BB) provides turn-key solutions, quality vessels and maritime personnel for the marine operations market. BB is currently operating 30 vessels for harbour assistance, project work and offshore support from headquarter at Lysaker, Norway.
- Miko Marine AS is a Norwegian registered company, established in 1996. The company specializes in products and services for oil pollution prevention and in-water repair of ship and floating rigs, and is further offering marine operation services for transport, handling and installation of heavy construction elements in the marine environment.
- Heyerdahl Arkitekter AS has in the last 20 years been providing architect services to major national infrastructural projects, both for roads and rails. The company shares has been sold to Norconsult, and the companies will be merged by 2020.
- Haug og Blom-Bakke AS is a structural engineering consultancy firm, who has extensive experience in bridge design.
- FORCE Technology AS is engineering company supplying assistance within many fields, and has in this project phase provided services within corrosion protection by use of coating technology and inspection/maintenance/monitoring.
- Swerim is a newly founded Metals and Mining research institute. It originates from Swerea-KIMAB and Swerea-MEFOS and the metals research institute IM founded in 1921. Core competences are within Manufacturing of and with metals, including application technologies for infrastructure, vehicles / transport, and the manufacturing industry.

In order to strengthen our expertise further on risk and uncertainties management in execution of large construction projects Kåre Dybwad has been seconded to the team as a consultant.

1.4 Project scope

The objective of the current project phase is to develop 4 nominated floating bridge concepts, document all 4 concepts sufficiently for ranking, and recommend the best suited alternative. The characteristics of the 4 concepts are as follows:

- K11: End-anchored floating bridge. In previous phase named K7.

- K12: End-anchored floating bridge with mooring system for increase robustness and redundancy.
- K13: Straight side-anchored bridge with expansion joint. In previous phase named K8.
- K14: Side-anchored bridge without expansion joint.

In order to make the correct recommendation all available documentation from previous phases have been thoroughly examined. Design and construction premises as well as selection criteria have been carefully considered and discussed with the Client. This form basis for the documentation of work performed and the conclusions presented. Key tasks are:

- Global analyses including sensitivity studies and validation of results
- Prediction of aerodynamic loads
- Prediction of hydrodynamic loads
- Ship impact analyses, investigation of local and global effects
- Fatigue analyses
- Design of structural elements
- Marine geotechnical evaluations
- Steel fabrication
- Bridge assembly and installation
- Architectural design
- Risk assessment

2 LOADS

2.1 Static loads

2.1.1 Permanent loads

The permanent loads include the selfweight of the girder, buoyancy of the pontoons and the pretension loads of the cable stays.

These loads are balanced in order to minimize the bending moments of the bridge girder and tower.

The steel weight of the girder is summarized in [1]. In addition to the steel weight, permanent equipment such as railings and asphalt are included in the self-weight of the girder, as shown in Table 2-1.

> *Table 2-1 Additional permanent weight included in bridge girder self-weight*

Additional permanent weight	Unit load	Equivalent line load
Asphalt, driving lanes	2.0 kN/m^2	4000 kg/m
Asphalt, pedestrian lanes	1.5 kN/m^2	600 kg/m
Permanent equipment	-	500 kg/m
Total:		5100 kg/m

2.1.2 Traffic loads

The design traffic load is according to the Eurocode traffic load system (LM1) from Eurocode 1991-2:2003+NA:2010, N400 and *«Forskrift for trafikklast på bruer, ferjekaier og andre bærende konstruksjoner i det offentlige vegnettet»*, ref. [2]. A summary of the traffic models is shown in Table 2-2 and Table 2-2.

> *Table 2-2 LM1 traffic load specification*

Lane	Width	Distributed area load	Axle load	Horizontal traffic load	α-factors
Lane 1	3 m	α_{q1} * 9.0 kN/m^2	α_{Q1} * 2x300 kN	900 kN	α_{q1}=0.6, α_{Q1}=1.0
Lane 2	3 m	α_{q2} * 2.5 kN/m^2	α_{Q2} * 2x200 kN	0 kN	α_{q2}=1.0, α_{Q2}=1.0
Lane 3	3 m	α_{q3} * 2.5 kN/m^2	α_{Q3} * 2x100 kN	0 kN	α_{q3}=1.0, α_{Q3}=1.0
Pedestrian Lane	3 m	α_{fk} * 2.5 kN/m^2	0	0 kN	α_{fk}=1.0

Remaining area	14 m	α_{qr} * 2.5 kN/m^2	0	0 kN	α_{qr}=1.0
Total:	23 m + 3 m	73.7 kN/m	1200 kN	900 kN	

> *Table 2-3 Lane specification and positions from CL-road for LM1 traffic load*

Alignment	Lane	Left offset from CL-road	Right offset from CL-road
Centre aligned	Lane 1	-3 m	0 m
	Lane 2	-6 m	-3 m
	Lane 3	0 m	3 m
	Pedestrian Lane	11 m	14.5 m
	Remaining area	-13 m to -6 m	3 m to 10 m
Left aligned	Lane 1	-13 m	-10 m
	Lane 2	-10 m	-7 m
	Lane 3	-7 m	-4 m
	Pedestrian Lane	11 m	14.5 m
	Remaining area	-4 m	10 m
Right aligned	Lane 1	7 m	10 m
	Lane 2	4 m	7 m
	Lane 3	1 m	4 m
	Pedestrian Lane	11 m	14.5 m
	Remaining area	-13 m	1 m

2.1.3 Temperature loads

> *Table 2-4 Maximum and minim air temperature for given return periods.*

Return period	Minimum temperature °C	Maximum temperature °C
1 year	-7	25
10 year	-12	29
50 year	-15	32
100 year	-17	33

For 100-year return period, temperature giving zero strain in the structure is assumed to be +8°C, with temperature variations of +/-25°C applied to the whole model (except structures below water surface).

2.1.4 Current loads

The extreme values of hourly sea currents (m/s) for four different locations at the planned bridge crossing is described in design basis along with relative factors for sectoral extreme speeds (every 45°) for return periods of 10, 50 and 100 years. Linear interpolation is used between the locations to calculate the current speeds at the pontoon locations (and assumed constant from end locations towards north and south shore).

The current forces acting on the pontoons are calculated according to the formulae given below:

$$F_{current} = \frac{1}{2} * \rho_{water} * U(x)^2 * \sum(C_d * A_p)$$

Where

ρ_{water} is the density of water, equal to 1025 kg/m^3

$U(x)$ is the current speed at pontoon X

$\sum(C_d * A_p)$ is the total projected drag area of the pontoon, which depends on the pontoon size, shape and orientation (the angle to the current)

The applied current loads applied to the pontoon nodes, for sector W for a 100-year return period, are shown in the below figure.

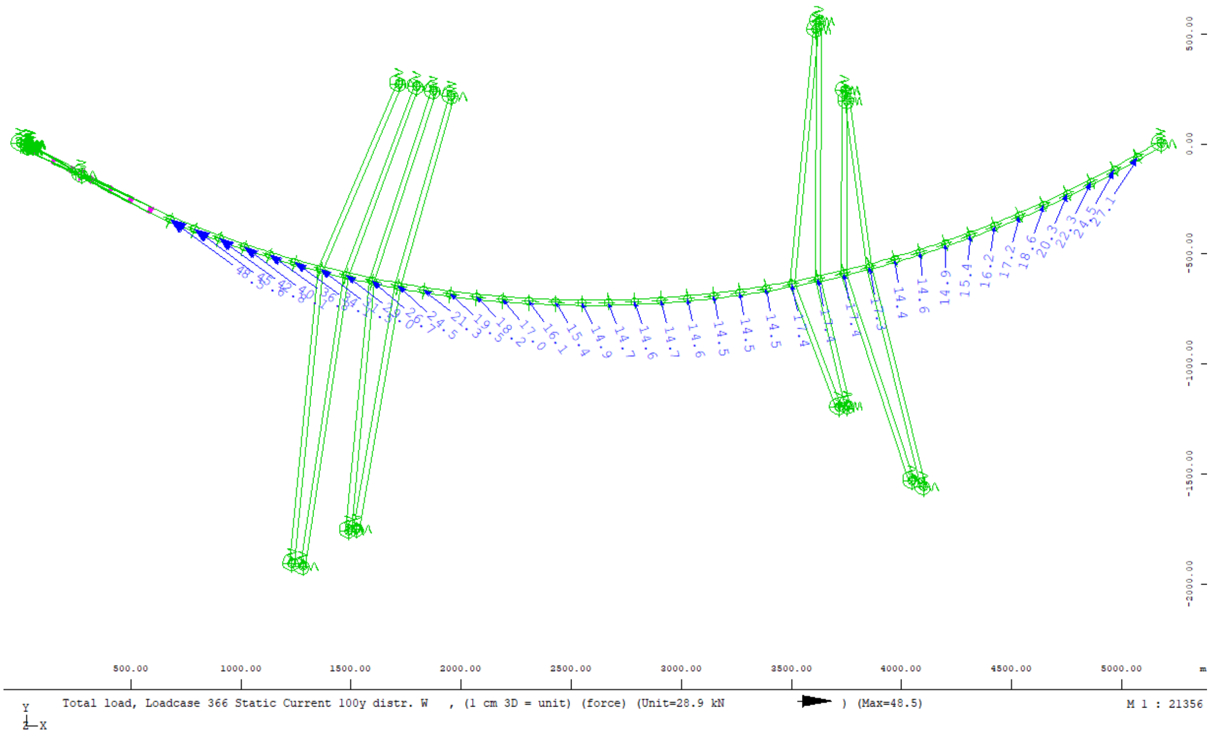


Figure 2-1 Static current load 100-year, sector W (in kN).

2.1.5 Static wind

Static wind cases mentioned here are not applied together with coupled analyses as the coupled analyses already have static wind contributions, but are used in some sensitivity studies and included here for completeness.

The design basis along with N400 and NS-EN 1991-1-4:2005+NA provide input for the wind loading.

The formulae for calculating the wind profile for 1h mean is available in design basis, and gives the following mean wind speeds at height 10 meters:

> *Table 2-5 1h and 10min mean wind speeds for given return periods at z=10m*

Return period (years)	Wind speed 1h mean (m/s)	Wind speed 10min mean (m/s)
1	21.4	22.9
10	25.8	27.6
50	28.5	30.5
100	29.6	31.7

10000	35.9	38.4

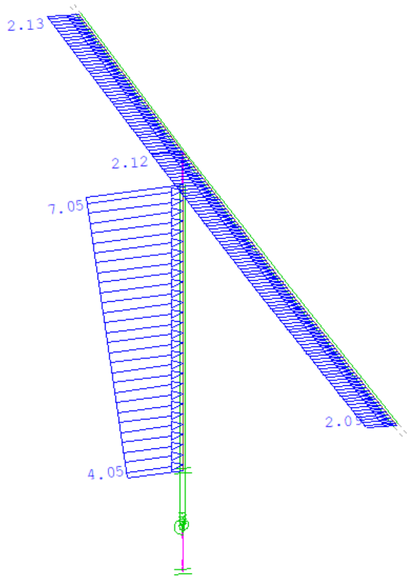
Reduction factors applies for sectors and are used to find the sectoral extremes. The factors are applied to the wind speeds. The directional reduction factors are found in design basis.

The mean wind can for strong winds be assumed to have the following distributions along the bridge axis (note that V means mean wind speed):

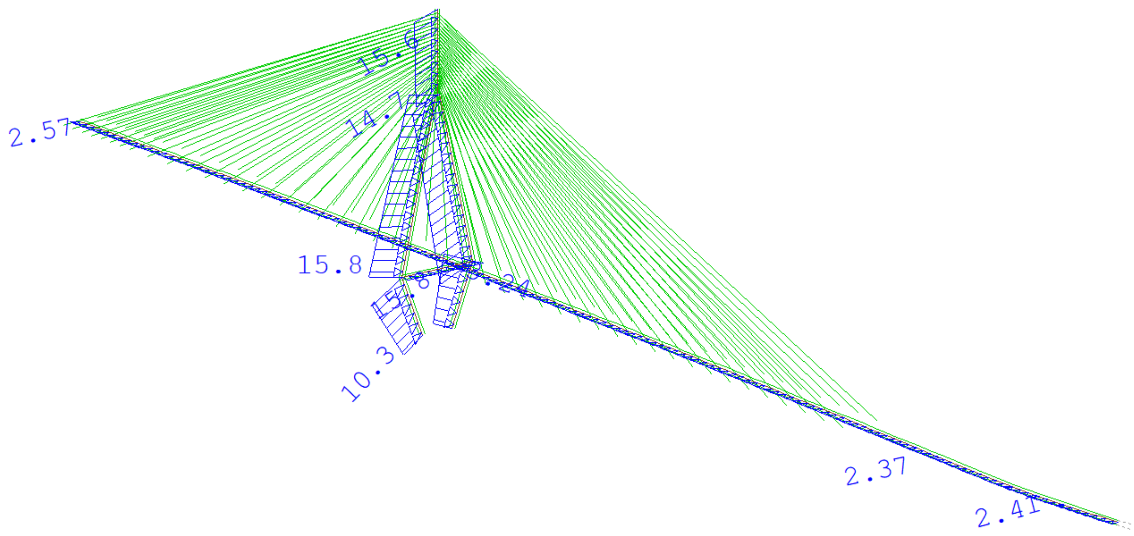
1) Constant
2) Linearly varying from 0.6 x V at one end to V on the other
3) Linearly varying from 0.8 x V at one end to V in the middle to 0.8 x V on the other end.

The wind load is applied as linearly varying line loads applied to each beam in the global element model, calculated from local wind speed where both location along the bridge span, height above the sea level, drag area and rotation of the element relative to the wind direction and wind speed distribution is accounted for.

Typical wind load application is shown in the figures below for concept K11:

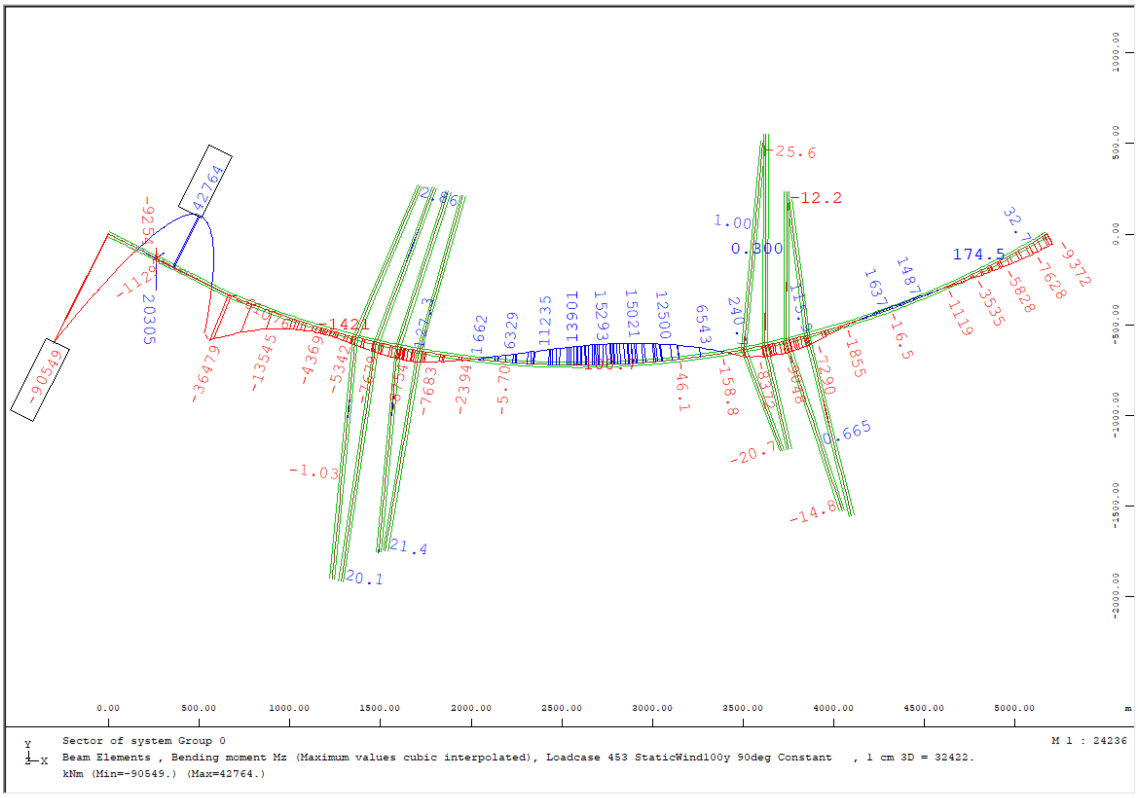


> *Figure 2-2 Typical wind load applied to bridge girder and pontoon tower (in this particular case concept K11, 100y constant wind sector 90 deg / wind from east, pontoon tower no. 8 from south)*

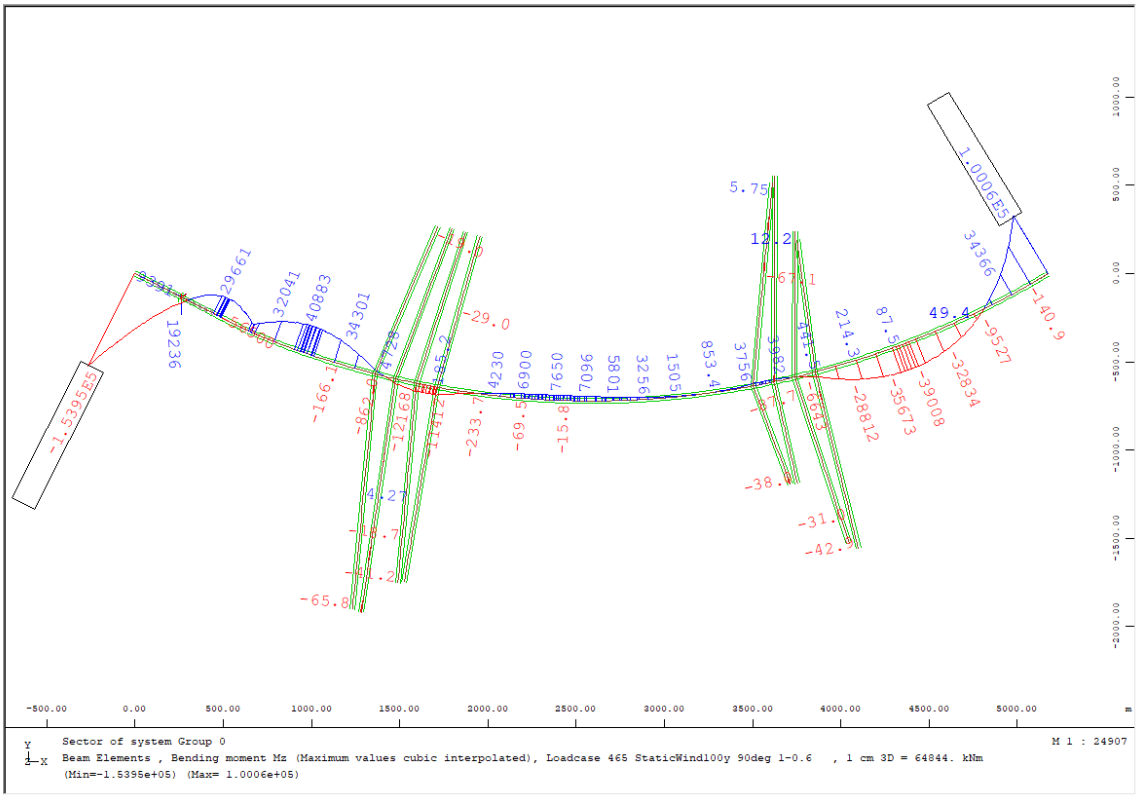


> *Figure 2-3 Typical wind load applied to bridge girder and land tower (in this particular case concept K11, 100y constant wind sector 90 deg / wind from east)*

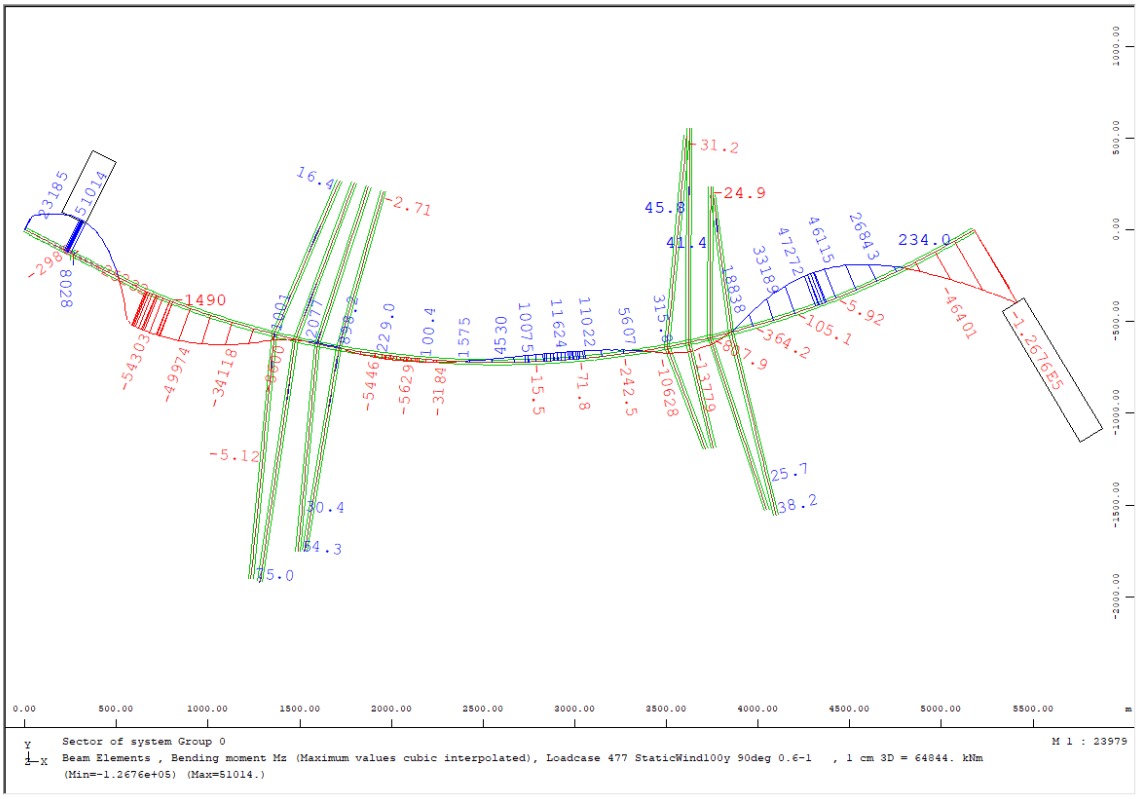
Typical wind distribution response (bridge strong axis moment, Mz):



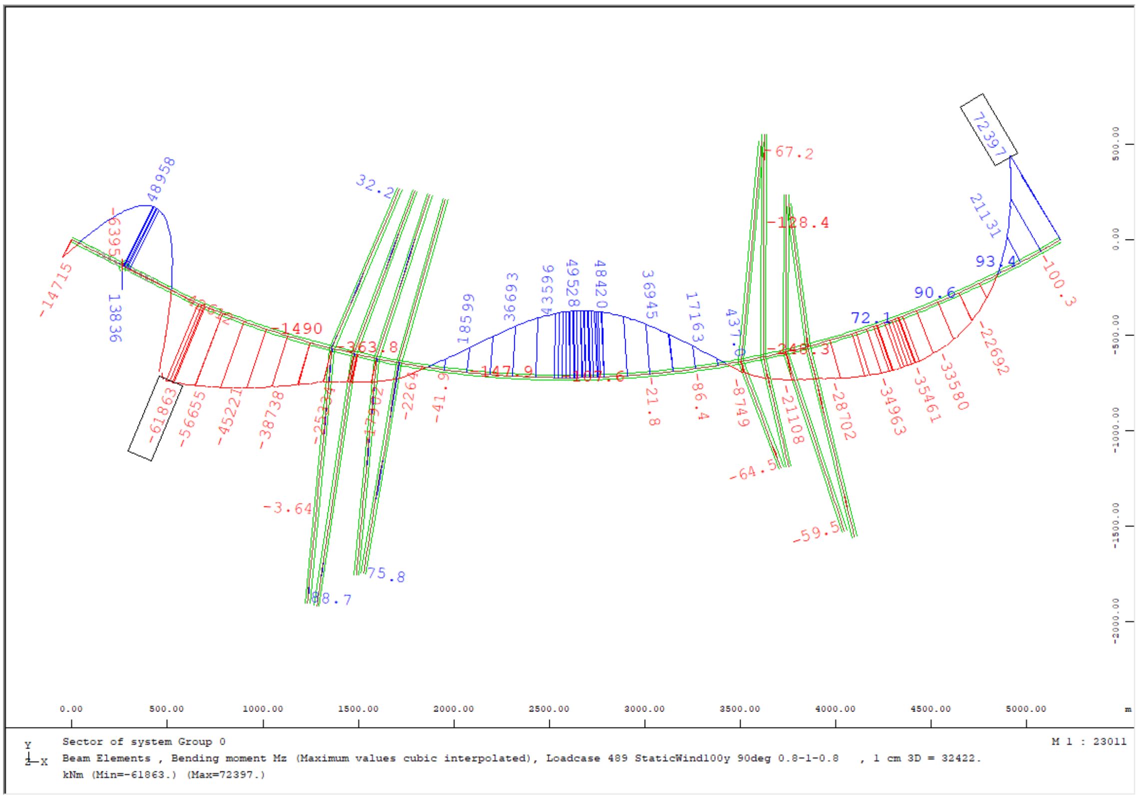
> *Figure 2-4 Typical wind response for constant wind speed along bridge, for wind from directly east*



> *Figure 2-5 Typical wind response for linearly varying wind speed from 100% at south, to 60% at north end of bridge, for wind from directly east*



> *Figure 2-6 Typical wind response for linearly varying wind speed from 60% at south, to 100% at north end of bridge, for wind from directly east*



> *Figure 2-7 Typical wind response for linearly varying wind speed from 80% at south, to 100% at center of bridge, to 80% at north end of bridge, for wind from directly east*

2.1.6 Tidal loads

> *Table 2-6 Tidal amplitudes*

Tidal amplitudes	Value	m
Lowest Astronomical Tide	0	m
Mean Low Water	0.39	m
Mean Sea Level	0.77	m
Mean high water	1.15	m
Highest Astronomical Tide	1.53	m

> *Table 2-7 Water level related to return periods relative to LAT.*

Return periods (years)	Highest water level (m)	Lowest water level (m)	Storm surge (m)
1	1.81	-0.20	+/- 0.235
10	1.97	-0.30	+/- 0.37
100	2.10	-0.50	+/- 0.535
10000	2.50	-0.65	+/- 0.81

Calculation of storm surge (100y) and combined factor:

Total tide	= (2.1-(-0.5))/2	= 1.3 m
Astronomical tide	= 1.53-0.765	= 0.765 m
Storm surge tide	= 1.3-0.765	= 0.535 m

According to design basis [3] the astronomical component is independent of the environmental conditions, whereas the surge component is governed by the atmospheric conditions. Since the confidence on the two components vary, that is, astronomical tide values are lot more predictable than the surge components are, appropriate safety factors can be applied separately on each component during further design.

A load factor for astronomical tide is not explicitly given in this design basis. However, in design basis [2] for phase 3 of the Bjørnafjorden it was suggested that it should have a load factor of 1.1. Thus, this has been applied in this phase as well.

Load factor astronomical tide = 1.1
Load factor storm surge part of tide = 1.6

Combined load factor 100 year conditions = 1.6*0.535 /1.3+1.1*0.765/1.3= 1.31

Tidal variations are applied as loads and applied to each pontoon as follow:

$F_{pontoon,i,100y} = +/- \rho_{water} * g * A_{pontoon,i,WL} * \Delta_{Tidal,100y}$

Tidal variations are applied as loads on each pontoon and summarized in table Table 2-8 for each pontoon type.

> *Table 2-8 Tidal loads, example from K12.*

Pontoon type	Area of pontoon at WL (m^2)	Tidal load (kN)
1	559	+/- 7301
2	665	+/- 8694
3	770	+/- 10064
4	873	+/- 11413

2.1.7 Marine fouling

Marine fouling is calculated in accordance with ref. [4]. The forces applied to the static model at the pontoon is summarized in Table 2-9 for each of the pontoon types.

> *Table 2-9 Pontoon loads from marine fouling. example from K12.*

Pontoon type	Area subjected to marine fouling	Submerged weight	Permanent loading from marine fouling
Pontoon 1	1259 m^2	468 N/m^2	589 kN
Pontoon 2	1378 m^2	468 N/m^2	645 kN
Pontoon 3	1496 m^2	468 N/m^2	700 kN
Pontoon 4	1611 m^2	468 N/m^2	754 kN

2.1.8 Water density

The effect of variation in water density is deemed negligible.

2.2 Coupled loads

The coupled loads concerns static wind loads, turbulent wind loads and wave loads can be found in Appendix-C K12 Coupled analysis

3 LOAD COMBINATIONS

3.1 Summary

In Table 3-1 below, the limit states and design criteria checks applied in this study are presented. The different load combination numbers define different combination rules, each with different load factors for different load contributions. For a more thorough description of the limit states and design criteria, see Chapter 3.2 through Chapter 3.7 and the design basis [3].

> *Table 3-1 Summary of combinations*

Combination number	Combination type	Description
21	SLS load combinations	- Traffic load dominating
22	SLS load combinations	- 1 year conditions with traffic
23	SLS load combinations	- 100 year conditions without traffic
24	SLS load combinations	- Infrequent
25	SLS load combinations	- Quasi-permanent
31	ULS load combinations	ULS 6.10a - 1 year conditions with traffic
32	ULS load combinations	ULS 6.10a - 100 year conditions without traffic
33	ULS load combinations	ULS 6.10b - 1 year conditions with traffic
34	ULS load combinations	ULS 6.10b - 100 year conditions without traffic
35	ULS load combination	DNV Mooring consequence class 1
36	ULS load combination	DNV Mooring consequence class 2
41	Static motion limitation	0.7x traffic
42	Static motion limitation	1-year static wind

3.2 Ultimate limit state

According to Design Basis [3], chapter 7.3.2, the bridge characteristic response in ULS shall be defined based on an environmental event with return period of 100 years, in which the bridge is assumed closed for traffic. Additionally, characteristic response from environmental and traffic loading shall be evaluated with an environmental event with a return period of 1 year. The events are evaluated with respect to load combination principles of EQU and STR. Load combination factors applied are according to Design Basis and presented in Table 3-2 to Table 3-7.

3.2.1 Ultimate limit state – EQU

In load combination set 31, the worst load combination according to Table 3-2 is combined with 1-year environmental loads and factor according to Table 3-3.

> *Table 3-2: ULS-EQU load combination factors COMB 31, $\gamma \times \psi_0$*

Load		Dominating load			
		G	**L**	**T**	**C**
Self-weight	G	**1.0/0.9**	1.0/0.9	1.0/0.9	1.0/0.9
Traffic	L	0.95	**1.35**	0.95	0.95
Temperature	T	0.84	0.84	**1.2**	0.84
Other loads	C	1.05	1.05	1.05	**1.5**

> *Table 3-3: ULS-EQU load combination factors COMB 31 environmental loads, $\gamma \times \psi_0$*

Load		E
Environmental, 1 year, with traffic	E	**1.6**

> *Table 3-4: ULS-EQU load combination factors COMB 32, $\gamma \times \psi_0$*

Load		Dominating load
		E
Self-weight	G	1.0/0.9
Temperature	T	0.84
Environmental, 100 year, without traffic	E	**1.6**
Other loads	C	1.05

3.2.2 Ultimate limit state – STR

In load combination set 33, the worst load combination according to Table 3-5 is combined with 1-year environmental loads and factor according to Table 3-3.

> *Table 3-5: ULS-STR load combination factors COMB 33, γ x ψ_0*

Load		Dominating load			
		G	L	T	C
Self-weight	G	**1.35/1.0**	1.2/1.0	1.2/1.0	1.2/1.0
Traffic	L	0.95	**1.35**	0.95	0.95
Temperature	T	0.84	0.84	**1.2**	0.84
Other loads	C	1.05	1.05	1.05	**1.5**

> *Table 3-6: ULS-STR load combination factors COMB 33 environmental loads, γ x ψ_0*

Load		E
Environmental, 1 year, with traffic	E	**1.6**

> *Table 3-7: ULS-STR load combination factors COMB 34, γ x ψ_0*

Load		Dominating load
		E
Self-weight	G	1.2/1.0
Temperature	T	0.84
Environmental, 100 year, without traffic	E	**1.6**
Other loads	C	1.05

3.2.3 Ultimate limit state – GEO

ULS load factors for geotechnical design are given by DNVGL OS C101 [5] Chapter 2 Section 10 and presented in Table 3-8.

> *Table 3-8: ULS-GEO load coefficients for consequence class 1 (COMB 35) and 2 (COMB 36)*

Load factor	ULS CC1	ULS CC2
γ_{mean}	1.1	1.4
γ_{dyn}	1.5	2.1

3.2.4 Ultimate limit state – FAT

ULS-FAT is evaluated according to the procedure established by DNVGL as described in [3], Chapter 7.3.2.

3.3 Accidental limit state

The accidental limit stat load combinations are described in [3], with load factors as shown in Table 3-9. The limit states described comprises two stages; the purpose of stage A is to control the magnitude of local damage for the bridge subjected to an accidental load, and stage B is to check the behavior of the bridge in a damaged condition. Load combination factors are presented in Table 3-9 and Table 3-10 and taken from Design Basis [3].

3.3.1 ALS Standard

> *Table 3-9: ALS load combination factors, ψ_2, stage A*

Load combinations	Stage A			
	Earthquake	Abnormal environmental loads	Fire and explosion	Ship impact
Self-weight	1.0	1.0	1.0	1.0
Traffic	0.5	0	0.5	0.5
Accidental loads				
Earthquake	1.0	0	0	0
Environmental loads, 10.000 year	0	1.0	0	0
Ship impact	0	0	0	1.0
Fire and explosion	0	0	1.0	0

> *Table 3-10: ALS load combination factors, ψ_2, stage B*

Load combinations	Stage B (damaged condition)		
	Pontoon filled with water	Lost mooring cable	Lost cable stay
Self-weight	1.0	1.0	1.0
Environmental loads, 100 year	1.0	1.0	1.0
Accidental loads			
Pontoon filled with water	1.0	0	0
Lost mooring cable	0	1.0	0
Lost stay cable	0	0	1.0

3.3.2 ALS – GEO

ALS load factors for geotechnical design are given by DNVGL OS C101 [5] Chapter 2 Section 10 and presented in Table 3-11.

> *Table 3-11: ALS-GEO load coefficients for consequence class 1 and 2*

Load factor	ALS CC1	ALS CC2
γ_{mean}	1	1.1
γ_{dyn}	1	1.25

3.4 Serviceability limit state

3.4.1 SLS characteristic

According to design basis chapter 7.3.1 [3] the characteristic SLS should be used to determine bearing displacements. Table 3-12 to Table 3-14 shows combinations and combination factors applied, based on Design Basis [3].

> *Table 3-12 Combination factors SLS [3] COMB21, ψ_0*

Load		Dominating load			
		G	L	T	C
Self-weight	G	**1.0**	1.0	1.0	1.0
Traffic	L	0.7	**1.0**	0.7	0.95
Temperature	T	0.7	0.7	**1.0**	0.84
Environmental, 1 year, with traffic	E_{1yr}	0.7	0.7	0.7	1.12
Other loads	C	0.7	0.7	0.7	**1.0**

> *Table 3-13 Combination factors SLS [9] COMB22, ψ_0*

Load		Dominating load
		E_{1yr}
Self-weight	G	1.0
Traffic	L	0.7
Temperature	T	0.7

Environmental, 1 year, with traffic	E_{1yr}	**1.0**
Other loads	C	0.7

> *Table 3-14 Combination factors SLS [9] COMB23, ψ_0*

Load		Dominating load
		E_{100yr}
Self-weight	G	1.0
Temperature	T	0.7
Environmental, 100 year, without traffic	E_{100yr}	**1.0**
Other loads	C	0.7

Characteristic SLS combinations considered are also presented in chapter 3.1.

3.4.2 SLS in-frequent

The in-frequent load combination shall be used for evaluation of minimum vertical navigation clearance, as stated in Design Basis [3], Chapter 7.3.1.

> *Table 3-15 Load combinations and factors in-frequent SLS [3], ψ_0*

Load		Dominating load			
		L	T	E_{50yr}	C
Self-weight	G	1.0	1.0	1.0	1.0
Traffic	L	**0.8**	0.7	0.7	0.7
Temperature	T	0.6	**0.8**	0.6	0.6
Environmental, 50 year, with traffic	E_{50yr}	0.6	0.6	**0.8**	0.6
Other loads	C	0.6	0.6	0.6	**0.8**

Table 3-15 shows an example of combinations to be evaluated, as presented in Design Basis.

3.4.3 SLS quasi-permanent

Combination factors for the quasi-permanent SLS condition are given in design basis [3] and presented in Table 3-16, with ψ_2 = 0 for variable loads.

> *Table 3-16: Combination factors for quasi-permanent SLS condition*

Load		Dominating load			
		L	T	E_{50yr}	C
Self-weight	G	1.0	1.0	1.0	1.0
Traffic	L	**0.2/0.5**	0.2/0.5	0.2/0.5	0.2/0.5
Temperature	T	0/0.5	**0/0.5**	0/0.5	0/0.5
Environmental, 50 year, with traffic	E_{50yr}	0/0.5	0/0.5	**0/0.5**	0/0.5
Other loads	C	0/0.5	0/0.5	0/0.5	**0/0.5**

3.5 Fatigue limit state

See SBJ-33-C5-OON-22-RE-016 Fatigue Assessment [6].

3.6 Static deflection, motion and comfort criteria

Bridge acceleration limitations shall be established based on driver comfort and are described in Design Basis Chapter 9.3 [3].

In Design Basis chapter 9.2 static deflection and motion criteria for the bridge is described, see Table 3-17 below.

> *Table 3-17 Static deflection and motion criteria taken from Design Basis [3]*

Motion limitation	Load scenario	Maximum motion
Vertical deformation from traffic loads	0.7xtraffic	$u_y \leq 1.5m$
Rotation about bridge axis from eccentric traffic loading	0.7xtraffic	$\theta_x \leq 1.0$ deg
Rotation about bridge axis from static wind load	1-year static wind	$\theta_x \leq 0.5$ deg

3.7 Freeboard/Stability criteria

In design basis chapter 9.1 the freeboard and stability criteria that should be maintained are described.

1. For structural parts that do not follow the tide, the freeboard should be maintained for the highest water level for a tide with a 100 year return period.
2. The stability of the bridge shall be evaluated with respect to ULS-EQU. For the 1-year condition, the change of mass and aerodynamic coefficients of the bridge girder due to presence of traffic shall be accounted for.

Also, according to N400 freeboard should be maintained during environmental conditions with a return period of 100 years.

Sensitivity studies of the robustness of the structure when freeboard is temporarily lost shall be conducted.

4 STRUCTURAL CAPACITY REQUIREMENTS

This chapter concerns the structural capacity of the bridge girder as well as global capacity requirement with regards to displacement, acceleration, deflections and stability.

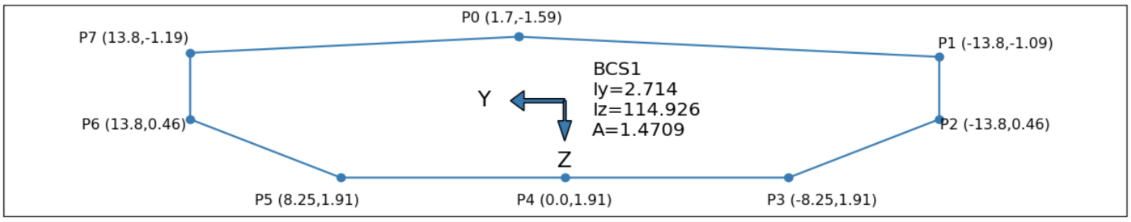
4.1 ULS stress response

The ULS response can be found in Appendix E – Characteristic and limit state response and on the webpage olavolsen.interactive.no [7] for K12 – Model 30.

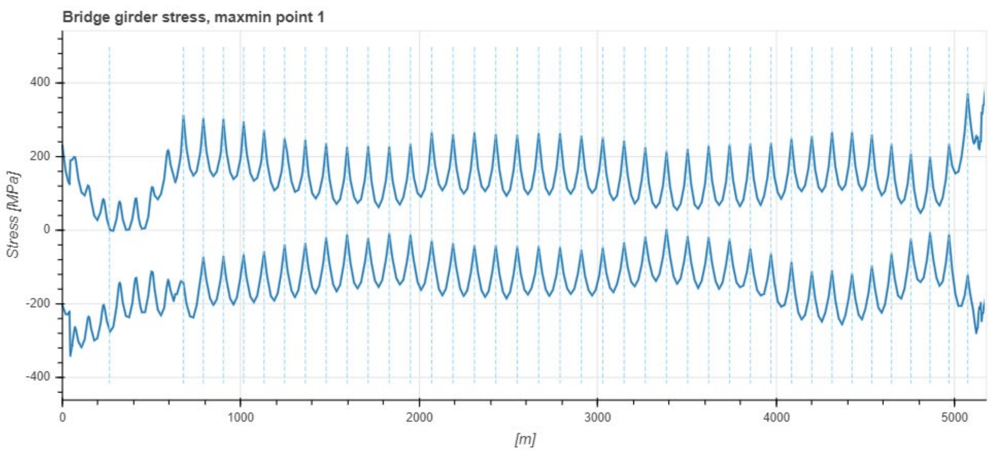
The most utilized stress points in the top deck (P1 and P7) are governed by the 100 year environmental condition with no traffic included, see *ULS 6.10b – 100 year conditions without traffic in Chapter* 3.2.2

The most utilized stress point in the bottom plate (P3 and P5) are governed by the dominant traffic condition, see *ULS 6.10b – 1 year environmental with traffic in Chapter* 3.2.2

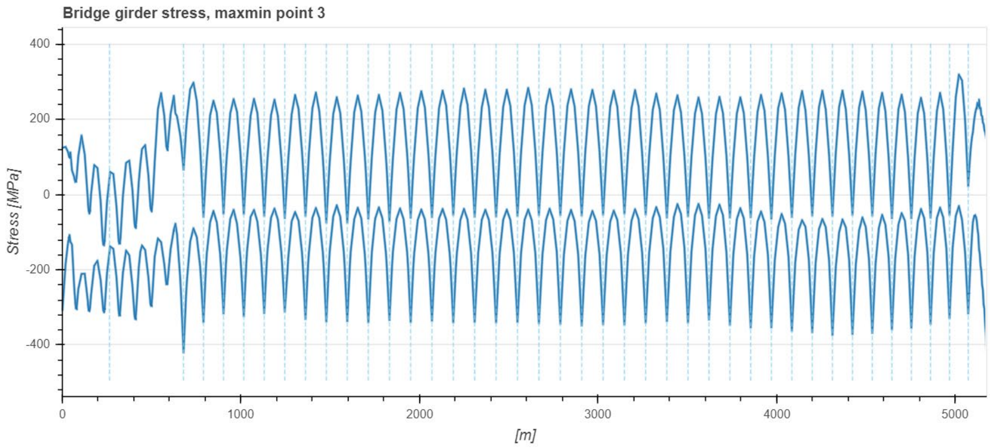
The normal stress for the 4 most utilized stress points are seen below. Where they are placed in the cross section can be seen in Figure 4-1.



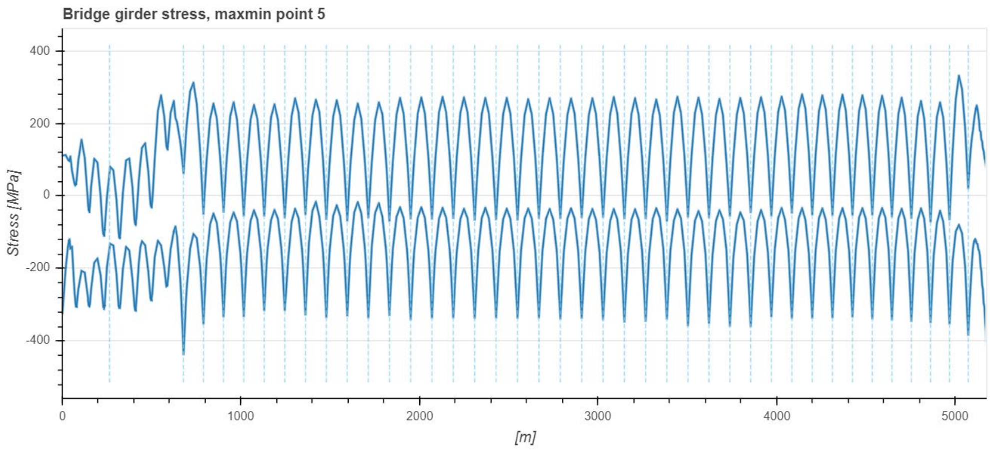
> *Figure 4-1 Stress points*



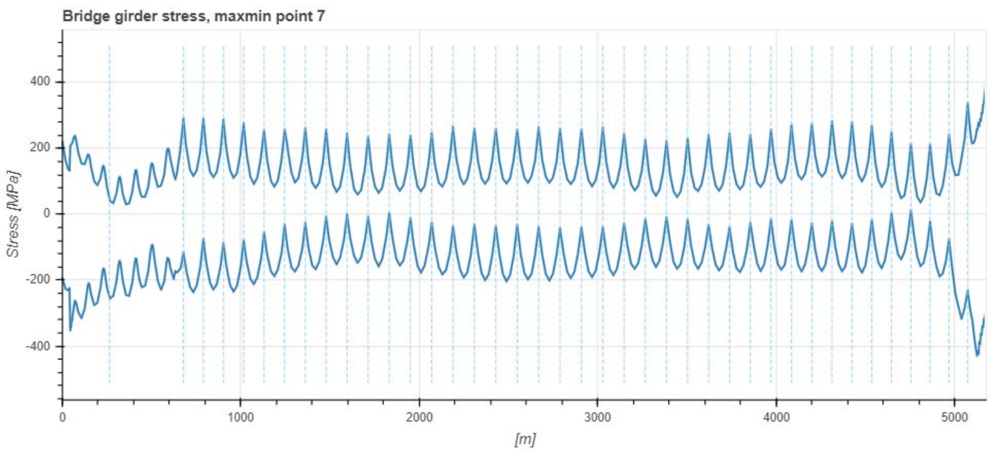
> *Figure 4-2 Stress point P1 - Combination 34 in Chapter 3.2.2 (ULS 6.10b – 100 year conditions without traffic)*



> *Figure 4-3 Stress point P3 - Combination 33 in Chapter 3.2.2 (ULS 6.10b – 1 year conditions with traffic)*



> *Figure 4-4 Stress point P5 - Combination 33 in Chapter 3.2.2 (ULS 6.10b – 1 year conditions with traffic)*



> *Figure 4-5 Stress point P7 - Combination 34 in Chapter 3.2.2 (ULS 6.10b – 100 year conditions without traffic)*

4.2 ALS response

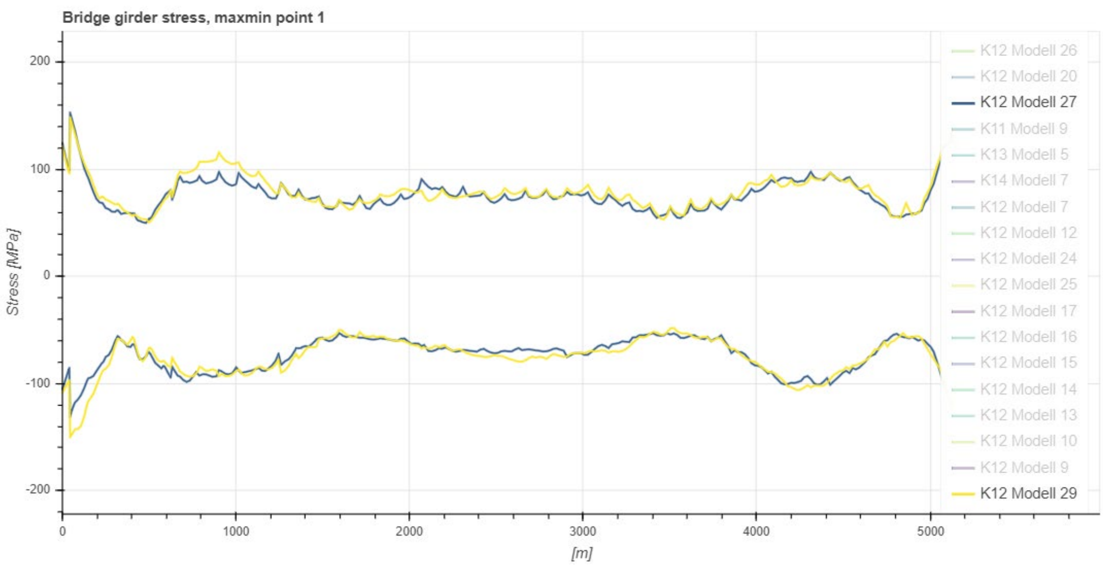
4.2.1 Extreme environmental response – RP=10000y

Global analyses for environmental conditions with a return period of 10000 years shows that the response about weak axis and typical weak axis stress points (P3 and P5) supersedes the load factor of 1.6 (closer to 1.7) compared to the response from environmental conditions with a return period of 100 years. With regards to strong axis and typical strong axis stress points (P1 and P7) the response does not supersede the 1.6 factor (closer to 1.5). Thus, both the 100 year and the 10000 year condition should be considered in design in the next phase.

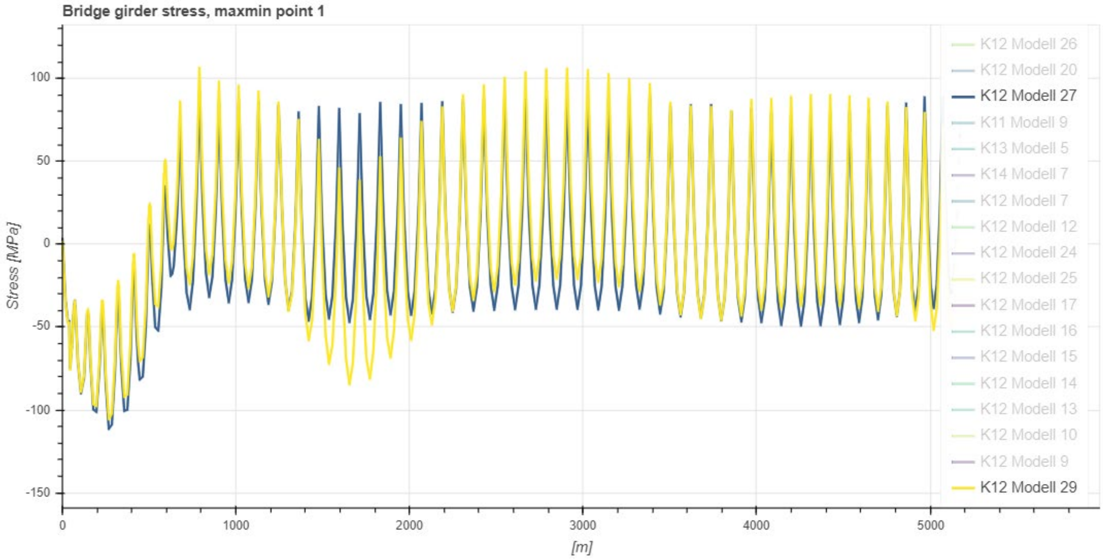
For further details, see SBJ-32-C5-OON-22-RE-005- App R K12 evaluation of 10000 RP conditions [8]

4.2.2 Loss of two anchor lines on same side of anchor group.

As one can see from Figure 4-6 and Figure 4-7 below, the loss of anchor has close to no impact on the environmental response, but makes some difference on the permanent load condition. However, the sum of the effects is small compared to the change in load factor between the two limit states. Thus, the anchor line loss situation is not governing with regards to the design of bridge girder.



> *Figure 4-6 Comparison of stress response in point P1 – Anchor line loss (model 29) vs Anchor intact (model 27)- Environmental response*



> *Figure 4-7 Comparison of stress response in point P1 – Anchor line loss (model 29) vs Anchor intact (model 27)- Environmental response*

The local effects with regards to this accidental state are handled in SBJ-33-C5-OON-22-RE-021-B- K12 - Design of mooring and anchoring [9].

4.2.3 Loss of stay-cable

The loss of a single stay-cable does not have a significant effect on the global bridge response during service after the loss, as long as the effect is treated right in the local design of the stay-cables. However, the transient effect right after the loss may give increased loads to the neighboring cables. This ALS situation is handled in SBJ-33-C5-OON-22-RE-019 Design of cable stayed bridge [10].

4.2.4 Damaged pontoon (from ship impact)

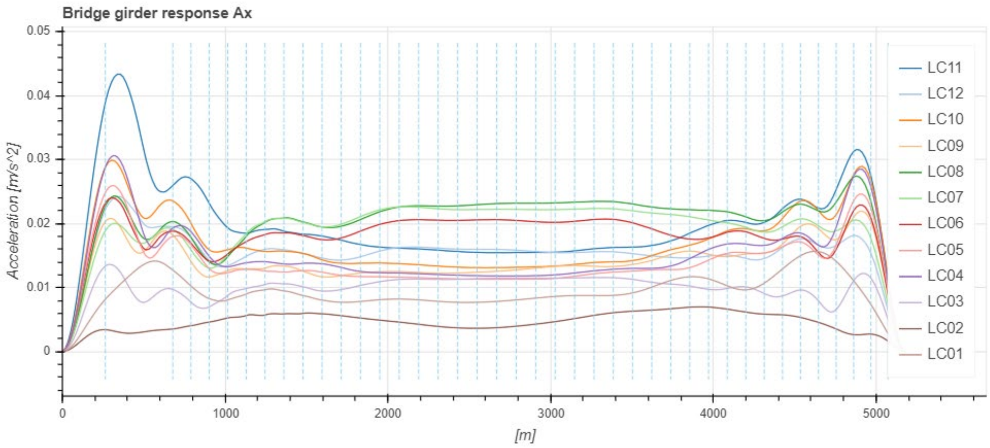
The effect of damaged pontoon is handled in SBJ-33-C5-OON-22-RE-013-015 Ship impact [11].

4.3 Accelerations, deflections and displacements

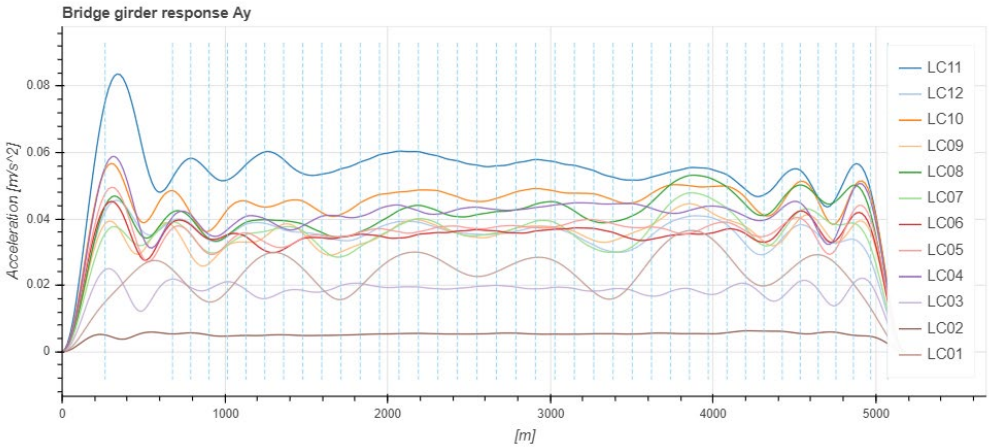
4.3.1 Accelerations of the bridge girder

The accelerations of the bridge girder for environmental conditions with a return period of 1 year and environmental conditions with a return period of 100 year are presented below, respectively (standard deviations).

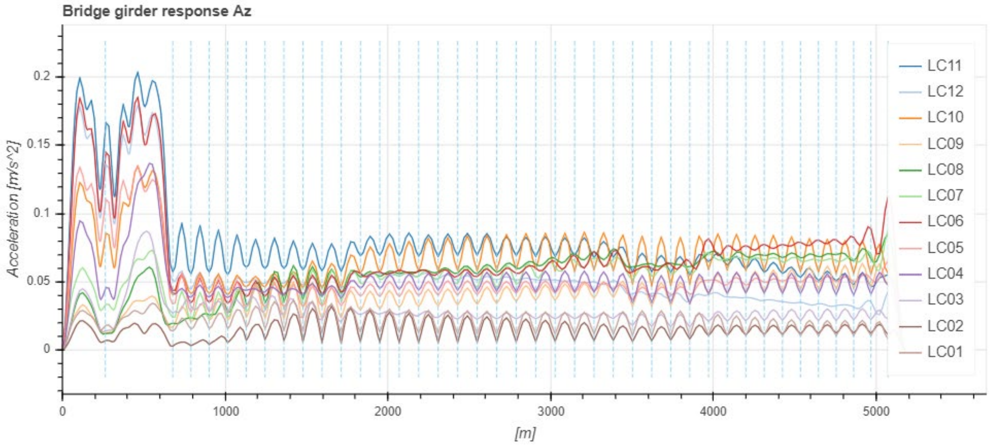
4.3.1.1 Environmental conditions with a return periode of 1 year



> *Figure 4-8 Accelerations in global X-direction (standard deviation)*

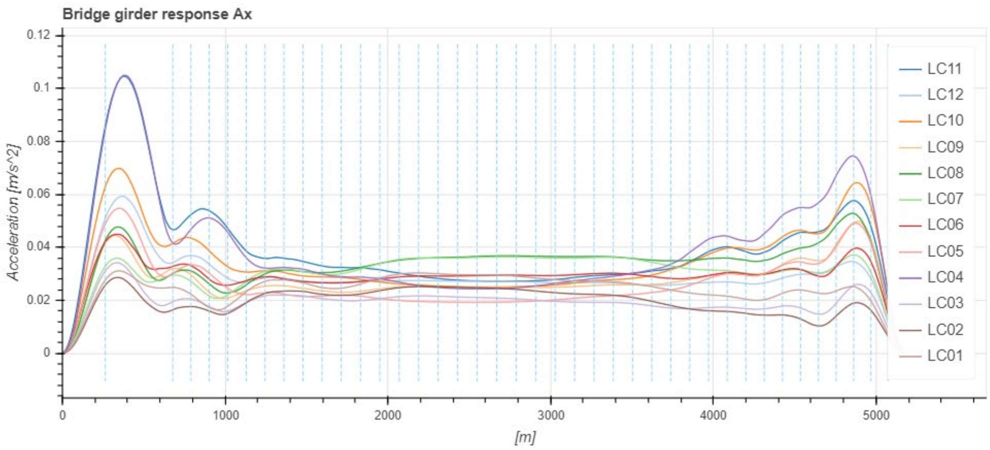


> *Figure 4-9 Accelerations in global Y-direction (standard deviation)*

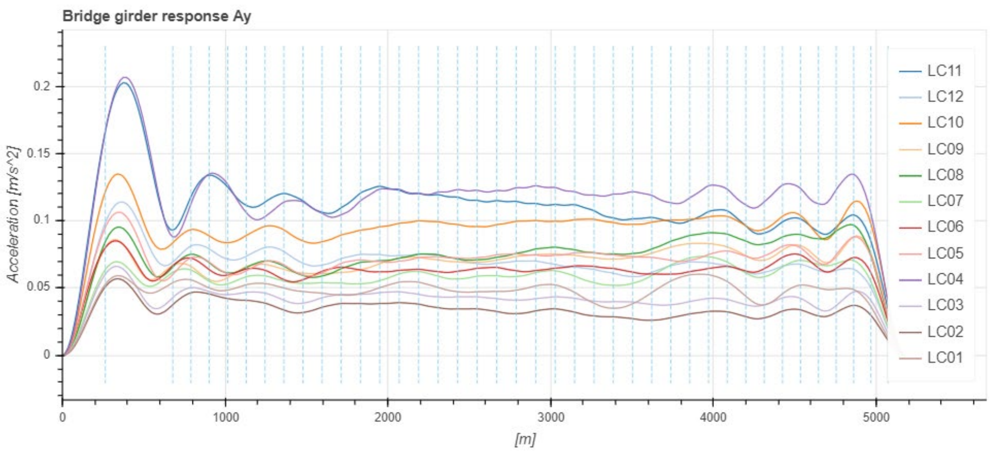


> *Figure 4-10 Accelerations in global Z-direction (standard deviation)*

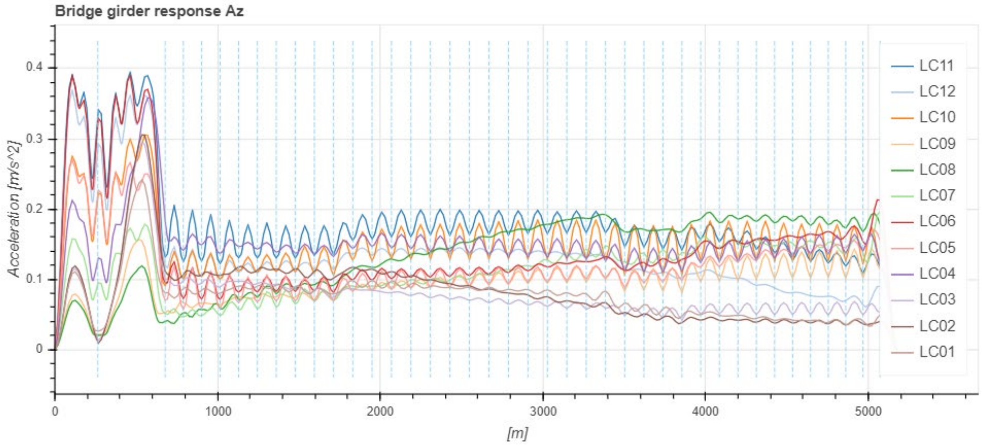
4.3.1.2 Environmental conditions with a return periode of 100 year



> *Figure 4-11 Accelerations in global X-direction (standard deviation)*



> *Figure 4-12 Accelerations in global Y-direction (standard deviation)*



> *Figure 4-13 Accelerations in global Z-direction (standard deviation)*

4.3.1.3 Summary

> *Table 4-1 Standard deviations of accelerations for RP=1y*

Accelerations	Global X-direcetion 1 year [m/s²]	Global Y-direcetion 1 year[m/s²]	Global Z-direcetion 1 year[m/s²]
Cable stayed bridge	0.044	0.081	0.2
Floating bridge	0.031	0.06	0.08

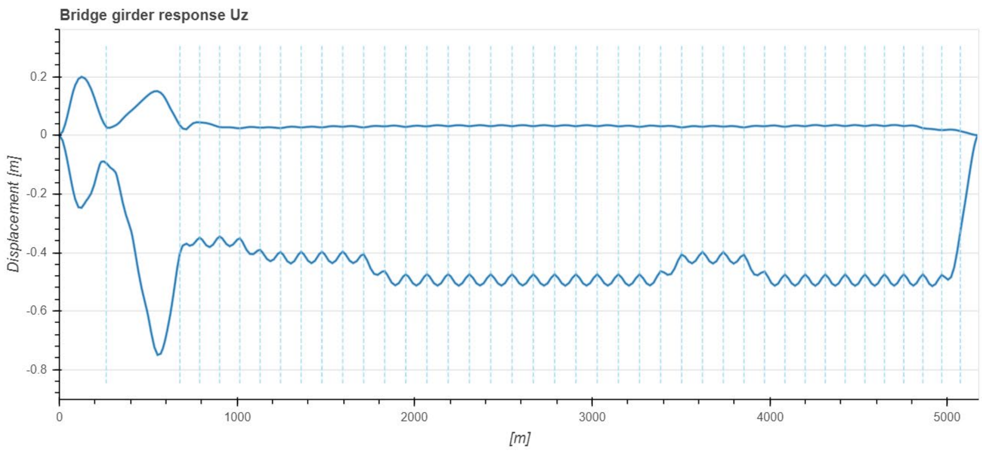
> *Table 4-2 Standard deviations of accelerations for RP=100y*

Accelerations	Global X-direcetion 1 year [m/s²]	Global Y-direcetion 1 year[m/s²]	Global Z-direcetion 1 year[m/s²]
Cable stayed bridge	0.105	0.205	0.4
Floating bridge	0.07	0.13	0.2

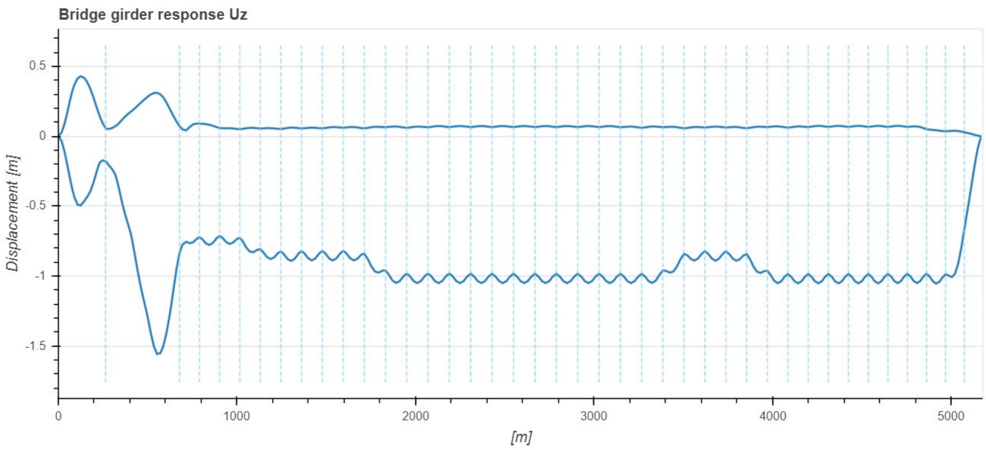
>

4.3.2 Vertical deformation from 0.7 x traffic loads

Figure 4-14 and Figure 4-15 shows the deformation from two different sets of reduction factors, one for traffic with a larger influence length (>1000m) , while the other is for influence lengths below 200m.



> *Figure 4-14 Vertical displacement- with reduction factors corresponding to an influence length >1000 m*



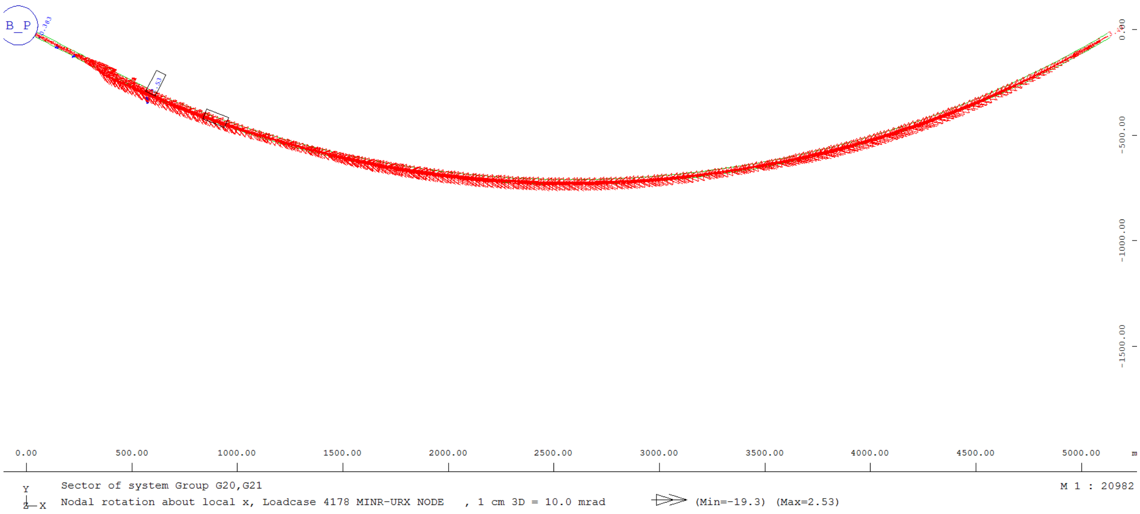
> *Figure 4-15 Vertical displacement – with no reduction factors (corresponding to influence length <200m), not valid for the cable stayed bridge.*

The vertical deformation for the cable stayed bridge is somewhere in between the two presented cases below. It is difficult to say exactly what it is but most likely it is below the deformation requirement of 1.5m.

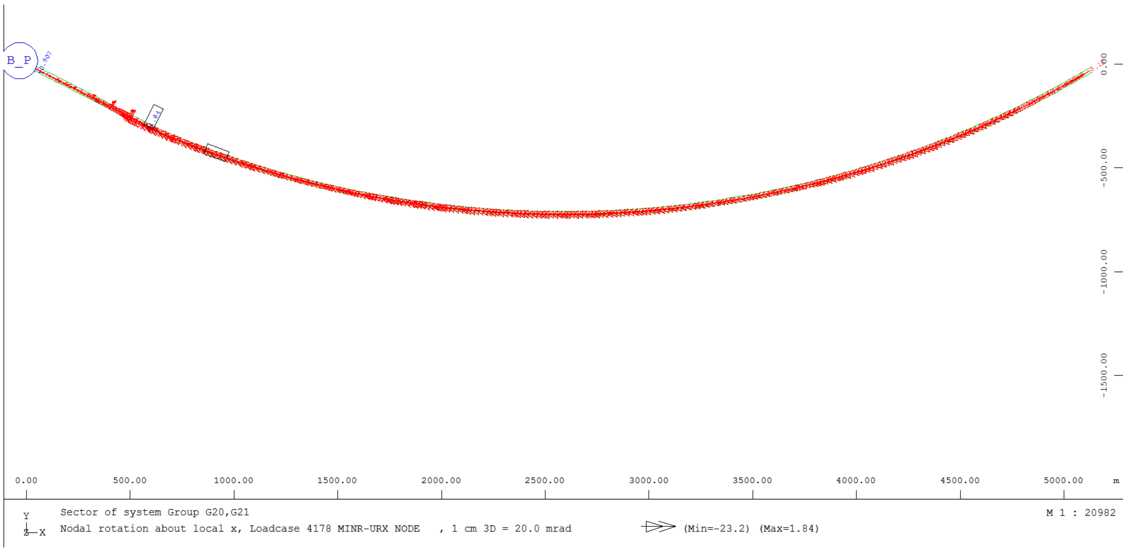
The floating bridge has influence lengths below 200m in vertical direction but the deformation is still within the requirement.

4.3.3 Rotation about bridge axis from eccentric traffic loading

The maximum rotation was extracted from local coordinate system (see Figure 4-16) for a the load situation 0.7 x ULS traffic load.



> *Figure 4-16 Rotation about local X-axis (AbsMax=19.2mrad) - Traffic x 0.7 – Factors corresponding to 1000m influence length*



> *Figure 4-17 Rotation about local X-axis (AbsMax=23.2mrad) - Traffic x 0.7 – Factors corresponding to 200m influence length*

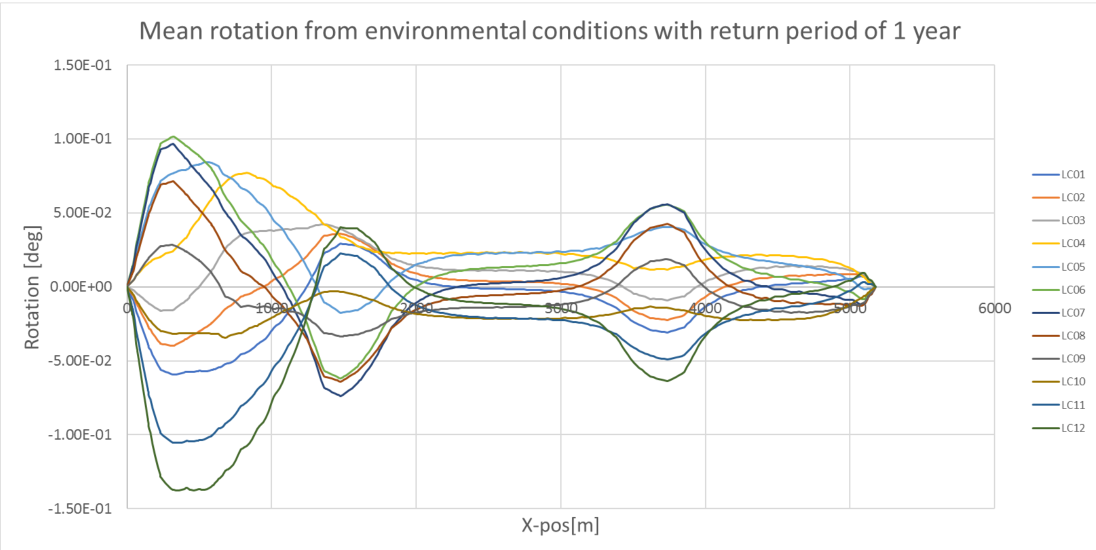
Maksimum rotation about bridge axis = (23mrad/1000)*(180/pi) = 1.3 deg

Most likely the torsional rotation should be controlled by influence lengths close to 1000m due to the long torsional modes.

The maksimum rotation for this case is about 1.1 degrees which is slightly above the requirement of 1.0 degrees. This can easily be accounted for in the next phase by increasing the length of the pontoon by a meter or two.

4.3.4 Rotation about bridge axis from static wind load

As seen from Figure 4-18 the rotation from mean wind never exceeds 0.15 degrees, which is far below the requirement in design basis of 0.5 degrees. The load conditions applied are similar to the governing sea states with a return period 100 years, but adjusted for the one year conditions.

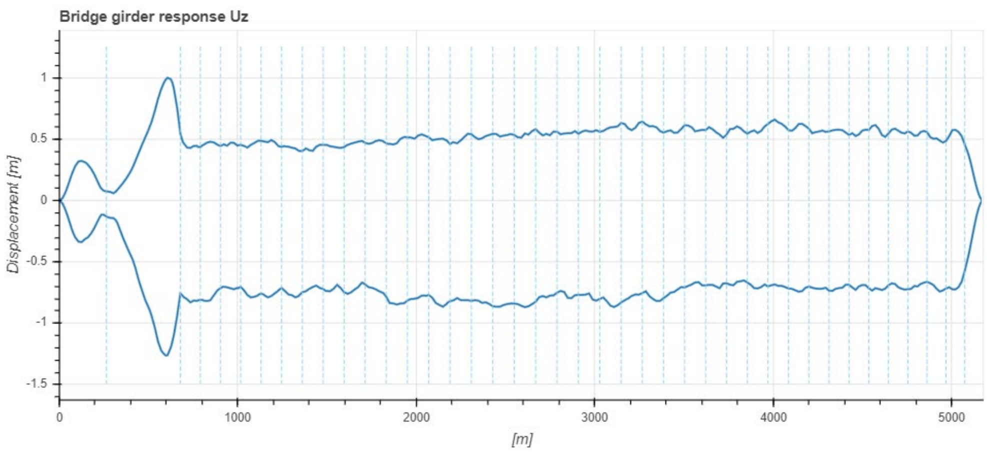


> *Figure 4-18 Mean rotation from environmental conditions (Mean wind)*

4.3.5 Check of free board

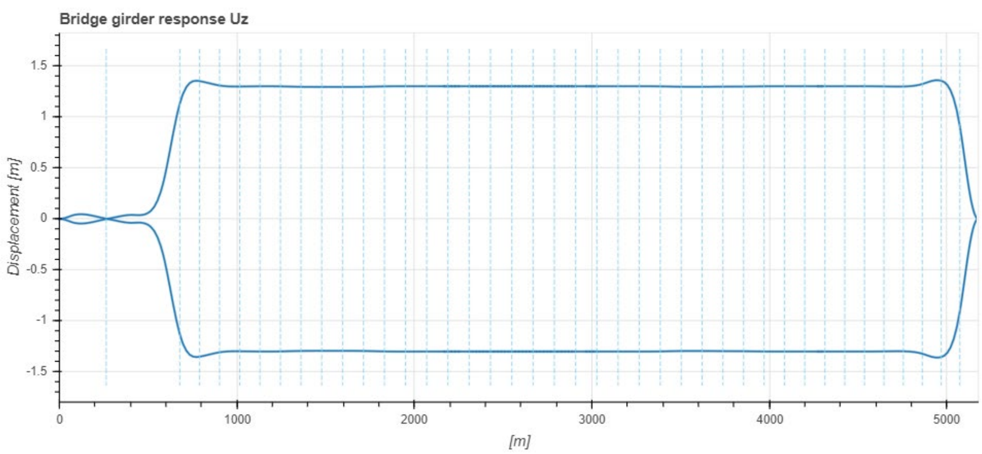
In all three contributions we have to check with regards to;

1.) Displacement from center of pontoon
2.) Displacement at ends due to rotation of the pontoon
3.) Tide
4.) Wave elevation



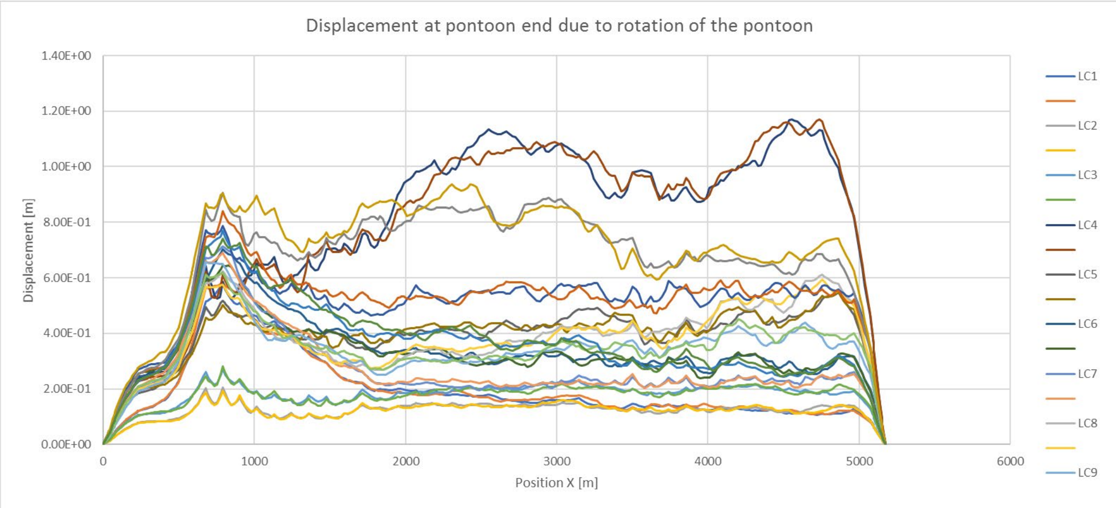
> *Figure 4-19 Maximum and minimum displacement 100 years conditions*

Maximum loss of draft is about 0.8m for the center of the pontoons.



> *Figure 4-20 Tidal response (tide=1.31m) 100 years return period*

The relative tidal displacement (RP=100 years) is about 0.4m on the pontoon closest to the landfall in north. The remaining pontoons follows the tide.

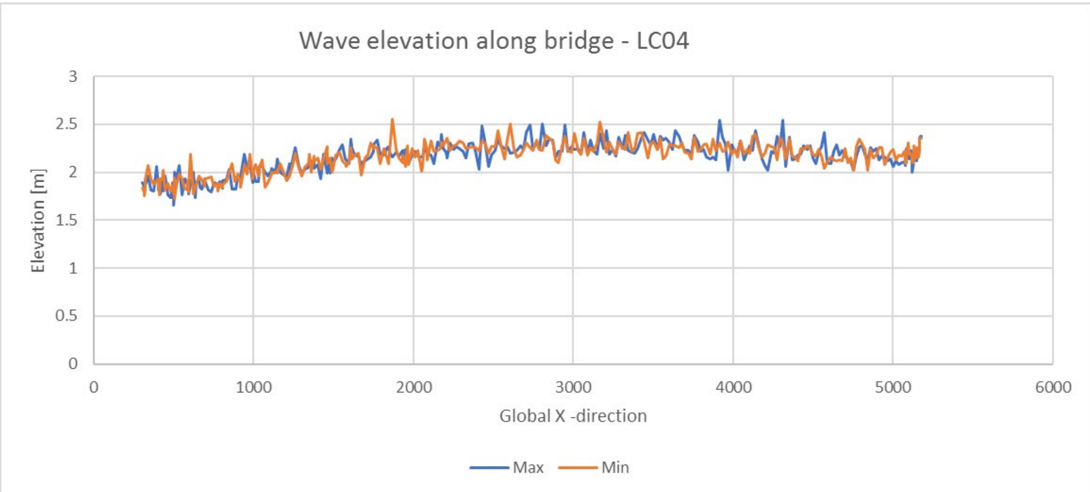


> *Figure 4-21 Displacement at pontoon end due to rotation of the pontoon*

The maximum rotation of the pontoon is about 2.37 degrees.

The rotation and displacement of the pontoon comes from excitations of different modes. Thus, they are uncorrelated. The pontoon affected by tide has a relatively small displacement from the environmental conditions and can be disregarded.

The pontoon in axis 38 (4700m in global X position) is the pontoon that based on the plot will have the smallest freeboard.



> *Figure 4-22 Wave elevation along bridge for load case 4 (governing with regards to free board)*

The maximum wave elevation seen from Figure 4-22 is about 2.5m.

How corelated the wave elevation is with the elevation at the pontoon edges is difficult to say for certain but most likely they are not very correlated. If there is any correlation this should be positive. (Pontoon elevation follows wave elevation). Thus, it will be conservative to assume that they are uncorrelated.

Total displacement = sqrt(1.2m^2+0.7m^2) = 1.39m

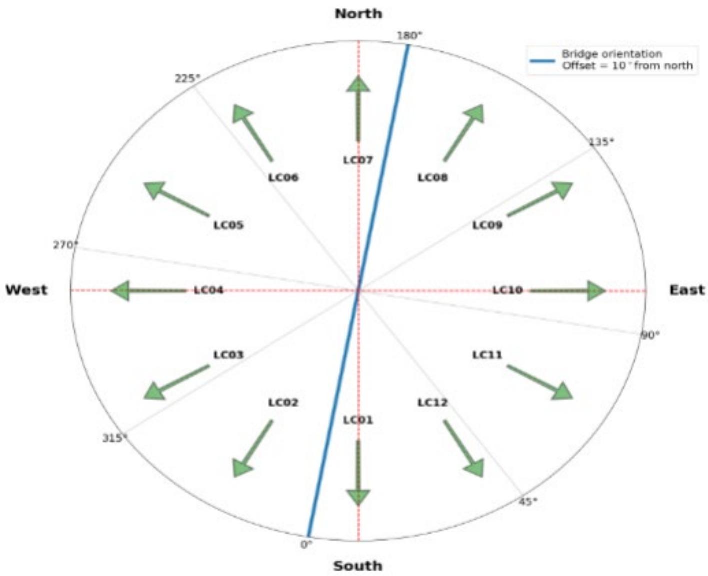
Freeboard = Draft – sqrt(Total displacement^2 + wave elevation^2)

Freeboard = 4m-2.86m = 1.14m > 0

The freeboard criteria is satisfied.

4.4 Comfort requirement

The definition of the ULS-load cases (LC) is according to the figure below.



4.4.1 Vehicle properties

To get an idea of the dynamic amplification the eigenfrequencies with associated eigenvectors are plotted.

> *Table 3 Eigenvectors and eigenfrequencies for the vehicle*

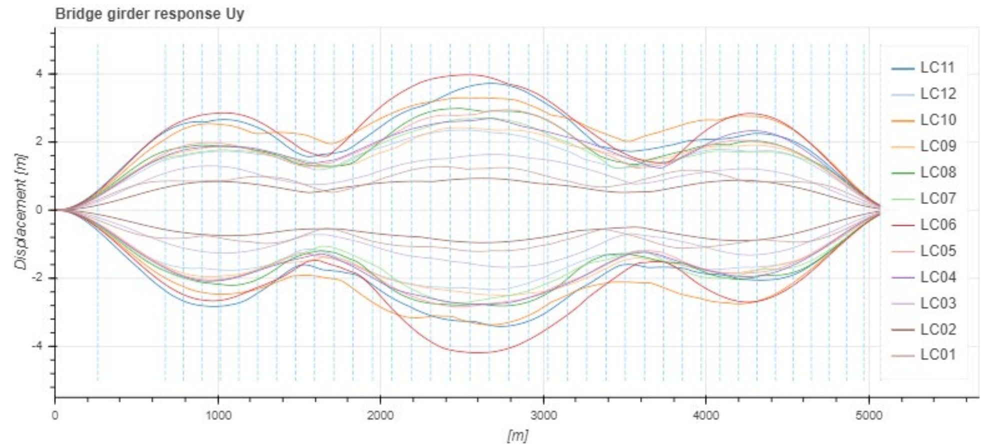
Eigen-frequencies	Eigenvectors										
[Hz]	Dof 1	Dof 2	Dof 3	Dof 4	Dof 5	Dof 6	Dof 7	Dof 8	Dof 9	Dof 10	Dof 11	Dof 12
0.81	0.99	0	0	0	0	0	0	0	0	0	0	0
1.00	0	0	0	-0.99	0	0	0	0	0	0	0	0
1.15	0	0	-1.00	0.00	0	0	0	0	0	0	0	0
1.44	0.06	-0.50	-0.05	0.08	0.50	-0.50	0.50	0	0	0	0	0

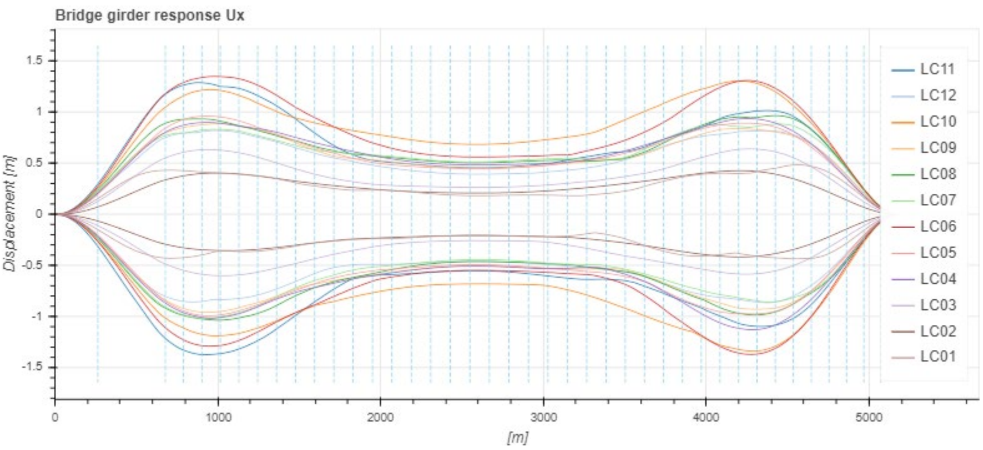
9.41	0.06	-0.50	0.05	0.08	-0.50	-0.50	-0.50	0	0	0	0	0
9.41	0.06	-0.50	-0.05	-0.08	-0.50	0.50	0.50	0	0	0	0	0
9.41	0.06	-0.50	0.05	-0.08	0.50	0.50	-0.50	0	0	0	0	0
9.57	0	0	0	0	0	0	0	0.65	0.03	0	0	0
10.95	0	0	0	0	0	0	0	0.38	-0.50	0	-0.12	0.86
10.95	0	0	0	0	0	0	0	0.38	-0.50	0.51	-0.59	-0.38
10.95	0	0	0	0	0	0	0	0.38	-0.50	0.29	0.79	-0.18
10.95	0	0	0	0	0	0	0	0.38	-0.50	-0.81	-0.08	-0.30

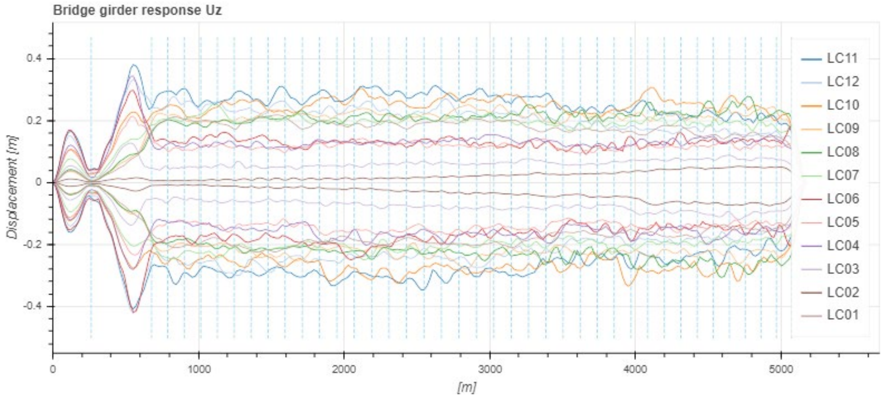
4.4.2 Bridge motion and acceleration from environmental loading

Bridge motion and acceleration 1-year return period

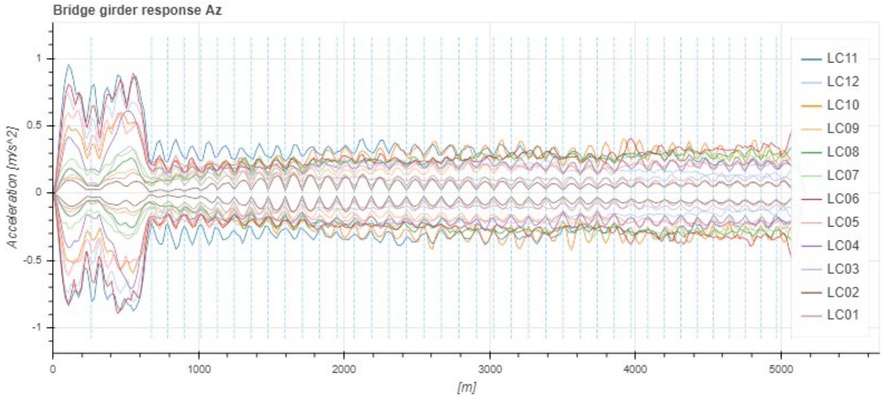
Acceleration from the bridge motion is found from the global analysis [12]. Note that x- is in-length, y- is sway, z- is in vertical direction. The following figures are max/min envelopes over 5 seeds of 1 hour.



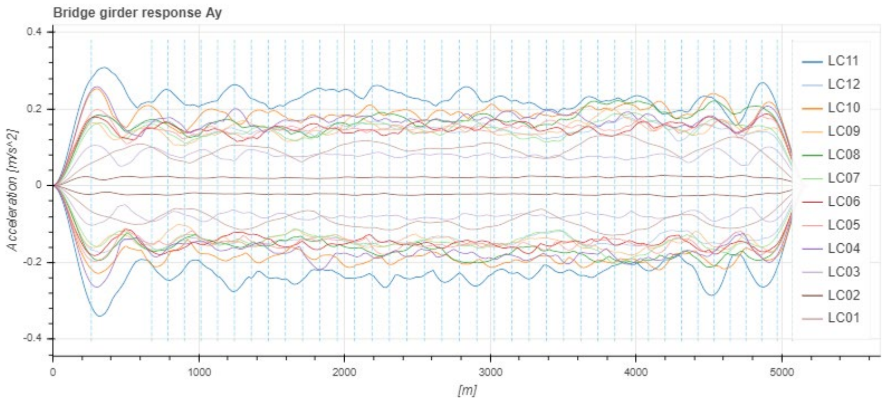




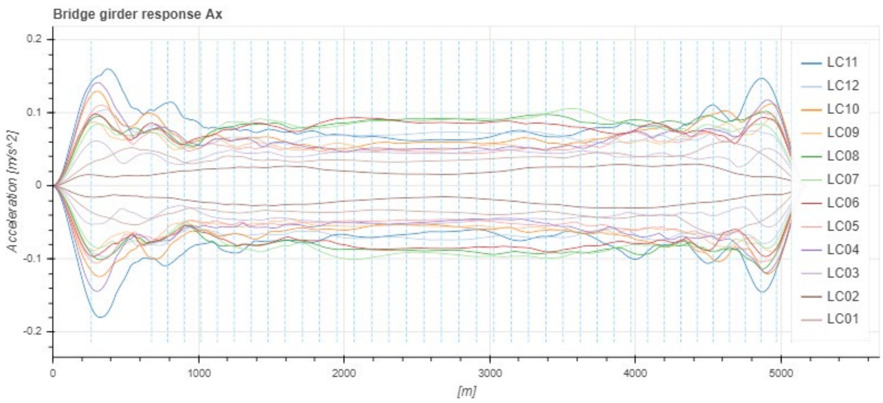
Bridge girder response Uz
Displacement [m]
[m]
LC11
LC12
LC10
LC09
LC08
LC07
LC06
LC05
LC04
LC03
LC02
LC01



Bridge girder response Az
Acceleration [m/s^2]
[m]
LC11
LC12
LC10
LC09
LC08
LC07
LC06
LC05
LC04
LC03
LC02
LC01



Bridge girder response Ay
Acceleration [m/s^2]
[m]
LC11
LC12
LC10
LC09
LC08
LC07
LC06
LC05
LC04
LC03
LC02
LC01



Bridge girder response Ax
Acceleration [m/s^2]
[m]
LC11
LC12
LC10
LC09
LC08
LC07
LC06
LC05
LC04
LC03
LC02
LC01

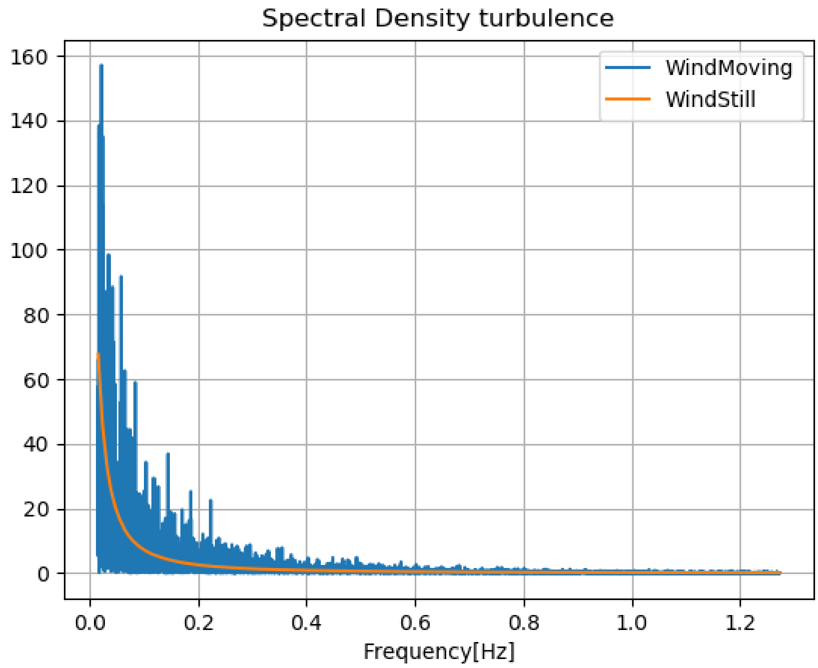
Acceleration on the moving vehicle

The table below show the RMS value for all 5x100runs for bridge acceleration loaded on the moving vehicle for each load-case.

	RMS Sway	RMS Heave	RMS Roll	RMS Pitch
LC1	1.52E-02	8.85E-03	4.76E-06	4.43E-06
LC2	4.80E-04	3.60E-03	8.80E-07	2.38E-06
LC3	2.86E-02	5.70E-02	1.11E-04	1.83E-05
LC4	2.22E-02	6.60E-02	1.05E-04	2.18E-05
LC5	2.16E-02	1.23E-01	8.25E-05	4.78E-05
LC6	2.28E-02	1.15E-01	9.20E-05	4.40E-05
LC7	2.01E-02	7.40E-02	3.81E-05	3.63E-05
LC8	2.95E-02	7.50E-02	7.30E-05	2.99E-05
LC9	2.10E-02	3.57E-02	6.40E-05	1.42E-05
LC10	3.43E-02	9.30E-02	1.29E-04	3.48E-05
LC11	5.40E-02	1.44E-01	1.96E-04	4.29E-05
LC12	2.24E-02	7.95E-02	5.65E-05	2.54E-05
MEAN	2.43E-02	7.28E-02	7.93E-05	2.68E-05

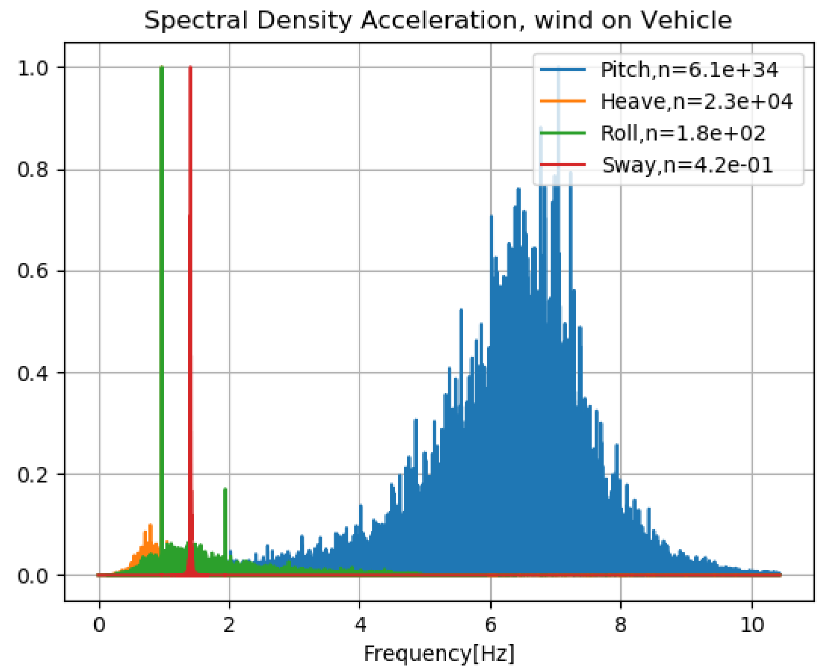
4.4.3 Wind properties

Results from the wind only parts are shown in this section. Figure below show spectral density plots of a single-point spectra vs. the moving point spectra of 70km/h moving vehicle. It is hard to tell the difference between the mean and the frequency shift, but an averaging has been performed and you could notice that the frequency components has an "upspeeding" effect. Note that in the region of eigenfrequencies, the wind energy is low.



> *Figure 4-23 Spectral density of the incoming single point wind and the incoming wind at a moving vehicle. For in wind turbulence*

Figure below is a normalized spectral density of the vehicle in wind. The normalization forces the maximum value to be 1 for each curve. In the legends the n-value is the scaling factor. Note that the maximum heave peak is at the same frequency as the roll-frequency.



> *Figure 4-24 Normalized spectral density of the response from wind on the moving vehicles, n=scaling factor so that the peak becomes 1. The larger the n the lower the actual response*

The table below shows the RMS-value from the different parts of the vehicle. This is for wind combinations only. You could tell that the sway-response is predominant.

> *Table 4 RMS from wind only (wind from LC6)*

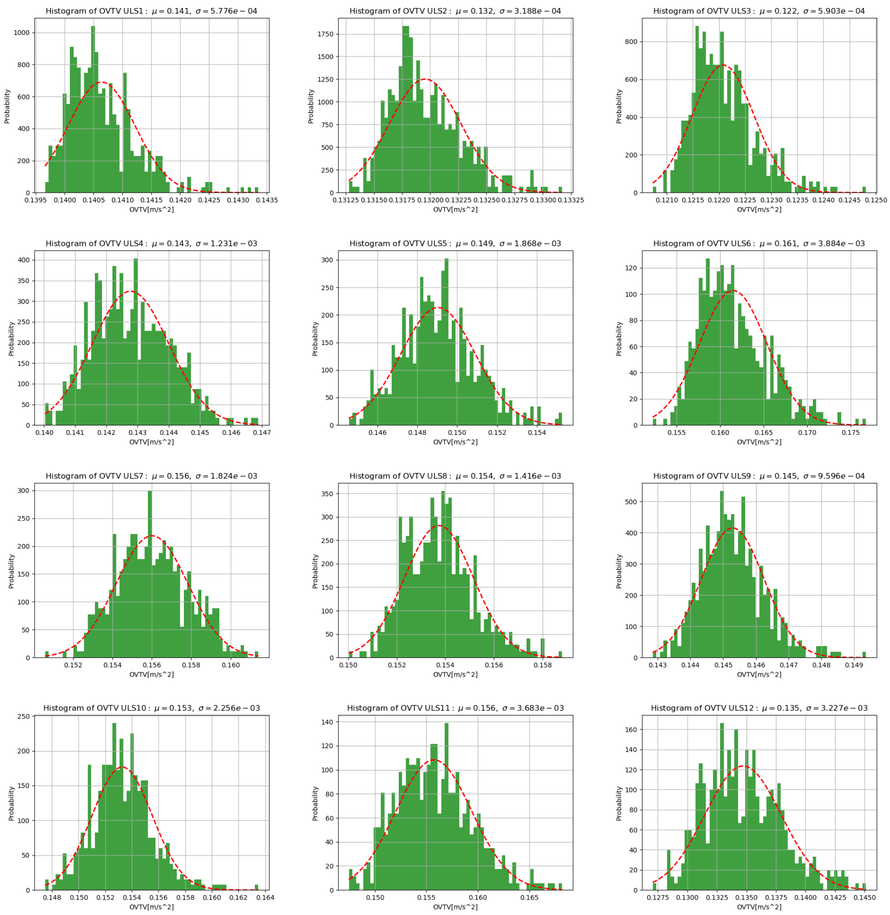
Direction	Description	Max RMS, wind only
RMSvs	RMS of vertical acceleration of seat	0.0008
RMSls	RMS of lateral acceleration of seat	0.1264
RMSps	RMS of pitch acceleration of seat	1.31E-18
RMSrs	RMS of roll acceleration of seat	0.0111
RMSvb	RMS of vertical acceleration of backrest	0.0011
RMSlb	RMS of lateral acceleration of backrest	0.1264
RMSvf	RMS of vertical acceleration of floor	0.0008
RMSlf	RMS of lateral acceleration of floor	0.0886

4.4.4 Combined results

The table below gives the mean and max values of for the total OVTV value from both wind and bridge accelerations. Red value indicated maximum over the column.

	OVTV, Mean of 500	OVTV, Max of 5 seeds	OVTV, Max of all
LC1	0.14064087	0.140825822	
LC2	0.13195506	0.132053678	
LC3	0.12208064	0.122489001	
LC4	0.14277632	0.142956072	
LC5	0.14903951	0.149239054	
LC6	0.16148528	0.162397012	0.1768
LC7	0.1560131	0.156591652	
LC8	0.1537384	0.154559694	
LC9	0.14529093	0.145530758	
LC10	0.15327323	0.153744572	
LC11	0.15573584	0.156413309	
LC12	0.13472076	0.135295977	

The figures below give the distribution of the OVTV in a histogram for all different load-cases. A normal-distribution with mean and standard-deviation is also show.



4.4.5 Conclusion

The OVTV values is well within the limit of $0.315 m/s^2$ for 1-year environment condition. The mean OVTV value is around $0.146 m/s^2$ and the maximum response found was $0.177 m/s^2$.

Note that acceleration from wind is larger than the acceleration from bridge motion.

4.5 Global stability

A global stability evaluation has been performed in SBJ-33-C5-OON-22-RE-012 App D K12 Global stability evaluations [13].

The global stability is very good, the metacentric height GM (distance from COG to meta center of the structure) is above 17 m for all intact situations. By taking one pontoon out of the equation (basically increasing the load span from 120m to 180m) one still maintains a good stability with a metacentric height above 7m.

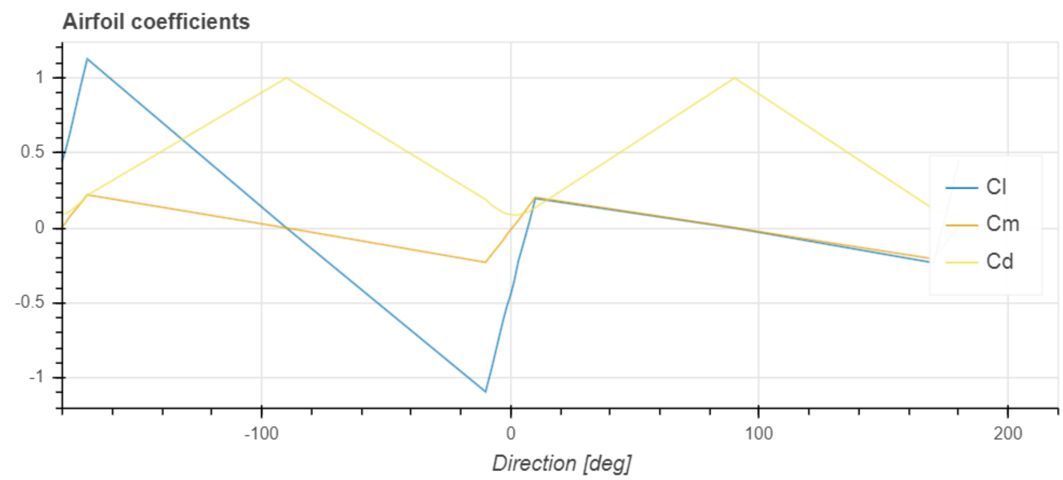
4.5.1 ULS-EQU stability

The stability of the bridge shall be evaluated with respect to ULS-EQU. For the 1-year condition, the change of mass and aerodynamic coefficients of the bridge girder due to the presence of traffic shall be accounted for.

The largest rotation found from the environmental analyses with a return period of 100 years is 2.3 degrees, see Figure 4-21. The bridge has to tilt beyond 8 degrees to lose water plane area, so the bridge is stabile with regards to the requirements.

Assuming a tilt from ULS traffic of 1 degree (design basis requirement), this increases the drag factor with 7% for wind with 0 degrees incident. See Figure 4-25. The moment factor increases from 0.013 to 0.045 for wind with 0 degrees incident angle which is not enough to introduce any significant further rotation of the girder. The airflow through the traffic increase the drag factor by 40%. Introducing 1 year environmental conditions reduces the wind load with a factor of 0.52 (26m/s)^2/(36m/s^2) as well as considerably lower rotations from waves.
The new mass is most likely not enough to cause considerably changes in the eigen modes of the structure.

The rotation from a ULS scenario including traffic is most probably smaller (0.52*1.4*1.07=0.78) than the case with traffic, and far away from losing the waterplane area and the global stability.



> *Figure 4-25 Drag factor with respect to bridge girder width*

4.5.2 Loss of freeboard

According to design basis sensitivity studies of the robustness of the structure when freeboard is temporarily lost shall be conducted. For the current design we are not very close to losing the freeboard. However, if we were close, a standard procedure for investigating this issue would be to examine the restoring moment for various tilt angles (GZ-curve). The area below this curve should be larger than the area below the corresponding wind heel moment for similar angles until the construction loses its stability.

Restoring moment area > *Safety factor* (1.3 in DNV-standards) * *Windheel moment area (0 deg ->instability deg)*

A further investigation finding the GZ-curve and compare it to the wind heel curve have not been done. However, the high GM factor indicates that the initial restoring moment will be very large. This can be looked further in to in the next phase but is most likely not an issue.

5 OTHER STUDIES

Numerous of studies has been performed to investigate and ensure robustness with regards to the chosen concept. A reference to the most relevant studies is listed below;

1. Wind spectrum sensitivity study
2. Anchor stiffness study K12 and K14
3. Number of anchor groups K12 and K14
4. Eigen mode anchor stiffness sensitivity
5. Skew wind from traffic study
6. Evaluation of critical wind directions
7. Discretization of mooring lines study
8. Sensitivity of wave spread
9. Evaluation of directional grid
10. Pushover analysis
11. Simplified anchor loss studies K12, K13 and K14
12. Influence of swell waves

A summary of all studies are found in SBJ-32-C5-OON-22-RE-005- Sensitivity studies [14] with further references to appendices.

6 REFERENCES

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[19] SBJ-30-C3-NOR-90-RE-102-2 Appendix 2 - Sensitivity studies.

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