INTERNATIONAL CONFERENCE

SUBMERGED FLOATING TUNNELS

May 29 - 30 1996
Sandnes, Norway

Preikestolen (Pulpit Rock) in Lysefjorden

Norwegian Public
Roads Administration

PREFACE

This is a report prepared mainly for those who took part in the conference. However, what came out of the conference was far more valuable than most of us had anticipated. Thus we would also like to have it documented for the future.

The good result is, of course, thanks to the participants and in particular to those who prepared and presented papers. As always, the good results come from the efforts of dedicated and competent individuals.

In order to keep the cost down for all parties, this report have been copied directly from the manuscripts. We apologise for the sometimes rather poor quality, but hope the message gets across.

Finally, I would like to thank all the participants, the sponsors and all who took part in the organisation and running of the conference. A special thanks goes to Jorunn Bokn and Kari Mehla for taking care of all the practical details.

Oslo, August 1996

Kaare Flaate

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	Italy	R. Carpaneto
	Norway	T. Søreide
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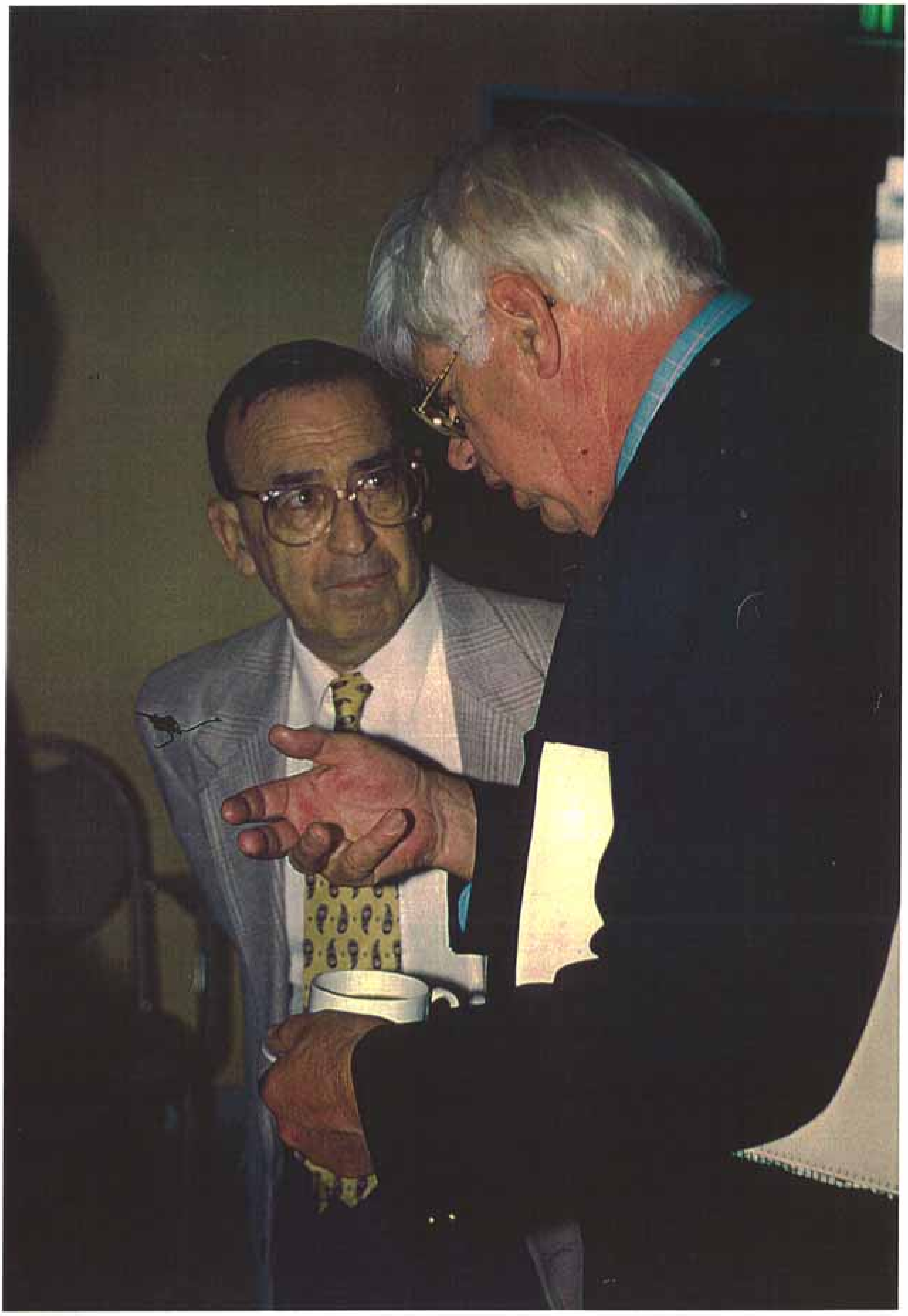


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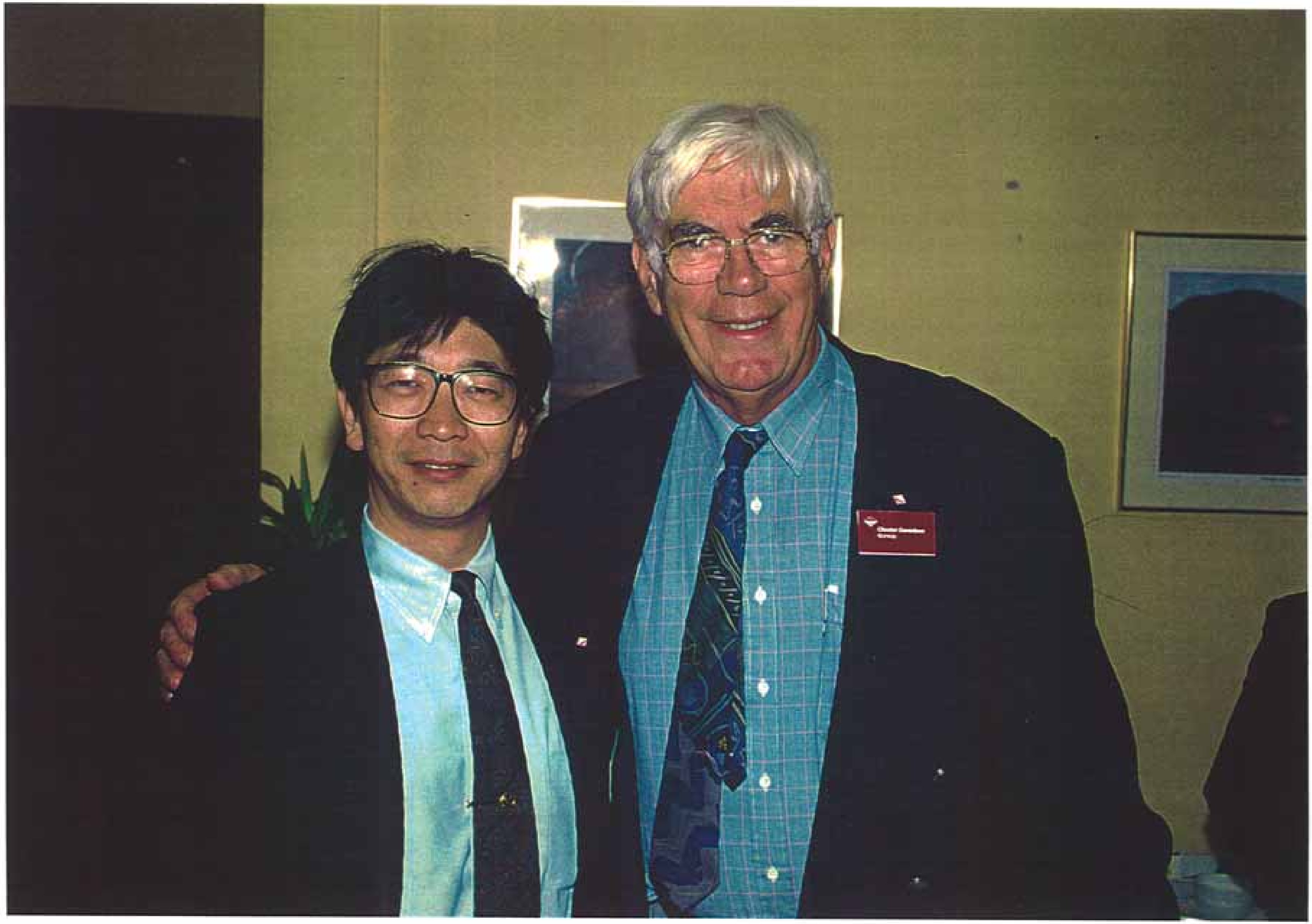

HØGSFJORD
SANDNES BUSS
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SANDNES BUSS


Four alternativ solutions across Hogsfjord

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

1. Welcome speech

1. O. Søfteland,
2. E. Matacena

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

1. Welcome speech

1.1 Welcome speech

O. Søfteland, Director General
Norwegian Public Roads Administration

Welcome speech

Olav Søfteland, General Director
Norwegian Public Roads Administration

Ladies and gentlemen,

Welcome to the International Conference on Submerged Floating Tunnels. One of the main questions you may ask yourself is, why do we have this conference here in Sandnes? Well, there are several reasons.

Firstly, there is tremendous interest in Norway for strait crossings as we are always looking for new ways to connect islands to the mainland or to cross some of our numerous fjords. As such, the submerged floating tunnel, or SFT for short, is a very interesting solution to many of these crossings.

In addition, we have a site, located not too far from here, where we have considered for a long time of building an SFT. It is the fjord: Høgsfjord. To build such a structure is an arduous task and we need all the assistance and co-operation that we can get from the international community to solve the remaining technical problems.

A second reason why we are here is a direct result of the Strait Crossings Conferences that have been held in Norway three times. At each of these conferences there was a growing interest in submerged floating tunnels. At the last one in Ålesund in 1994, it was felt that greater international contact was needed.

On that occasion, in what was possibly a rash word of enthusiasm, Chester Danielsen promised to organise a pure SFT conference within two years. Now, anybody who knows Chester knows that he doesn't pledge anything lightly and this conference is a direct result of that promise. So thank you Chester for gathering us all here today.

Not all of you may know this, but the European Union has taken a keen interest in submerged floating tunnels and an analysis project of limited scope and dimension is under way. This project is administrated by FEHRL with working participants from Denmark, Italy and Norway. The result will be a state-of-the-art report with recommendations for further R&D projects.

Norway is pleased to have the chairmanship of this group and we are particular interested in future research projects with practical applications.

I have underlined the need for international co-operation and this conference is part of that co-operation. However, at one stage in the planning of this conference we asked ourselves, what is going to be the next step? What kind of forum can we have to exchange ideas? In addition, we felt that conferences like this should be taken care of by one of the existing international organisations.

With this in mind, the International Tunnelling Association was approached and I was pleased to see that within only 2-3 months, they decided to establish a sub-group on SFT. We believe that this is a good way forward, as from now on the questions just raised will be the responsibility of an international committee.

For this prompt action, I wish to thank the president of ITA, Professor Sebastinò Pelizza and all the others that have contributed to this solution. I can promise on behalf of the Norwegian Public Roads Administration that we will participate in future activities.

To conclude:

Initially we thought we would be lucky if we succeeded in attracting 40 top engineers to this conference. So you can imagine our reaction when over 70 inscribed. Personally I am looking forward to the various sessions, from the general design concepts today to the «nuts & bolts» tomorrow. As I have mentioned already, one of the main objectives of this conference is to promote co-operation on an international level. I would like to remind you that the success of this co-operation depends upon you.

I will like to end off by extending a special thanks to those of you who have travelled a great distance and sacrificed both time and money to be here.

Thank you for your kind attention.

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

1. Welcome speech

1.2 Welcome speech

E. Matacena, President
Ponte di Archimede

DEAR COLLEAGUES,

MY NAME IS ELIO MATACENA.

I AM ITALIAN.

I ADDED, TO MY SHIPOUNER'S ACTIVITY, SINCE MORE THAN TWENTY YEARS, AN INTENSIVE PROMOTION OF THE ARCHIMEDES BRIDGE, CARRIED OUT IN GREECE, ITALY, UNITED STATES, SPAIN, TURKEY AND GERMANY.

I GIVE YOU MY WELCOME WITH A SENTENCE OF ISAAC NEWTON THAT INVOLVES A STIMULATING ACKNOWLEDGEMENT OF OUR PIONERISTIC INVOLVEMENT: "MEN BUILD TOO MANY WALLS AND NOT ENOUGH BRIDGES".

THESE SIGNIFICANT WORDS REPRESENT, FOR MANY CONSIDERATIONS, THE EMBLEMATIC EXPRESSION OF OUR CONFERENCE. WE ARE SUPPORTERS OF AN INNOVATIVE TYPOLOGY AIMING TO REALIZE A NEW KIND OF BRIDGES, WITH THE WISH THAT THEIR NUMBER WILL OVERTAKE THE WALLS ONE.

OUR CONFERENCE WAS PROMOTED ALSO FOLLOWING THE AUTHORITATIVE SUPPORT OF THE EUROPEAN UNION TO THE SFT TYPOLOGY. A GROUP OF SCANDINAVIAN AND ITALIAN PARTNERS OBTAINED A CO-FINANCING FROM THE EUROPEAN UNION FOR THE STUDY OF A PRACTICAL APPLICATION OF THE SFT CONCEPT.

THE WORDS OF NEWTON I REPORTED HAVE A FURTHER METAFORIC VALUE BECAUSE THEY ARE INCLUDED IN THE EUROPEAN UNION DOCUMENT ISSUED TO PROMOTING, FACILITATING AND ENCOURAGING THE COOPERATION BETWEEN ENTERPRISES OF DIFFERENT STATES FORMING EUROPEAN ECONOMIC INTERESTING GROUPS.

THE THREE PARTNERS OF THE SCANDINAVIAN-ITALIAN GROUP INVOLVED IN THE STUDY OF THE SFT TYPOLOGY, COST OF WHICH IS CO-FINANCED BY THE EUROPEAN UNION, ARE AIMING TO EXTEND THE MEMBERSHIP TO THE NATIONS INTERESTED IN COLLABORATING TO

THIS STUDY. OUR TARGET IS TO JOIN TOGETHER IN A FORUM THE NATIONS WHICH ARE INTERESTED TO OFFER THEIR TECHNOLOGICAL CONTRIBUTION. ANY MEMBER STATE WILL, ON REQUEST, PARTICIPATE WITH ONE DELEGATION IN THE RESEARCHES, BROADENING THE FORUM FIELD OF ACTION.

IN A SECOND STAGE MEASURES AND FACILITATIONS SHALL BE IMPLEMENTED TO ALLOW THE ENTERING IN THE FORUM GREAT FAMILY ALSO TO REPRESENTATIVES OF EXTRA-COMMUNITY NATIONS.

WE ARE GRATEFUL TO THE NORWEGIAN COLLEAGUES FOR THEIR KIND HOSPITALITY AND WE ADDRESS THEM OUR WARM COMPLIMENTS AND BEST WISHES FOR THE FIRST IN THE WORLD SUBMERGED FLOATING TUNNEL THEY WILL REALIZE TO LINK LAUVIK WITH OANES.

THE HOGSFJORD PERMANENT CROSSING SHALL DEMONSTRATE THE FEASIBILITY AND RELIABILITY OF THE SUBMERGED FLOATING TUNNEL CONCEPT AND WILL COVER THEREFORE A GREAT IMPORTANCE IN THE HISTORY OF THE HUMAN PROGRESS.

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

2. State of the Art "International Tunnelling Association"

C. Hakkart
Delta Marine Consultants bv

International Conference on Submerged Floating Tunnels Sandnes May, 1996

STATE of THE ART
[International Tunnelling Association]

Ch. J.A. Hakkaart

Delta Marine Consultants
P.O. Box 268
2800 AG Gouda
The Netherlands

29 mei 1996

The overheads and the presentation are based on the publication:

International Tunnelling Association
Working Group
Immersed and Floating Tunnels
State of the Art

Tunnelling and Underground Space Technology
1993

SUBMERGED

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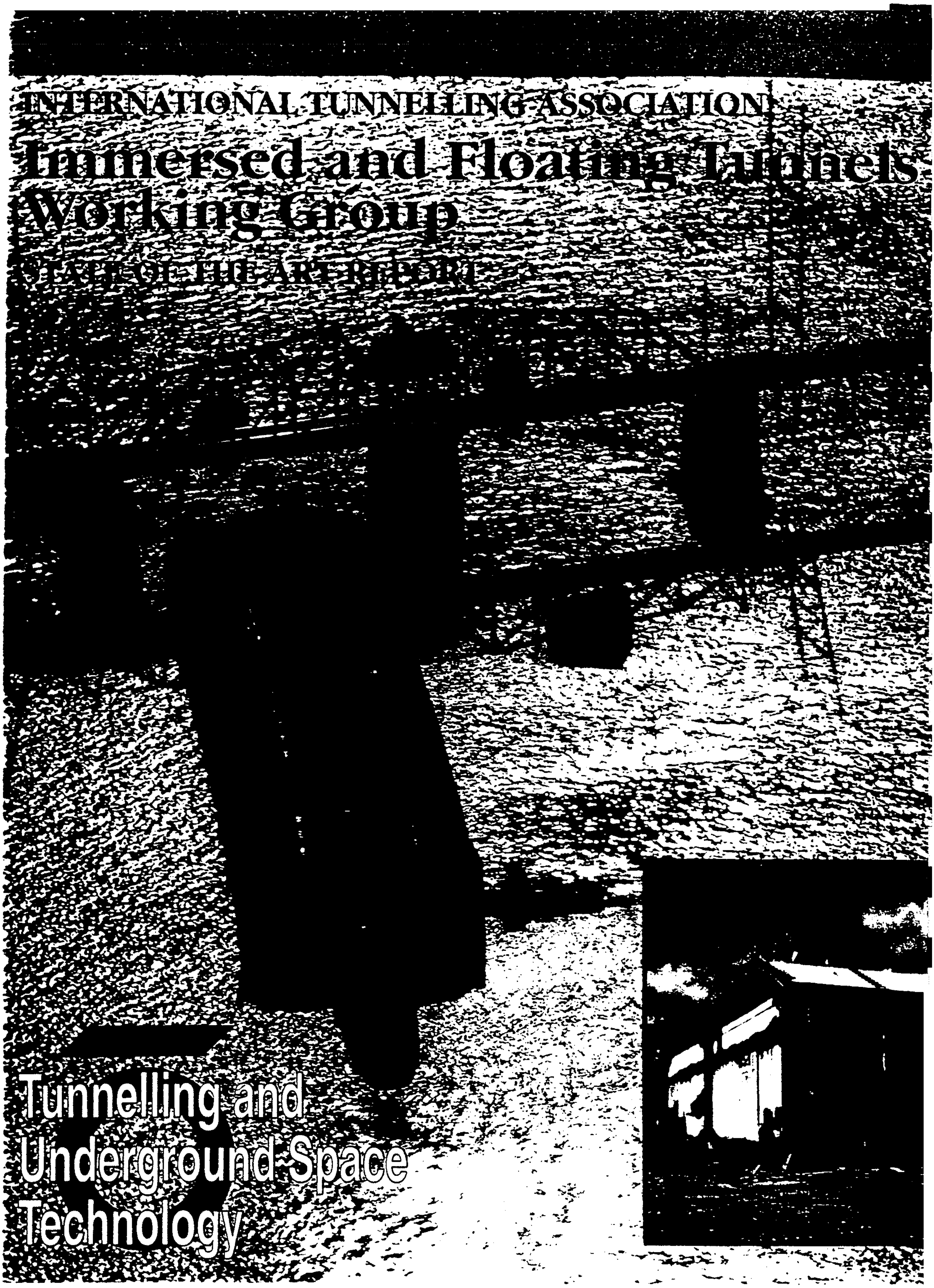
INTERNATIONAL TUNNELLING ASSOCIATION

Immersed and Floating Tunnels Working Group

STATE OF THE ART REPORT



Tunnelling and Underground Space Technology

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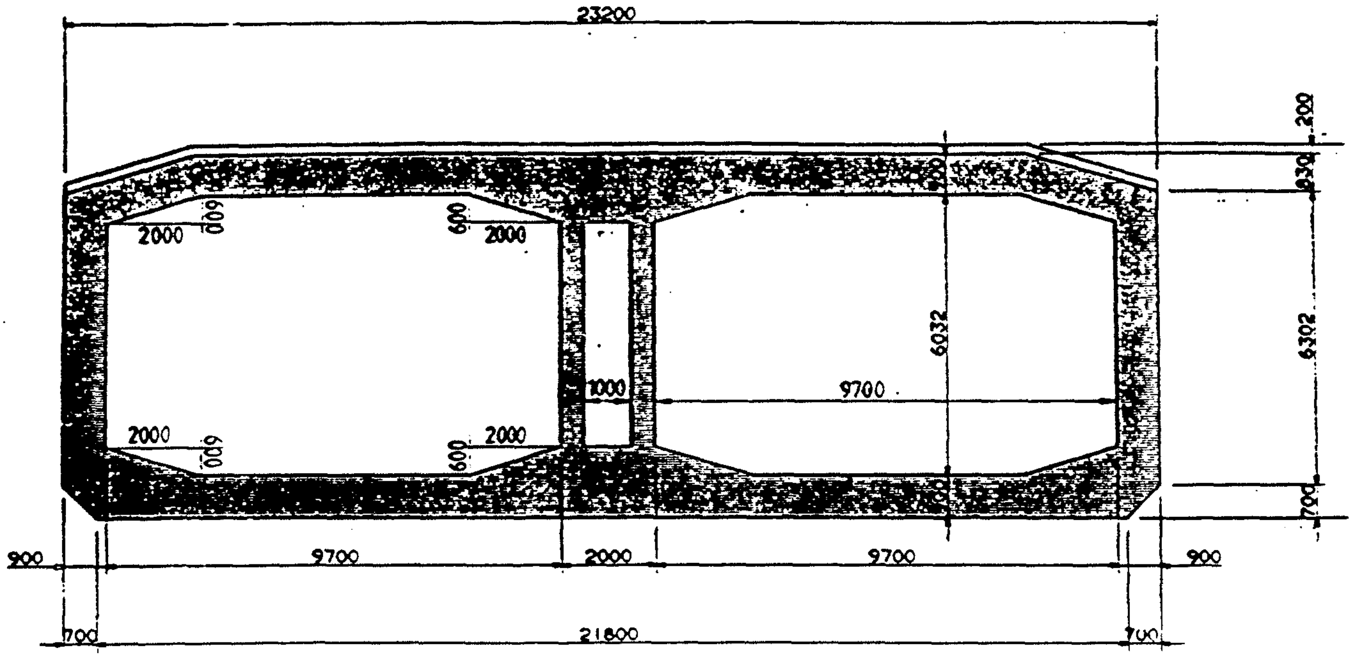
TWO TYPES OF TUNNELS

* STEEL TUNNEL — USA TYPE

* CONCRETE TUNNEL

- WITH LINING
- WITHOUT LINING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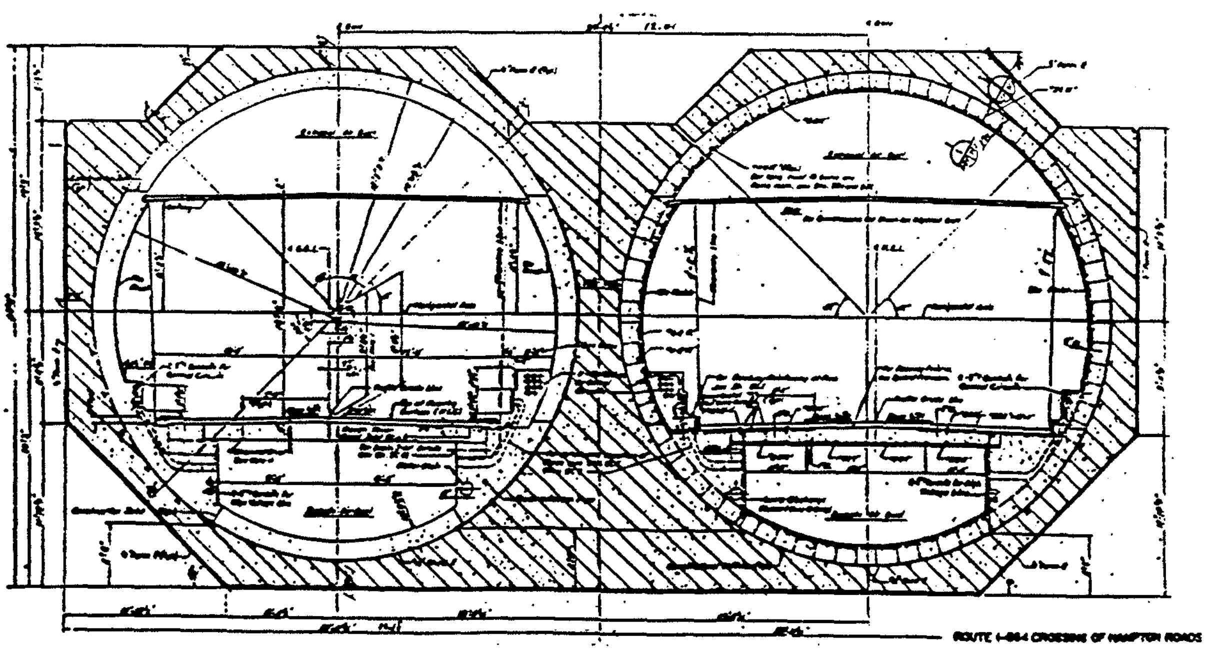
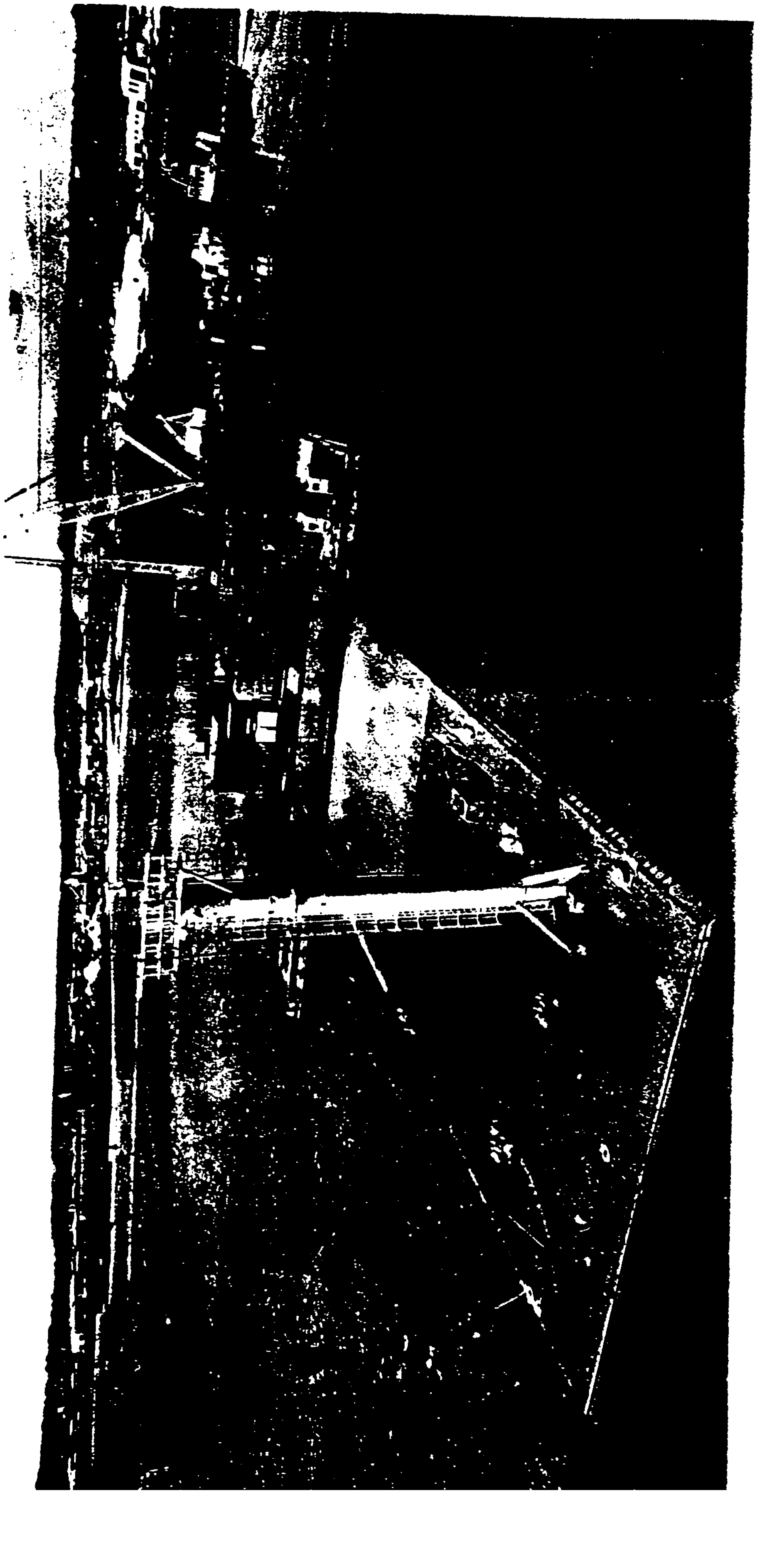
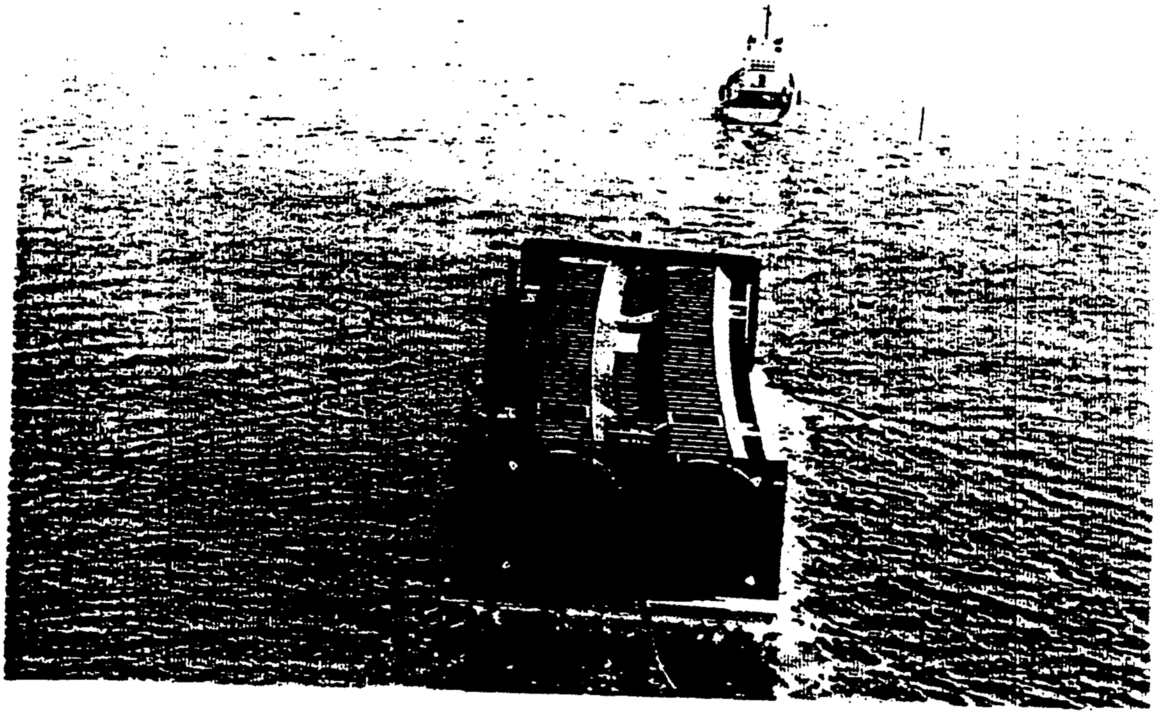
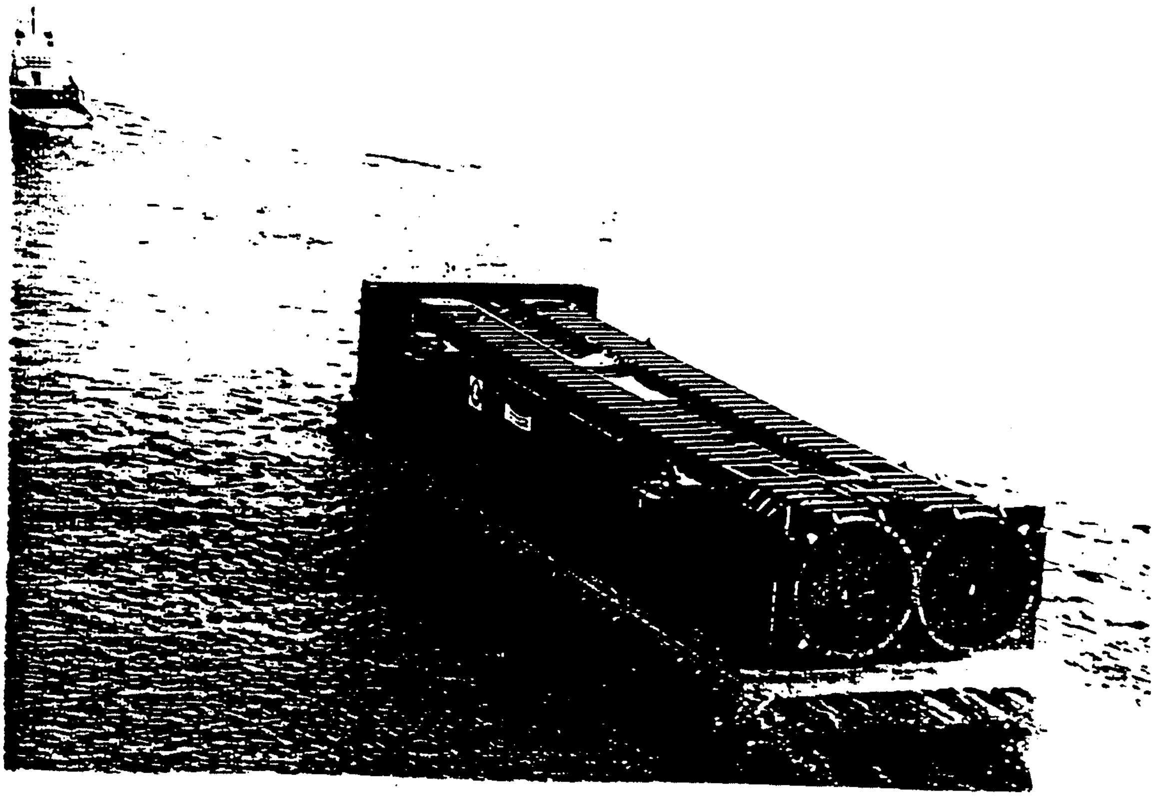
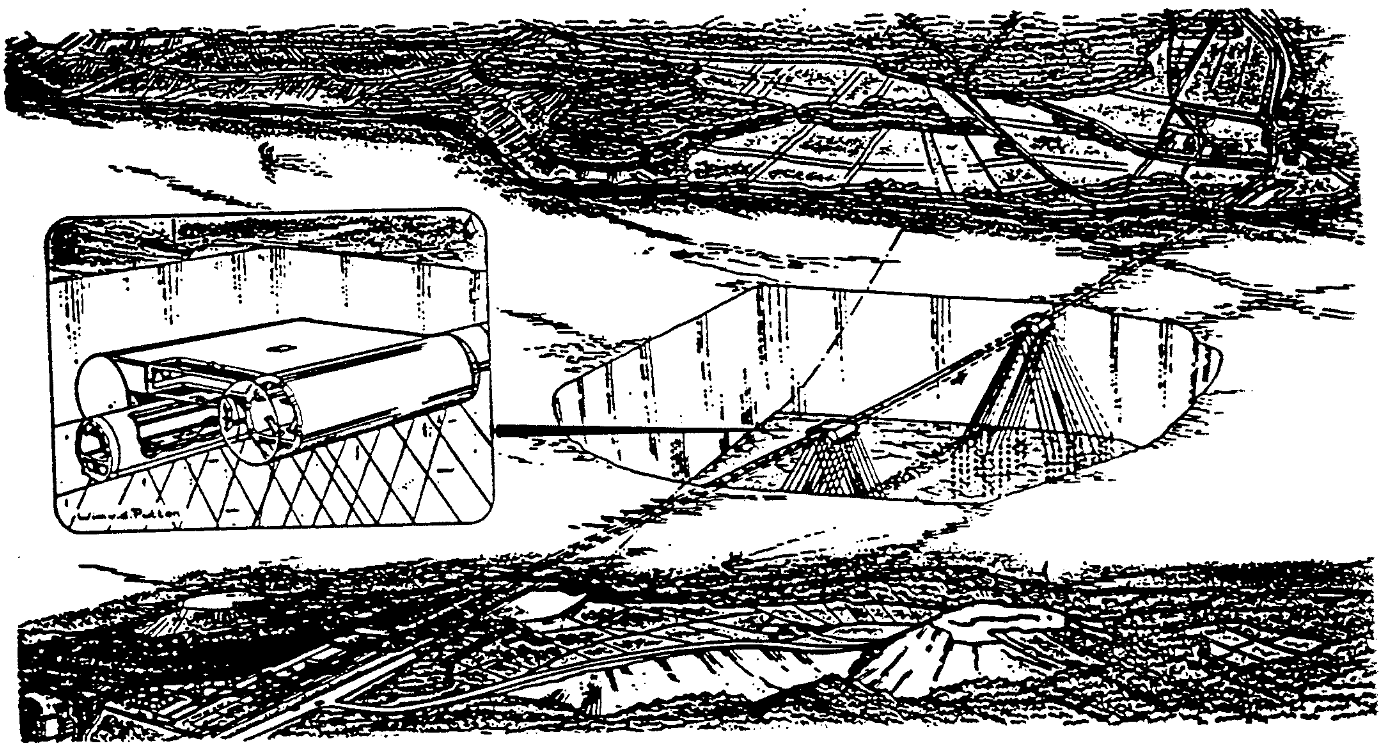

ROUTE I-664 CROSSING OF HAMPTON ROADS









SUBMERGED FLOATING TUNNEL HISTORY

18??	SFT patent in Norway?
1923	SFT patent in Norway
1947	SFT patent in Norway
?	Construction and short time use of small SFT in Eastern Europe?
1970 - now	Studies Strait of Messina
1985-1994	Studies Høgsfjord
1989	ITA Working Group Immersed and Floating Tunnels

SUBMERGED FLOATING TUNNEL HISTORY

1993	Publication of ITA "state of the Art report"
1994	Society of Submerged Floating Tunnel Technology Research in Hokkaido
1994-	Studies SFT in Lakes in Italy
1996[april]	ITA accepted as platform for SFT.
1996[may]	First SFT congress in Sandnes

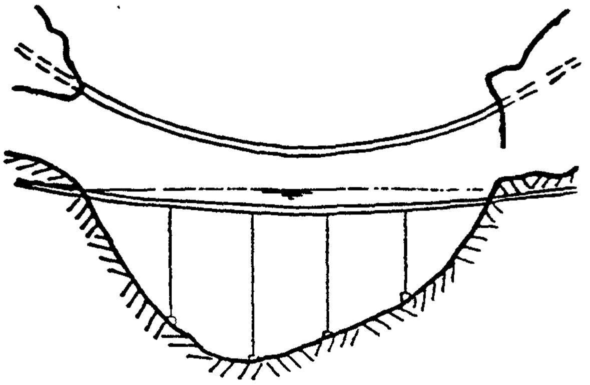


Figure 6-19. Arch shape horizontal support.

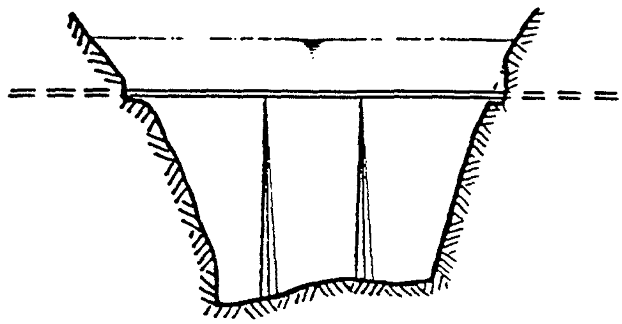


Figure 6-20. Anchor cables in groups.

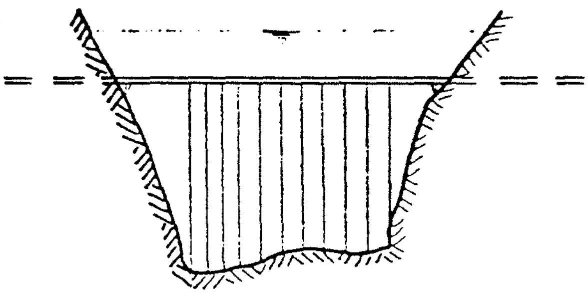


Figure 6-21. Anchor cables along the tunnel.

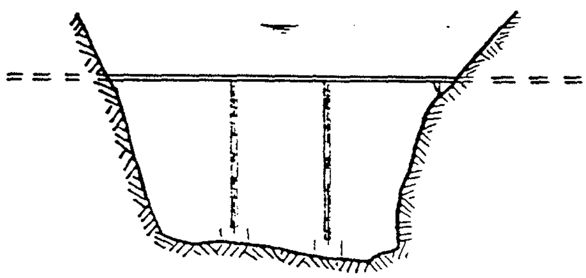


Figure 6-22. Fixed supports.

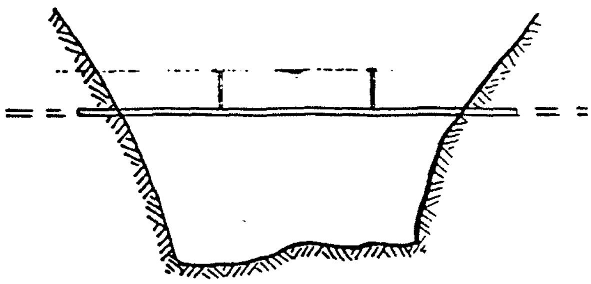


Figure 6-23. Pontoon support.

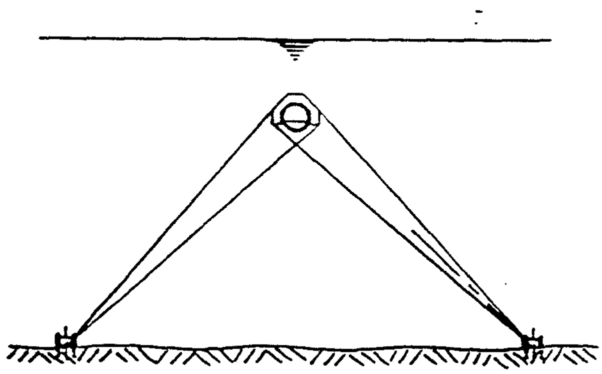
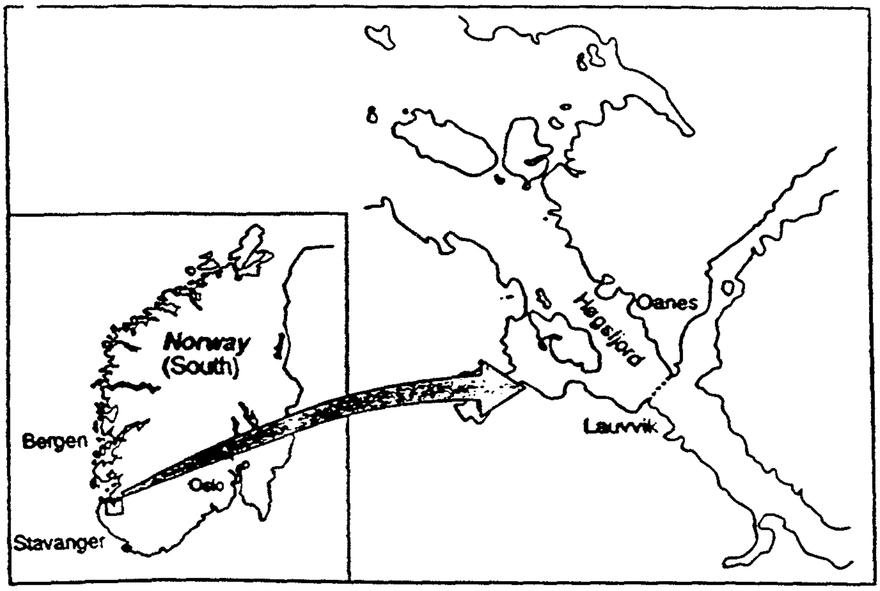


Figure 6-24. Combined horizontal and vertical support.

COMPARISON WITH OTHER FIXED CROSSINGS

Type	Distance	Water depth	Sea State
Bridge	Yes	Yes	No
Ponton Bridge	No	No	Yes
Floating Bridge	No	No	Yes
Bored Tunnel	No	Yes	No
Immersed Tunnel	No	Yes	No
Submerged Floating Tunnel	No	No	Limited

HOGSFJORD in Norway



Some relevant site conditions are:

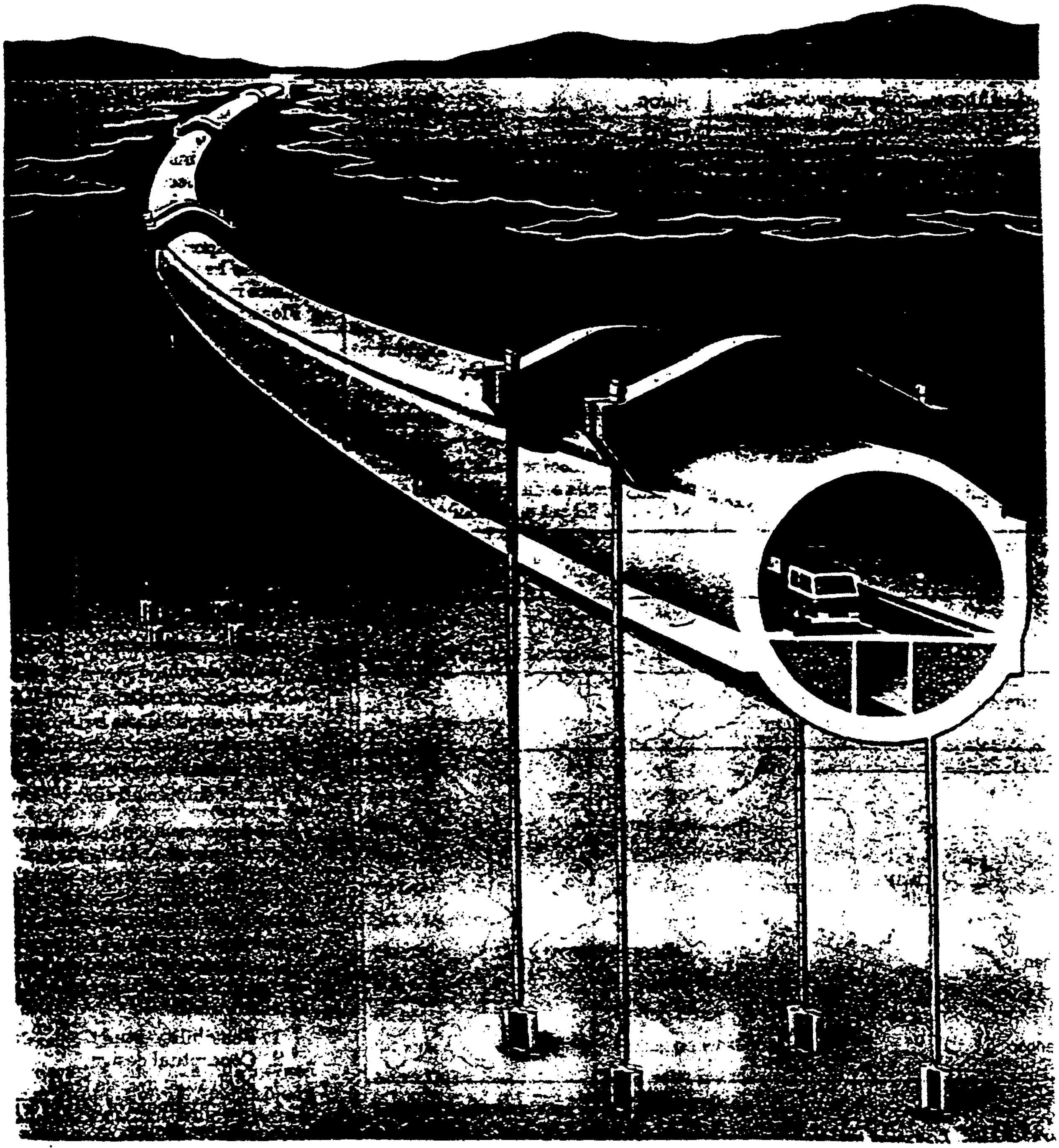
* width of waterway: 1400 m
* water depth: 150 m
* average currents: 0.6 m/s
* water depth above tunnel: 20 m
* significant waveheight: 1.5 m
* shipping traffic is low

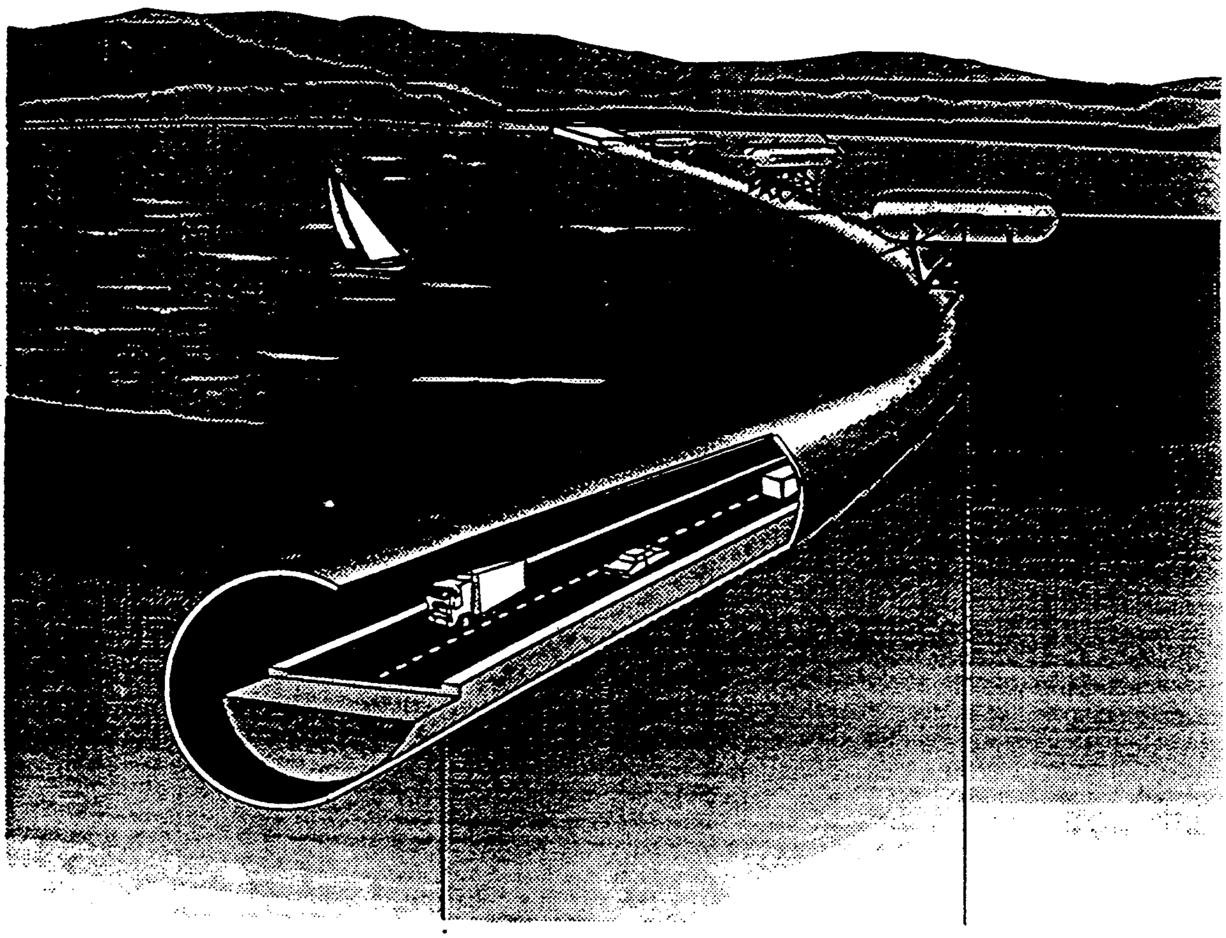
Study period: 1987 - 1991

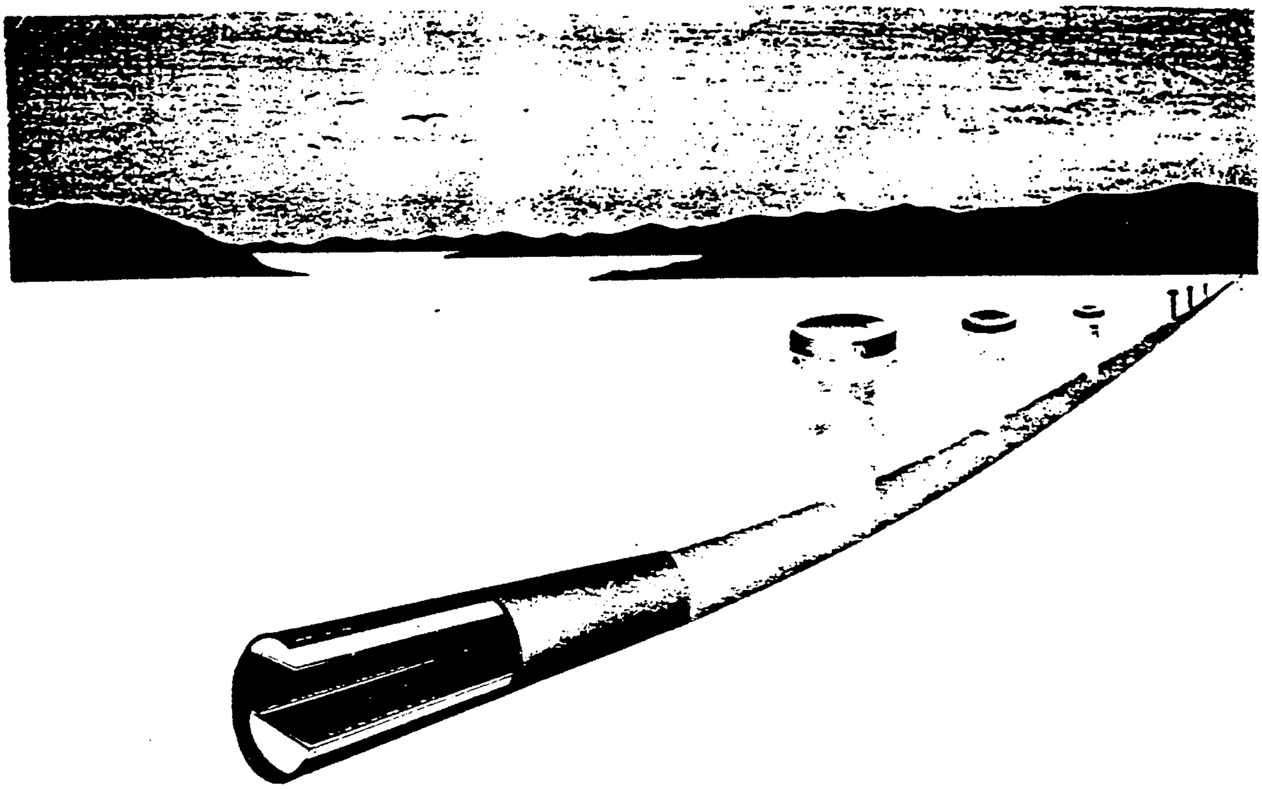
Estimated cost :

US$ 80 to 100 million (1991 level).

Construction Time: about 3 years







Strait of MESSINA in Italy

Some relevant site conditions are:

* width of waterway:	3000	m
* water depth:	350	m
* currents:	1 to 2	m/s
* water depth above tunnel:	55	m
* significant waveheight:	9 to 16	m

* shipping traffic is high
* seismic loading to be expected

Estimated costs:

Each of the three tunnels

US $ 2.5 billion (1991 level).

Construction Time: about 7 years

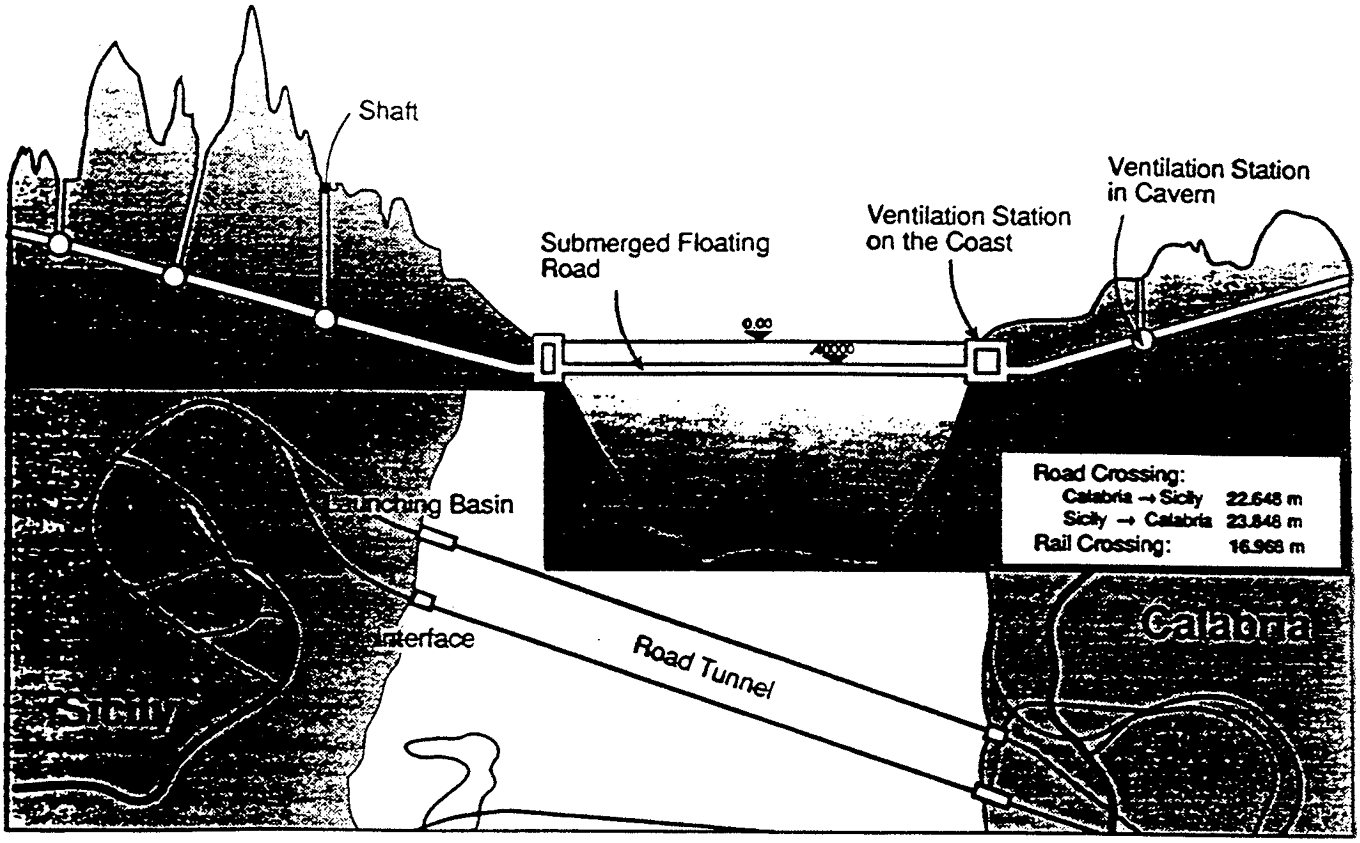
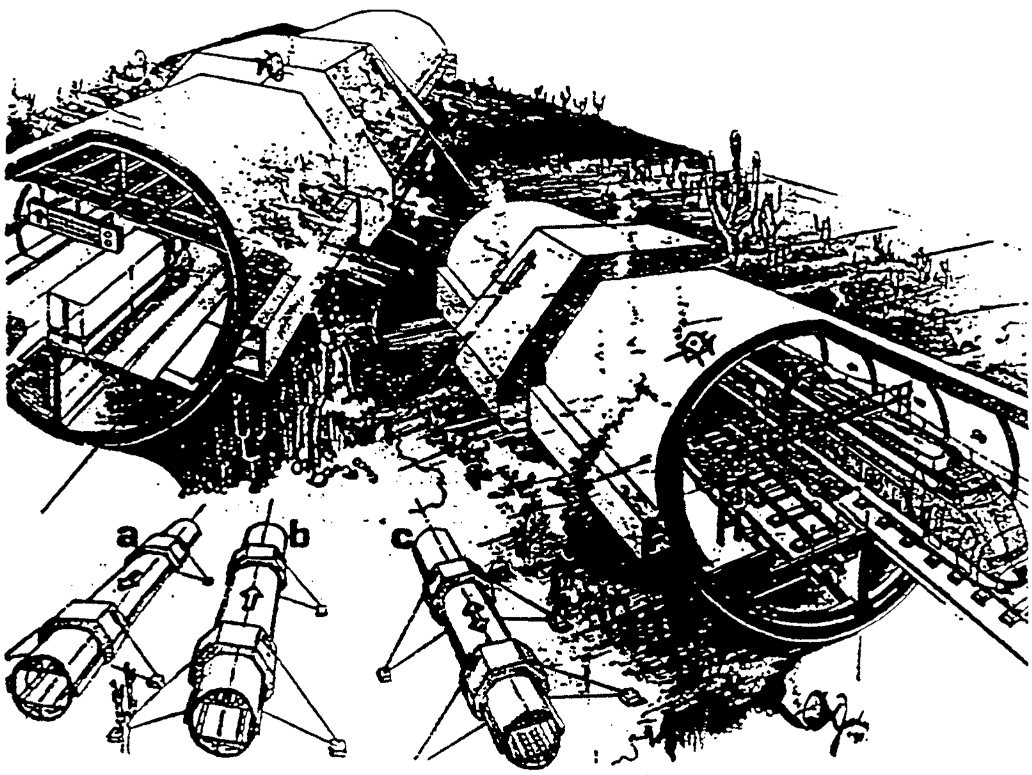


Figure 6-16. Location of the Strait of Messina.



DESIGN

LOADS

Permanent loads
- Structural dead weight
- Hydrostatic pressure
- Buoyancy

Functional loads
- Loads due to traffic
- Loads due to changes in ballast conditions
- Variable loads during construction

Deformation loads
- Shrinkage
- Creep and relaxation
- Post- or pre-tensioning
- Differential settlements
- Temperature variations
- Remaining internal loads due to the construction method

Environmental loads
- Loads due to wave action
- Static loads caused by current
- Dynamic loads caused by vortex of current
- Loads due to tidal variation
- Loads caused by floating ice on the water surface
- Loads due to change of water density
- Response due to earthquake
- a slow tectonic slip between Sicily and Italy.
- a slow relative movement of the several faults present in the area.

Accidental loads
- Loads due to traffic accidents
- Loads due to ship collisions, anchor dragging
- Falling ship anchors and other debris
- sinking ships
- Explosions inside or outside the tube
- Fire from burning cars or fluids
- Loss of buoyancy
- Failure within the support system

MATERIALS

LIMIT STATES

SLS: Serviceability Limit State
ULS: Ultimate Limit State
PLS: Progressive Collapse Limit State
FLS: Fatigue Limit State

ANALYSIS Static and Dynamic

Global Static Analysis

Local Static Analysis:
- the joint between SFT and abutment
- the tether connections
- the pontoon connections.

Dynamic Analysis:
- the mass and expected added mass of the structure and its individual components in relation to each other.
- the stiffness of the structure and its individual components in relation to each other.
- the damping of the structure and its individual components in relation to each other.

Dynamic Response due to Marine Loading

Dynamic Response due to Seismic Loading:
- vertical exitation
- transversal excitation
- longitudinal excitation

Dynamic Response due to Passing Trains

Instrumentation

Durability and Maintenance

SAFETY

TARGET:

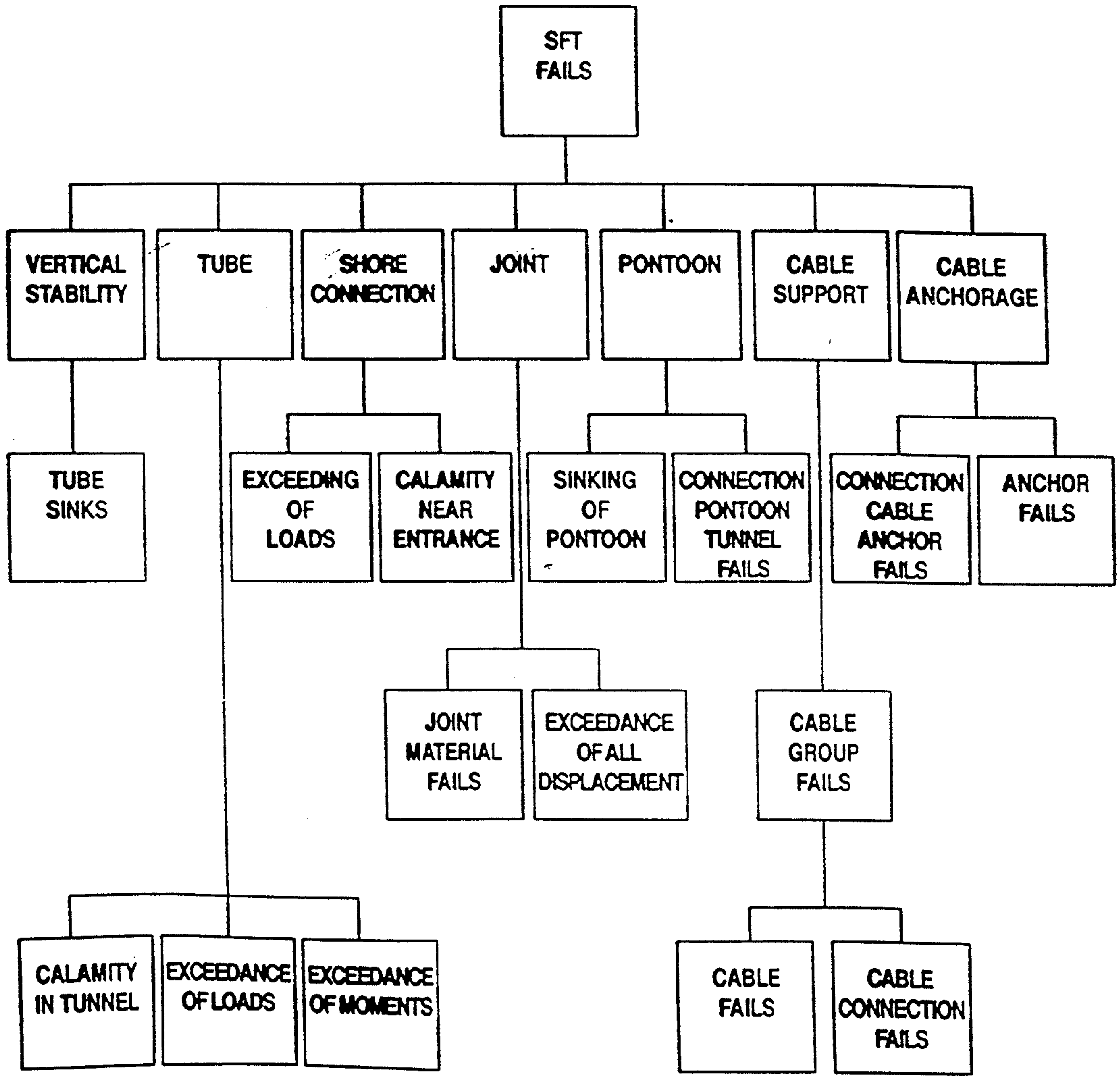
The level of safety required for an SFT will be the same as that required for comparable structures, e.g. bridges

ASPECTS:

-Technologized safety
- * design rules
- * experience
- * measures
- * backups, etc

-Psychological safety
- * movements below limits of human observation
- * ....

RISK ANALYSIS and SAFETY



LEAKAGE IN TUNNELS

* IF IT OCCURS, THEN MOSTLY ON JOINTS

* LEAKAGE IN THE MAIN BODY ONLY INCIDENTICALLY

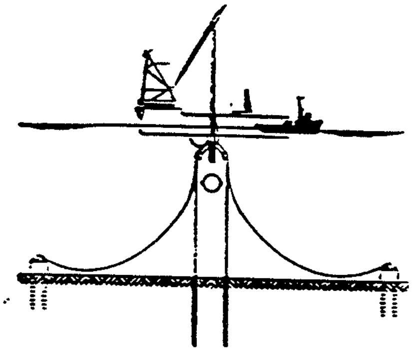
* NO DIFFERENCE BETWEEN STEEL TUNNELS AND CONCRETE TUNNELS

* LEAKAGE OCCURS MOSTLY IN THE PORTALS

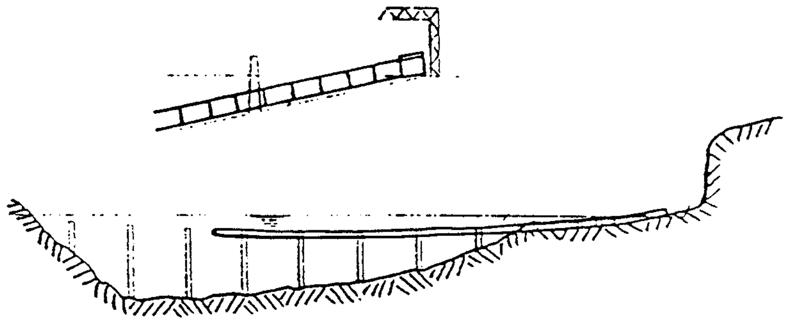
hbw

CONSTRUCTION METHODS

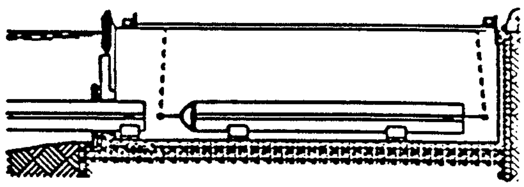
Construction in elements

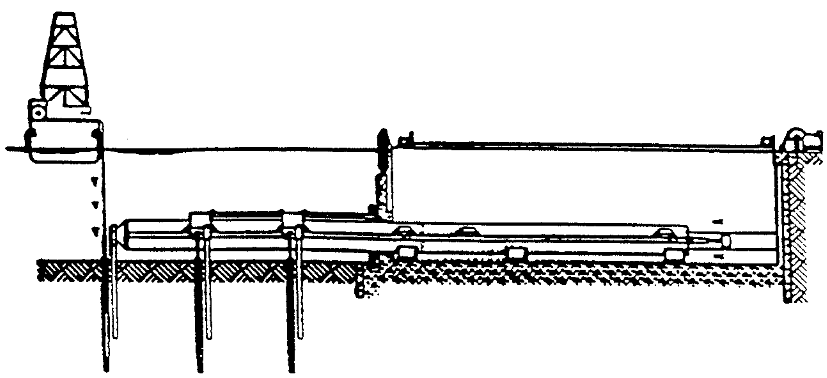


Incremental Construction and Launching



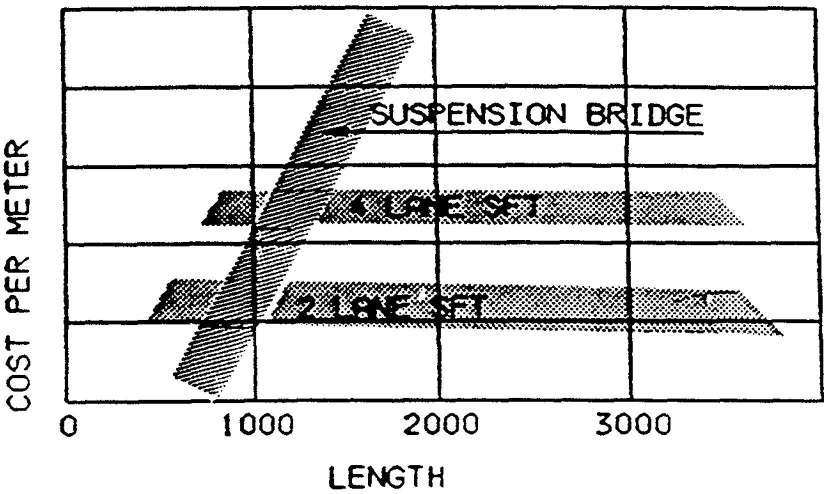
Element construction and Incremental Launching





COSTS

Construction Costs



Operational Costs

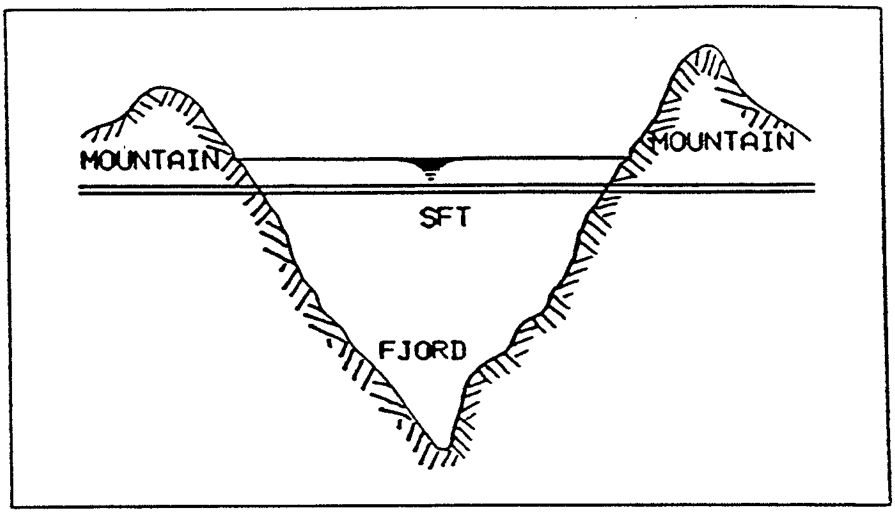
<u>usual running and maintenance costs:</u>

- lighting
- ventilation
- interior cleaning
- drainage
- traffic control, etc.

<u>costs particular to an SFT structure:</u>

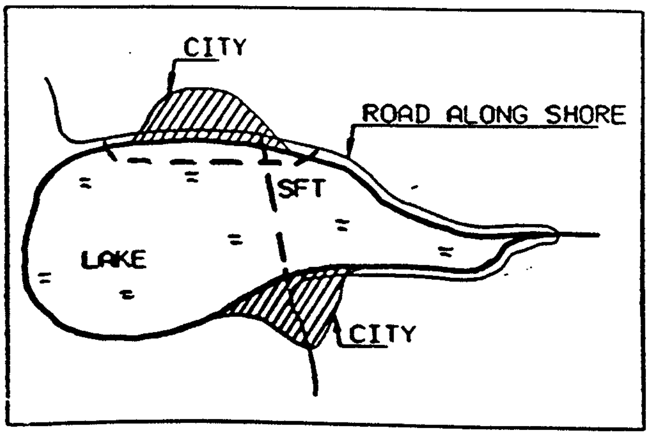
- instrumentation and monitoring
- inspection
- reporting
- verification of the design.

DEEP and WIDE WATERWAYS

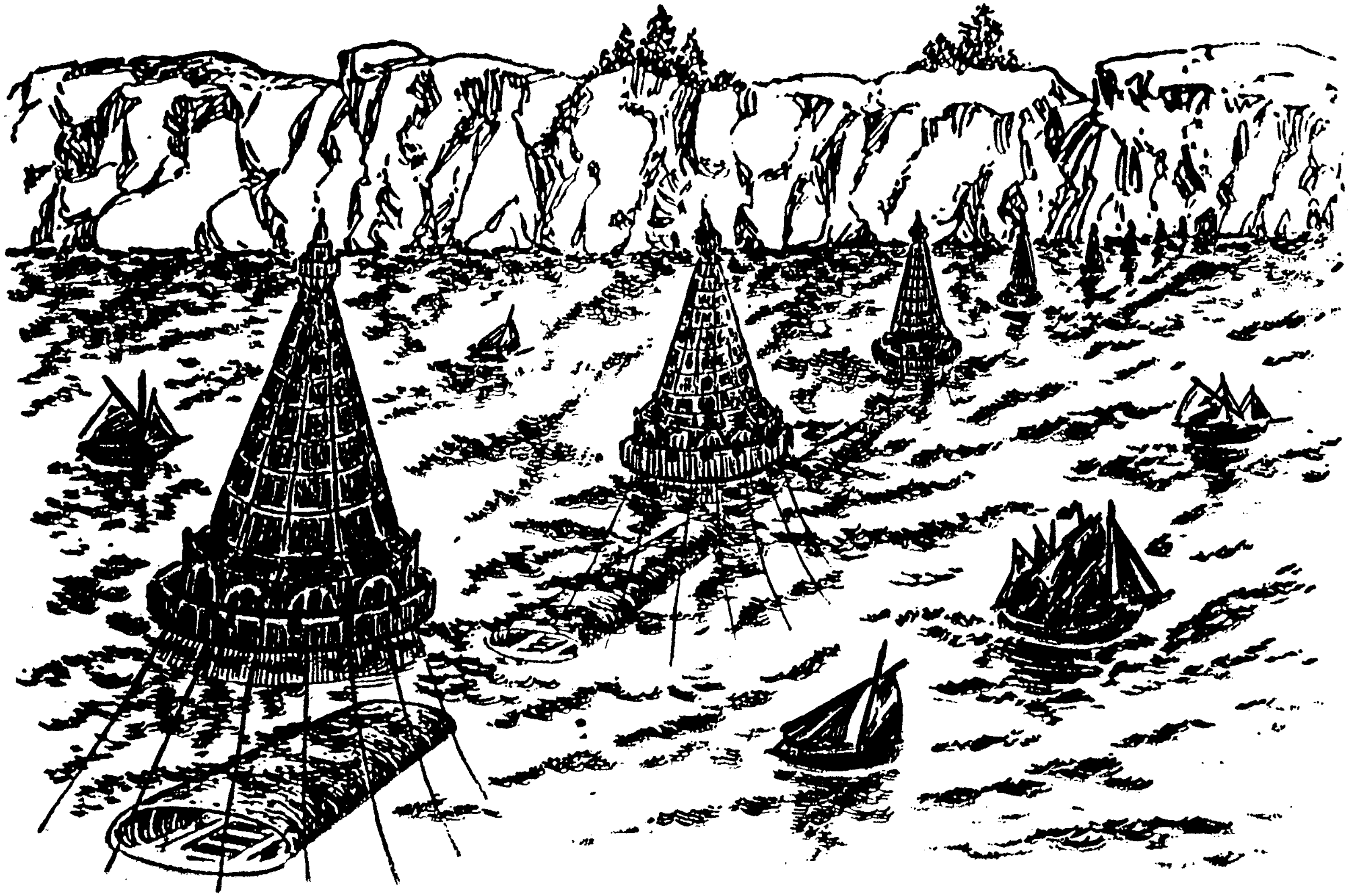


Country	Possible location for Submerged Floating Tunnels
Norway	Many fjords
Italy	Straits of Messina
Greece	Mainland to islands
Turkey	In-between continents; between mainland and islands
Spain/Morocco	Strait of Gibraltar
France	Gironde
U.S.A.	Fjords on west coast
Alaska	Bering Strait
Canada	Fjords on west coast
South China Sea	Between islands
Coast of South-east Asia	Mainland to islands
Japan	Mainland to islands; between islands

ENVIRONMENT



Country	Location of Lake
EUROPE:	
Italy	Como/Lecco
Italy	Maggiore
Italy	Lugano
Italy	Iseo
Italy	Garda
Switzerland	Neuchatel
Switzerland	Vierwaldstettersee
Switzerland	Zürichsee
France, Switzerland	Geneve/Leman
Germany, Austria, Switzerland	Bodensee
Sweden	Vättern
Portugal	Rio Tejo
THE AMERICAS:	
Canada, U.S.A.	Superior
Canada, U.S.A.	Huron
Canada, U.S.A.	Erie
Canada, U.S.A.	Ontario
U.S.A.	Michigan
Nicaragua	Managua
Peru, Bolivia	Titicaca
ASIA:	
Israel, Jordan	Dead Sea
Japan	Biwa Ko
Ukraine	Azov
OCEANIA:	
New Zealand	Taupo
New Zealand	Wakatipu



International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

3. Hokkaido Crossing

1. K. Sato
2. S. Kurita
3. T. Fujii
4. S. Kagaya
5. T. Iijima

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

3. Hokkaido Crossing

3.1 The Purpose of Submerged Floating Tunnel Plan in Hokkaido, Japan

K. Sato
Hokkaido University

The Purpose of Submerged Floating Tunnel Plan in Hokkaido, Japan

Keiichi SATO
Professor, Hokkaido University

This paper describes the purpose of submerged floating tunnel plan in Hokkaido, and shows some effect of the tunnel.

I show the air network of New Chitose Airport in Hokkaido in Fig.1. The New Chitose Airport has two 3000m runways. It is 70 min. drive from Sapporo and 1 hr 30 min. flight from Tokyo. The number of domestic routes is the second largest in Japan after Haneda, Tokyo.

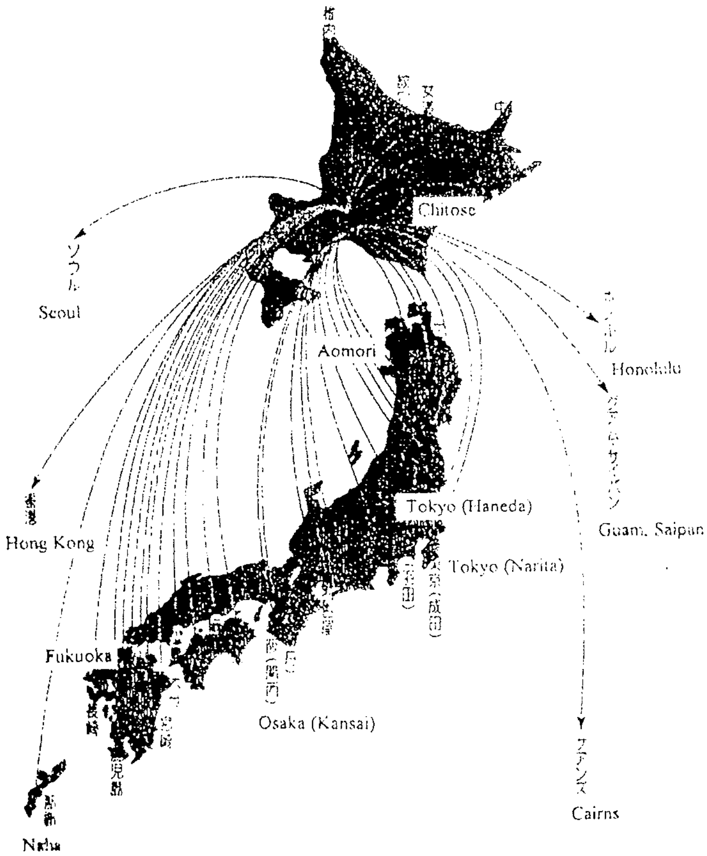


Fig.1 New Chitose Airport Air Network

We made a plan to construct a submerged tunnel to cross the mouth of this bay, which is only 30 km, or 18.63 miles, across. But if we construct a route around the bay, it is 130 km, or 80.73 miles. (Fig.2)

Many people asked such questions as "why should it be a submerged tunnel and not an undersea tunnel?", "who is going pay for it?", We are sure that many of you here in this room have the same questions. In my paper I will give some answer to these questions.

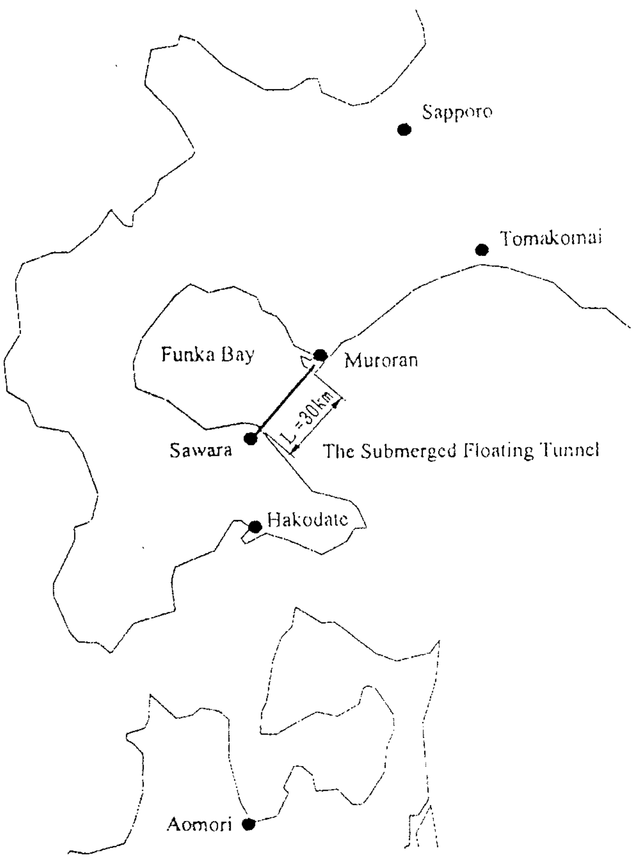


Fig.2 The Route of the Submerged Floating Tunnel

Currently in Japan, the transfer of the functions of the capital is being examined. In other words, the central government and Houses of Representatives and Councilors moving out from Tokyo. The background of these discussions are:

(1) To ease the over congestion and polarization to Tokyo.

(2) To improve the functions of central government

(3) To separate economic activities and politics

(4) To further decentralize the administrative function

Several sites have announced themselves as candidates for the new capital. One of them is the New Chitose Airport Vicinity Area of Hokkaido. A tract over 10,000 ha or 247,110 acre is already prepared, without fear of large-scale earthquakes, and there is plenty of high quality clean water. However, the distance from Tokyo is the problem.

Especially, the Shinkansen has not been constructed yet. So, we have proposed a new route plan for the Hokkaido Shinkansen using a submerged tunnel.

Fig.3 is a geological map and the longitudinal section of Funka Bay. The maximum depth is 100m, or 328 feet, deep.

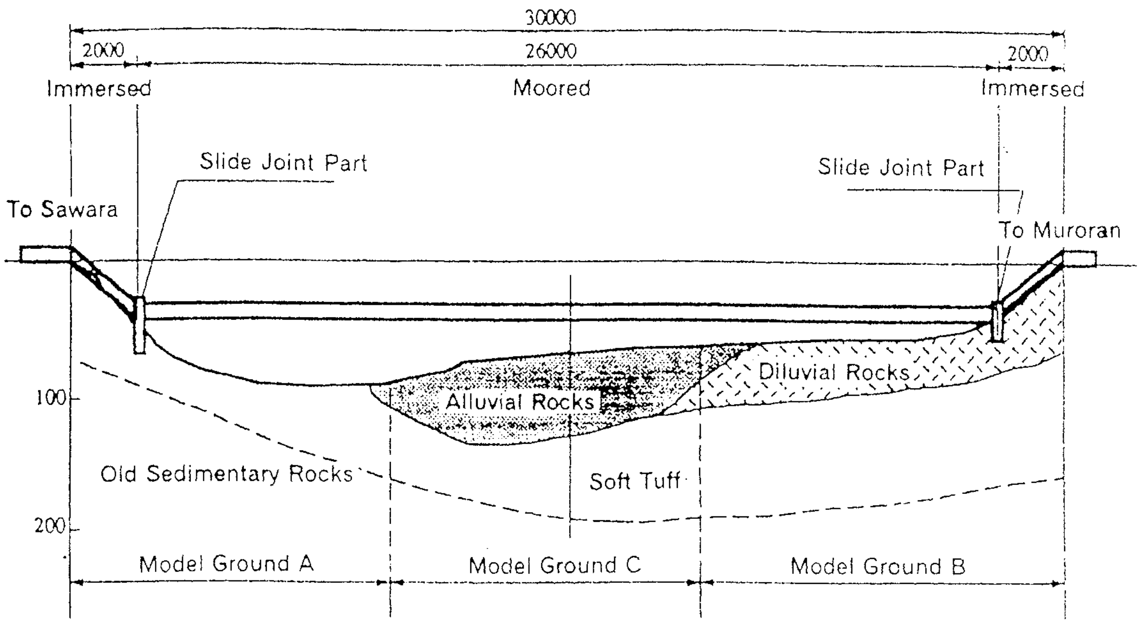


Fig.3 Longitudinal Section of the Funka Bay Submerged Floating Tunnel (unit: m)
(Shinkansen Single Track: Type A)

ie construction costs of the submerged tunnel with a Shinkansen-size double-track railway is estimated to be 500 billion yen, or 5 billion US dollars. It seems very expensive, but compared to the land purchase cost of other parts of Japan, it is not so expensive. As you know land in Japan costs so much and it seems extraordinary. The Japanese government estimates the cost for land acquisition for a new capital to be 5 trillion yen, or 500 billion US dollars.

Compared to parts of Japan, land in Hokkaido is much less expensive. To buy land in the vicinity of New Chitose Airport, mentioned before, will cost only 500 billion yen, or 5 billion US dollars. It is obvious that with 5 trillion yen, or 500 billion US dollars, we can make a submerged tunnel and construct the Shinkansen, and still there will be some money remaining. By using the submerged tunnel, traveling between the new capital in Hokkaido and Tokyo will be only 3 hours by Shinkansen. This is about the same as between Tokyo and Osaka today.

We selected a submerged tunnel rather than an undersea tunnel because of the following two reasons.

First, in the case of an undersea tunnel, the approach sections to the tunnel will be rather long and the tunnel cannot be accessed from the two cities located at the ends of the tunnel, namely Muroran and Sawara.

Second, this area is one of the major volcanic zones in Japan. There is a danger of hitting magma while digging a tunnel.

Fig.4 is a conceptual drawing of the new capital at completion. It will be contracted in harmony with the natural surroundings, disaster-proofed, and as a full-scale international political administrative city.

Materializing the submerged tunnel will be the key for Hokkaido to be selected tunnel. My fellow colleagues will explain to you the details of our research.

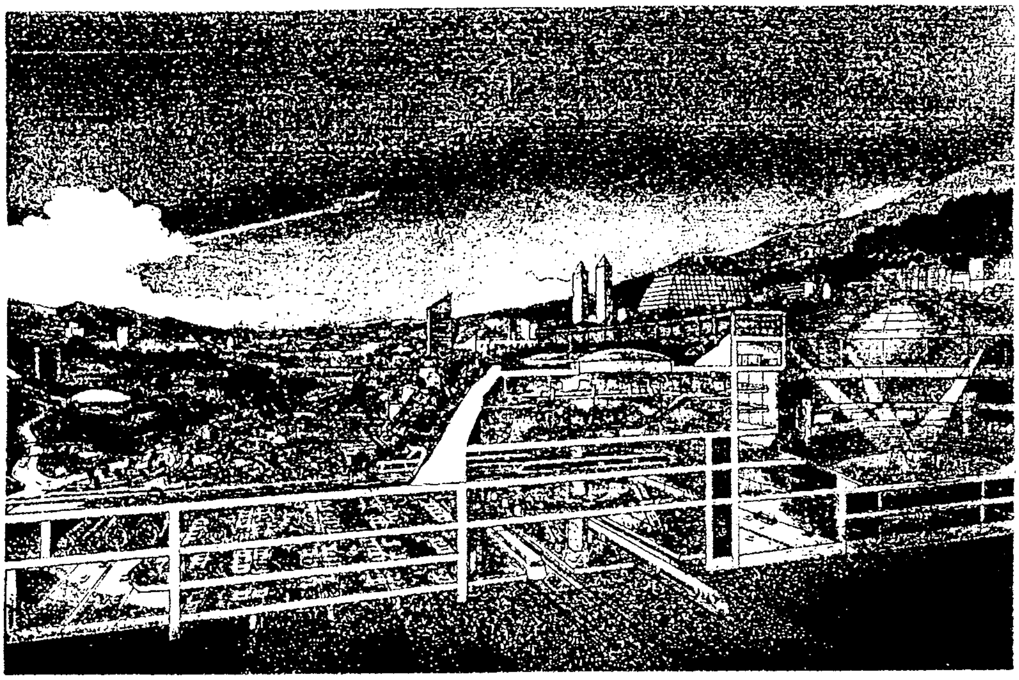


Fig.4 Conceptual Drawing of New Capital

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

3. Hokkaido Crossing

3.2 Summarize the Activity of the Society of SFT Technology Research in Hokkaido in Last Half Decade, and Next Step in Future

S. Kurita
Hokkaido Development Bureau

TITLE :
Summarize the Activity of The Society of Submerged Floating Tunnels(SFT) Technology Research in Hokkaido in Last Half Decade, and Next Step in Future.

Satoru KURITA
Port construction Division,
Port Department,
Hokkaido Development Bureau

1. The Summary of the Activities of the Study in Last Half Decade

1. History of The Society
   - a. The Society of Submerged Floating Tunnel Technology Research in Hokkaido was jointly inaugurated in 1990 by hokkaido's industrial, government and academic sectors.It is a public interest corporation and in 1992 was designated as a corporate juridical person.
   - b. Seven research subcommittees have been studying the establishment of comprehensive techniques for the planning, design, construction and other related factors regarding a submerged floating tunnel.
   - c. Having published the fruits of its research, in Japanese, "Submerged Floating Tunnel", in 1995, the Society is now proceeding to the next step, full scale verification tests.
2. Organization of the Society
   - a. the board of directors
   - b. the steering committee and the technical committee
   - c. A subcommittee is under the steering committee, planning
   - d. Seven subcommittees are under the technical committee, publish, study of structure, study of fluid force, study of material, study for soil condition of sea bed, construction, feasibility study , and study of maintenance and circumstance.
3. Members of the Society

a. Member of the Society are classified two categories, special and general member.
b. Special members are consisted of 10 governmental organizations and 2 universities.
c. General members are consisted of 66 private companies.

4. Contents of the Activities
    a. As the submerged floating tunnel is a unique structure, various analyses have been conducted concerning the tunnel itself, its mooring system and foundation.
    b. Thorough environmental impact assessments are underway.Driving simulations are being conducted to learn drivers' attitudes and reactions to the experience of using the SFT.
    c. When designing structures, particularly offshore facilities, it is necessary to examine through hydraulics model tests the dynamic behavior of fluid.
    d. Because of its unique structure, stability tests of the tunnel using a wave flume and basin, tension tests of mooring cables, repeated pulling tests of the pile foundation and various other tests have been conducted in conditions simulating various terrains.
    e. Furthermore, indoor tests in the sea for corrosion resistance and water tightness, and exposure tests in the sea have been conducted for various materials likely to be used in construction.

2. Introduction of Fruit

1. The Manual, "Submerged Floating Tunnel"
    a. The purposes of the manual are to pick problems up ,to sort them out and to summarize technical difficulty of them.
2. Summary of the Case Studies
    a. Main case was F-bay and two sub cases were studied.
    b. sty case F-bay - Next Speaker will explain this case in detail
        i. This case study is one of main activities in our Society. It was insert in the manual "Submerged Floating Tunnel".
        ii. This case study is the test design in deep water.

c. 2nd case M port

i. This case study is the test design in shallow water as a port facility for access.

d. 3rd case Lake T

i. This case study is also the test design without fluid forces.

3. Explanation M-port case in detail

a. This case study is the test design in shallow water as a port facility for access. Though the SFT has some problems to adopt in shallow water area, it has high potential as the facility for access in port areas.

b. The target root is between a pier and a small island in the entrance of port along the breakwater. This island is expected to use a center of marine leisure and recreation. The distance is about 900m. The length of SFT is 120m. The type of utilization is on foot.

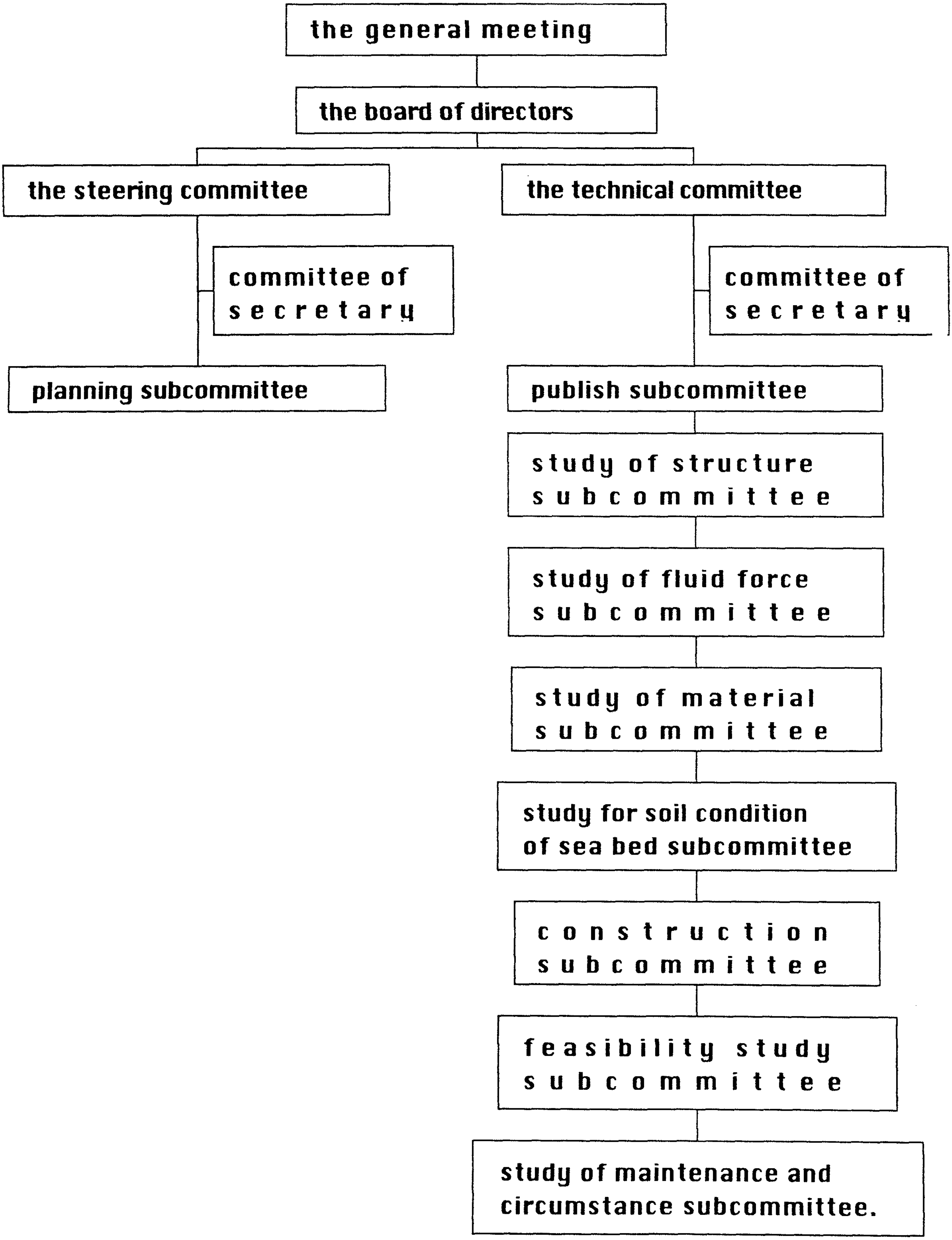
c. SFT is located at a narrow entrance for small ships and yachts.

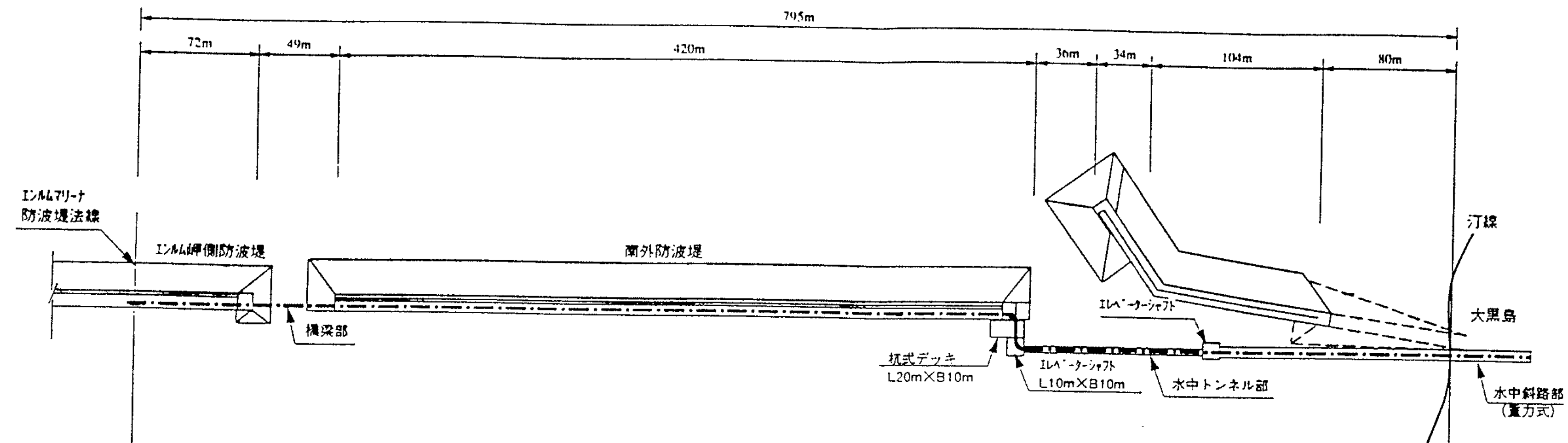
d. The diameter is 4m and the water depth is 10m. The both height and width of passage inside are 2m and a half.

3. The Next Activities in Future

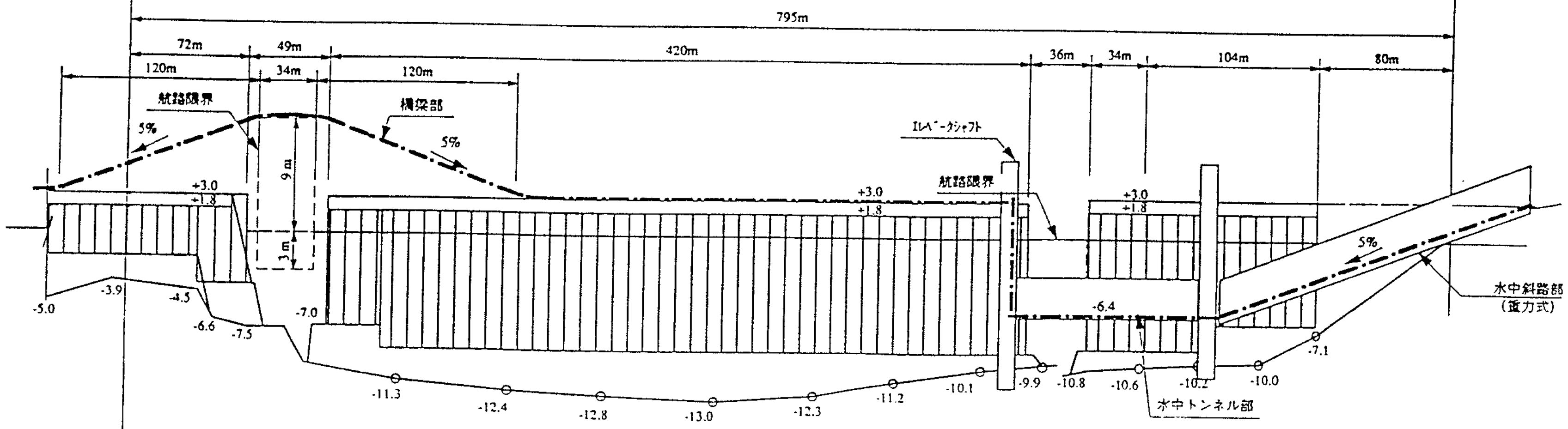
1. Preparation for full scale verification test on the basis of the end case study
2. Full scale verification test for shallow water area
3. Design of verification plant for shallow water area
4. Public relation for national and local government

ORGANIZATION CHART of THE SOCIETY



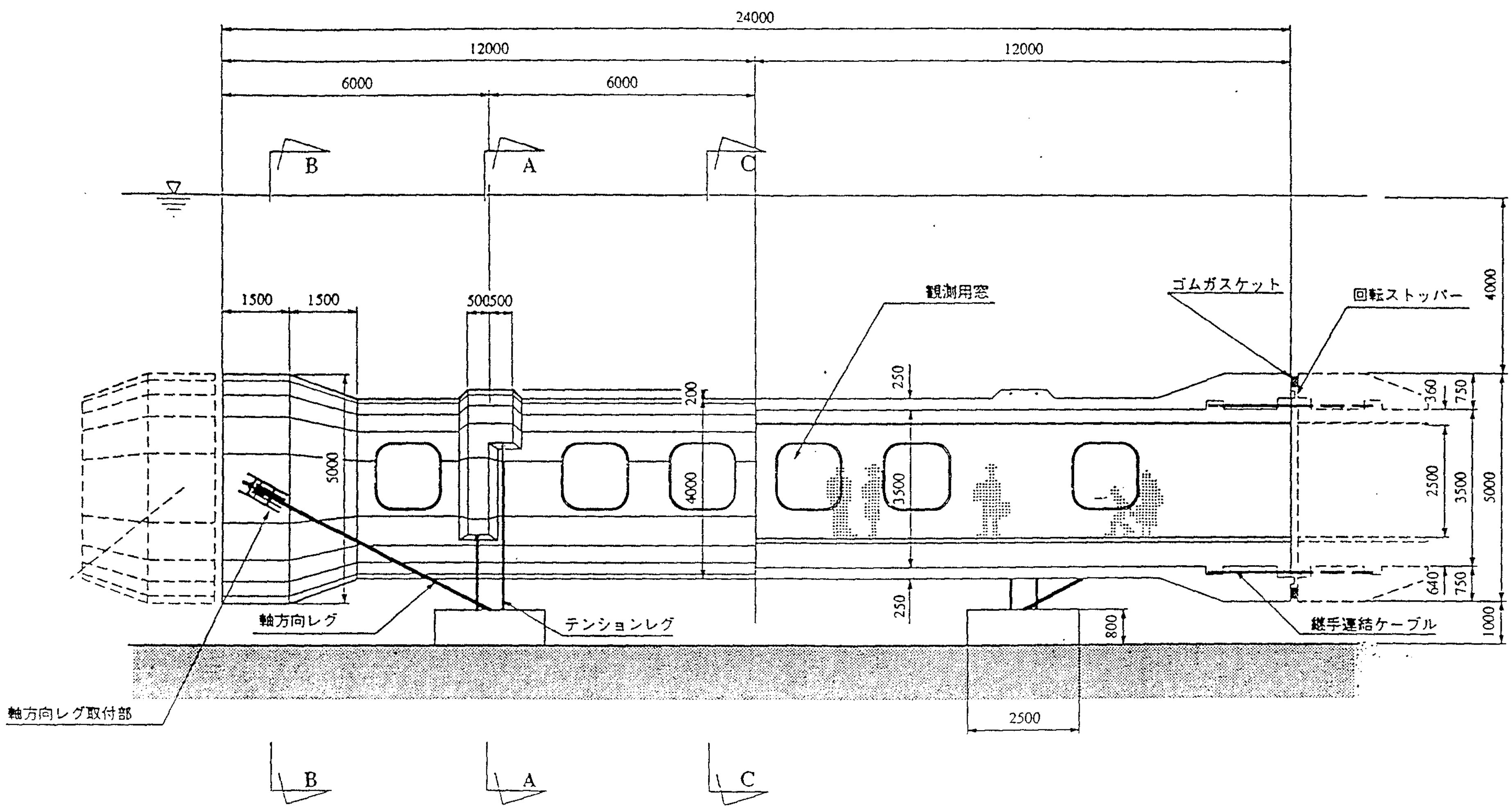


平面図



側面図

Case 4: 橋梁+水中トンネル案 (3)


24000
12000
12000
6000
6000
B
A
C
1500
1500
500500
観測用窓
ゴムガスケット
回転ストッパー
4000
250
200
360
750
5000
4000
3500
2500
3500
5000
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640
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軸方向レグ
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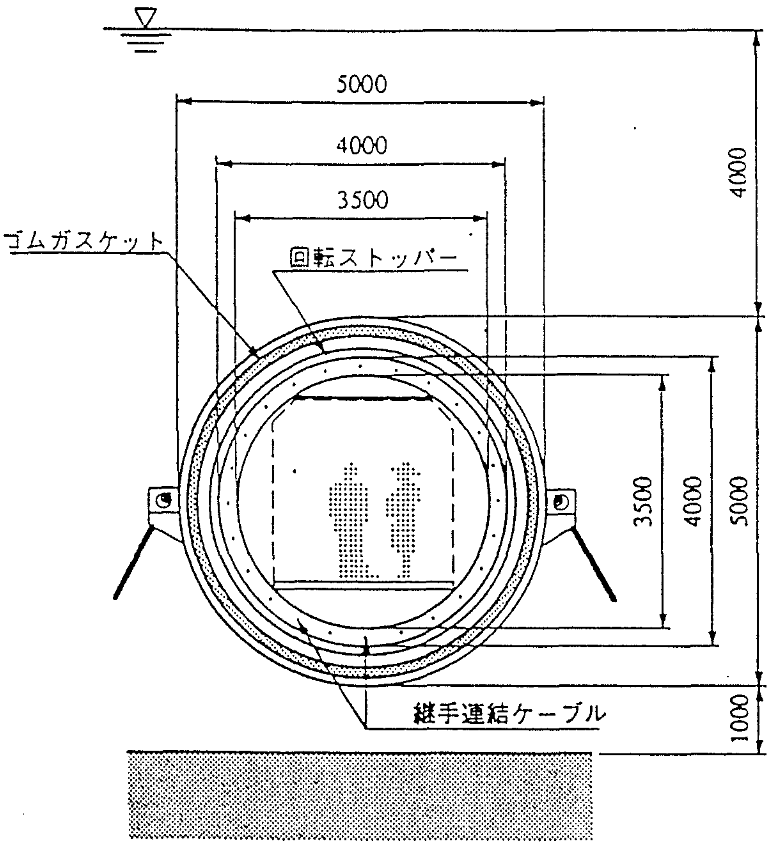
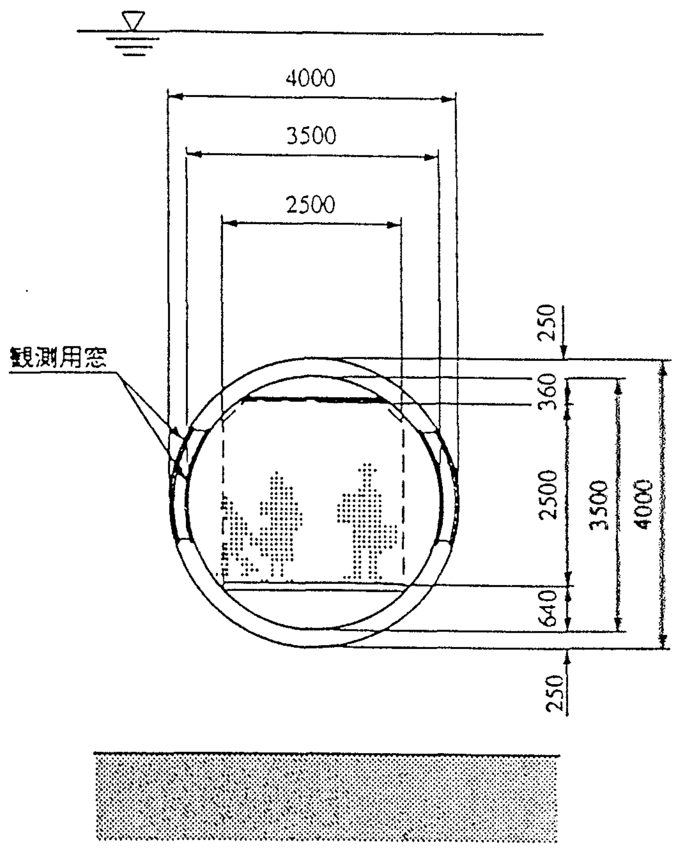
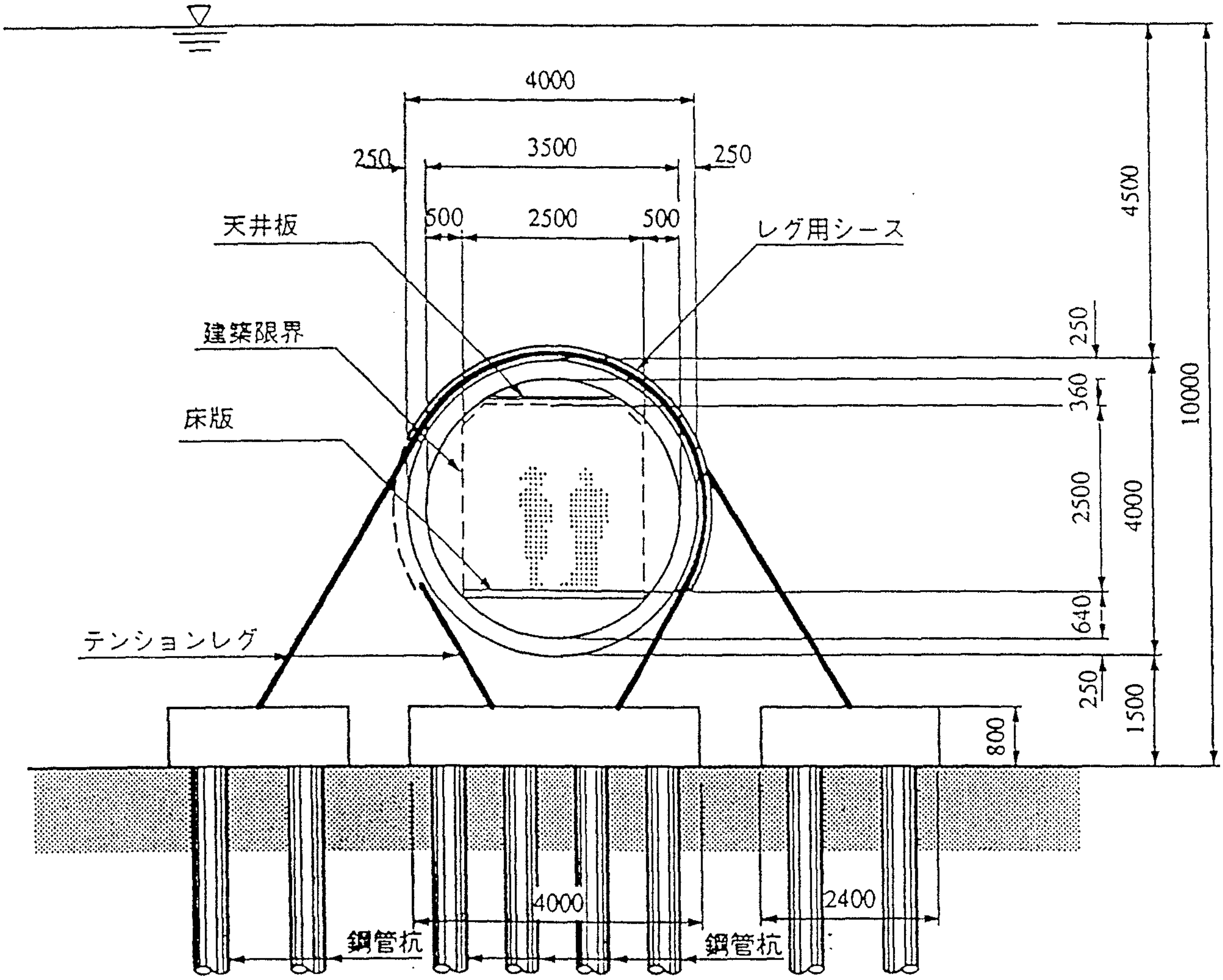
軸方向レグ取付部
2500



ジョイント部断面図（B-B）



一般部断面図（C-C）


4000
250
3500
250
500
2500
500
天井板
レグ用シース
建築限界
床版
テンションレグ
鋼管杭
鋼管杭
4000
2400
4500
10000
250
360
2500
4000
640
250
1500
800

レグ取付部断面図（Ａ－Ａ）

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

3. Hokkaido Crossing

3.3 Submerged Floating Tunnels Project in Funka Bay Design and Execution

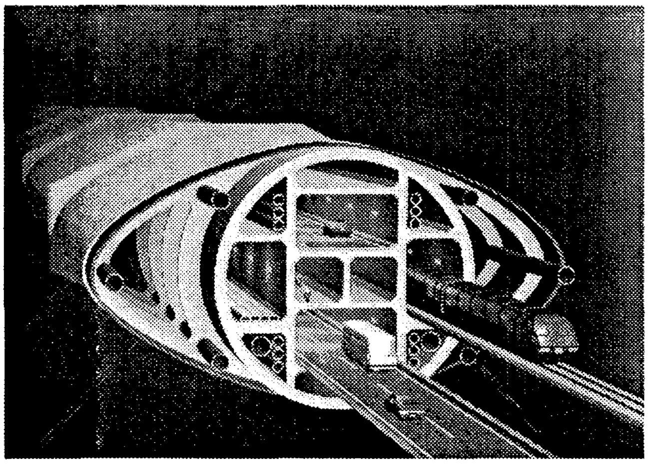
T. Fujii
Nishimatu Construction Co. Ltd.

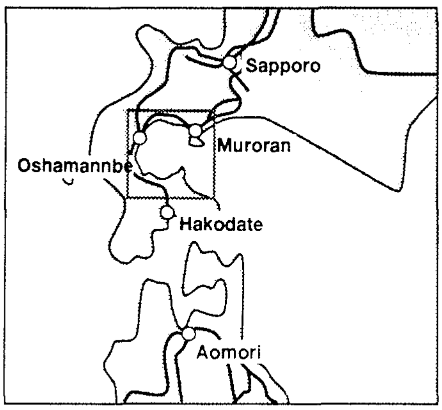
Submerged Floating Tunnels Project in Funka Bay Design and Excution

Toshiyuki Fujii

Nishimatu Construction Co.,Ltd. Tokyo,Japan

The case study is carried out as a project at Funka Bay in Hokkaido , Japan. Funka Bay , with 40km in it's diameter and 2,000km^2 in the area , is located in southern part of Hokkaido. Figure-1 shows the location and water depth of Funka-Bay. If the Funka Bay Project becomes reality, This can strengthen the axis of Hokkaido by connecting 3 big cities : - Hakodate , Muroran , and Sapporo. Gate length of Funka Bay is about 30km , and water depth is from 50 to 90m near the center. Seabed slope is steep at Sawara side , and modelate at Muroran side. In general , Funka Bay has more shallow and more modelate seabed slope than Fjord of Norway.





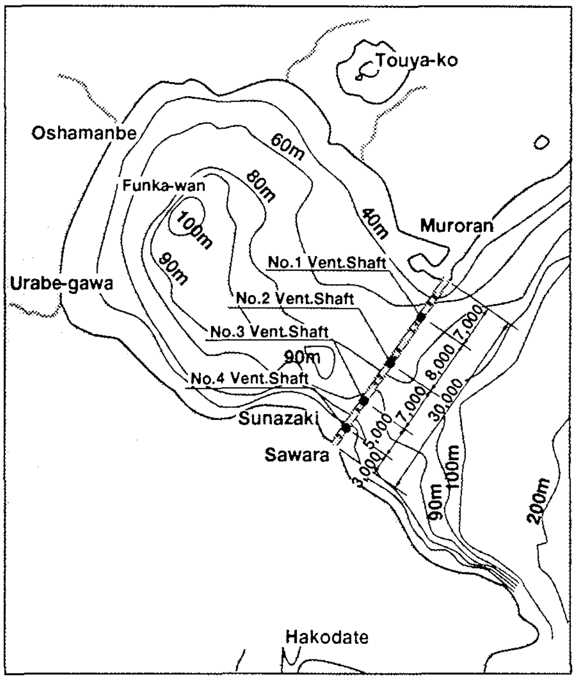


Fig.-1 Location and water depth of Funka bay S.F.T.

Fig.-2 shows the longtudinal Section of Funka-Bay S.F.T. . Structure of Funka Bay S.F.T. is finalized judging from water depth and ground conditions. Along 30km of Funka Bay S.F.T. in total length , bridge is adopted in a part of 7km of Muroran side because of shallow depth and high frequency of ship passing. Then pile-bed type S.T. is adopted in a central part of 8km of Muroran side and 3km of Sawara side because of shallow depth. And tension leg type S.F.T. is adopted in a central part of 12km because of deep depth and preferable ground conditions.

Clearance between sea water level and top of S.F.T. is determined as 30m for ships to be able to pass freely.

We plan four ventilation shafts on this route. Four shafts are sufficient for ventilation to accommodate expected traffic of road tunnel built on this route .

My report forcuses on the tension leg type S.F.T.

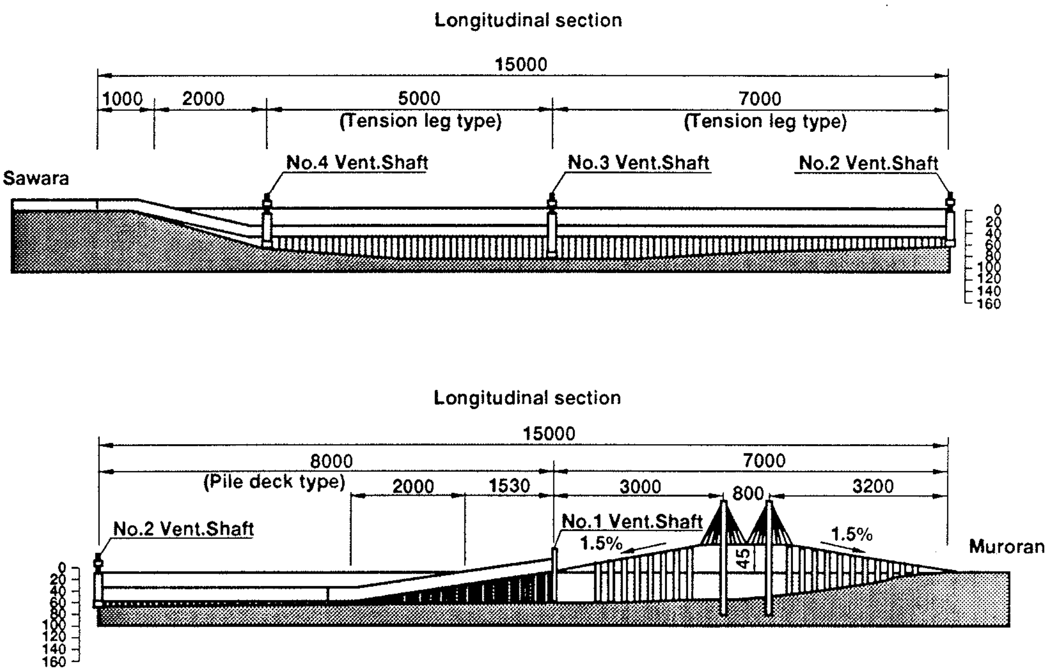


Fig.-2 Longitudinal section of S.F.T.

Figure-3 shows the design section of Funka-Bay S.F.T. . Section of Funka Bay S.F.T. is determined to provide a four lane road , a double track super-express railway , a emargency pass , a utility space and a ventilation space.

We adopt circular section for S.F.T. , which is most stable structure hydrostatically. As a result of our study , tunnel diameter is 23m with 1.0m - thick - outer - wall and 0.6m - thick - inner - wall. The tunnel is considered to be a concrete structure.

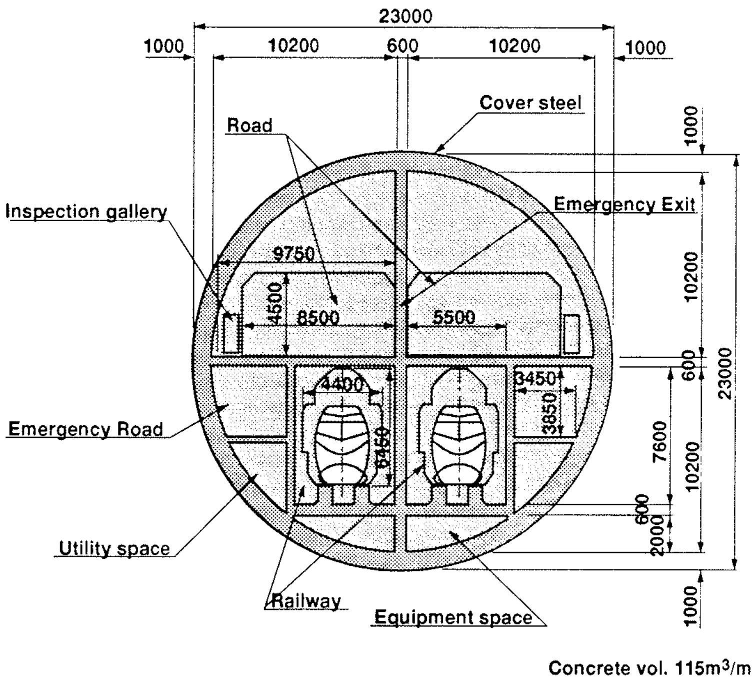


Fig.-3 design Section

Among natural conditions in the design of Funka Bay S.F.T. , wave and seismic conditions are most critical. As the other conditions , there are tidal current , tsunami and density current. But all of those are not decisive in the design of S.F.T. . Table-1 shows the design wave conditions to S.F.T. , and table-2 shows the design seismic acceleration to it.

For level 1 of ultimate limit state , we adopt a wave condition which has a return period of 145 years , a significant wave height of 9.7m and the maximum wave height of 19.4m. For level 2 , the return period is 950 years , and significant wave height is 24.4m. For serviceability limit state , a return period is 2 years , significant wave height is 5.2m , and the maximum wave height is 10.4m.

Design seismic acceleration of 240 gals is adopted in horizontal direction and 80gals in vertical direction for ultimate limit state - level 2 , 120 gals in horizontal direction and 40 gals in vertical direction for ultimate limit state - level 1. Those conditions are based on the result of seismic occurrence analysis in the Funka Bay area. And the seismic spectrum is presumed from the past data which have been recorded in this area.

Table-1 Design wave condition

Limit Condition	Ultimate Limit Condition		Serviceability Limit State
	Level1	Level2	Serviceability
Return Period(Year)	950	145	2
Significant Wave Piriod(m)	11.7	9.7	5.2
Max.Wave Hight(m)	23.4	19.4	10.4
Wave Period(Sec)	13.0	13.0	10.0

Table-2 Design Seismic Acceleration condition

Limit Condition	Ultimate Limit Condition	
	Level1	Level2
Return Period(Year)	950	145
Horizontal(gal)	240	120
Vertical(gal)	80	40

We employed two different types of tension leg arrangements in the design of Funka Bay S.F.T. A type has vertical two tendons , and C type has four diagonal tendons. Figure-4 shows the anchoring system of Funka Bay S.F.T. .

Diagonal angle of C type is 30 degrees , A type shows large horizontal displacement , but snapping hardly occurs , and it is economical. C type allows only very small displacement , but snapping is more likely to occur and it is not economical.

Therefore , we consider that C type is suitable only for the portion close to ventilation shafts , and A type is good for the other ordinary parts.

The length of tunnel unit is finalized as 200m , because two tension legs for one tunnel - unit are more beneficial for stabilization and constructability.

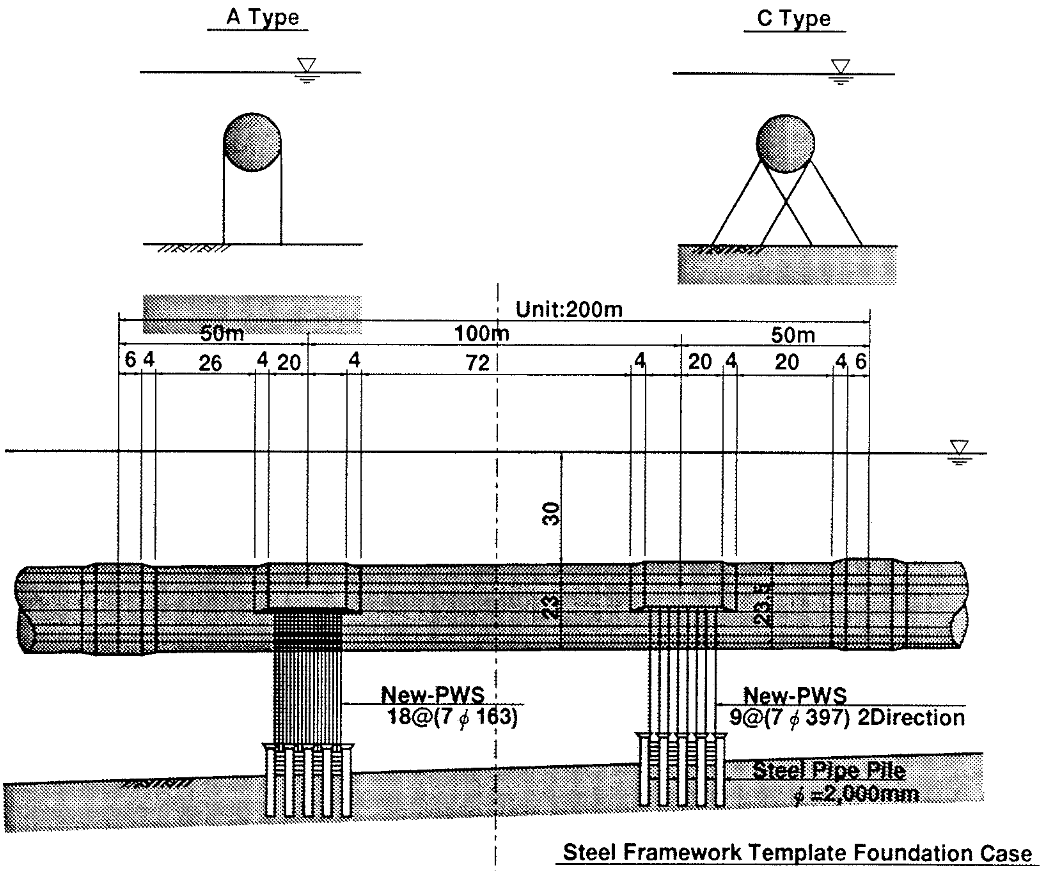
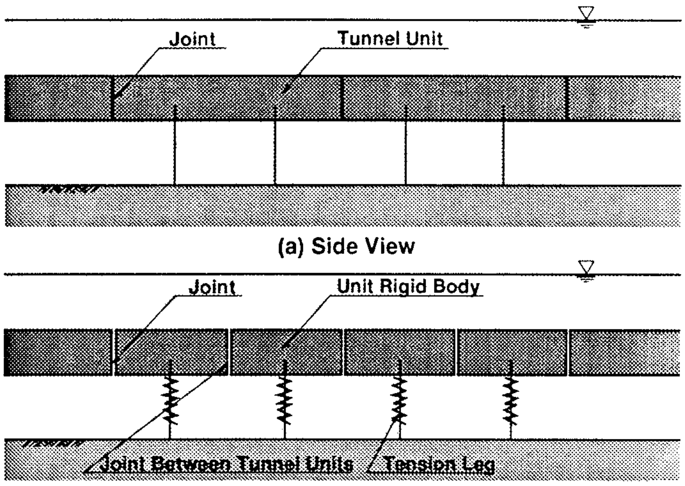


Fig.-4 Anchoring System

In the design of S.F.T. system we have to consider dead loads , live loads , wave load , seismic load , tidal current load and tsunami-wave load. This paper says the design of wave load and seismic load because these loads are most dominant in the design of Funka Bay S.F.T.

In the design of wave load , we modeled the S.F.T. as continuous rigid bodies with elastic springs equivalent to the restoring force by tension leg. So the whole length is 7km between two ventilation shafts. The S.F.T. is composed of 200m long tunnel units which are connected with rolling spring between themselves. Figure-5 shows the wave resistant analysis model of S.F.T. .

In this design , we change the incidental wave angle from 0 to 90 degrees. Analysis conditions are shown in the table-3 , which are determined by the pre-study. Wave height is also shown in the table-2 , but wave period is assumed as regular one.



(b) Analysis Model

Fig.-5 Wave - resistant Analysis Model

Table-3 Analysis Condition

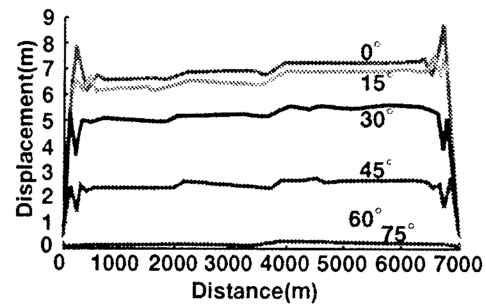
Rigid Body	Mass	3367(ton)
	Rotational Inertra Moment(x-axis)	2.8×10^6(ton-m^2)
	Rotational Inertra Moment(y-axis)	8360(ton-m^2)
	Rotational Inertra Moment(z-axis)	2.8×10^6(ton-m^2)
Joint	Shear Spring	1.01×10^6(tf/m)
	Rotational Spring	0(tf-m)
	Axial Spring	3.57×10^6(tf/m)
Joint Between Tunnel Units	Axial Spring	1.01×10^6(tf/m)
	Axial Spring	1.30×10^6(tf-m)
	Axial Spring	3.57×10^6(tf-m)

(Unit Tunnl Length=200m)

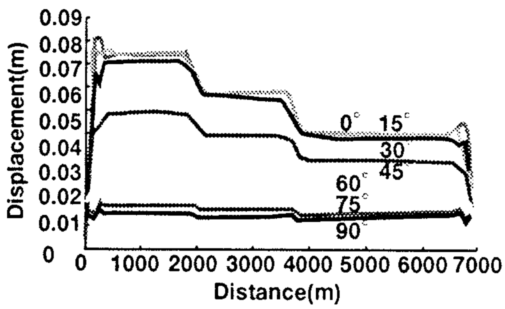
The result of analysis of wave resistance shows that horizontal deflection and acceleration , and vertical deflection and acceleration are maximum when incidental wave angle is zero.

The figure-6 shows the result of analysis of A type between No.2 and No.3 ventilation shafts in the case of ultimate limit state - level 2.

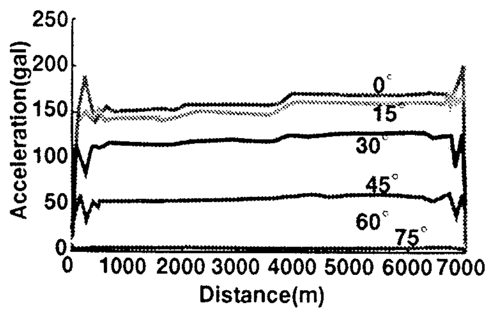
The maximum horizontal deflection is about 8m , and the maximum horizontal acceleration is about 180gal.



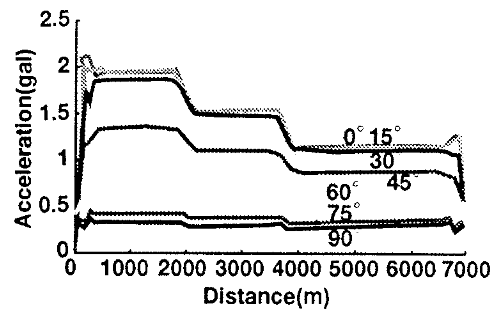
Max.Horizontal displacement (Ultimate Limit Stale Level 2)



Max.Vertical Displacement (Ultimate Limit Stale Level 2)



Max.Horizontal Acceleration (Ultimate Limit Stale Level 2)



Max.Verticall Acceleration (Ultimate Limit Stale Level 2)

Fig.-6 Design results of wave resistance

The table-4 shows the result of serviceability limit state by using the same analysis model and method as in the case of ultimate limit state analysis.

The maximum horizontal deflection is 155.2cm , and the maximum horizontal acceleration is 61.9gal respectively. All of those have satisfied both Japanese Standard and International Standards.

Table-4 Checking on Displacement & Acceleration

Legend	Checking State	Direction	displacement			Acceleration		
			Allowable Value	Observed Value	Judge	Allowable Value	Observed Value	Judge
Railway	Serviceability	Horizontal	170cm*1	155.2cm	O.K	100gal*2	61.9gal	O.K
		Vertical	7.7cm	3.9cm	O.K	130gal*2	1.6gal	O.K
Road	Serviceability	Horizontal	——	155.2cm	——	82gal*4	61.9gal	O.K
		Vertical	——	3.9cm	——	125gal*4	1.6gal	O.K

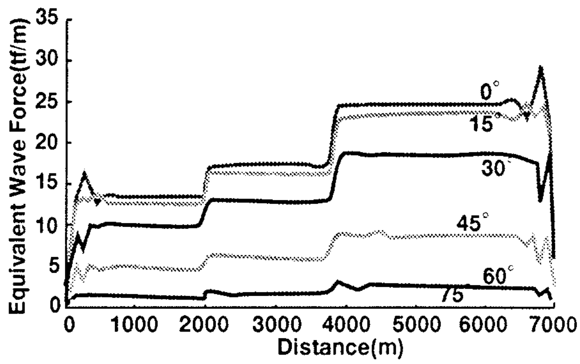
*1 It is based on critical state of Rapid Railway service in Japan.
Horizontal frequency 0.1(=1/10 sec) is assumed.

*2 Those are based on the SPEC for Rapid Railway Service in Japan.

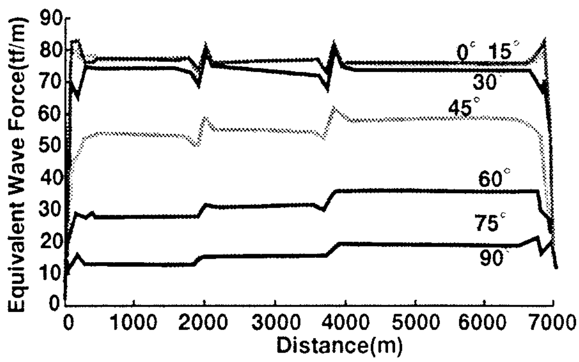
*4 Allowable limit for human body against vibration defined by ISO is applied.
Exposed time 1hour to go through the tunnel is assumed.
Freguency is not specified.

The figure-7 shows the maximum horizontal and vertical wave forces of the S.F.T. in the case of ultimate limit state - level 2 under the same analysis against deflection. The table-5 shows a summary of these results.

In the case of two-dimensional analysis , we employ the modified Mollison's equation to obtain wave force in the design of S.F.T. . Both results are very similar . We have reached a conclusion after our experimental and analytical study that the modified Mollison's equation can provide wave forces with sufficient accuracy.



Horizontal Max. Equivalent Wave Force
(Ultimate Limit State Level 2)



Vertical Max. Equivalent Wave Force
(Ultimate Limit State Level 2)

Fig.-7 Equivalent wave force of S.F.T.

Table-5 Checking on Design wave Force
(Ultimate Limit StateLevel 2)

Return Period	Direction	2D-analysis (tf/m)	3D-dynamic analysis(tf/m)
950years	Horizontal	28.1(③section)	30.6(③section)
	Vertical	80.5(①section)	82.7(①section)
145years	Horizontal	23.3(③section)	25.3(③section)
	Vertical	66.8(①section)	68.6(①section)

In the design of S.F.T. for wave , it is one of the most important point that neither slack nor snap occurs in tension leg. To satisfy this condition , we determined the conversion gravity of tunnel and the pretension force of tension leg for A type as 0.77 and 5,000tf respectively. They are 0.51 and 6,030tf for C type respectively. The check results have proved that Funka Bay S.F.T. is sufficiently safe.

Tension Force Range of Tension Leg

Ultimate Limit State Level 2 : $0 \leqq T \leqq Ty$

Ultimate Limit State Level 1 : $0.2 \leqq Ti \leqq Te$

Ty : Yield Limit Tension
Te : Elastic Limit Tension

Type	Specific Gravity	Initial Tension Force (Ti)
A	0.77	5000 tf
C	0.51	6030 tf

Result of Ultimate Limit State Level 2

Max. Tensile Force : Twv = Ti + Fwv × 100m / 2nos
= 5,000 tf + 82.7 × 100/2
= 9,140 tf < Ty = 13,400 tf

Min. Tensile Force : Twv = Ti - Fwv × 100m / 2nos
= 5,000 tf - 82.7 × 100/2
= 865 tf > 0

Fig.-8 Check results to tension force of tension leg

In the design for seismic condition , the same analysis model and method are employed as in the design against wave . Table-7 shows seismic condition of Funka Bay S.F.T. and Figure-9 shows design seismic acceleration observed at ground surface. We analyzed dynamically the section forces of tunnel along its axial direction and at joint locations.

Table-7 Seismic Condition

Limit State	Ultimate Limit State	
	Level2	Level1
Return Period	950 years	145 years
Max. Horizontal Acceleration	189 gal	121 gal
Max. Vertical Acceleration	53 gal	35 gal

Input Seismic Acceleration for Dynamic Response Analysis.

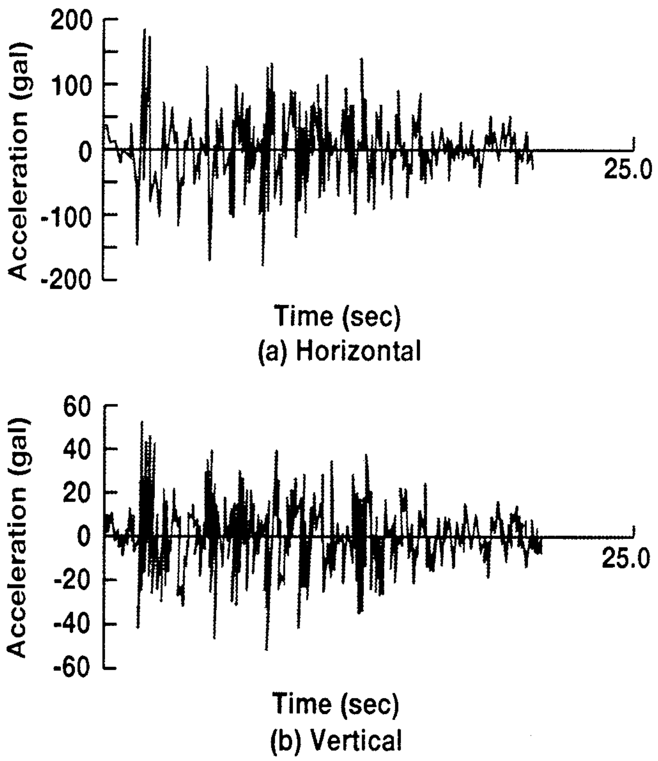
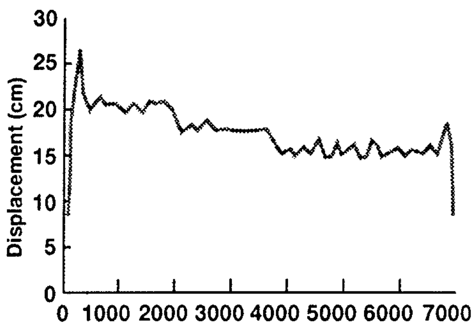
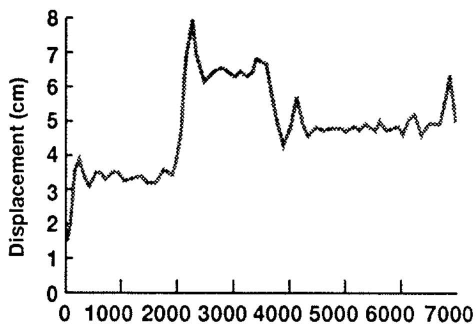


Fig.-8 Design Seismic Acceleration
Observed at Ground Surface
(Ultimate Limit State Level 2)

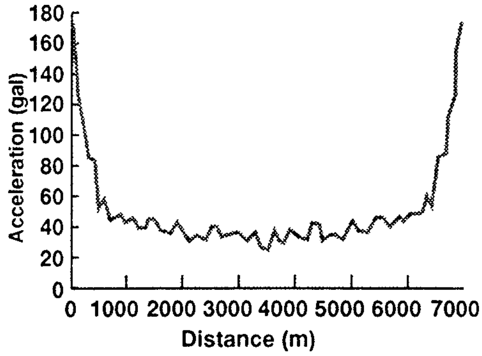
The figure-10 shows the result of analysis in the case of ultimate limit state - level 2. Horizontal deflection is between 12 and 26cm , horizontal acceleration is between 40 and 180 gal. Vertical deflection is between 3 and 8cm , and vertical acceleration is 120 gal.



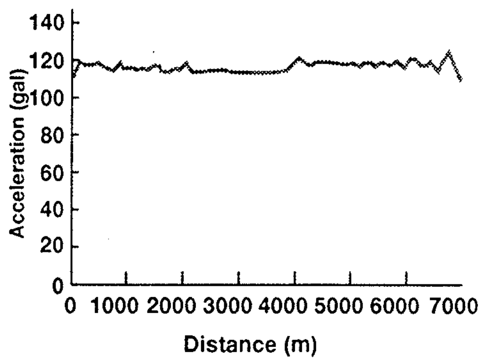
Max. Horizontal Displacement
(Ultimate Limit State Level 2)



Max. Vertical Displacement
(Ultimate Limit State Level 2)



Max. Horizontal Displacement
(Ultimate Limit State Level 2)



Max. Vertical Displacement
(Ultimate Limit State Level 2)

Fig.-10 Displacement of S.F.T. in seismic condition.

The table-8 shows the check results of deflections and accelerations. The standard for deflection and acceleration in the case of earthquake has not been established , but we made a judgment from the standard for safety operation of Japanese Shinkansen .

Table-8 Checking on Displacement & Acceleration

Legend	Checking State	Direction	Displacement			Accelaration		
			Allowable Value	Observed Value	Judge	Allowable Value	Observed Value	Judge
Railway	Service-ability	Horizontal	27cm *1	26.5cm	O.K	100gal *2	172.3gal	N.G
		Vertical	7.7cm	7.6cm	O.K	130gal *2	111.3gal	O.K

* 1 : It is based on critical state of Rapid Railway Service in Japan.4Hz (T = 0.25 sec) of Horizontl frequency is applied according to the predominant frequency of earthquake.
* 2 : Those are based on the SPEC for Rapid Railway Service in Japan.

The figure11 shows the maximum horizontal and vertical forces of tension leg in the case of earthquake.

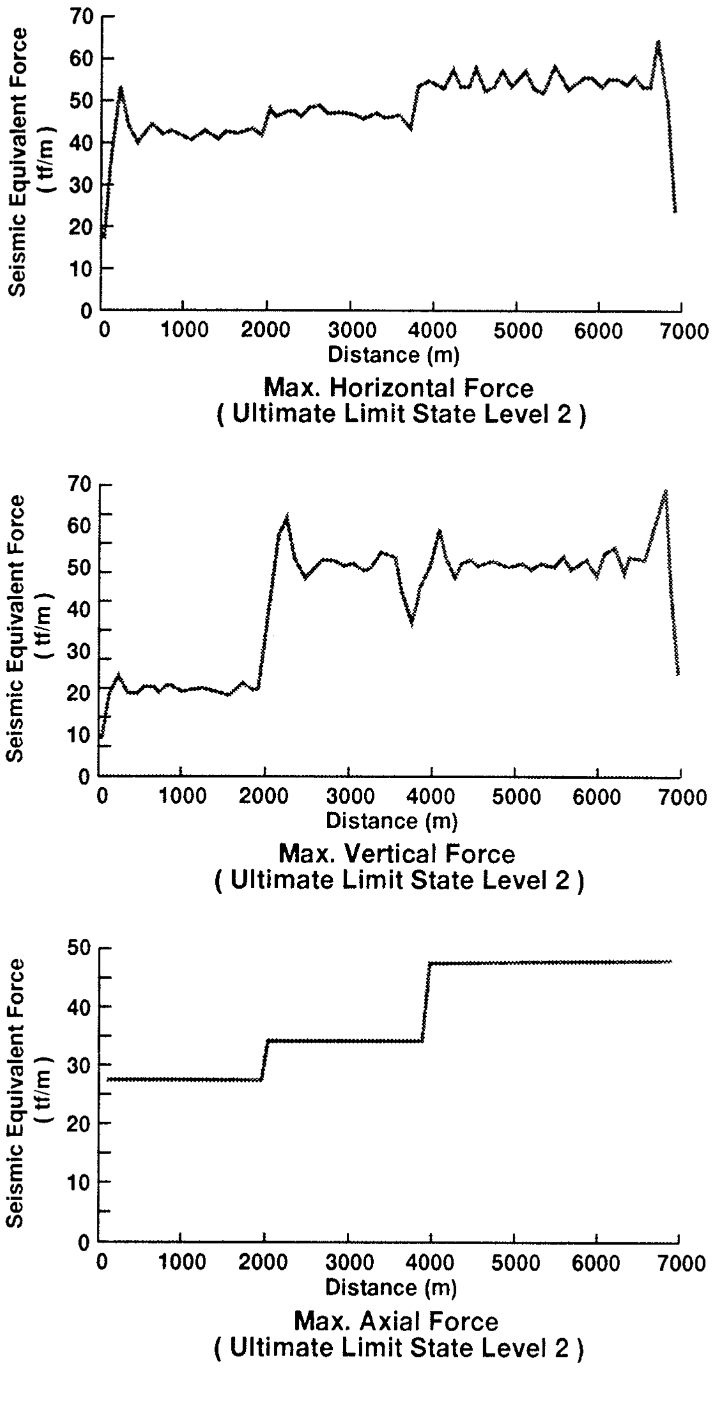


Fig.-11 maximum forced tension leg

The table-9 shows a summary of check results against seismic load. The results of two-dimensional model in the table are obtained through a frequency domain analysis using a modified spring-mass model. In this analysis , seismic hydro-mechanical load is computed by using the modified Mollison's equation , and acceleration of water particle is computed by using duplicate reflection theory to take into account compressibility of fluid.

The results by three-dimensional model show larger loads than those by two-dimentinal one. Similar to the check against wave load , we make sure that neither snap nor slack occurs in legs in order to keep the safety of S.F.T.

Table-9 Checking on Design Seismic Condition

Return Period	Direction	2D-Analysis (tf/m)	3D-Dynamic Analysis (tf/m)
950 years	x	2.4 (①section)	0.7 (①section)
	y	0.8 (③section)	0.5 (③section)
	z	67.0 (②section)	88.7 (②section)
145 years	x	1.6 (①section)	0.4 (①section)
	y	0.4 (③section)	0.3 (③section)
	z	44.2 (②section)	58.5 (②section)

Checking on Snap & Slack of Tension Leg

Max. Tensile Force : $T_{ev} = T_i + F_{ev} \times 100\text{m} / 2\text{Legs}$

$= 5000 + 88.7 \times 100 / 2$

$= 9440 \text{ tf} < T_y = 13440 \text{ tf}$

Min. Tensile Force : $T_{ev} = T_i - F_{ev} \times 100\text{m} / 2\text{Legs}$

$= 5000 - 88.7 \times 100 / 2$

$= 565 \text{ tf} > 0$

Tension leg and its foundation to fix the tension leg into ground are designed against pre-tension force , wave load and seismic load to S.F.T.

Design load of A type of Funka Bay S.F.T. is shown in the table-10. Initial tension of leg is 5,000tf , maximum design force is 9,000tf in the case of level 2 and 8,340tf in the case of level 1. The table-11 shows the check results of level 1. We have found that tension legs are safe against fatigue limit state by using the spectrum method. We have also found that repeated stress creates a critical condition.

Table-10 Design Load for Tendon & Foundation

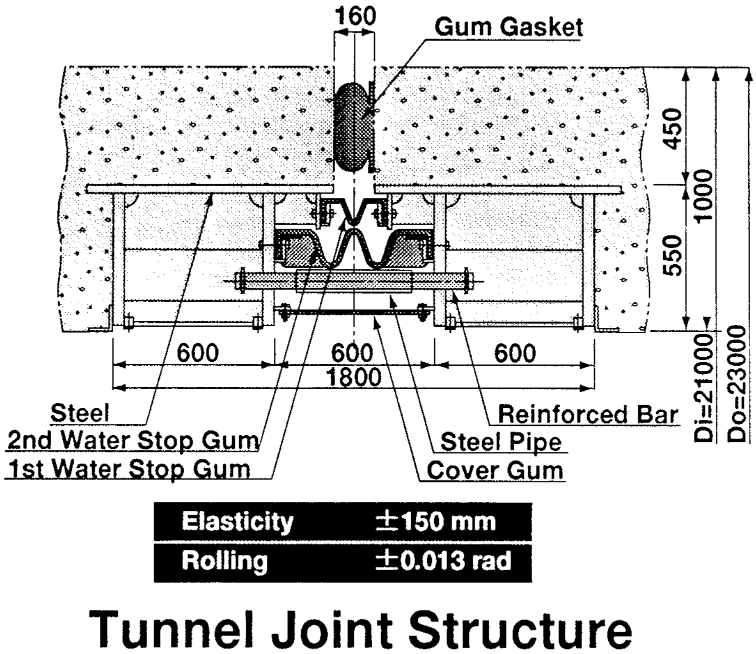
Loading Case *1			$\boxed{1}$ Pre tension	$\boxed{2}$ Live Load	$\boxed{3}$ Wave	$\boxed{4}$ Earth-quake	Total max.	Total min.
Ultimate	Level2	$\boxed{1}+\boxed{3}$	5,000		±4,025		9,025	975
		$\boxed{1}+\boxed{2}+\boxed{3}+\boxed{4}$	5,000	-239	0	±3,350	8,350	1,411
	Level1	$\boxed{1}+\boxed{3}$	5,000		±3,340		8,340	1,660
		$\boxed{1}+\boxed{2}+\boxed{3}+\boxed{4}$	5,000	-239	0	±2,210	7,210	2,551

*1 : All the case shown above is the one of vertical displacement

Table-11 Checking on Level 1

Section Force	Tmin (tf)	2,551
	Tmax (tf)	8,340
Structural analysis force	ν a	1.00
Design Section Force	Tdmin = Tmin / ν a (tf)	1,660
	Tdmax = Tmax / ν a (tf)	8,340
Section Resistance	Te (tf)	10,752
Material Factor	ν m	1.00
Member Factor	ν b	1.15
Design Section Resistance	Tde = Te / ν m / ν b	9,350
Structural Factor	ν i	1.00
Initial Tensile Force	Ti (tf)	5,000
Checking Equation	0.2Ti ≦ Tdimin (tf)	1000 < 1,660
	ν i·Tdmax / Tde ≦ 1.0	0.892 < 1.0
Judge		O.K

We employ a flexible joint in order to absorb deflections of S.F.T. when wave load or sensmic load is working. Figure-12 shows tunnel joint structure of S.F.T. and Waterstop gum gusket used at joints.



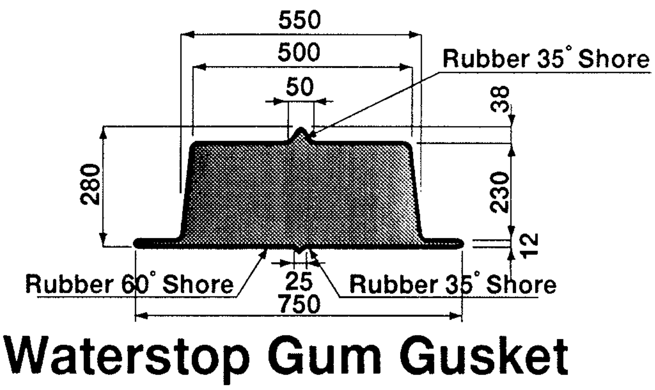


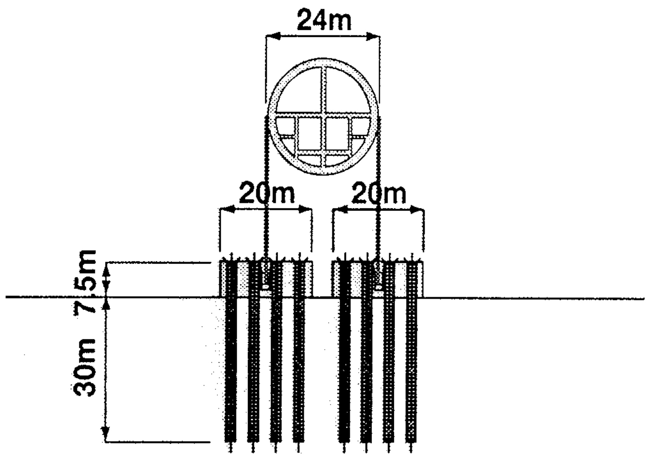
Fig.-12 Joint structure and gum gusket

Both pile foundation and ground anchor foundation can be applied. The figure-13 shows the case of pile foundation.

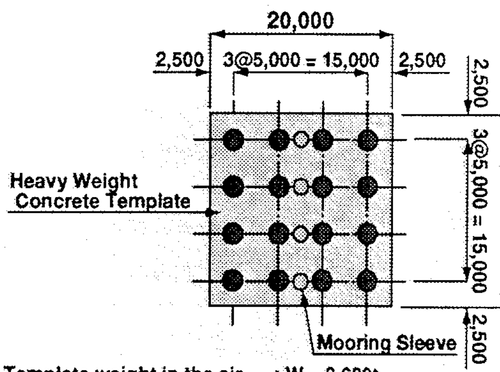
16 nos. of steel pipe pile , 2,000mm diameter and 30m long, have to be provided for one foundation of Funka Bay S.F.T.In this design , we take into account both non-linearity of ground and grouping effect of piles.

Dimension of Pile Foundation

Template	Steel Frame Work + Heavy Weight Concrete $20m^B$ x $20m^L$ x $7.5m^H$
Piles	Steel Pipe Piles ϕ2000 x 30^t L=30m



Side View



Template weight in the air : W = 8,600t
Template weight under water : We = 6,100t

Plan

Fig.-13 Foundation of S.F.T. (Pile foundation type)

In the case study of Funka Bay S.F.T. , we investigated construction method , whole construction schedule , and cost estimation.Those are shown in table-12. It is sufficiently possible to achieve this project by using the present construction techniques that have been established through construction of many submerged tunnels , large sea-bridges and man-made islands.Whole construction schedule of Funka Bay S.F.T. is 14 years , and total cost is estimated as 2,000 billion yen.

Table-12 Schedule and cost of Funka - Bay S.F.T.

Schedule to Complete	**14ys**
Total Cost	**2,000 Bil. yen**
(SFT	**62mil yen / m)**

Finally , some important points to realize this project are shown in table-13.

First, assessment of sea environment : In Japan , water depth in littoral zone is shallower than those in Norway. Therefore , the influence by construction of S.F.T. is more significant. We have to assess this point.

Second , establishment of maintenance technique : Damage of tunnel or tension leg will cause a serious problem . We have to consider the effective system to check and repair S.F.T.

Third , cost : This is one of the most important points to realize this project.

Forth , consensus of society : We need consensus on:-Why we have to realize the project , why the project should be in Funka Bay , why S.F.T. system is employed , and etc.

Fifth , as one of the technical points : deflection and bearing capacity of ground due to repeated load. Our case study is insufficient on this point .

Sixth , establishment of design system : We have to study much more deeply the load factors , factors of material strength and etc.

Table-13 Important points to realization

1. **Assessment of Sea Environment**

2. **Establishment of Maintenance Technique**

3. **Cost**

4. **Consensus of Society**

5. **Technical Point : Deflection and Bearing Capacity of Ground due to Repeated Load**

6. **Establishment of Design System**

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

3. Hokkaido Crossing

3.4 An Application of Ecological Growth Model to the Environmental Impact of the Construction of SFT

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An Application of Ecological Growth Model to the Environmental Impact of the Construction of SFT

Seiichi Kagaya

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1. INTRODUCTION

The environmental problems originating from marine construction projects are quite significant and require adequate consideration. These problems can perhaps be best analyzed from an ecological point of view. The sea is the habitat of many living organisms which form a biotic community. Each organism interacts with its environment and with other organisms. The interaction with other organisms leads to competition and predatory relation [Nijkamp 1977].

This study aims to examine the environmental impacts of the construction of a long submerged floating tunnel across the bay. This study has been motivated by the apparent scant data and research works on the likely environmental consequences of such a large-scale marine project. The main concern is with the development of techniques to predict likely environmental impacts. The main premise is that the construction of the submerged floating tunnel across the bay will result in changes in tidal patterns. These changes may initially affect the immediate surroundings of the projects site but eventually they are likely to have widespread effect.

Ideally, these changes may be advantageous to some organisms but disadvantageous to others. From this viewpoint, we propose an ecological model for the environmental assessment. Our model represents an ecological growth process including a self-organization and some influences of environmental change in some niches.

The simulation technique was used as a major tool for this study. The main procedures are summarized in Figure 1.

Collect the information on the characteristics of the sea area

↓

Predict the upwelling with/without the strait crossing structure(tunnel)

↓

Predict the nutrient or sediment behavior

↓

Predict the generation of plankton

↓

Predict the marine living products

Fig.1 Procedure of this study

2. ECOLOGICAL GROWTH MODEL AND ITS APPLICATION

2.1 Sea current within a bay

In natural upwelling of the sea the seabed water containing the rich nutrient is supplied to the upper layered fluid receiving the sunlight. Thus, the basic production capacity in the sea is improved. This area is, therefore, available as the fishing ground.

In this study, we predict the upwelling phenomenon generated by the submerged structure which is like a floating tunnel. First of all, we examine the sea current in the bay using multi-level density flow model which is applicable to the stratified flow fields. Next, we predict the effect and influenced area by the simulation. This approach is based on the estimation the effects of the submerged floating tunnel which would generate the upwelling phenomenon.

We can obtain various results by altering the parameters of the condition. Thus it is necessary to prove the reliability of the results by using experimental data or actual indicated data. This approach has already been validated by other cases and their simulation results [Fujiwara et al.]. In this paper we only used the results of this model simulation as the inputs to the following ecological model.

2.2 Concepts of an Ecological Model

There are both logical and operational reasons for studying phenomena at the level of the ecosystem. Modeling is done to aid the conceptualization and measurement of complex systems and,sometimes, to predict the consequences of an action that would be expensive, difficult, or destructive to do with the real system.

The modular generally starts with a conception of what the important parts in this system are and some idea of how these parts interconnect. At this stage the development of the model is heavily dependent on the modeler's experience and intuition. In general, a knowledge of what the important components and interactions are may be obtained directly from field experience or earlier studies on other systems [Hopkinson Jr. et al. 1977] .

The relations are used to construct difference or differential equations that predict changes in the system through time. Analytical and numerical techniques for solving difference and differential equation models are examined[Shoemaker 1979].

2.3 Mathematical model of ecosystem

One of the first steps in modeling an ecological system is to define variables. Those states or conditions of an ecological system are called state variables[Haberman 1977]. Examples of state variables include nutrient concentrations and population densities. The symbol Z_i denotes the i-th state variable. State variables referring specifically to numbers of individuals are denoted by N_i [Zeiglar 1979].

A model that has more than one state variable is called multidimensional. If there are N state variables, they can be referred to as $Z_i, i=1,....,N$. A multi-dimensional model of an ecosystem includes state variables describing both the abiotic and biotic components of the system.The rate of movement of materials or biomass between factors described by state variables of a system are called flow rates. The movement of individuals from one age class to another within a population is also a flow rate. The symbol, G(i,j), denotes the flow rate of material from Z_j.

In a system consisting of a single variable Z_1, the only flow rate is G(1,1). If Z_1 is a population, then G(1,1) is the net increase in the population biomass. The net increase is the net growth rate r per individual times the number of individuals. If r is a constant, then

$$G(1,1)= r \cdot Z_1 \qquad (2.1)$$

In many cases r, the net growth rate per individual, decreases as the population size Z_1 increases because of increased stresses from crowding, food shortages, or limited reproductive sites.
In many cases, data have indicated that the decrease in the net growth rate can be modeled by replacing the constant growth rate r with the term

$$r'= r((K-Z_1)/K)=r(1-Z_1/K) \qquad (2.2)$$

The constant K is called the carrying capacity. It is the maximum population which the environment can support. The net growth rate r' is zero when the population density equals the carrying capacity. However, when Z1 is much smaller than K, r' is approximately equal to the maximum net growth rate r. Substituting r' for r in Eq.2.1, we obtain

$$G(1,1)=r' Z_1 = r Z_1((k-Z_1)/K) \qquad (2.3)$$

Assume that Z_1 is a prey population and Z_2 is a predator population that feeds upon Z_1. then G(1,2) is the rate where prey biomass Z_1 is being consumed by the predator Z_2. G(1,1) is the net growth of the prey population, and G(2,2) is the net growth of the predator population without prey consumption. Since the population would decrease without food, G(2,2) is negative. The term G(2,1) is zero since no material moves from a predator population back to the prey population.

The predation rate G(1,2) has a different form because it is a function of both predator and prey densities. The simplest and most widely used description of predation assumes that the number of prey consumed is a fixed fraction of the number of prey encountered. The number of prey encountered by a single predator is the searching rate multiplied by the density of prey in the searched area. The total number of consumed prey is the number of prey encountered by each predator multiplied by the predator density multiplied by the fraction of encountered prey that are consumed.That is,

$$G(1,2)= S_p Z_1 Z_2 \alpha = a Z_1 Z_2 \qquad (2.4)$$

where α is the fraction of encountered prey that are consumed, S_p is the searching rate of the predator, Z_1 and Z_2 are the prey and predator densities, and $a = \alpha S_p$.

The total rate of change of a state variable Z_i is the sum of the effects due to flows into Z_i from other variables minus the flows out of Z_i. Thus

$$F_i = \sum_j \alpha_{ji} G(j,i) - \sum_{K\,(k \neq i)} G(i,k) \qquad (2.5)$$

where F_i:total rate of change of Z_i, α_{ji}: the conversion ratio of material Z_i to material Z_j, $G(j,k)$: flow rate from Z_i to Z_k.

The rates of change F_i are the basis of the difference and differential equations that describe and predict the behavior of an ecological system. In this case, if Z_1 is the concentration of nutrient in milligrams per liter and Z_2 is the plankton biomass in milligrams per liter, then α_{21} is the fraction of plankton biomass that is nutrient and is in units of grams of nutrient per gram of carbon.

Rates of change of a two-dimensional system are constructed as follows:

If $G(2,1)=0$ and $N=2$, From Eq.3.5 we can obtain:

$$F_1 = G(1,1) - G(1,2) \qquad (2.6)$$

$$F_2 = \alpha_{12} G(1,2) + G(2,2) \qquad (2.7)$$

Since the predator population Z_2 decline without a food source Z_1, $G(2,2)$ is negative.

If the birth and death rates of predator and prey populations are density independent and the consumption rate of the predator is proportional to prey density, then $G(1.2)$ has the form given in Eq.2.4 and $G(1,1)$ and $G(2,2)$ have the form corresponding to Eq.2.1.

Thus the rates of change of Z_1 and Z_2 are

$$F_1 = r_1 Z_1 - a_1 Z_1 Z_2 \qquad (2.8)$$

$$F_2 = r_2 Z_2 + a_2 Z_1 Z_2 \qquad (2.9)$$

where $a_2 = \alpha_{12} a_1$ and $r_1 > 0$. Since $G(2,2) < 0$, $r_2 < 0$.

If the prey population growth $G(1,1)$ is indicated by the density dependent Eq.2.2, then the corresponding rates of change are

$$F_1 = r_1 Z_1 ((K - Z_1) / K) - a_1 Z_1 Z_2 \qquad (2.10)$$

$$F_2 = r_2 Z_2 + a_2 Z_1 Z_2 \qquad (2.11)$$

2.4 Predictive equations

Predictive equations incorporate the rates of change F_i into equations that predict the values of the state variables $Z_i(t)$ through time. If time is divided into discrete units such as days, generations, or years, the value of state variable Z_i at time $t+\Delta t$ is the value of the state variables Z_i at time t plus the rate of change F_i multiplied by Δt, the number of time units that have elapsed

The mathematical representation of this relationship is

$$Z_i(t+\Delta t) = Z_i(t) + F_i(t)\Delta t \qquad (2.12)$$

Equation 2.12 is called a difference equation. Usually the units of t are chosen so that Δt equals one. In this case, the difference equation is

$$Z_i(t+1) = Z_i(t) + F_i(t) \qquad (2.13)$$

The rate of change F_i, as a function of flow rates $G(j,i)$, substitutes into the difference from Eq.2.13, we obtain

$$Z_i(t+1) = Z_i(t) + \sum_j \alpha_{ji}\, G(j,i) - \sum_{k(k\neq i)} G(i,k) \qquad (2.14)$$

Difference equations which have the form of Eq 2.14 predict changes at discrete points in time. Predictive equations that describe dynamics of a system continuously through time are called differential equations. This type of equation is based on the derivative of $Z_i(t)$, which is the instantaneous rate of change of $Z_i(t)$ and is denoted by the symbol $dZ_i(t)/dt$. Using Eq.2.12, the derivative is defined to be

$$dZ_i(t)/dt = \lim_{\Delta t\to 0} (Z_i(t+\Delta t)-Z_i(t))/\Delta t = \lim_{\Delta t\to 0} F_i(t) = F_i(t) \qquad (2.15)$$

The last equality follows because $F_i(t)$ does not depend upon Δt.

Thus the derivative equals the rate of change F_i defined in Eq.2.15, that is,

$$dZ_i(t)/dt = F_i(t) = \sum_j \alpha_{ji}\, G(j,i) - \sum_{k(k\neq i)} G(i,k) \qquad (2.16)$$

These models most widely used to describe the growth of a single species are obtained

$$dZ/dt = F = r\,Z \qquad (2.17)$$

and

$$dZ/dt = F = r\,Z((K - Z) / K) \qquad (2.18)$$

Equation 2.17 is called the exponential growth equation,and Eq.3.18 is the logistic growth equation. Exponential and logistic growth also have difference equation forms

$$Z(t+1) = Z(t) + r\,Z(t) \qquad (2.19)$$

and

$$Z(t+1) = Z(t) + r\,Z(t)\,((K - Z(t) / K) \qquad (2.20)$$

Models predicting the behavior of two interacting species can be constructed in a similar fashion. The derivative dZ_i/dt of each state variable is calculated by substituting the flow rates $G(j,i)$ into Eq.2.16. For example, the dynamics of predator and prey populations can be predicted from Eq.2.16 by the following set of differential equations:

$$dZ_1/dt = F_1 = r_1 Z_1 - (r_1 / K_1) Z_1^2 - a_1 Z_1 Z_2 + e_1 Z_1 \quad (2.21)$$

$$dZ_2/dt = F_2 = r_2 Z_2 + a_2 Z_1 Z_2 - p_2 Z_2 \quad (2.22)$$

where e_1 is the fraction of influence to the prey population (plankton) caused by the marine construction, p_2 is the coefficient of fishery product of the predator(scallop), r_1,a_1,and $a_2>0$ and $r_2<0$.

In this case, a term of the influence to the plankton (prey) caused by the marine construction is included in Eq.2.21. Another term of fishery product is also included in Eq.2.22.

Equations 2.21 and 2.22 are defined as the extensive Lotka-Volterra equations.

If the prey population has a growth rate that decreases with increased prey density, the set of differential equations obtained by substituting rates of change F_1 and F_2 defined in Eqs.2.10 and 2.11 would be a more appropriate model.

By using the same method of model const-ruction, the predator-prey model considered above can be extended to incorporate other species and substances.

To illustrate the application of the modeling techniques described above, we discuss in detail a two-dimensional model to predict the response of scallop population to nutrient loads caused by the construction of the submerged floating tunnel across the entrance of the bay.

The first step in building a model of the ecosystem is to develop equations for each of the flow rates G(i,j). Even though some of these flows will be zero, other flows are not zero, for instance, the rate of feeding on plankton by scallop. Scallop feed by filtering plankton from water. The amount of water filtered by each scallop multiplied by the concentration of plankton in the filtered water is the amount of plankton ingested by each scallop. Multiplying this amount by the number of scallop gives the total amount of plankton consumed by scallop, that is,

$$G(1,2) = \text{filtering rate} \times \text{plankton concentration} \times \text{number of scallop} = Cg \times (Z_1/V) \times Z_2 \quad (2.23)$$

or

$$G(1,2) = aZ_1Z_2 \quad (2.24)$$

where V=volume, Cg=filtering rate, a= Cg /V, and Z_1 and Z_2 are the number of plankton and scallop, respectively. Note that Eq.2.24 has the same mathematical form as the general expression for predation.

In this case, the relation between the amount of nutrients and plankton population should be considered. But the increase rate of nutrient is directly proportional to the growth rate of the plankton population. Thus we use the results of the simulation of the sea current prediction model. Therefore, in the ecological growth model we only consider the relationship between the plankton behavior depending on the change of nutrient (change of sediment).In an aquatic ecosystem, nutrients are returned to water by the excretions and death of plankton. But the flow of this material from the plankton population is also no considerable in this study.

2.5 Solution Techniques

The purpose of predictive equation is to determine the values of Z_i at each point in time t. An expression that gives the values of $Z_i(t)$ is called the solution of the predictive equations.

To solve difference equations numerically the values of variables at time t are used to calculate values at time t+1, which in turn are used to calculate the values at time t+2. This process starts with $t=t_0$, for which an initial value $Z_i(t_0)$ is given. For instance, consider a difference equation model of a predator and a prey population whose rates of change are defined by Eqs. 2.21 and 2.22. Then from Eq.2.13,

$$Z_1(t+1)=Z_1(t)+r_1Z_1(t)-(r_1/K_1)Z_{21}(t)-a_1Z_1(t)Z_2(t)+e_1Z_1(t) \quad (2.25)$$

$$Z_2(t+1)=Z_2(t)+r_2Z_2(t)+a_2Z_1(t)Z_2(t)-p_2Z_2(t) \quad (2.26)$$

To numerically solve the equations, initial values of the state variables and parameter values must be specified.

2.6 Dynamic and Steady-state Models

Dynamically changing systems may eventually reach a state at which the values of the variables remain steady. A system in which all the state variables stay constant through time is said to be in steady-state or in equilibrium.

Let $Z^*=(Z^*_1,...,Z^*_n)$ be the equilibrium values of the state variables.

Since the values of the variables do not change, the rates of change equal zero, that is,

$$F_i(Z^*)=0 \quad i=1,...,n \quad (2.27)$$

Eq. 2.27 is used to calculate equilibrium solution Z^*. To calculate the equilibrium solution to the logistic Eqs.2.18 and 2.20, substitute Eq.2.2 into Eq.2.27 to obtain

$$F_1(Z^*_1)= rZ^*_1((K-Z^*_1)/K)=0 \quad (2.28)$$

By dividing both sides of Eq.2.28 by rZ^*_1/K and adding Z^*_1 to both sides, we obtain

$$K=Z^*_1 \quad (2.29)$$

Thus once Z_1 reaches the level K, it will remain at that level.

Meanwhile, to obtain the equilibrium solution to the equations proposed here, substitute the rate of change Eqs.2.8 and 2.9 into the equilibrium Eq.2.25 to obtain

$$F_1(Z^*_1,Z^*_2)= r_1Z^*_1-(r_1/K_1)Z^*_{21}-a_1Z^*_1Z^*_2+e_1Z^*_1=0 \quad (2.30)$$

$$F_2(Z^*_1,Z^*_2)=r_2Z^*_2-a_1Z^*_1Z^*_2-p_2Z^*_2=0 \quad (2.31)$$

By dividing Eq.2.30 by Z^*_1, dividing Eq.2.31 by Z^*_2 and rearranging terms, we obtain the equilibrium points

$$Z^*_2 = (r_1+e_1 / a_1)\{1-(r_2-p_2)/a_2K_1\} \quad (2.32)$$

$$Z^*_1 = -(r_2 - p_2)/\ a_2 \qquad (2.33)$$

An equilibrium point such that the system will return to it after small perturbations is called stable.

In general,the equilibrium point Z^*_1 of the differential logistic growth equation is stable, but Z^*_1 and Z^*_2 of these equations are not stable.

2.7 Combination between the multi-level sea current model and the ecological growth model

The simulation results obtained by the multilevel sea current model revealed the spatial pattern and the dynamic behavior of sediment transported by the current. The behavior of sediment,for instance, was determined in each vertical cells which are found at intervals of ten meters as the depth increases. The x-y axis is used to show the distribution of sediment in the whole bay.The distribution of sediment in each cell affects the growth of scallops.

It is, therefore, calculated and used as an important input for evaluating the ecological behavior of the sea area. The multilevel sea current model is appropriate for determining the spatial distribution of sediments resulting from the behavior of the sea current.The ecological growth model, on the other hand, cannot be used directly to determine the behavior of the spatial distribution of sediment. It is suitable for predicting the dynamic ecological behavior at each point of the sea since it is a dynamic simulation model.

For the purpose of the simulation some sites were selected at random. It was also assumed that the concentration of nutrients is proportional to the concentration of sediment.

3.Empirical study

3.1 Objective area of the study

We selected the Volcanic Bay to test the validity of the model. The Volcanic Bay Area is located along the south-eastern coast of the northern Japanese island of Hokkaido.As the name suggests, the Volcanic Bay was formed as the result of volcanic activity.Therefore, the entrance of the bay is narrow and its shape is like a huge bowl. Some scholars and engineers belonging to the public and private organizations have been discussing the possibility of constructing a strait crossing tunnel across its entrance.

The main product from the bay is scallops. The fishery ground,basically the cultivated area continues along the sea-shore. The total area is 212m2 and the haul of scallop is about 50 thousand tones every year. Therefore, it is important to simulate the behavior of scallop and to predict the likely impact of the environmental changes.Fig 3 displays annual product in the whole bay area for ten years. The output of the cultivated scallops has been increasing, but the capacity of the cultivation can come close to the stable level. The carrying capacity is assumed to be 0.37 billion population at the age of zero, 0.19 billion population at the age of one and 0.06 billion population at the age of two. The rough sketch of cultivated seashore is shown in Fig.2.

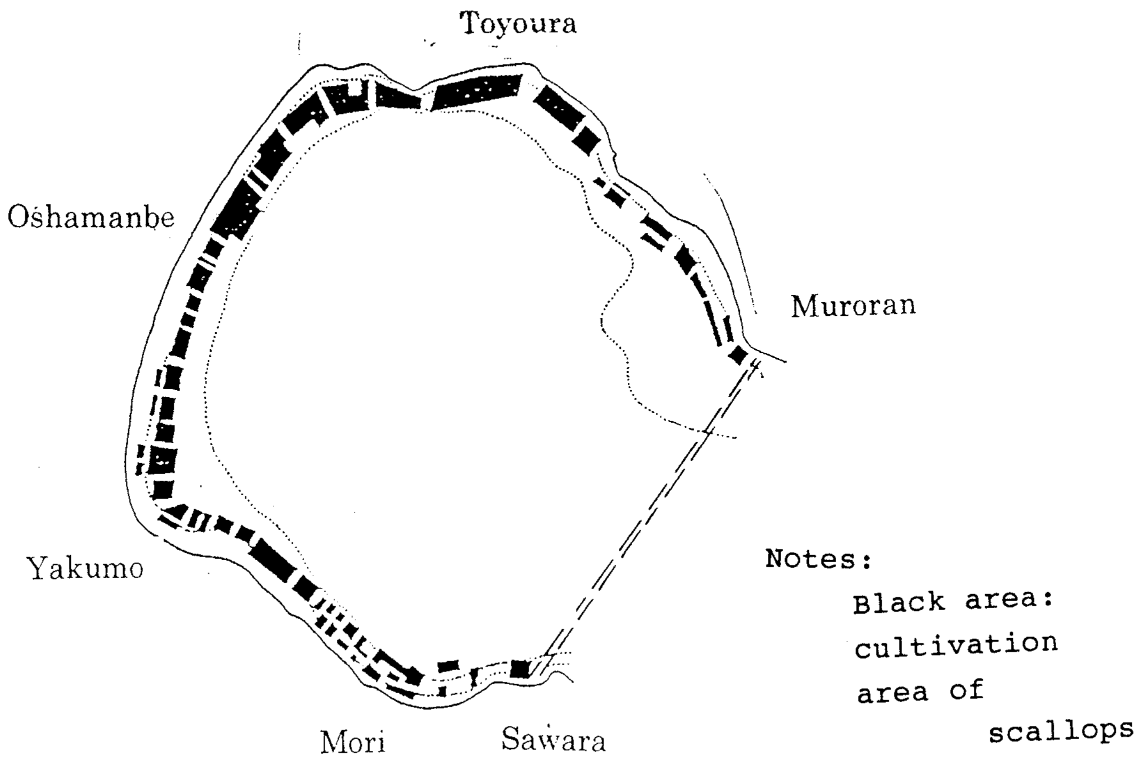

Fig.2 Outline of the study area

3.2 Results of the simulation by the ecological growth model

The simulation results obtained by using the sea current model mentioned above, became the input data for the ecological growth model. The simulation was carried out at each arbitrary point within the bay.

We considered the fourth cell, that is, the depth of 20.0 m-30.0 m. This is the cell in which most scallops are cultivated. It was realized that in this cell, the construction of the tunnel will result in large changes in the concentration of sediment, particularly along the sides of the structure. Around the seashore more or less changes occurred due to the construction. These were in the range of 0.8 to 1.2 times as much as the usual case which can be obtained by the sea current prediction model. In this case the simulation was by combining some of the conditions.

Figure 3 illustrates the potential movement of both plankton and scallop population due to the influences of the construction. The degree of the influence by the construction is indicated as the ratio E. When E is 1.0, the sea environment can be influenced by the construction. When E is more than 1, the influences is positive.When E is less than 1, it is negative. According to our model, the behavior of both plankton and scallop varied with time, characterized by a swinging pattern at the beginning which tends to approach a stable state. For instance, firstly the potential for the scallop population approached 490 percentage as the peak value and then a few attenuated vibration were observed in the case where E= 1.2. Finally it reached at the stable point in that same case. Most of the cases of the simulation exhibited a similar behavior. Based on the results of the simulation, the stable states are illustrated in Table 2.

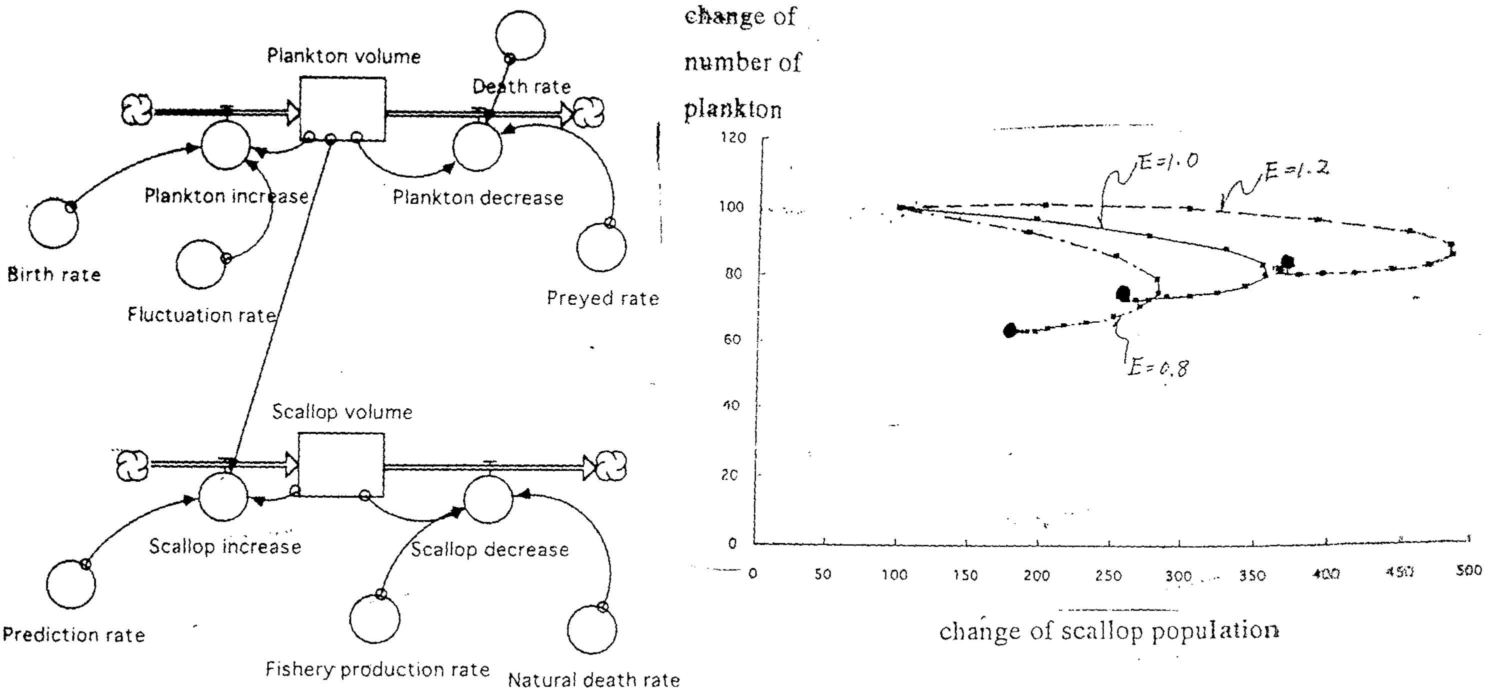

Fig 3. Diagram of simulation model and the simulation results

Table 2 the stable state obtained by the simulation without the tunnel

population movement	plankton S	plankton M	plankton L	scallop S	scallop M	scallop L
case 1	54			450		
case 2	68				312	
case 3	74					256
case 4		74		774		
case 5		82			548	
case 6		85				447
case 7			83	1056		
case 8			87		770	
case 9			89			631

Notes: S is small movement,M is medium and L is large movements.

A real system usually has an element of uncertainty, so we considered the estimation with a range from small to large. The actual movements of plankton and scallop can occur within the range as shown in Table 2. Considering these cases, it can be inferred that the movement of plankton population is likely to be small but that of scallop is expected to be large.

Table 3 The stable states influenced by the construction (in case 3 and case 6)

ratio between with tunnel& without tunnel	results of simulation			
	case 3		case 6	
	plankton	scallop	plankton	scallop
E=0.8	84	68	85	43
E=1.0	100	100	100	100
E=1.2	112	146	108	233

Notes:in the case where E=1.0, both values =100.

Considering these results, it can be said :

1.This simulation model illustrates the interaction between plankton and scallop within the Volcanic Bay. If the scallop mortality increase, its population can be reduced and the number of plankton can be increased correpondingly.

2. If the plankton behavior is activated, the scallop population can be increased.

3.The environmental impact of the construction on plankton is not expected to be so large. However, the effect on scallop is likely to be considerably large.For instance, the coastal area where the construction is likely to cause a 120 percentage increase of the sediment (nutrients) will have 1.12 times the increase of the plankton and 1.46 times the increase of the scallops in case 3.

4.CONCLUSION

We have discussed the likely environmental impacts of the construction of a submerged floating tunnel. Two models have been proposed namely, the sea current model and the ecological growth model. We applied these models to the actual study area. The results may be summarized as follows:

1.The influences on the sea environment due to the construction of the tunnel are predicted to be small, owing to the fact that the bay area is so wide compared with the area where tunnel seems to give some influences to the sea current.

2.In the immediate surroundings of the tunnel a large impact on the sea environment is predicted. This is based on the fact that the top and bottom of the tunnel, the sea current will be faster and on both sides it will be slower than the present situation. Hence, both sides of the tunnel will possibly become remarkable for fishery because of accumulation of nutrients which will result

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3. Hokkaido Crossing

3.5 Flow-Induced Vibration Control of SFT

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FIVC (Flow-Induced Vibration Control) of SFT

- Flow induced vibration on SFT
- Design guides of FIVC on SFT
- Problems and Topics

May 1996 at Sundness in Norway

TORU IIJIMA

Center for Cooperative Research and Development

Muroran Institute of Technology

FIVC (Flow-Induced Vibration Control) of SFT

Consideration points of FIVC of SFT

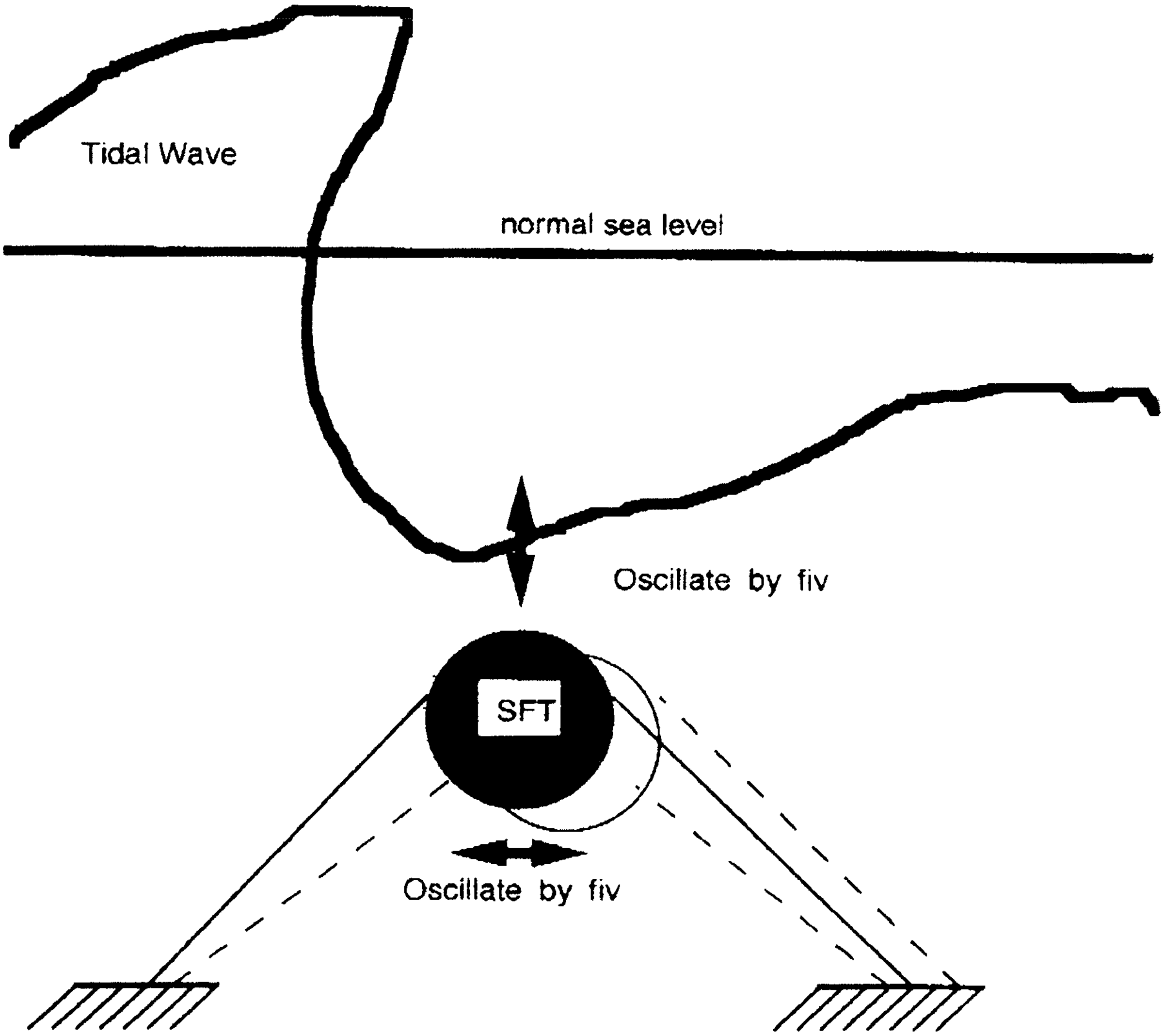
1. most higher safety mergins are demanded more than normal tunnel

2. Vibration source
   - Loads
   - Fluid force
   - Earthquake

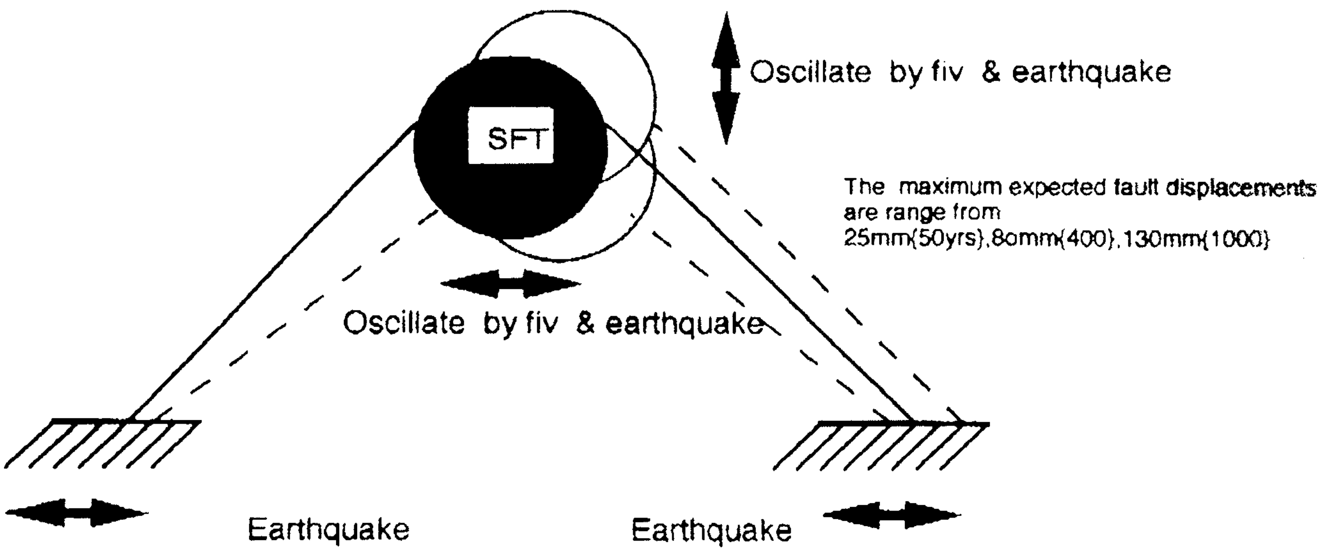
3. Simple system {maintenance free to the utmost}

4. Serfacial condition of structuaral parts; changed gradualy:

   Shape and serface of structure: adherance-sea-creatures



Tidal wave ocasion



Earthquake ocasion

How to FIVC of SFT

Condition

Passive or Symple System of Active Control with maintenance free to the utmost

Changing the surface condition of shape by creatures in the sea

FIVC method

{Control Physical condition}

Damping ; to increase damping effect

Elasticity ; to control tention of sapporting

Mass ; to decrease added mass effect

1. Give actuator energy for source of ocsillational one-
{complicate and needs to maintenance}

Damping ; Dash Pot-type Actuator

Elasticity ; Tention Control Type Actuator to use cable-support
{active stiffness control of cable vibration'; Fujino1993}

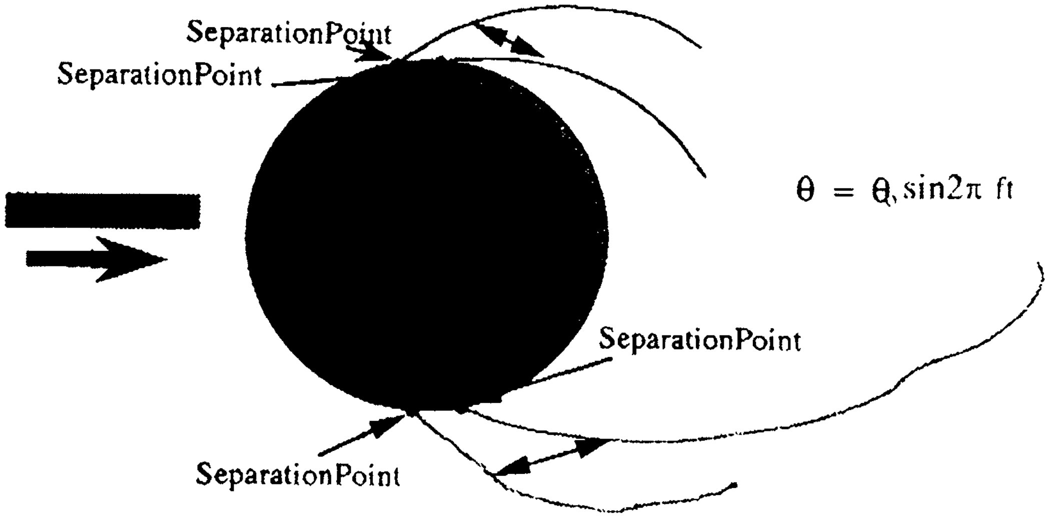
Mass ; Dynamic damper {F.Nicolussi 1994 }

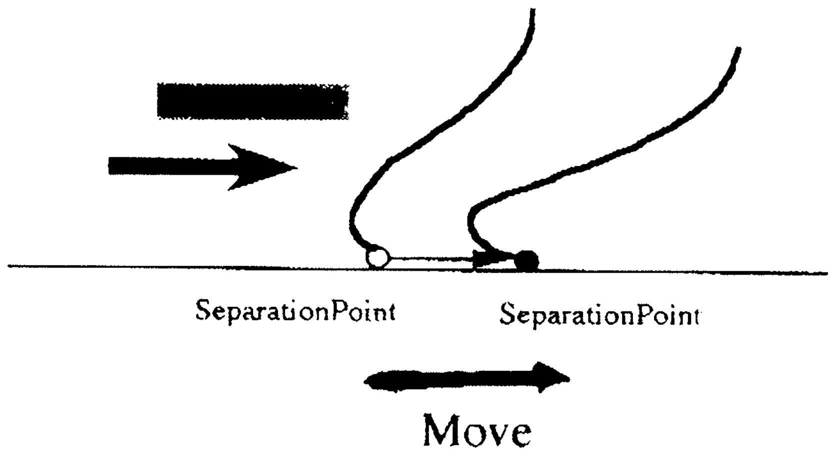
2. Interfairance of ocsillation mechanism
{symple system and less energy for control}

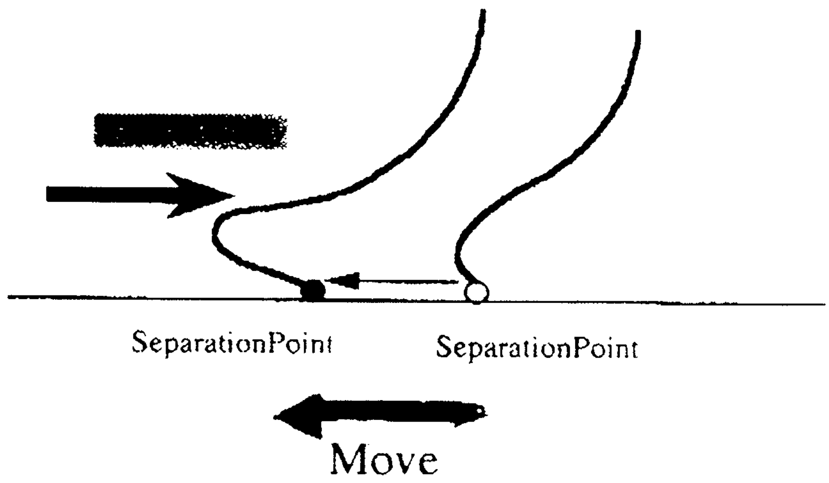
Damping ; Adding Damping

Elasticity ; Control boundary condition of body-surface
Interfering development of vorticies

Mass ; Control boundary condition of body-surface


SeparationPoint
SeparationPoint
$\theta = \theta_0 \sin 2\pi ft$
SeparationPoint
SeparationPoint


SeparationPoint
SeparationPoint
Move


SeparationPoint
SeparationPoint
Move

This SFT's lift force control is very difficult. Actuator was subjected to high water pressure and maintenance is very difficult. So we must innovate less maintenance free actuator and prove the problem of high water pressure.

And this kind of actuator is to be less energy against fluid force and to be the type of CM control because this structural natural frequency is very low.

So we suggest an idea of flip-flop-type actuator. We examined the actuator's ability of this experimentally. And we show the concept of flip flop actuator in Fig. 1.

We show the figures from F. 2 and 3.

Fig. 1 is for CL intensity against St. number and Fig. 2 for CD intensity against St. number.

β is Fluid force intensity (β=CL or CD/ CL or CD(without control)).

From these results, ability of flip-flop type actuator is enough.

When we use this actuator at large sinusoidal oscillate, 15% of CL was decreased.

In this type of actuator was moved by flip-flop type actuator by sinusoidal motion.

and this energy is less than 1/10 of lift force.

This actuator's ability is depend on surface condition and phase delay of lift motion.

At last, I think we must innovate to decrease the emergency lift force caused by tidal wave and earth quake to realize SFT at such condition.

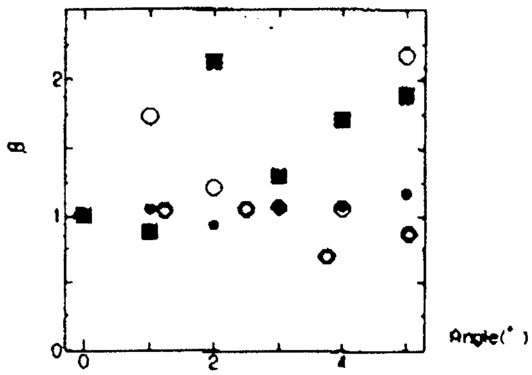


Fig. 2 Acceleration Angle vs. C_L –Intensity (S_t =0.05, 0.06, 0.1, 0.15)

● : S t =0.05
○ : S t =0.06
■ : S t =0.1
◎ : S t =0.15
□ : S t =0.2
◆ : S t =0.3
◇ : S t =0.4
▲ : S t =0.5
△ : S t =0.6
▼ : S t =0.75
▽ : S t =0.9

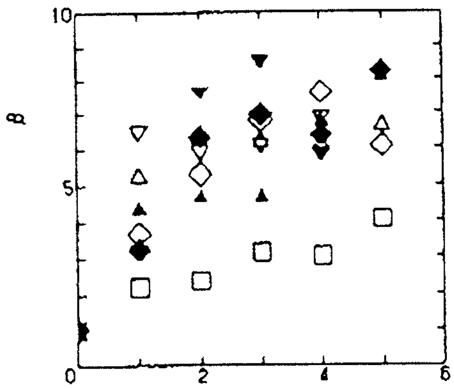


Fig. 3 Acceleration Angle vs. C_D –Intensity (S_t = 0.2, 0.3, 0.4, 0.5, 0.6, 0.75, 0.9)

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4. Messina Crossing

1. F. Ziliotto
2. A. Fiorentino
3. V. Di Tella
4. R. Robino

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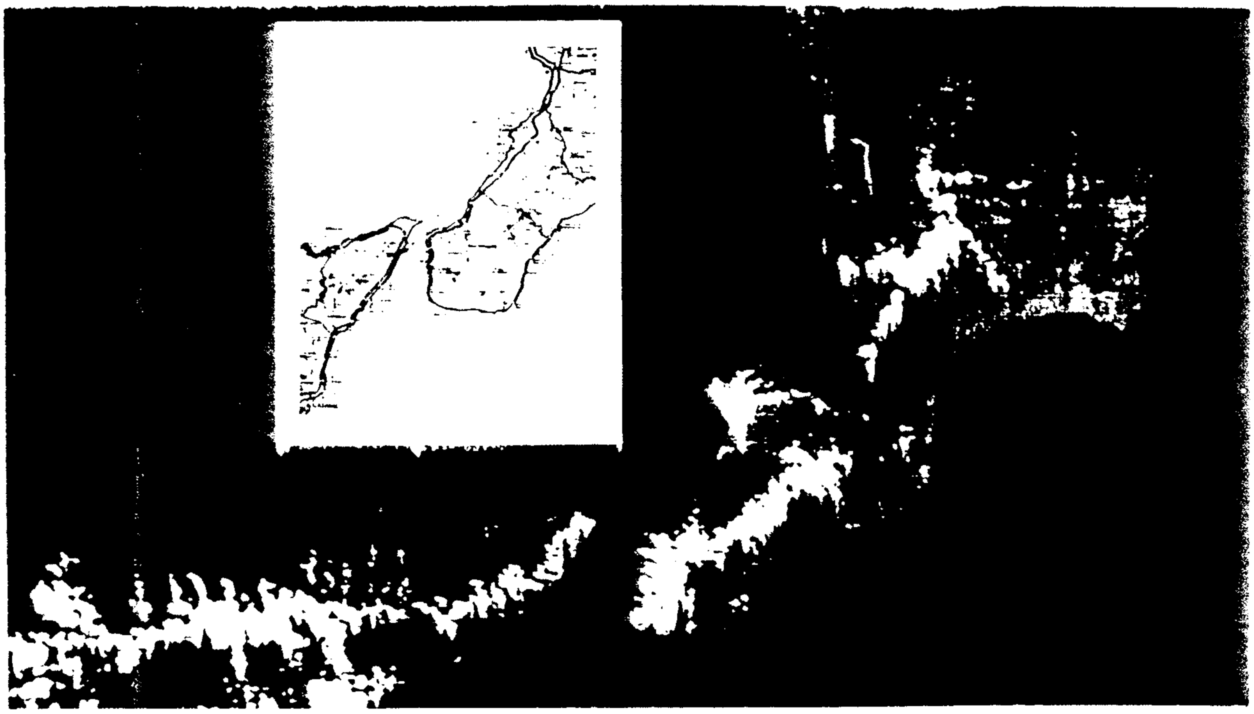
4. Messina Crossing

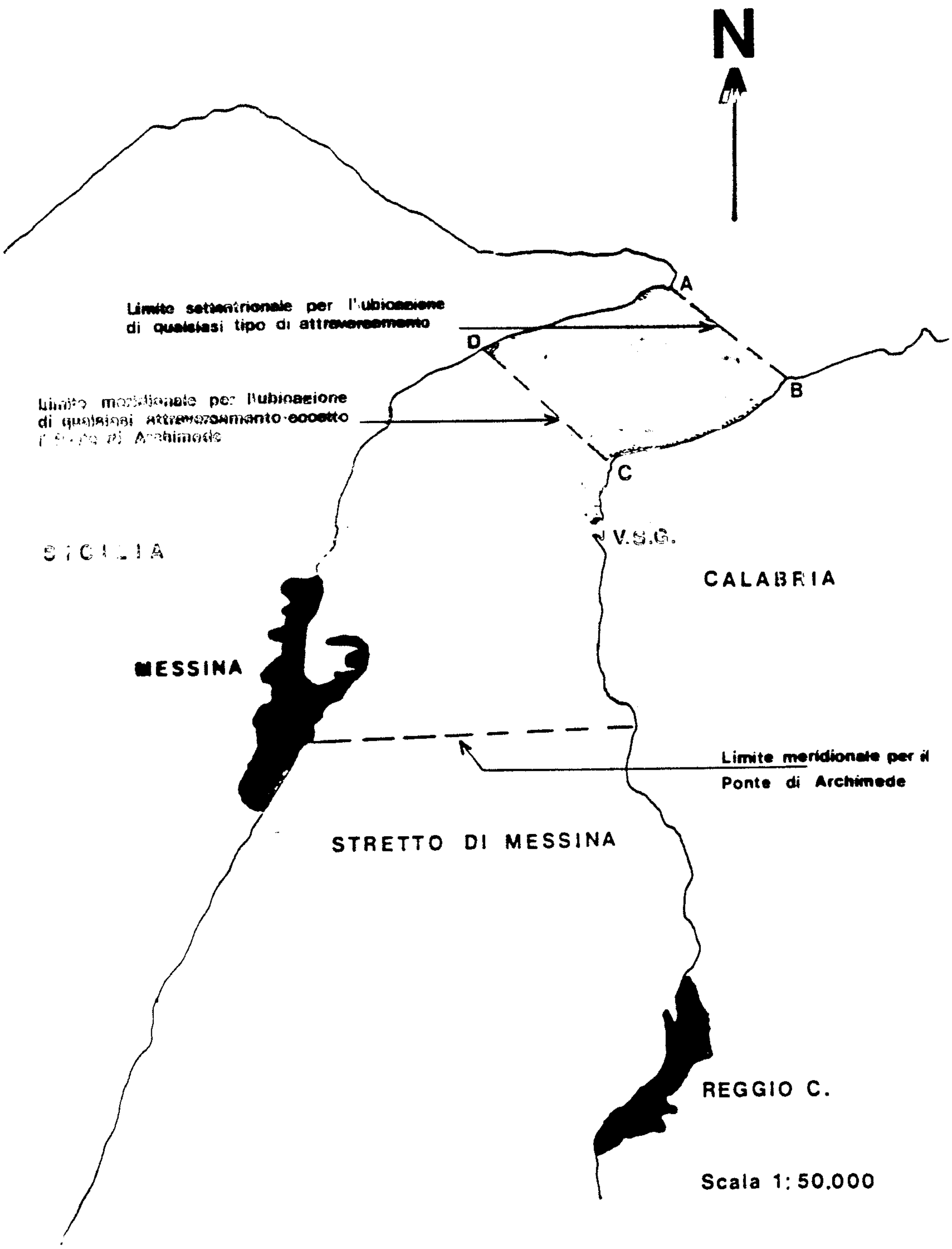
4.1 The Messina Strait

F. Ziliotto
Registro Italiano Navale

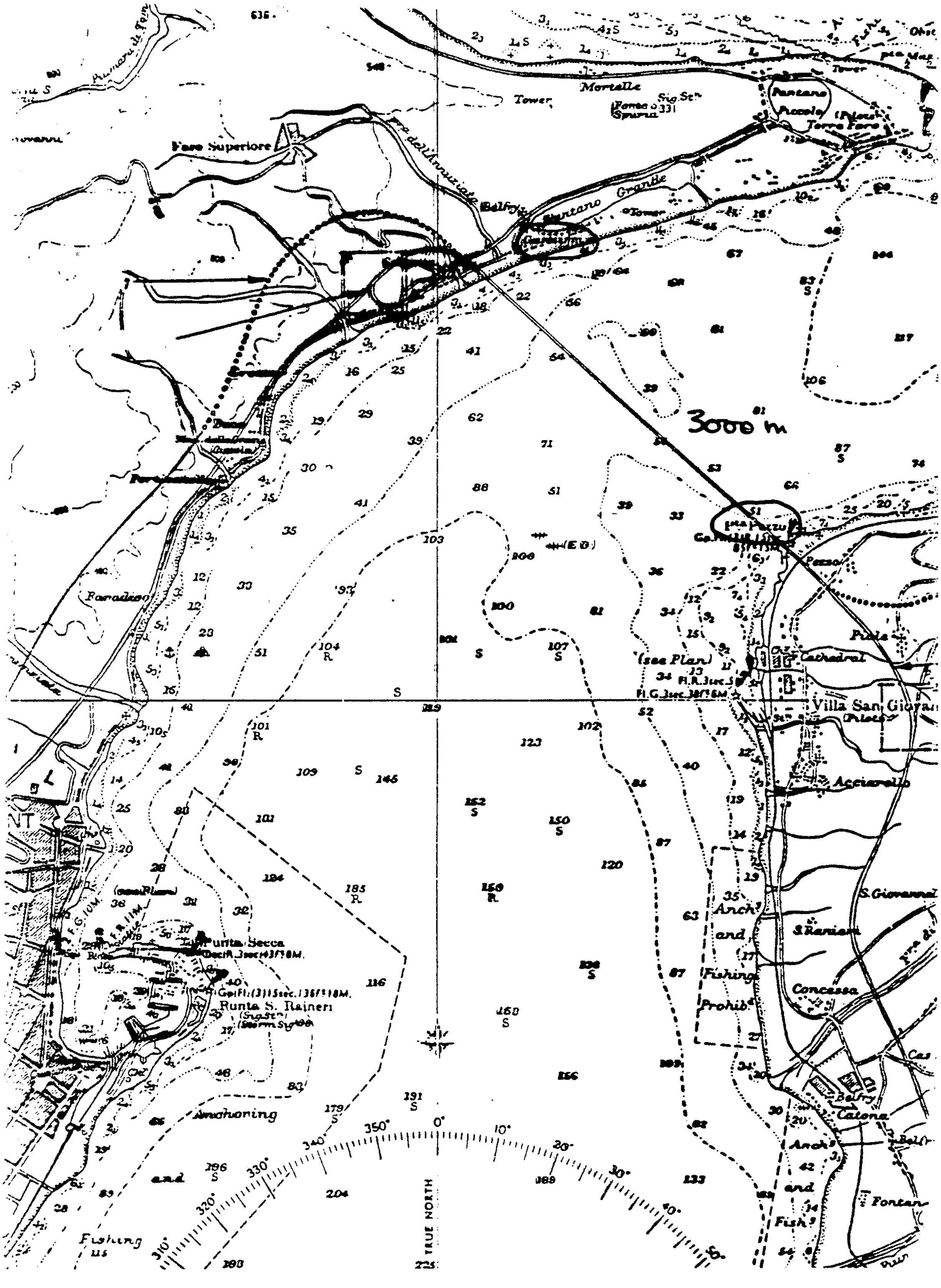
RINA

The Messina Strait



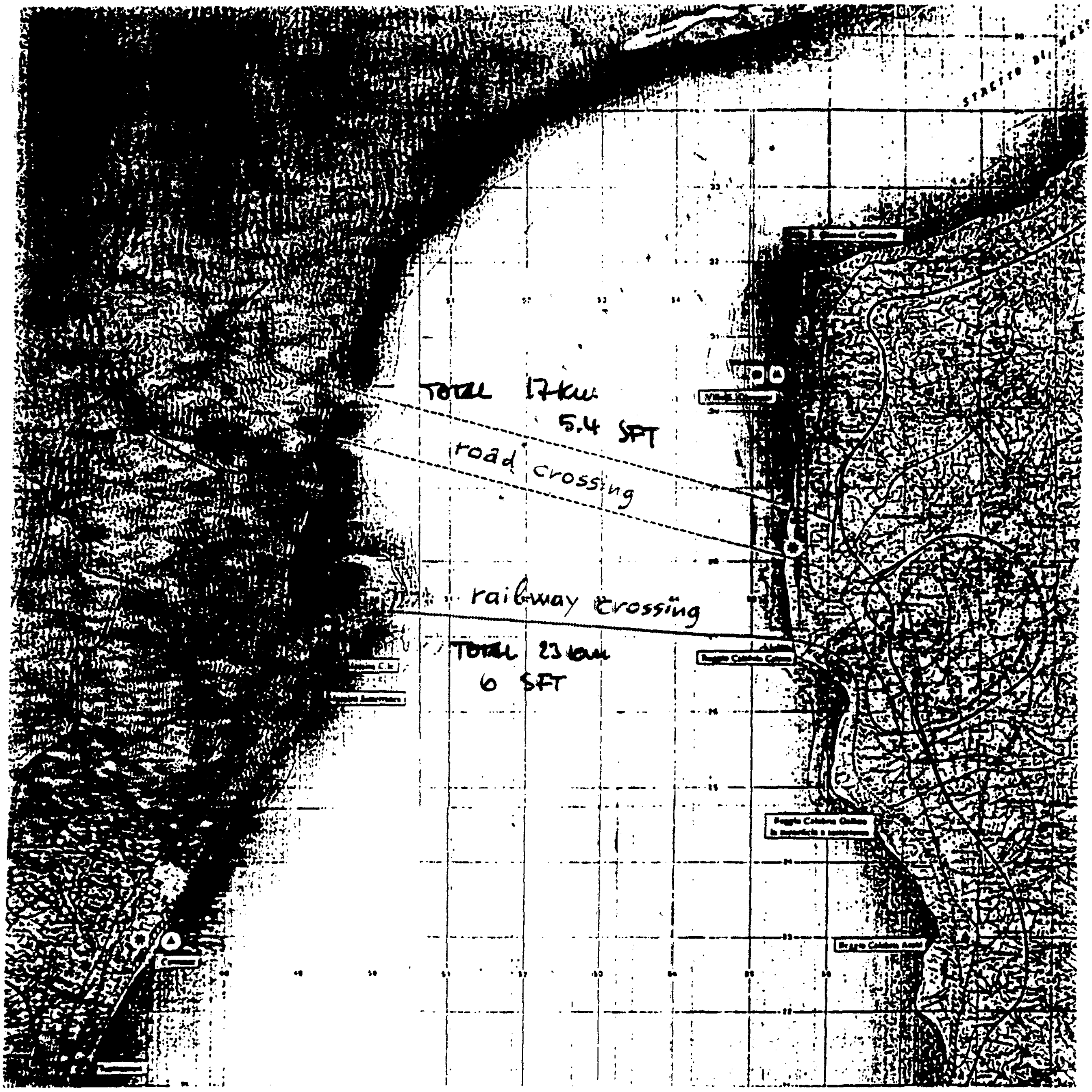


Limited areas for the crossing



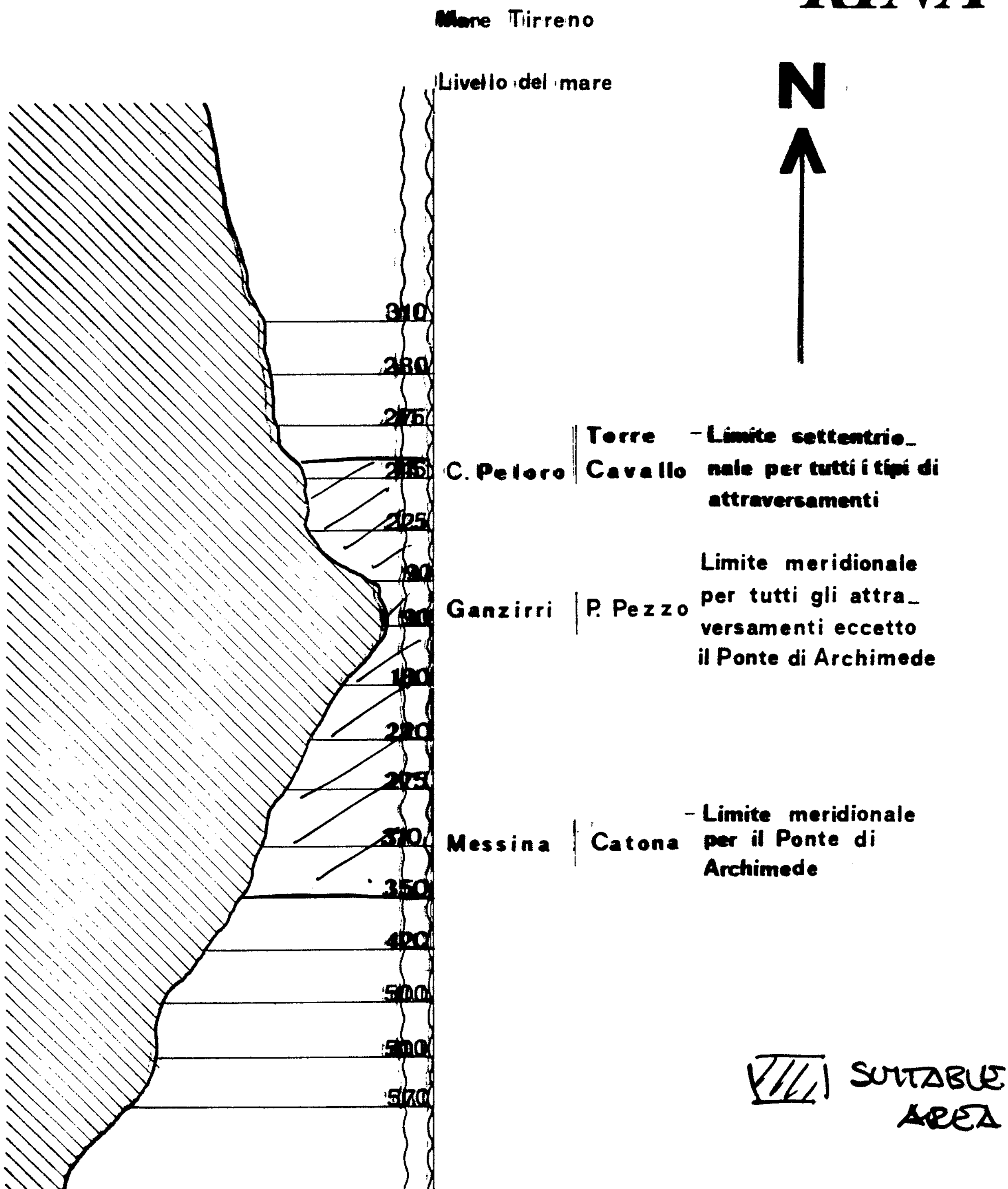
Original proposal "Ponte di Archimede"

SHORTER WAY BUT VERY SENSITIVE TO CURRENTS, TURBOLENCES AND RELATED NATURAL PROBLEMS



ENI permanent crossing proposal

RINA



Scala Vert. 1:50.000

Scala Orizz. 1:11.250

maximum depth contour North-South

ENVIRONMENTAL CONDITIONS

- ❑ **current**
  (about 3 m/sec associated to 400 years)
- ❑ **waves**
  (max 11.5 m with return period 400 years)
- ❑ **earthquake**
  (an earthquake of IX° Mercalli Scale with return period 100 years)
- ❑ **faults**
  (a complex system of faults is present on the sea bottom)

TRAFFIC CONDITIONS

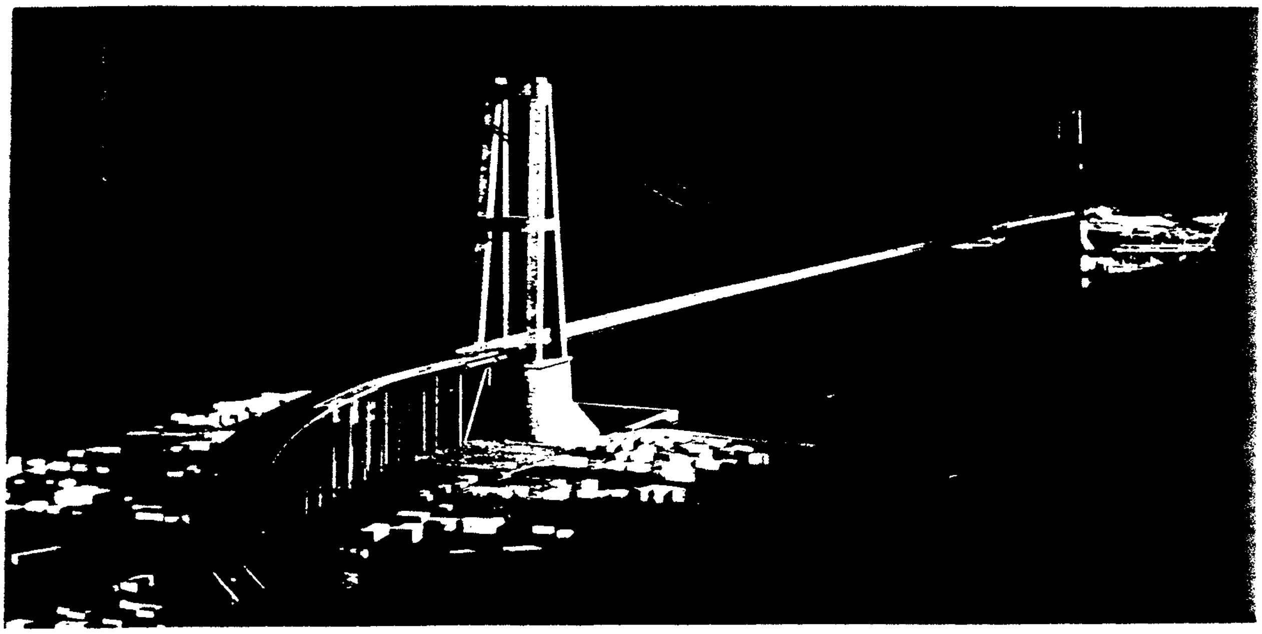
- ❑ **ro-ro ferries**
- ❑ **cargo ships**
- ❑ **fishing vessels**

⇩

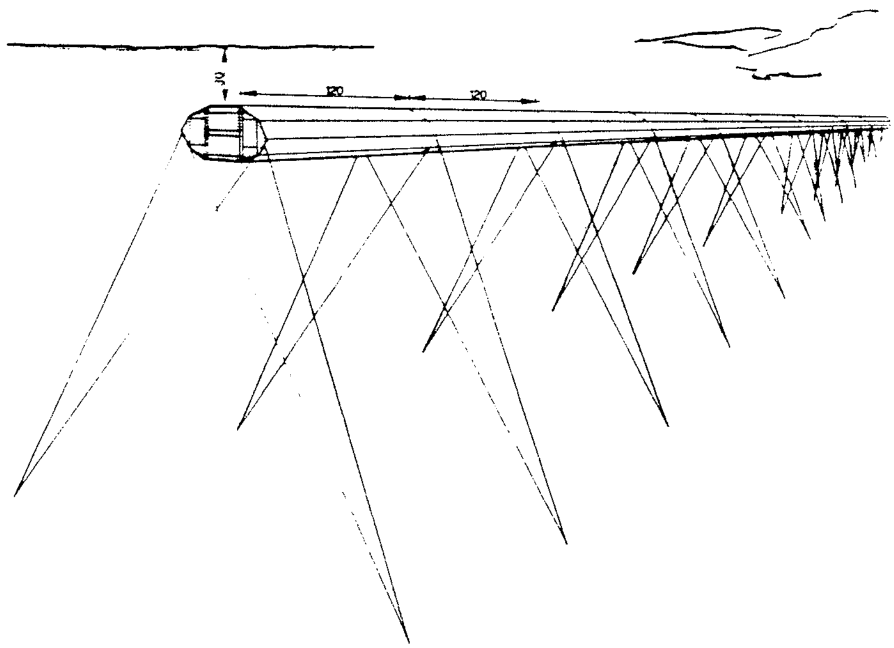
60,000 axial crossing / year
130,000 transverse crossing / year

PROJECTS for the PERMANENT CROSSING

suspended one span bridge



submerged floating tunnel



- Ponte di Archimede (1984) *feasibility study*
- ENI Project (1988) *feasibility study*
- ENI Project (1994) *basic design*

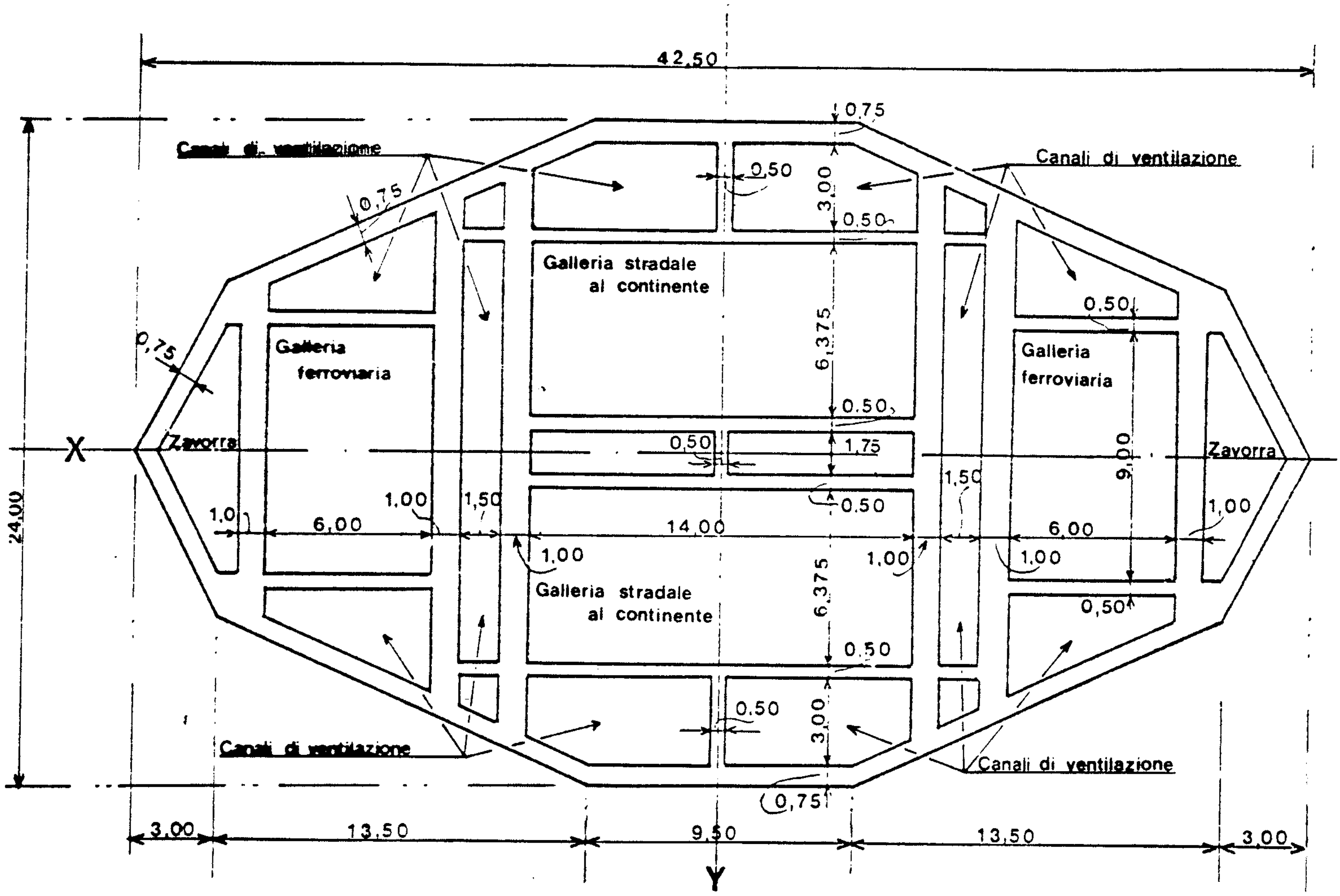
SUSPENDED BRIDGE MAIN DATA

- VERTICAL HANGERS TO PREVENT OVERSTRESS IN THE HANGERS THEMSELVES
- MASSIVE FOUNDATIONS : BLOCKS 150 x 150 m, 90 m IN HEIGHT
- ACCESS : 64 Km TUNNELS, 13 Km VIADUCTS
- SUSPENSION CABLES : D = 1.6 m, W = 280,00 t
- BRIDGE DECK : TWO RAILWAY TRACKS
  TWO TRIPLE MOTORWAY LANES
- BRIDGE DECK WEIGHT : 130,000 t
- STEEL TOWERS WEIGHT : 220,000 t.

RINA certification experience of submerged tunnels

- feasibility evaluation of the project "Ponte di Archimede" (1984)
- study of the minimum tunnel depth for the safety of navigation (1986)
- evaluation of the feasibility study for submerged tunnels
  - ENI project (1988)
- design guidelines for submerged buoyant tunnels
  - ENI project (1992)
- certification of the basic design of submerged buoyant tunnels
  - ENI project (1994)

Project PONTE DI ARCHIMEDE

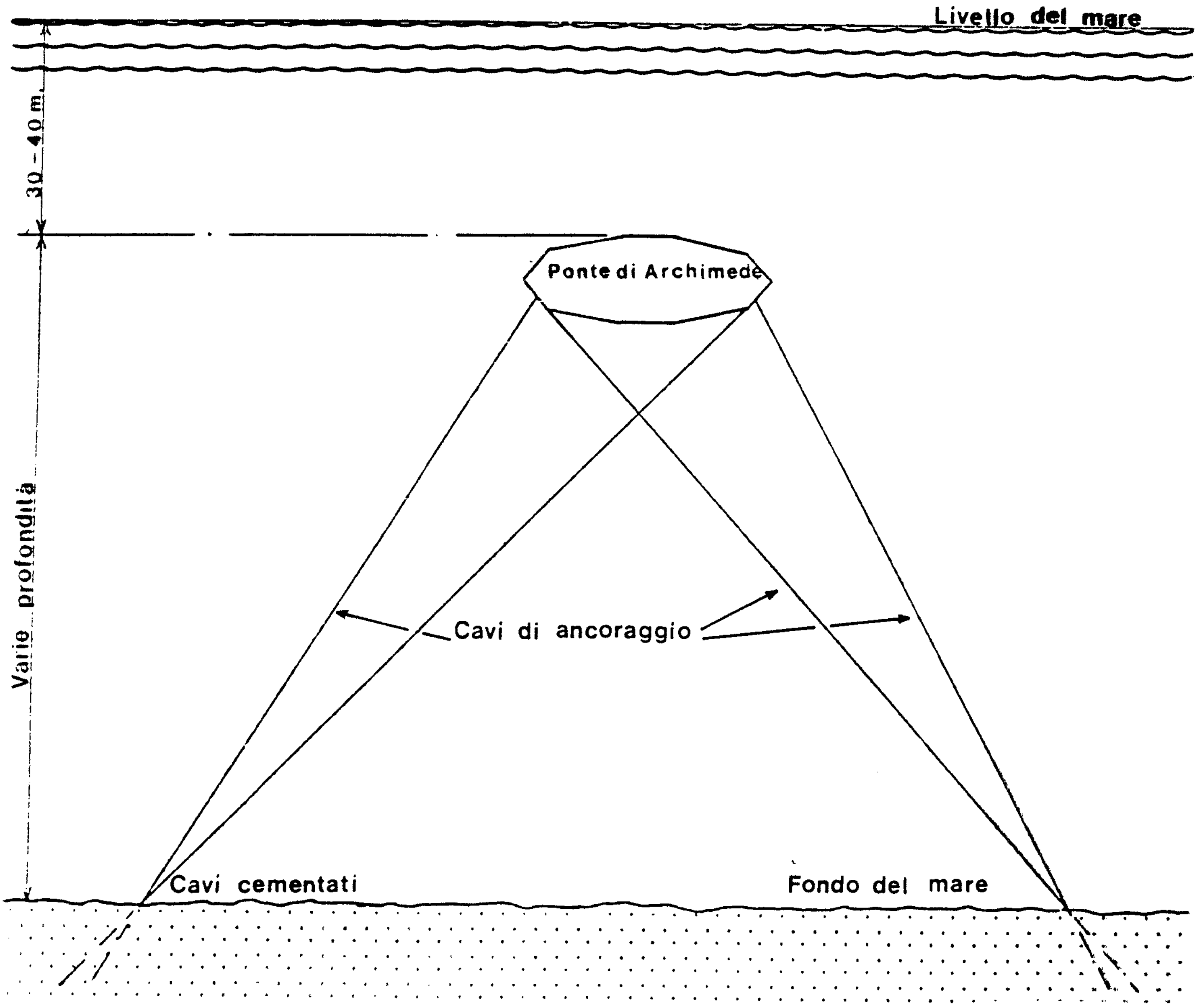


PONTE DI ARCHIMEDE

- underwater buoyant post-stressed concrete tunnel
- cross-section of the tunnel for road and rail traffic

RINA

Project
PONTE DI ARCHIMEDE



- buoyant tunnel structure supported by a system of cables anchored to the sea bed
- design value of the residual buoyancy force of about 1.9 MN/m

Project
PONTE DI ARCHIMEDE

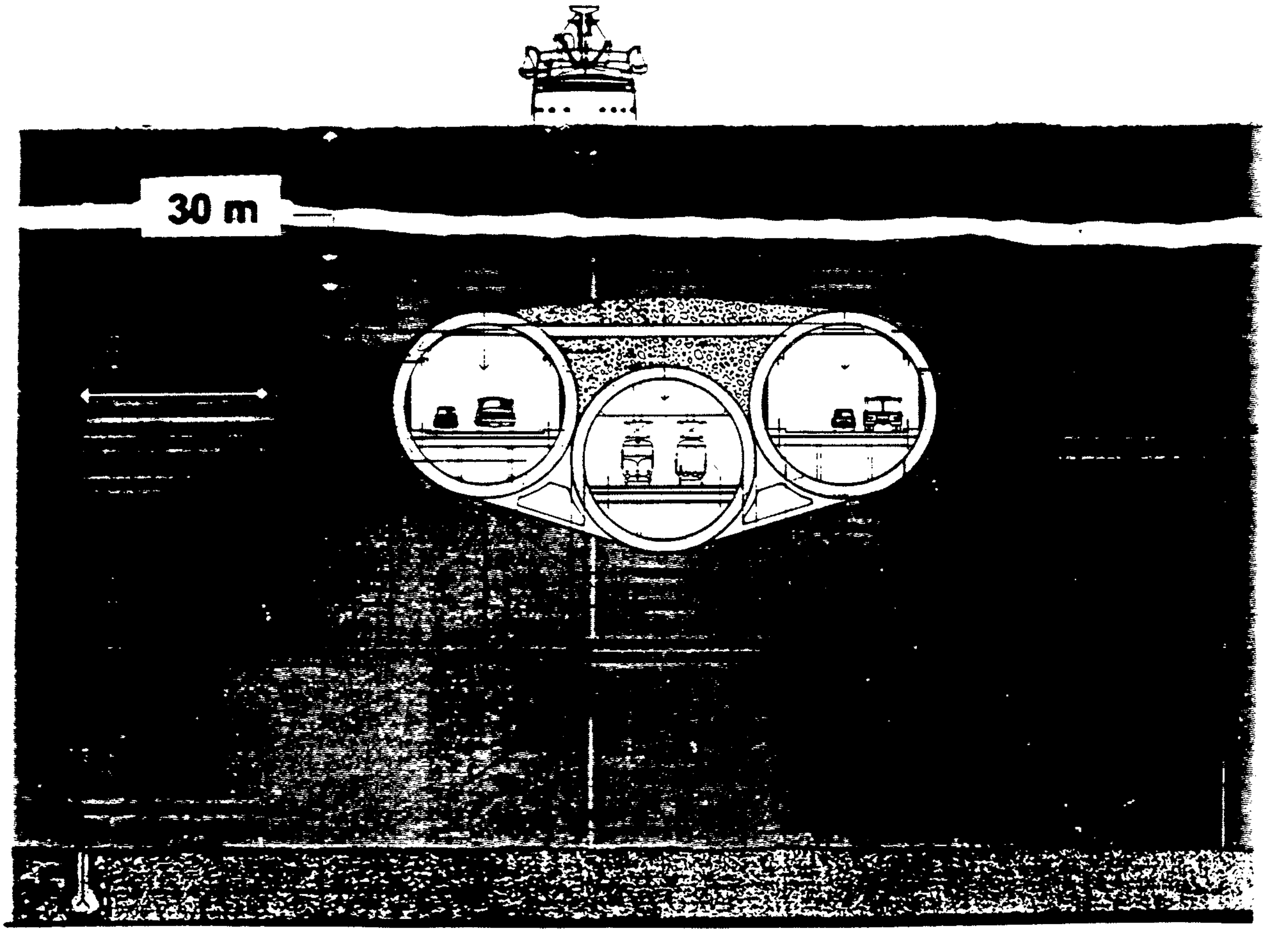
VALIDATION STUDY CARRIED OUT BY RINA

(1984)

- longitudinal hydrodynamic and seismic analyses;
- fault displacement;
- transverse strength checks;
- analysis of the capacity of the structure in partially damaged conditions;
- loss of an anchoring system;
- partial flooding of the tunnel for external leakage;

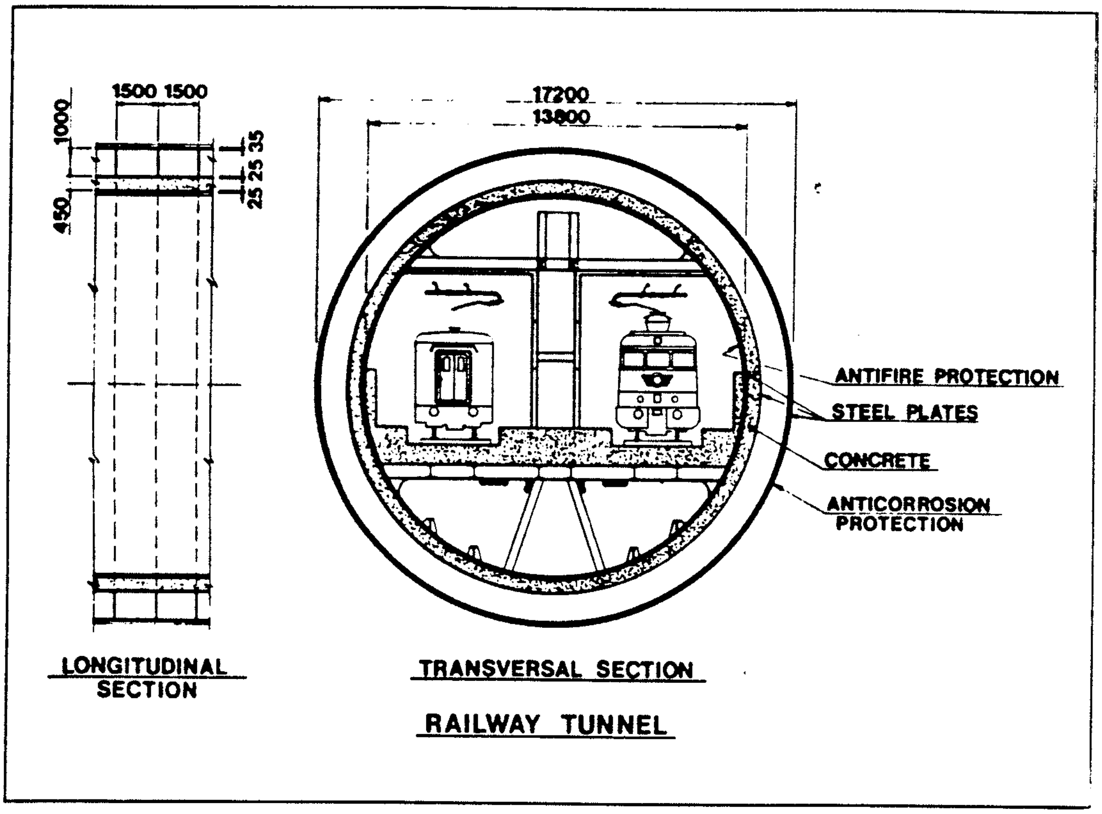
Messina Strait Crossing

MINIMUM TUNNEL DEPTH FOR THE SAFETY OF NAVIGATION



- ☆ seakeeping evaluation of the most probable extreme pitch + heave motions
- ☆ definition of the minimum tunnel depth from the sea level ➲ 30 m
- ☆ endorsement by the Maritime Safety Committee of the International Maritime Organisation upon request by the Italian Administration

ENI Project



- 3 underwater buoyant tunnels — 1 railways crossing, 2 road crossing
- double-hull cross section
- steel-concrete sandwich structure
- prefabricated modules of the primary structure (72 m)
- steel tethering system — D = 2050 mm THK = 68 mm, design load = 11,000 t
- gravity + pile foundations
- anti-seismic joints at the end-sections

ENI Project
(1988)

VALIDATION CARRIED OUT BY RINA AND DnV

- project premises;
- material selection;
- analyses:
  - static
  - hydrodynamic
  - seismic
  - traffic
- foundation design;
- inspection, maintenance and repair;
- construction and fabrication.

ENI Project
(1994)

↓

BASIC DESIGN

↓

certification carried out by RINA

in compliance with specific "Guidelines Proposed for Design of underwater Floating Tunnels"

- ✦ **fitness of the system for the functional purposes**
- ✦ **verification of the construction and assembling operations**
- ✦ **inspection, maintenance and repair plan**
- ✦ **equipments assessment**
- ✦ **safety and vulnerability analyses**

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

4. Messina Crossing

4.2 Brief history of Archimedes bridge in the Strait of Messina

A. Fiorentino
Universita Di Napoli

Antonio FIORENTINO

BRIEF HISTORY OF ARCHIMEDES BRIDGE IN THE STRAIT OF MESSINA

On March 28, 1968 the Italian Government published an international competition for ideas of a permanent crossing in the Strait of Messina, both for road and railway transit.

147 projects were presented and a first prize ex-aequo was awarded to Mr. Alan Grant, a British engineer, who submitted a completely new kind of crossing, based on SFT technology, that he christened "Archimedes Bridge" as a homage to the scientist who in the III century b.C. discovered the laws of buoyancy.

Five other projects deserved the first price ex-aequo, all of them based on traditional technology.

According to article 7 of the competition regulation, to the Author the intellectual property was granted, none the less Mr. Grant applied for patents in Italy (patent n.917716) in United Kingdom (patent n.1342343) in the United States (patent n.3738112) and in Greece (patent n.47282).

***

In 1972 Mr. Grant submitted his project to the Italian Ministry of Transports for a railway crossing in the Strait of Messina, by means of a circular shaped tunnel.

***

In 1973 a project for an Archimedes Bridge was proposed to the Greek Government for a road crossing of the Strait of Corinth. Later on, when an international tender was published, a detailed financial program was proposed.

On that occasion, and for the first time, such a structure was submitted to the examination of a Naval Register (in this case the Lloyd's Register of Shipping)

***

On June 11, 1981 the Italian Government created a public company named "Stretto di Messina S.p.A." appointed to project, to build and to run the most suitable permanent crossing among those proposed.

***

On November 23, 1983 a private company was founded, named "Ponte di Archimede nello Stretto di Messina S.p.A." which purchased from Mr.Grant the patents of his project of SFT and, on the same time, became holder of the intellectual property granted by the above mentioned article 7 of the competition published in 1968.

***

In 1984 the same Company "Ponte di Archimede" charged Registro Italiano Navale to perform a feasibility study of the project "Archimedes Bridge".

***

On November 2, 1984 Mr. Grant's project was officially submitted to the public company "Stretto di Messina S.p.A."

***

A dispute arose between the companies "Stretto di Messina" and "Ponte di Archimede". The former stated that any project of submerged tunnel had to comply with the prescription that the free water above the tunnel should not be less than 40 meters

The latter therefore applied to IMO (International Maritime Organization) to have their project approved with a depth of 30 meters. IMO stated that this depth was enough to permit free and safe passage of vessels in the Strait of Messina and therefore the company "Ponte di Archimede" was the first one to be entitled for a SFT at less than 40 m depth.

***

On September 26, 1990 "Ponte di Archimede" together with "Tecnomare", in their name and in the name of other Companies grouped in joint venture, submitted to the public company "Stretto di Messina" an offer for the realization and delivery, ready to operate, of a permanent crossing, SFT technology, in the Strait of Messina.

The public Company just answered with an acknowledgement of receipt.

***

The project of an Archimedes Bridge in the Strait of Messina was registered on April 8, 1993 at the office of the intellectual property of the Presidency of the Council. The only difference compared to the one submitted by Mr. Grant to the Ministry of Transports in 1972, is that now a second tunnel is proposed for road crossing, remaining the first one for railway.

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

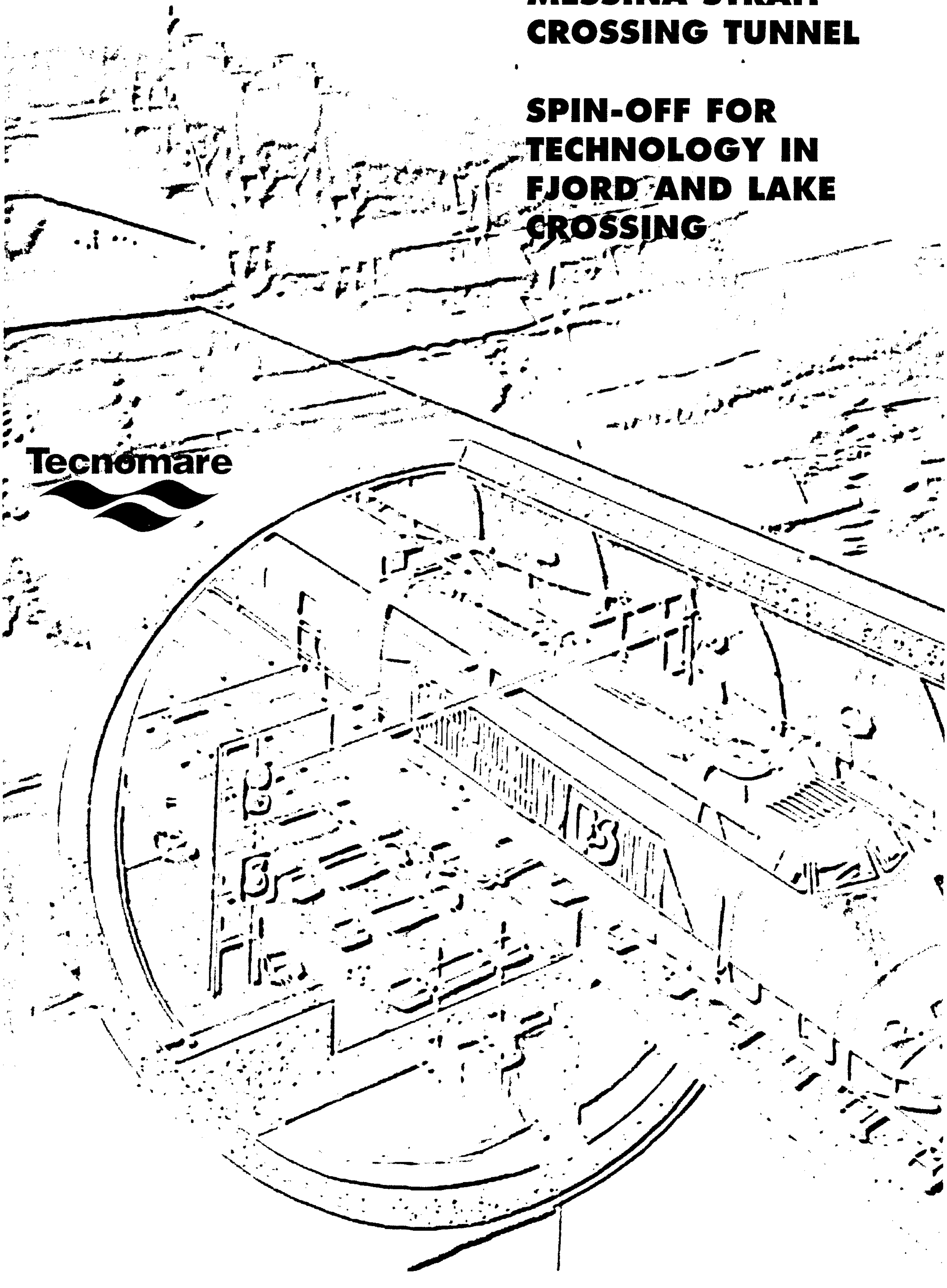
4. Messina Crossing

4.3 Messina Strait Crossing Tunnel
Spin-off for technology in fjord and lake crossing

V. Di Tella
Technomare S.p.A. Venezia

MESSINA STRAIT CROSSING TUNNEL

SPIN-OFF FOR TECHNOLOGY IN FJORD AND LAKE CROSSING



Tecnomare

Tecnomare

MESSINA STRAIT CROSSING TUNNEL

SPIN-OFF FOR TECHNOLOGY IN FJORD AND LAKE CROSSING

V. Di Tella, Tecnomare S.p.A., Venezia
F. Taddei, Snamprogetti S.p.A., Milano

INTRODUCTION

During 1992-93 Saipem, Snamprogetti and Tecnomare developed the project for crossing the Messina Strait by underwater buoyant tunnels on behalf of ENI - Consorzio per lo Stretto di Messina (Consortium for Messina Strait). The project was an outcome of a feasibility study undertaken by the said companies within a R&D project, and the results were presented during the First International Congress on Strait Crossing in Stavanger.

At the Third Symposium on Strait Crossing in Alesund in June 1994, the project was widely illustrated with three papers covering all aspects of this exciting project, which was then presented to the Italian Authorities for consideration in case the Messina Strait Crossing project will start.

The project envisages three tunnels for the complete connection of Sicily to the main land: one for the railway connection and two as motorways (three-lanes for each direction).

The project is characterised by very severe design conditions mainly for the environmental data and seabed morphology with the relevant earthquake conditions.

In addition to this, potential attack scenarios were considered and collisions with damaged sinking ships were evaluated and analysed.

This paper does not deal with the description of the Messina Strait project, but only gives a reference of the main design conditions and features of the tunnels which are summarised in the following figures:

FIG. 1 Design data of traffic for passengers and goods
FIG. 2 Environmental conditions
FIG. 3 Potential attack scenarios
FIG. 4 An example of results of damages due to external contact charge
FIG. 5 Statistical analysis of ships passing the Messina Strait

FIG. 6 Statistical analysis of different class ship for transit and shuttle traffic in order to define the design impact energy on the three tunnels

FIG. 7 Sea current pattern

FIG. 8 Position of faults in the interested area

FIG. 9 Earthquake epicentres

FIG. 10 Seabed morphology

FIG. 11 The general configuration of the railway tunnel with its mooring tether and the interface module between shore and on-land tunnel is shown

FIG. 12 Shows the cross section of the railway tunnel which is similar to the motorway tunnel and is characterised by the main internal sandwich cylinder (steel-concrete-steel) and the external (35 mm thickness) cylinder which role is the protection against possible external attack and ship collision.

The results of this basic design and the envisaged structural configuration fully satisfy the extremely critical design conditions which characterise the site and the very stringent safety requirements against accidental events due to ship impact and sabotage.
Moreover, the solution adopted for construction and installation of the tunnel elements and for the technological aspects of the system (i.e. modules design and installation, tethers and their installation tools, seismic joints and gasket system), are directly based on proven technologies both in the offshore and in the buried tunnel fields.

Because of these peculiarities, the proposed submerged tunnel configuration represents the solution for permanent crossing of areas characterised by different scenarios in terms of operative loads, environmental and geotechnical conditions, water depth and length of the link.

APPLICATION OF THE TECHNOLOGY TO DIFFERENT SCENARIOS

The technological know-how of the ENI Group Companies and specific design methods developed for the Messina Strait project can be much more easily applied to different operative scenarios like the ones of the Norwegian fjords and lakes.
Two possible applications are presented and discussed in the following .

HØGSFJORD

During the Third Symposium on Strait Crossing in Alesund (Norway) in June 1994, the paper dealing with the Hϕgsfjord motorway crossing was presented by
- A.Cannavacciuolo, R.Faldini, F.Casola.

The paper outlines the main features of the envisaged technical solution with particular attention to the system configuration, design criteria, structural components, and procedures for construction and installation. Please apply to this paper for more detailed technical information. Here we just represent the figures from 13 to 19 showing the general configuration of the tunnel, the cross section, the anchoring system and the installation procedure, for a due comparison.

LUGANO LAKE

The underwater buoyant tunnel, due to its flexibility for the selection of the route, and for the compatibility with respect to the environmental impact is particularly adapted for the crossing of lakes where the constraint imposed by the respect of the environment and the landscape and the water depth are relevant.
Lugano lake is an example for this kind of application.
The Swiss Authorities have decided to realise a high speed railway connecting North and South passing via Lugano (Project Alp Transit): several alternative routes are under evaluation.
Saipem, Snamprogetti and Tecnomare have developed a solution for the railway lake crossing which was presented to the Authorities for consideration. The main design conditions and results achieved are presented in the following.
Fig. 20 represents a possible route for the lake crossing but the concept characteristics is such that the route can be selected without constraints and may fit very well with urban plans of the two adjacent regions Canton Ticino and Lombardia.
Fig. 21 shows the main design conditions assumed for this feasibility study, which could be better defined when the route will be selected and the geotechnical information of the site will be available.
Fig. 22 shows the general configuration of the floating tunnel which is 1056 m in length between the two galleries on land in Terricolo and Olivella.
The tunnel has an internal circumference of 11 m and its axis is at -14 m from the M.W.L. and contains a double rail.

The minimum immersion level of the tunnel elements is -7,5 m in order not to interfere with the lake navigation.

The bottom of the lake in the interested area is characterised by a limited slope in the central part while close to the shore inclines at 30÷40 degrees.

The characteristics of the soil are that of a volcanic rock covered by a surface layer of 50 m of sediments.

The floating tunnel is mainly constituted by:

- stayed buoyant tunnel
- 2 interface modules at Terricolo and Olivella fixed on the lake bottom.

Tunnel

The tunnel is composed of 10 modules each 100 m long each with mooring stations at 50 m distance. The general configuration is shown in Fig. 22.

Each module is composed of a main cylindrical shell with an internal diameter of 11 m and thickness 25 mm (see Fig. 23).

The cylinder is stiffened by external T rings 475 mm height and spaced 2500 mm. The structure is completed with an external shell 8 mm thickness welded to the plate of the ring stiffness.

A concrete filling between the internal and external shell realises a sandwich structure with a high degree of resistance and safety barrier against sea water. Fig. 24 represents the tunnel cross section.

Two couples of strands are foreseen at each mooring station: one vertical in steel pipes and the second in kevlar fibre with an inclination of 45° with respect to the vertical.

While the steel strands are installed together with the tunnel module, the strands in kevlar are installed later with a pretension of 1100 kN (see Fig. 23).

Each strand is connected to the lake bottom by means of a foundation pile having a diameter of 1212.2 m and thickness ranging from 38.1 to 50.8 mm.

The piles length range from 15 m near the shore approach and 60 m in the central part of the lake.

Interface Modules

The connection between the floating part of the tunnel and the parts on land is made by two interface modules.

They are composed of a structural sandwich cylinder with internal diameter of 11 m connected to a box shaped structure which is piled to the lake bottom.
Both modules are 28 m length but with different foundation and structural configurations.
The starting interface module is structurally connected to the floating tunnel and has also a temporary access to the tunnel above the water level, which is utilised during the installation operation and is removed at the end of operations (see Fig. 25).
The second interface module is connected to the floating tunnel via a special structural joint allowing displacement and rotation respectively +/- 0.5 m and +/- 0.01 rad (see Fig. 26).

Structure Behaviour

The tunnel configuration was developed through a preliminary dimensioning of the main components and a verification of the loads to which the overall structure and its component are subjected during construction, installation and operative life, in this particular case the earthquake and the railway loads are the ones relevant for this design.
The analyses were performed on an overall structural models simulating the tunnel structure with its mooring system and a proper F.E. definition of major components of the tunnel.
The unit net buoyancy 400 kN/m is determined by operating and seismic loads.
A very important loading condition in this case is the fatigue due to railway traffic.
The daily traffic of 450 trains during the 200 year life induces approximately 30 million cycles on the structure and on the vertical strands.
The structural analysis of all the components of the structure were performed for conditions with return period of 400 years and 10,000 years for extreme and fatigue loads.
In particular railway traffic analysis was performed in order to verify the safety and comfort of passengers also during seismic events; all the results were satisfactory.

Weight Estimate

On the basis of the analysis performed, a weight calculation of the tunnel main components was performed.
Fig. 27 gives the weight breakdown.

Construction and Transportation

The fabrication of modules is foreseen in 2 phases:
- Prefabrication of main elements in qualified shops and their transport, possibly via railway, to the assembling yard
- Assembling of the module in a proper dedicated yard, close to the installation site.

Fig. 28 shows a typical lay-out of the assembling site.
The modules completed and equipped with temporary bulkheads, are skidded and launched. Small tugs will tow the floating modules to the installation site.

Installation

The installation operations will start with the foundations piles and the vertical steel strand inserted and grouted in the proper sleeves.
At the same time the trenches for the interface modules will be prepared.
Fig. 29 shows the installation of the interface modules that will be piled together with the foundation caissons.

The modules towed to the installation site will be temporary moored vertically over the corresponding 4 vertical strands already installed (see Fig. 30).
Special followers with a proper clamping system will be used for the immersion of the module to the appropriate draft.
Once the module has been positioned close to the previously installed module (interface or current), the compartment between the two temporary bulkheads of the two adjacent modules will be deballasted. The depression caused by the deballasting operations will generate the necessary contact force between the module ends which permits the close positioning and the welding of the main shells of the tunnel, from the internal.
For this operation special attention has been given to reduce as much as possible the use of heavy floating vessels. All operations are performed using small barges and existing vessels in the lake with a lifting capacity of 3000 kN.

Environmental Impact

In the definition of the general configuration of the crossing and of the construction , installation procedures, particular attention has been paid to all the aspects related to environmental impact according to the Swiss Federal Authority issue OEIA, Oct. 19, 1988. The study was conducted in four phases:

- identification of possible impact source due to specific technology
- characteristic of the environment at the installation sites
- identification of possible interference tunnel/environment during different phases: construction, installation and operation
- program for the detailed environmental impact analysis.

The following components were considered:

- soil
- hydraulic lake environment
- lake soil characteristics
- lake and coastal echo-system
- atmosphere
- vibrations and rumour
- landscape and archaeological zones.

Fig. 31 shows the potential impact matrix of the proposed crossing project.
As a result of the study, it was confirmed that the environmental impact of this project is very limited if compared with other possible crossing solutions and in any case well within acceptability.
A possible problem rose during the construction phase due to vibration and rumour at the assembling site, but the company's experience of similar cases leads to say that with the assumption of a proper mitigation system this problem can easily be solved.

CONCLUSIONS

As a result of the work performed by the ENI Group of Companies involved in these experiences of strait crossing with floating tunnel technology, we can definitely say that:

- The solution proposed for the Messina Strait and the application of the same technology to different scenarios demonstrated the flexibility of the concept which can be adapted case by case to specific design conditions.
- The technology is safe, feasible and proven even if there is no application of this technology for crossing, components, operation, construction and installation procedure are all well proven in the offshore field for environmental conditions much

more severe than ones assumed as design conditions for the proposed solution even in the case of Messina.

- The technology is very well compatible with the environment both in the phase of construction and during operative life.
- The flexibility of the concept and the different applications show that it is quite independent from the length of the crossing and from the water depth and gives the designer the maximum possible degree of freedom in the selection of the route specially when urban and landscape constraints are relevant.
- The modularity and prefabrication techniques have a very limited environmental impact during the construction phases and permits to utilise existing construction infrastructures as much as possible.
- The solution has shown very good behaviour with respect to the railway crossing and particularly for the need high speed railways.
- This solution is efficient from the point of view of cost/benefit ratio taking into account the possibility of straightening the route of the crossing.

FIG.1 - DESIGN DATA OF TRAFFIC FOR PASSENGERS AND GOODS

PASSENGER TRAFFIC PREVISIONS WITH FIRM LINK (milion of unit)

	2005	2010	2021	
RAILWAY	7325	8325	11064	
ROADWAY	8531	9862	13598	NATIONAL
TOTAL	15856	18187	24662	
RAILWAY	27025	27061	28117	
ROADWAY	5977	23525	22375	LOCAL
TOTAL	33002	50586	50492	

GOODS TRAFFIC PREVISIONS WITH FIRM LINK (milion of tonnes)

	2005	2010	2021	
RAILWAY	5420	6468	9000	
ROADWAY	8993	10971	16982	NATIONAL
TOTAL	14413	17439	25982	
RAILWAY	122	146	202	
ROADWAY	809	3600	3544	LOCAL
TOTAL	931	3746	3746	

FIG.2 - ENVIRONMENTAL DATA

WAVES

Significant height (m)	Return period (y)
5.0	50
6.5	400
8.5	1000

CURRENTS

Maximum speed (m/s)	Return period (y)
3.62	50
3.72	400
4.02	1000

EARTHQUAKE

Horizontal accel. (g)	Duration (s)	Return period (y)
0.127	10	50
0.360	15	400
0.650	25	2000

WIND

Velocity (km/h)	Gust (km/h)	Return period (y)
90	10	1
125	15	100

FIG.3 - POTENTIAL ATTACK SCENARIOS

External attack scenario	Structural objective	Equivalent TNT Charge weight (kg)	Standoff (m)
depth charge	external shell	100	5 - 10
driver	external shell and anchoring system	15	contact charge

Internal attack scenario	Structural objective	Equivalent TNT Charge weight (kg)	Standoff (m)
car bomb	tunnel internal wall	400	2
truck bomb	tunnel internal wall	1000	2 - 3
bomb carried on a train	tunnel internal wall	20	2
bomb thrown from a train	tunnel internal wall	20	1
foot mode bomb	tunnel internal wall	20	contact charge

FIG.4 - STRUCTURAL DAMAGE DUE TO EXTERNAL CONTACT CHARGE

CONTACT CHARGE WEIGHT (equivalent TNT) = 20 KG

ES	Breaching; collision of ES fragments against SES
SES	Low plastic deformation
C	Crushing
SIS	Very low plastic deformation

CONTACT CHARGE WEIGHT (equivalent TNT) = 40 KG

ES	Breaching; collision of ES fragments against SES
SES	Medium plastic deformation
C	Crushing
SIS	Low plastic deformation

CONTACT CHARGE WEIGHT (equivalent TNT) = 60 KG

ES	Breaching; collision of ES fragments against SES
SES	Partial tearing
C	Crushing
SIS	Low plastic deformation

CONTACT CHARGE WEIGHT (equivalent TNT) = [illegible] KG

ES	Breaching; collision of ES fragments against SES
SES	Complete tearing
C	Crushing
SIS	Medium plastic deformation

Legend

ES	External structure
SES	Sandwich External Shell
C	Concrete
SIS	Sandwich Internal Shell

FIG.5 - REFERENCE SHIP CHARACTERISTICS

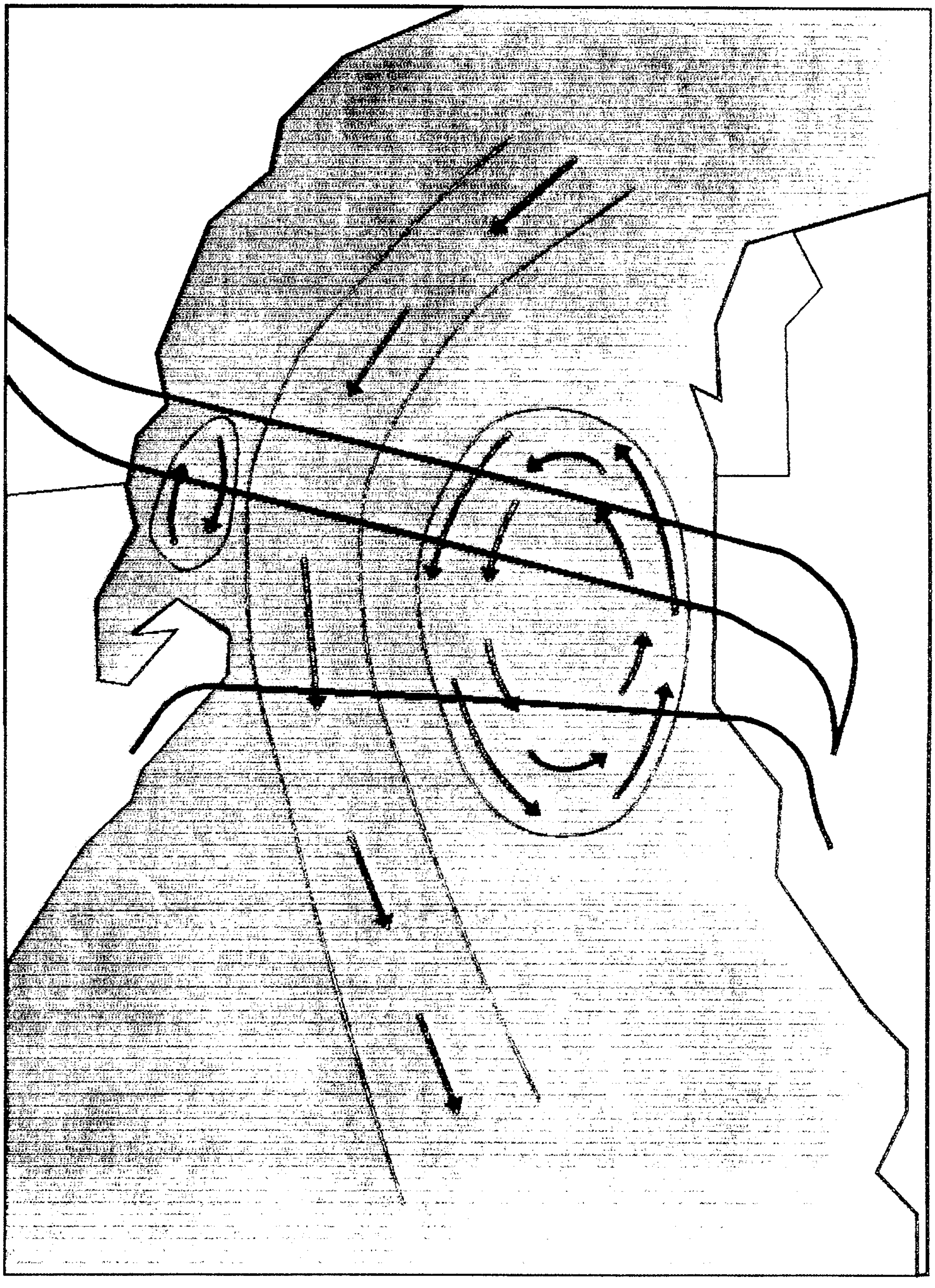
SHIP TYPE	LOAD DISPLACEMENT (t)	LENGTH (m)	WIDTH (m)
I Class	60	40	10
II Class	2,100	65	13
III Class	11,600	115	17
IV Class	70,000	190	28
V Class	160,000	300	48
Ferryboat type 1	2,400	100	17
Ferryboat type 2	7,200	150	20
Hydrofoil boat	240	30	7

FIG.6 - REFERENCE FLEET DISTRIBUTION OF SHIPS

TRANSIT TRAFFIC	SHIPS WITH COMPARTMENTS	SHIPS WITHOUT COMPARTMENTS
I Class Ship	39.8 %	60.2 %
II Class Ship	8.7 %	91.3 %
III Class Ship	6.9 %	93.1 %
IV Class Ship	14.5 %	85.5 %
V Class Ship	100 %	0 %

SHUTTLE TRAFFIC (without compartments)	FERRYBOATS TYPE 1	FERRYBOATS TYPE 2	HYDROFOIL BOATS
Road Tunnels	60.5 %	38.1 %	1.4 %
Railway Tunnels	19.7 %	42.8 %	37.5 %

Eni
CONSORZIO
PER LO STRETTO DI MESSINA

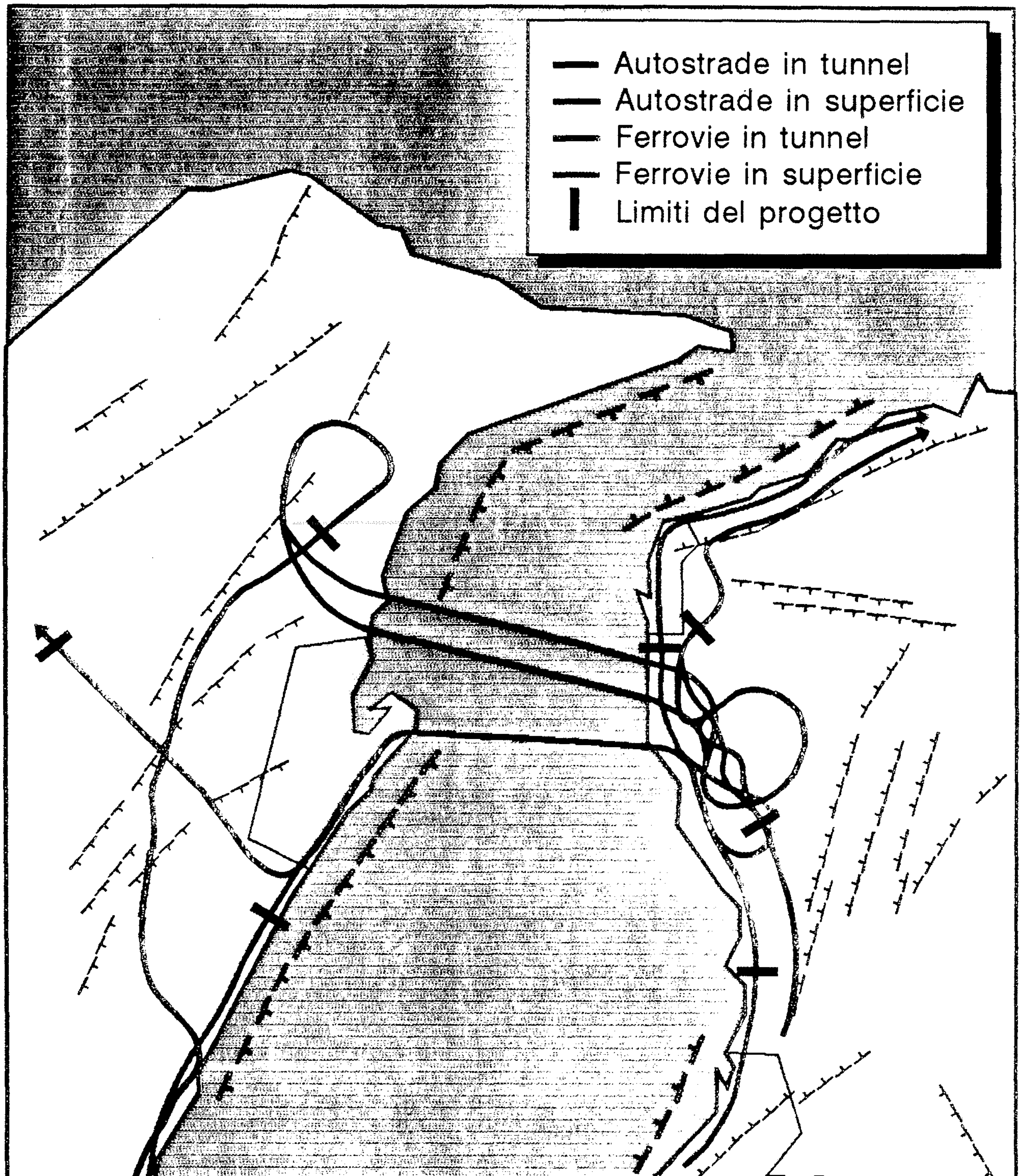


SCHEMATIC DIAGRAM
OF SEA CURRENTS

CURRENTS CH3

Eni

CONSORZIO
PER LO STRETTO DI MESSINA

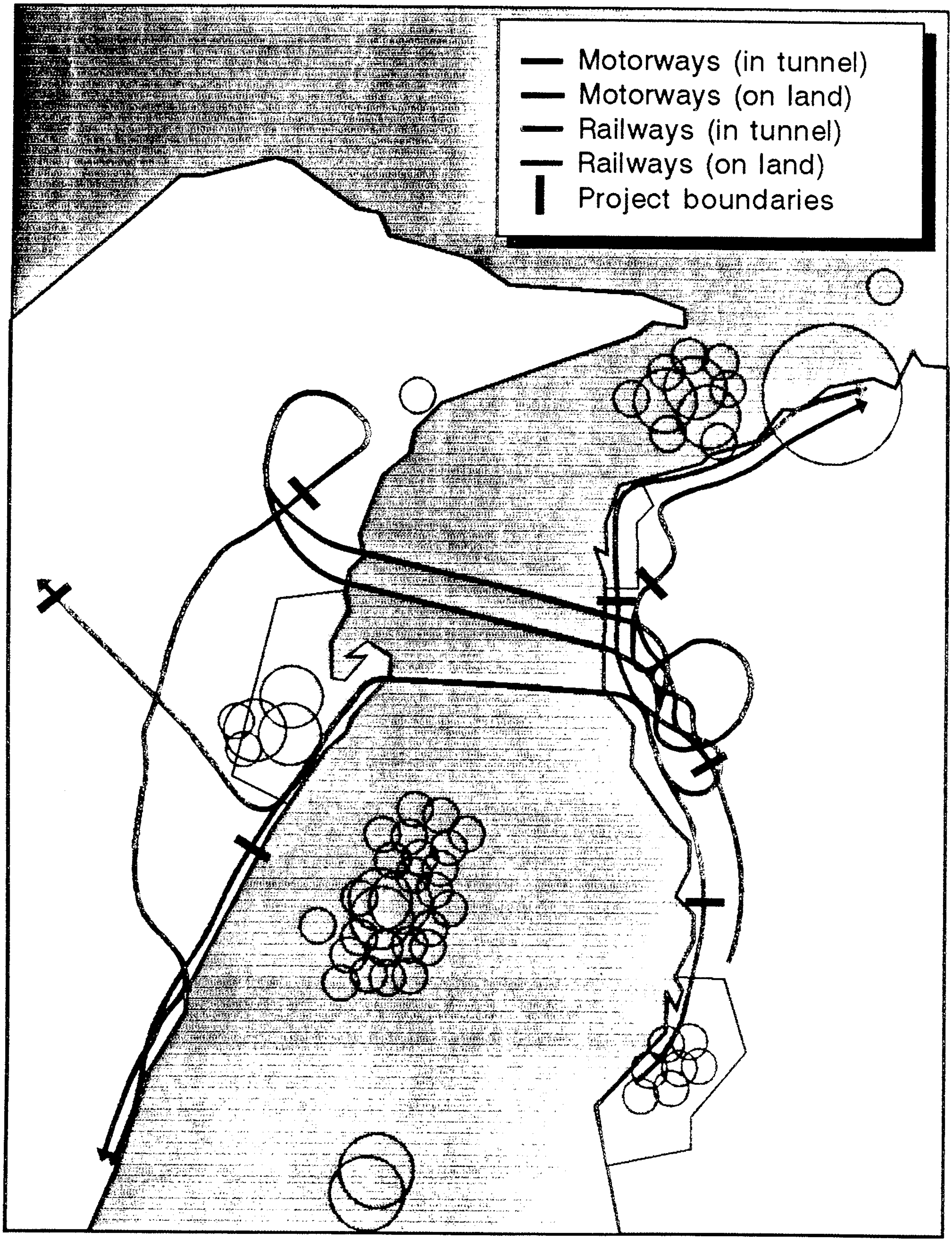


DISPOSITION OF FAULTS

FAULTDSP.CH3

Eni

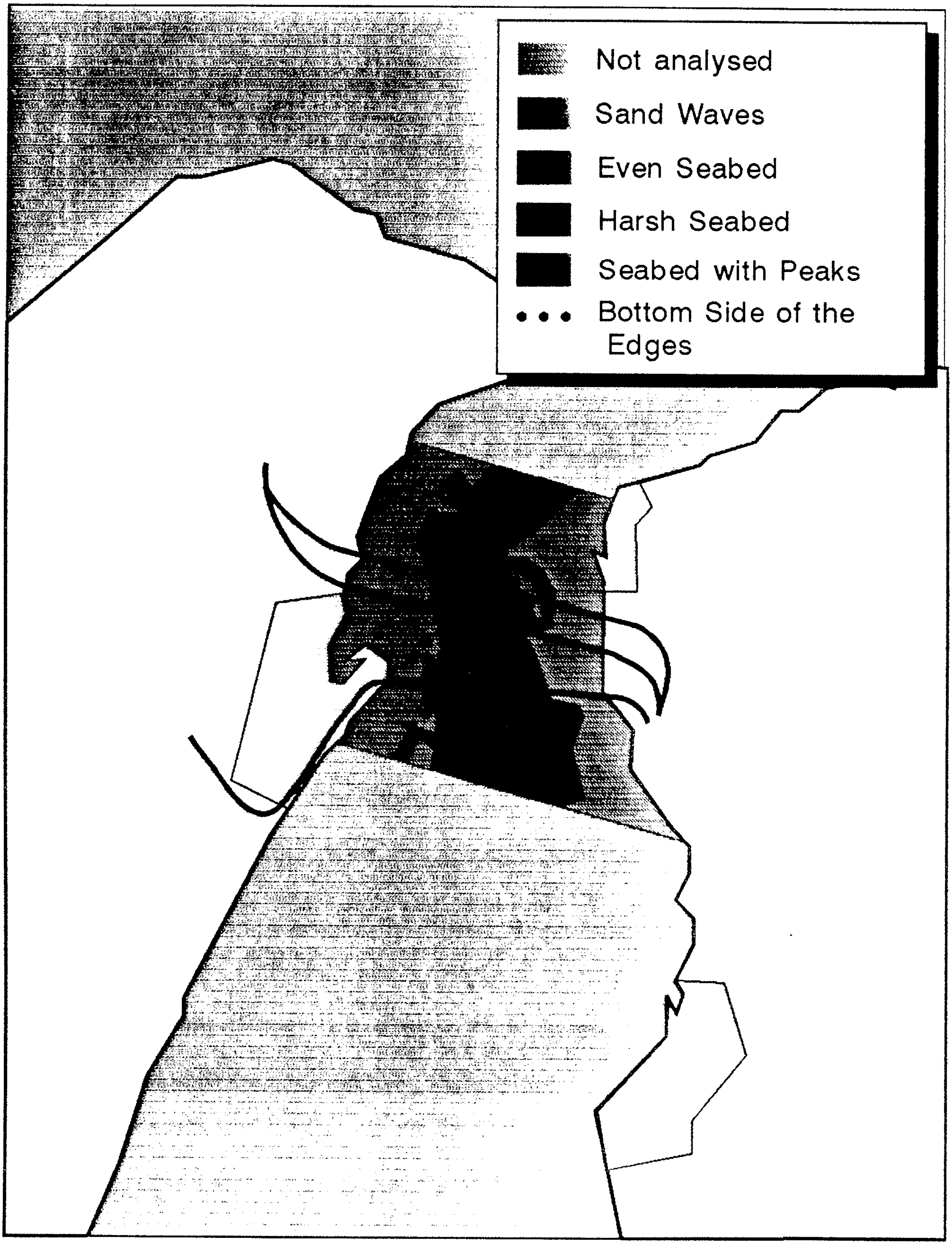
CONSORZIO
PER LO STRETTO DI MESSINA



EARTHQUAKE EPICENTRES

Eni

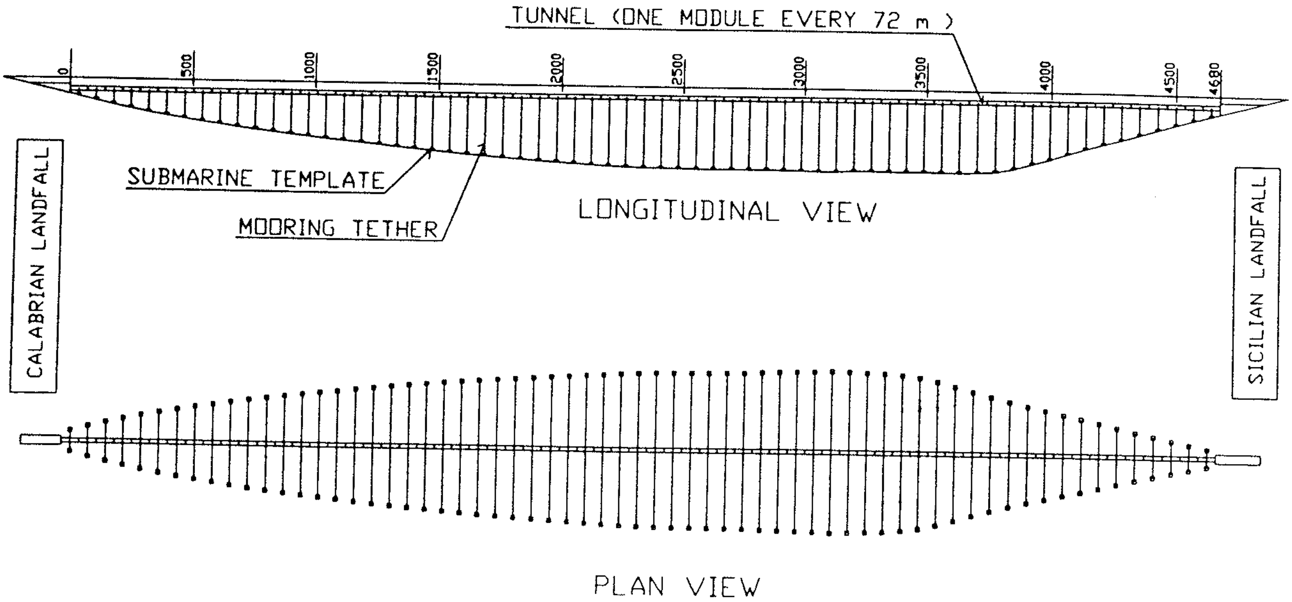
CONSORZIO
PER LO STRETTO DI MESSINA

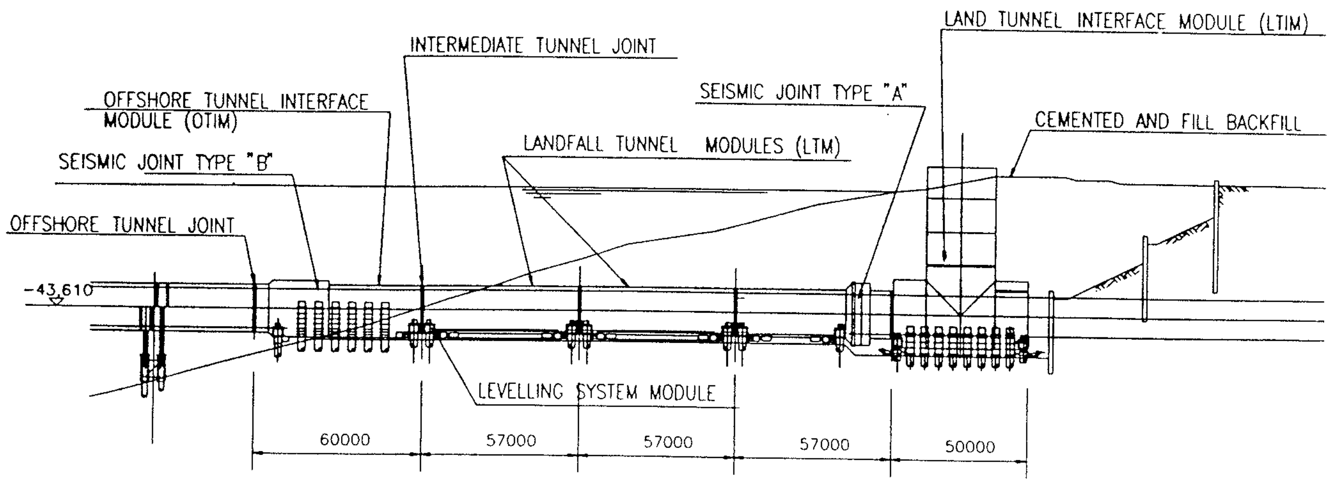


SEABED MORPHOLOGY
(FROM LITERATURE)

MORPHMAP.CH3

CONSORZIO
PER LO STRETTO DI MESSINA





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CONSORZIO
PER LO STRETTO DI MESSINA

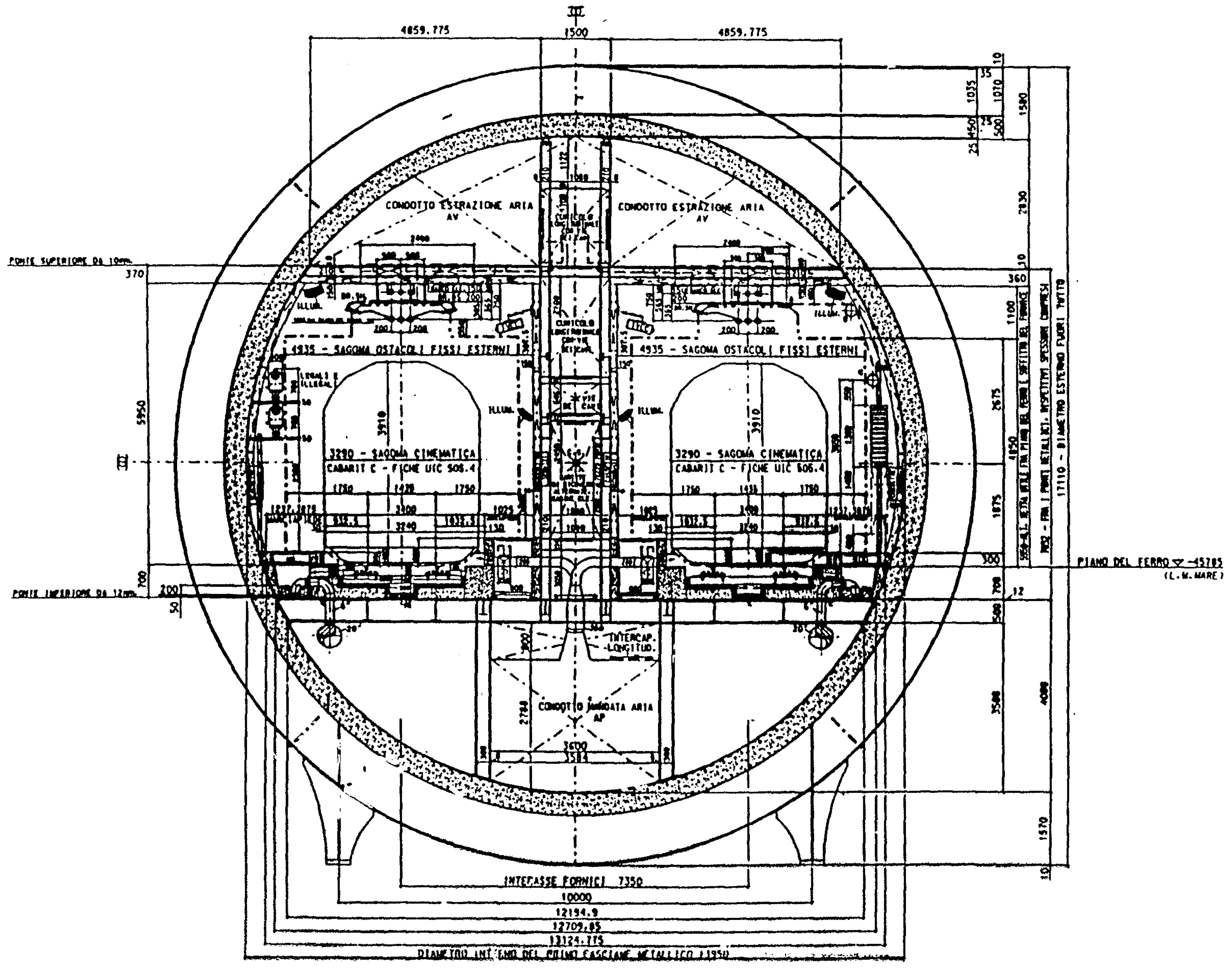


Fig.13 - GENERAL CONFIGURATION OF THE CROSSING

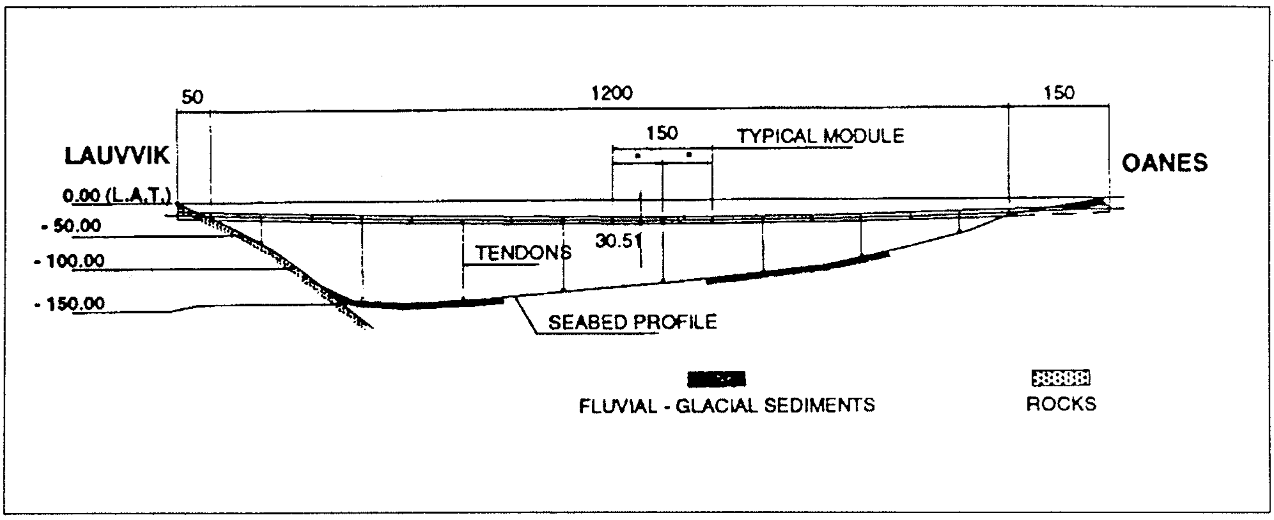


Fig.14 - TUNNEL CROSS SECTION

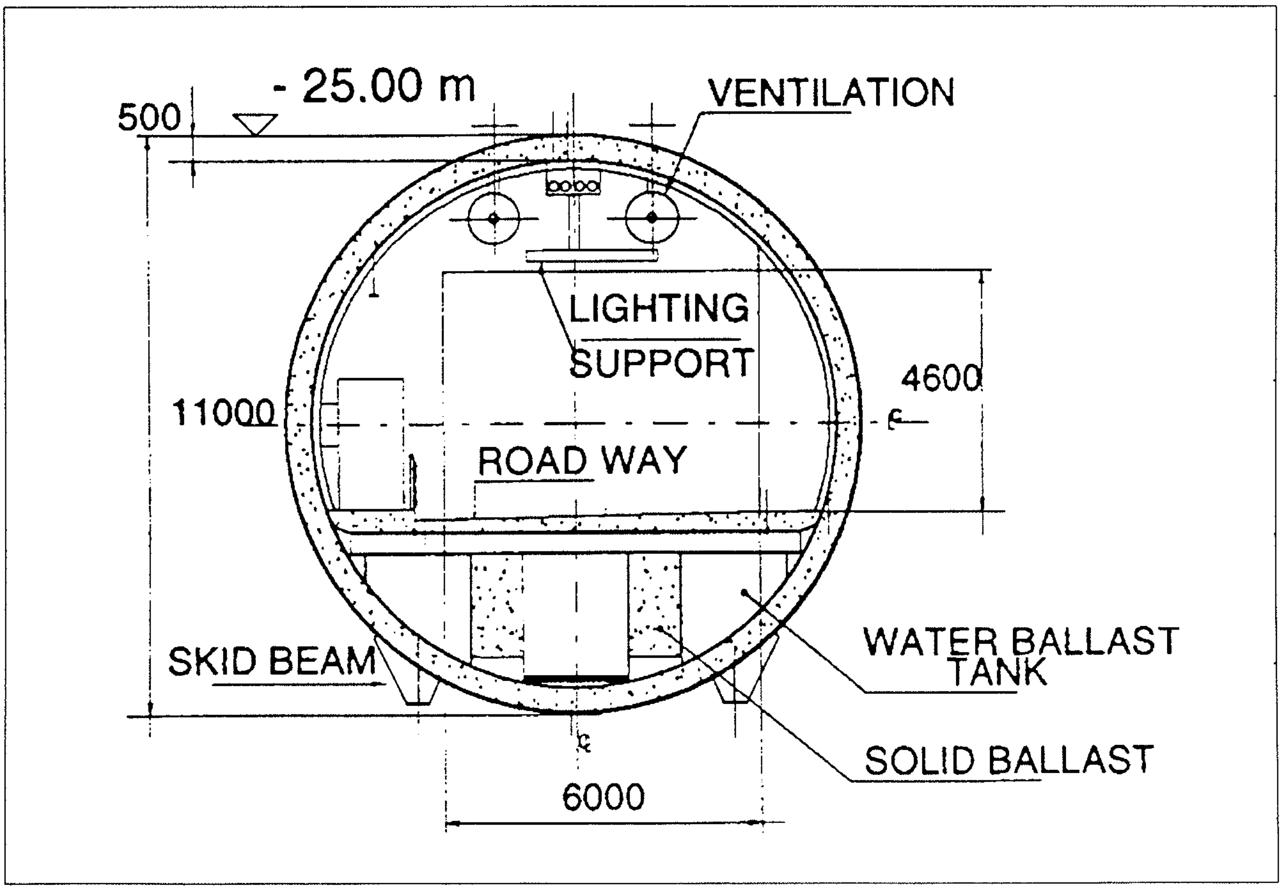


Fig.15 - ANCHORING STATION CONFIGURATION AND PLAN VIEW OF A TYPICAL ANCHORED MODULE

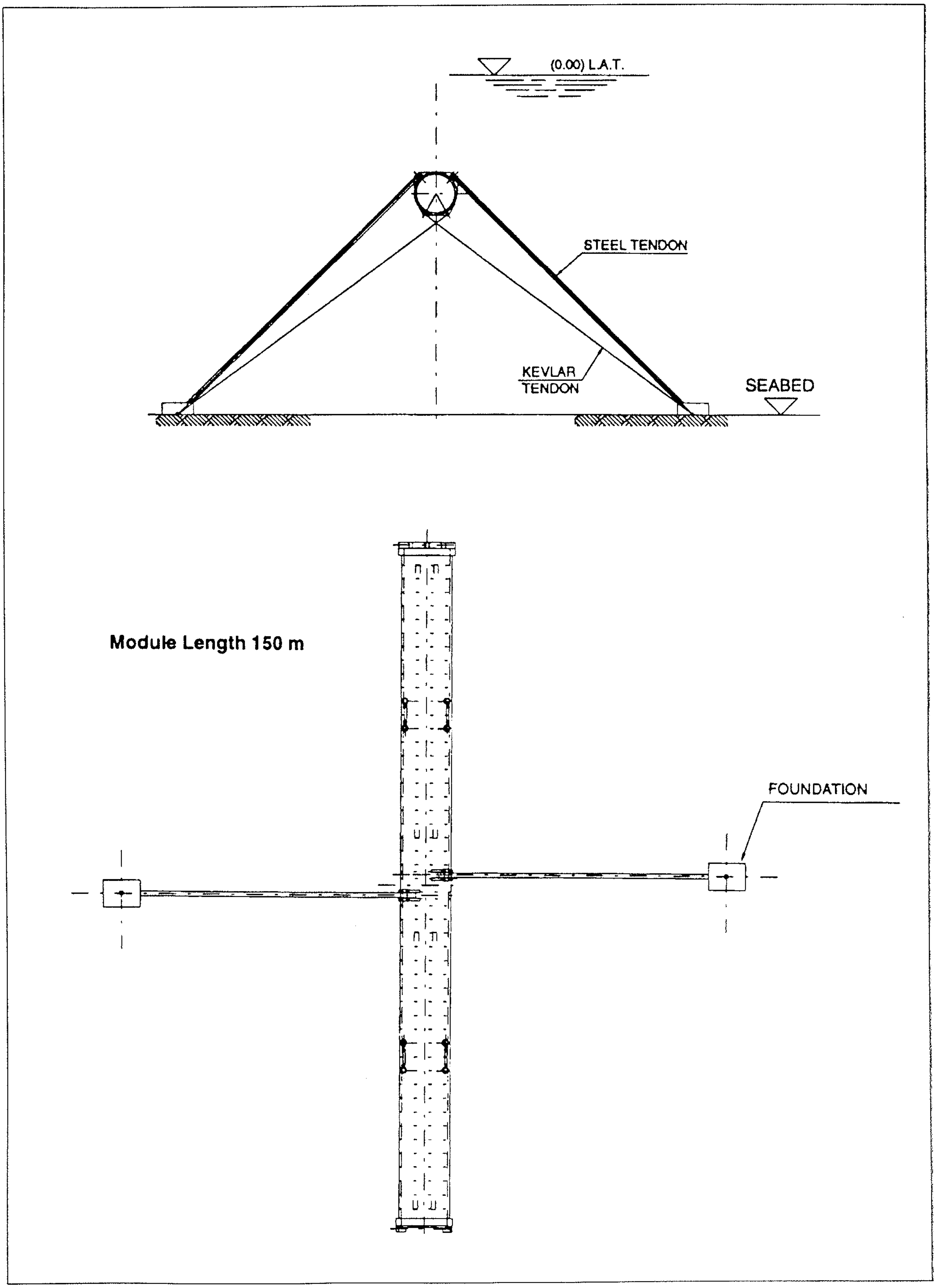


Fig.16 - MODULE IMMERSION

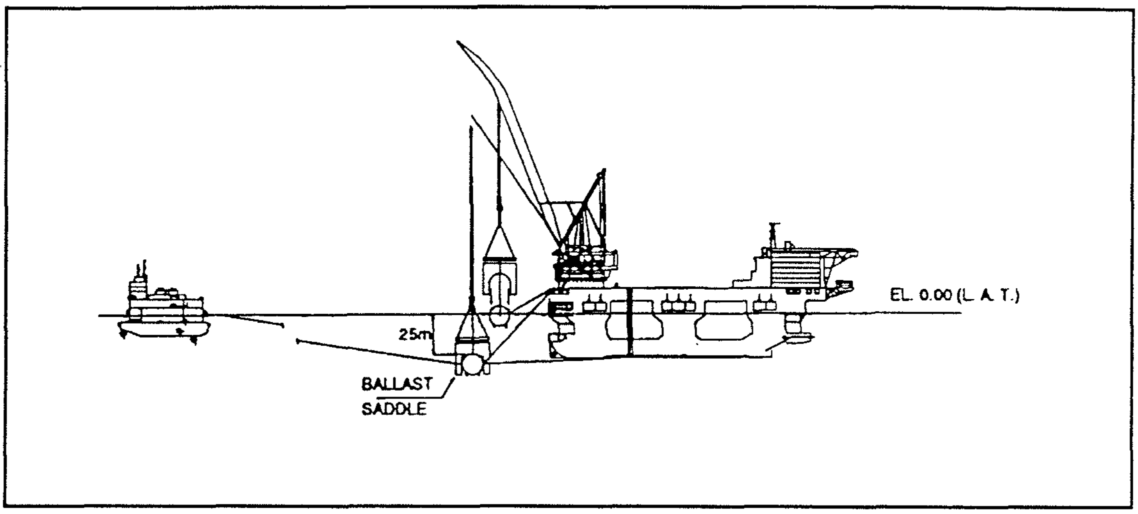


Fig.17 - MODULE ALIGNMENT AND DOCKING

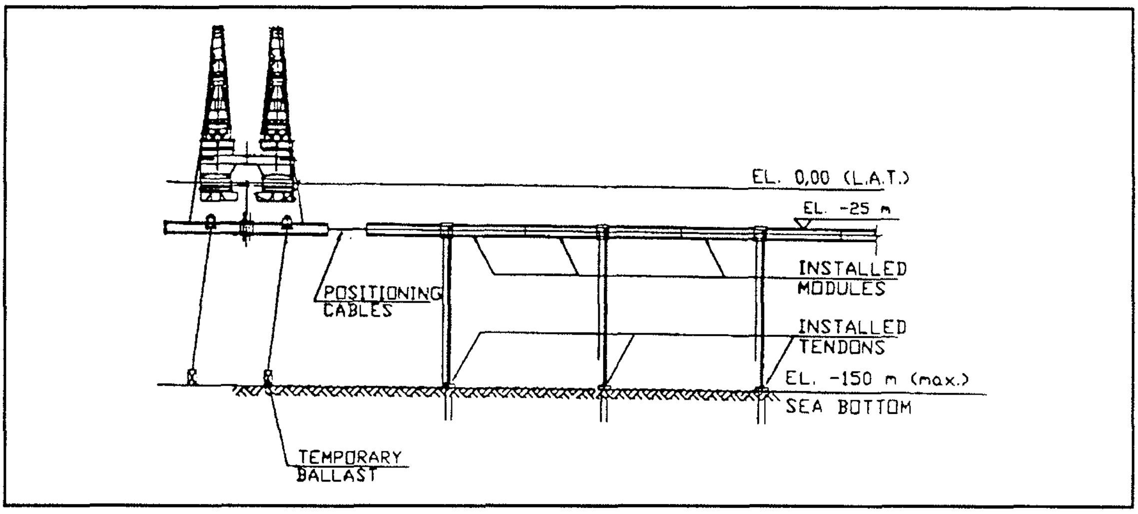


Fig.18 - STEEL TENDON INSTALLATION

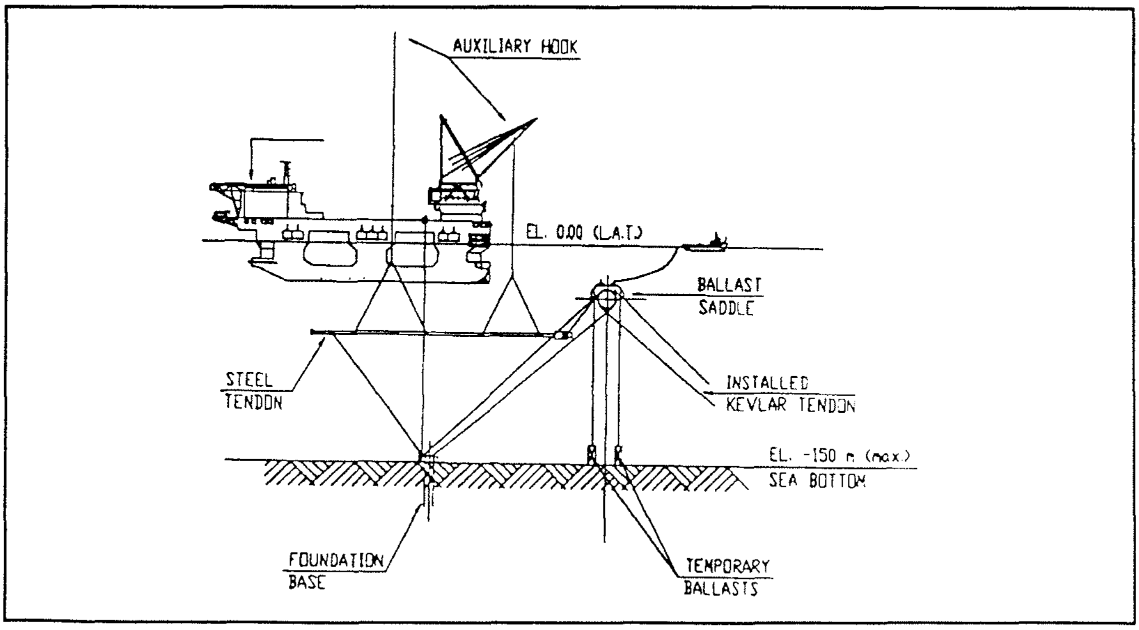


Fig.20 - REFERENCE ROUTE

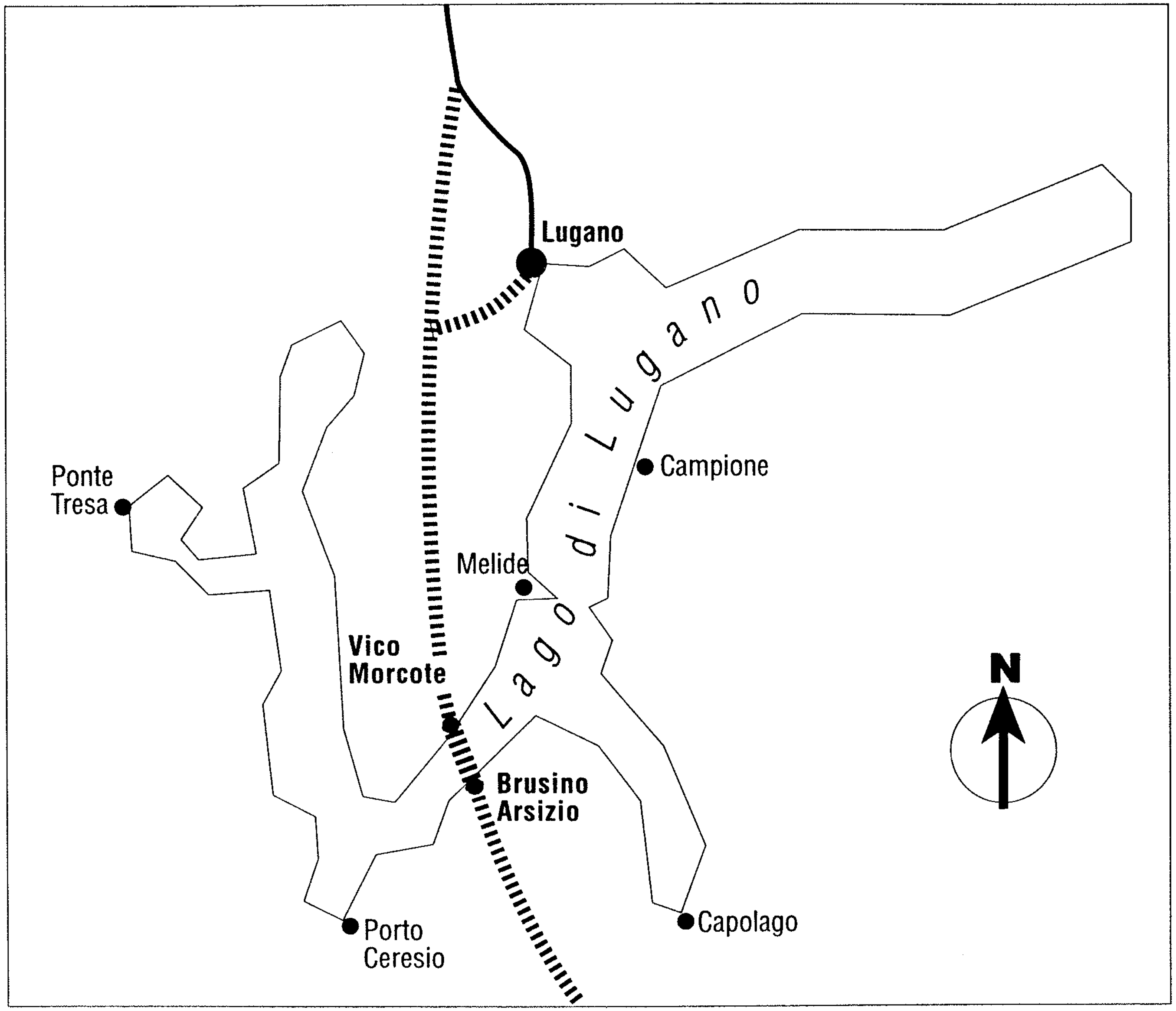


Fig.21 - ENVIRONMENTAL DATA

WAVES

Significant height (m)	Return period (y)
0.60	10
0.70	50
0.85	400
1.10	10000

CURRENTS

Maximum speed (m/s)	Return period (y)
0.1	operative
0.2	10
0.3	50
0.4	400
0.5	10000

EARTHQUAKE

Horizontal accel. (g)	Return period (y)
0.05	200
0.10	2000

WIND

Velocity (km/h)	Return period (y)
25	1
40	100

Fig. 22 - TUNNEL LONGITUDINAL VIEW

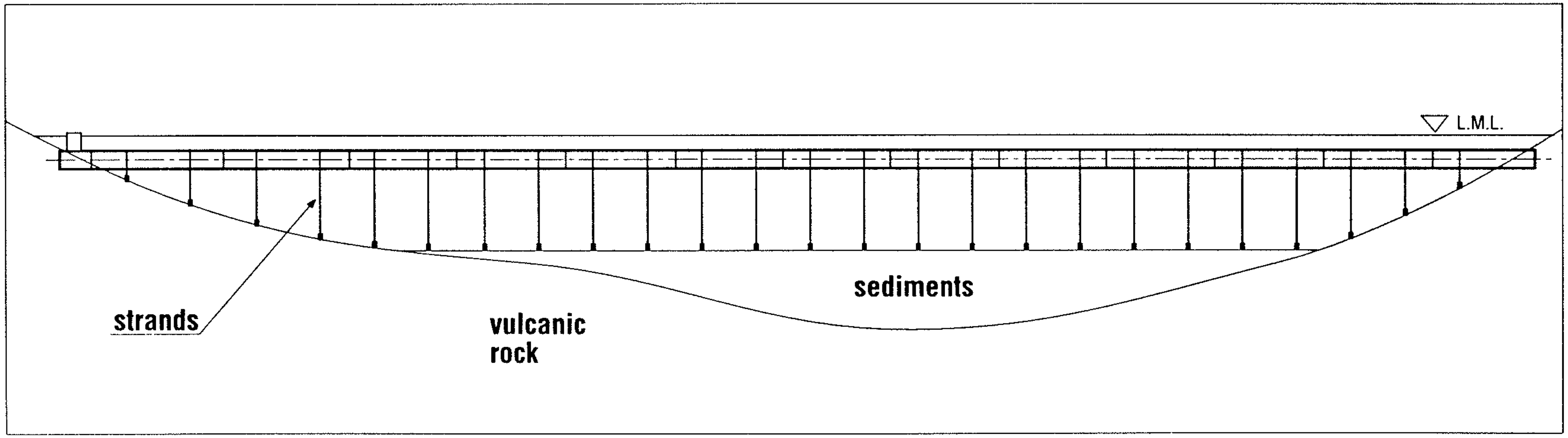


Fig. 23 - MODULE AND MOORING SYSTEM

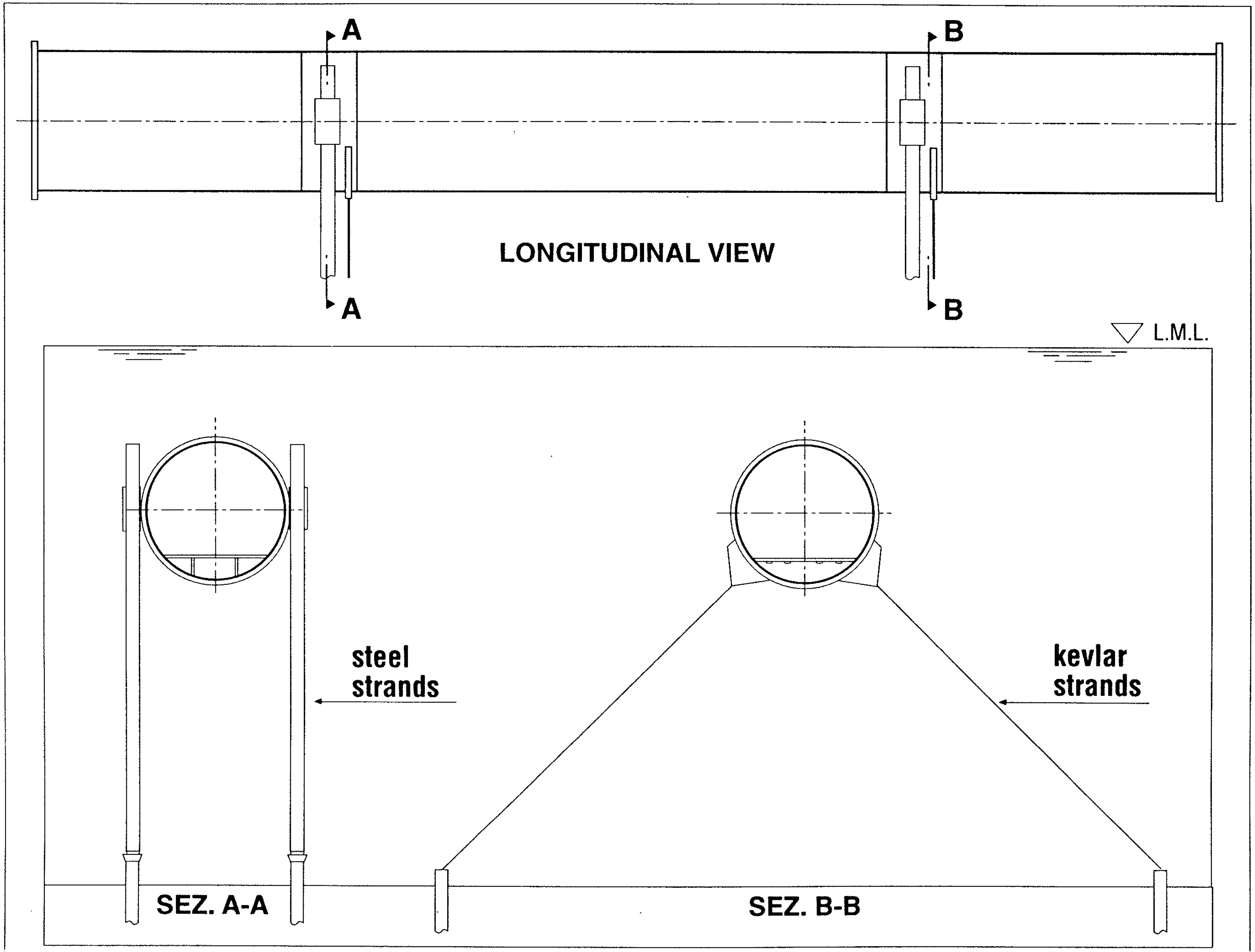


Fig. 24 - TUNNEL CROSS SECTION

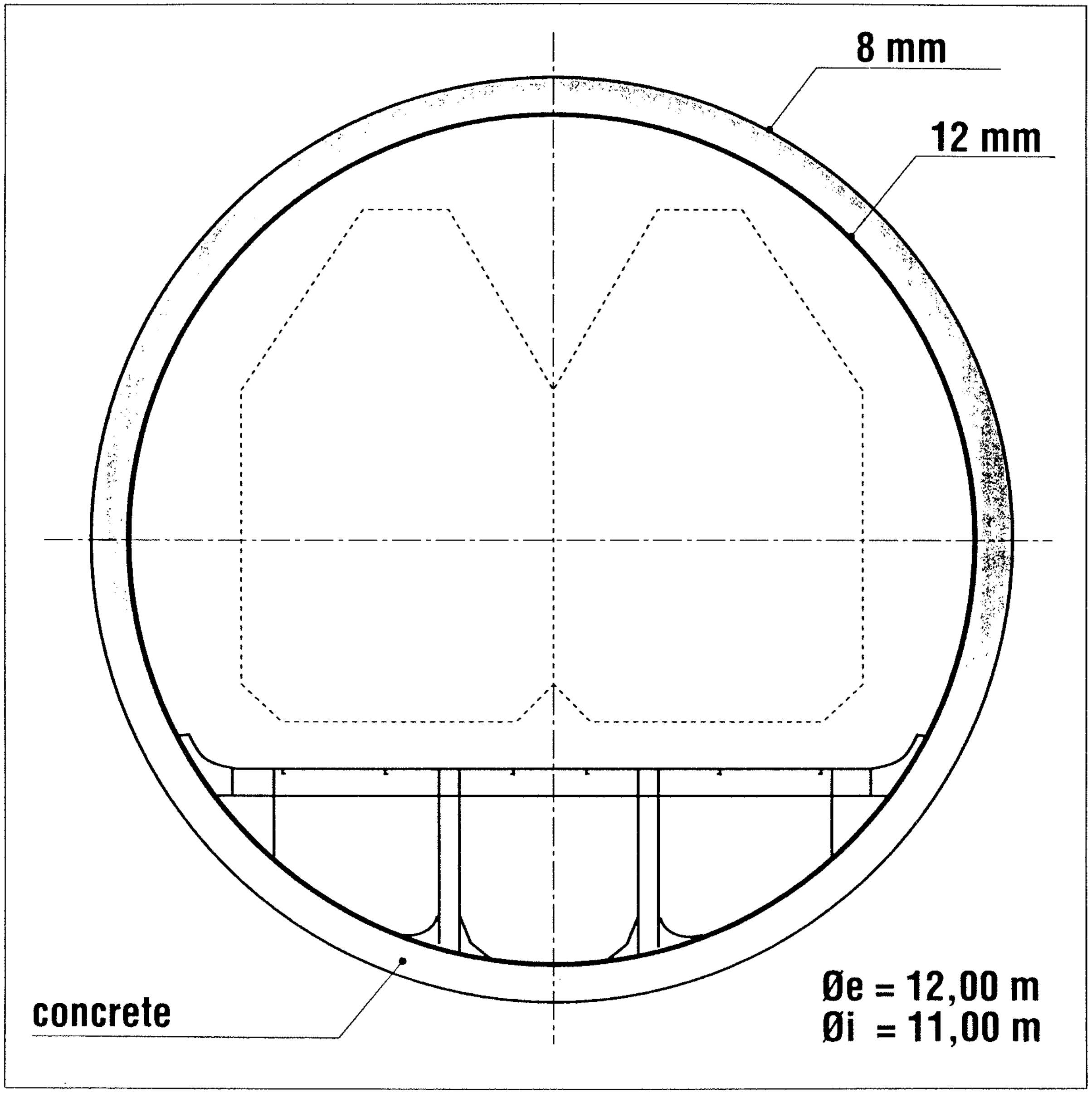


Fig. 25 - STARTING INTERFACE MODULE

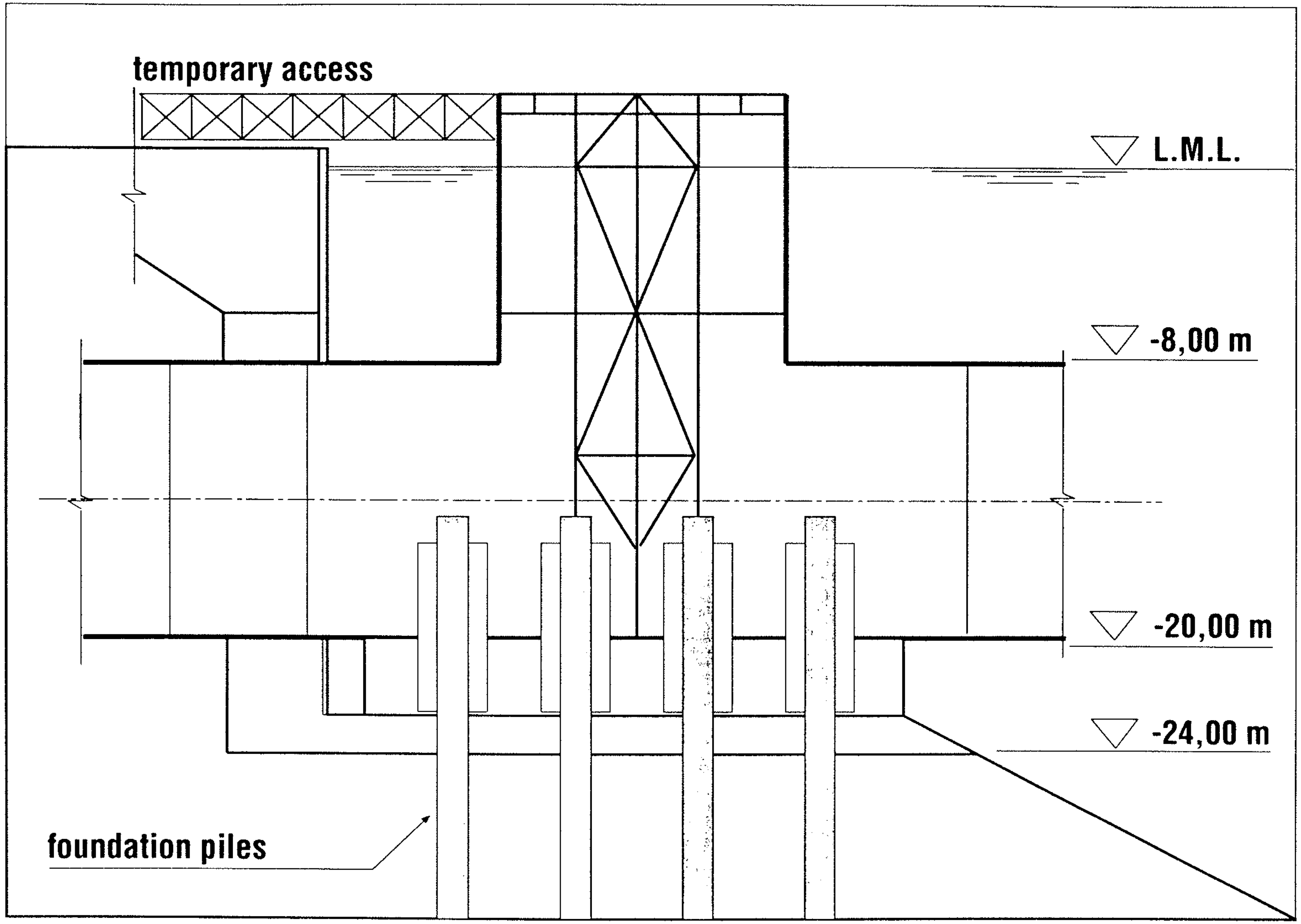


Fig. 26 - INTERFACE MODULE

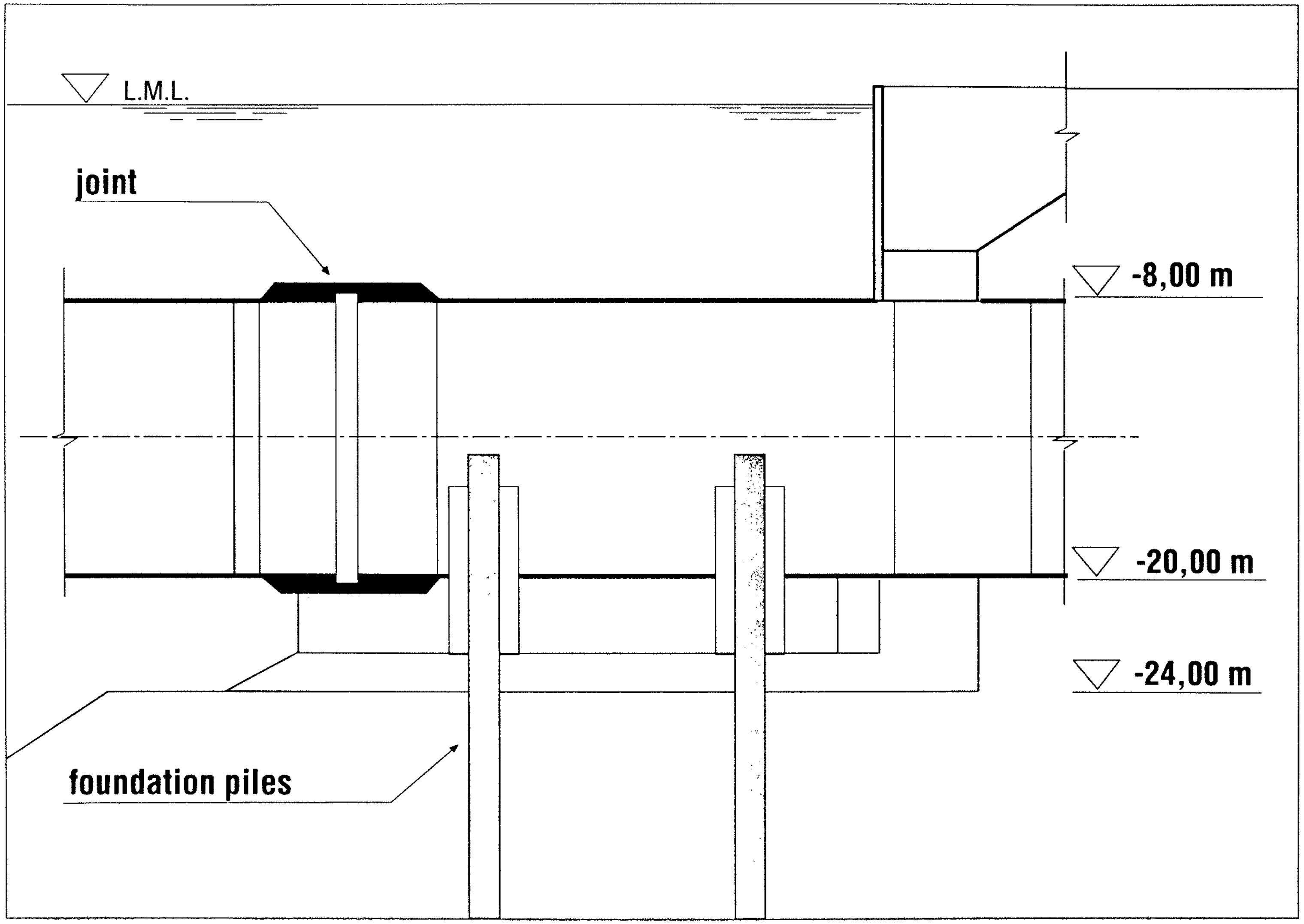


Fig.27 - TUNNEL ESTIMATE WEIGHTS

TUNNEL	
Internal shell	6797 t
External shell	2872 t
Ring stiffness	2736 t
Connection modules and strand connections	3100 t
Rails support structure	3000 t
TOTAL structural steel	18505 t
Internal plant	5730 t
Ballast	9828 t
Concrete (38.241 t/m)	38124 t
STRANDS	
Steel	2290 t
Kevlar	17.5 t
FOUNDATIONS	
Steel piles	4300 t
Piles sleeve grouting	8300 t

Fig.28 - MODULES PREFABBRICATION AREA

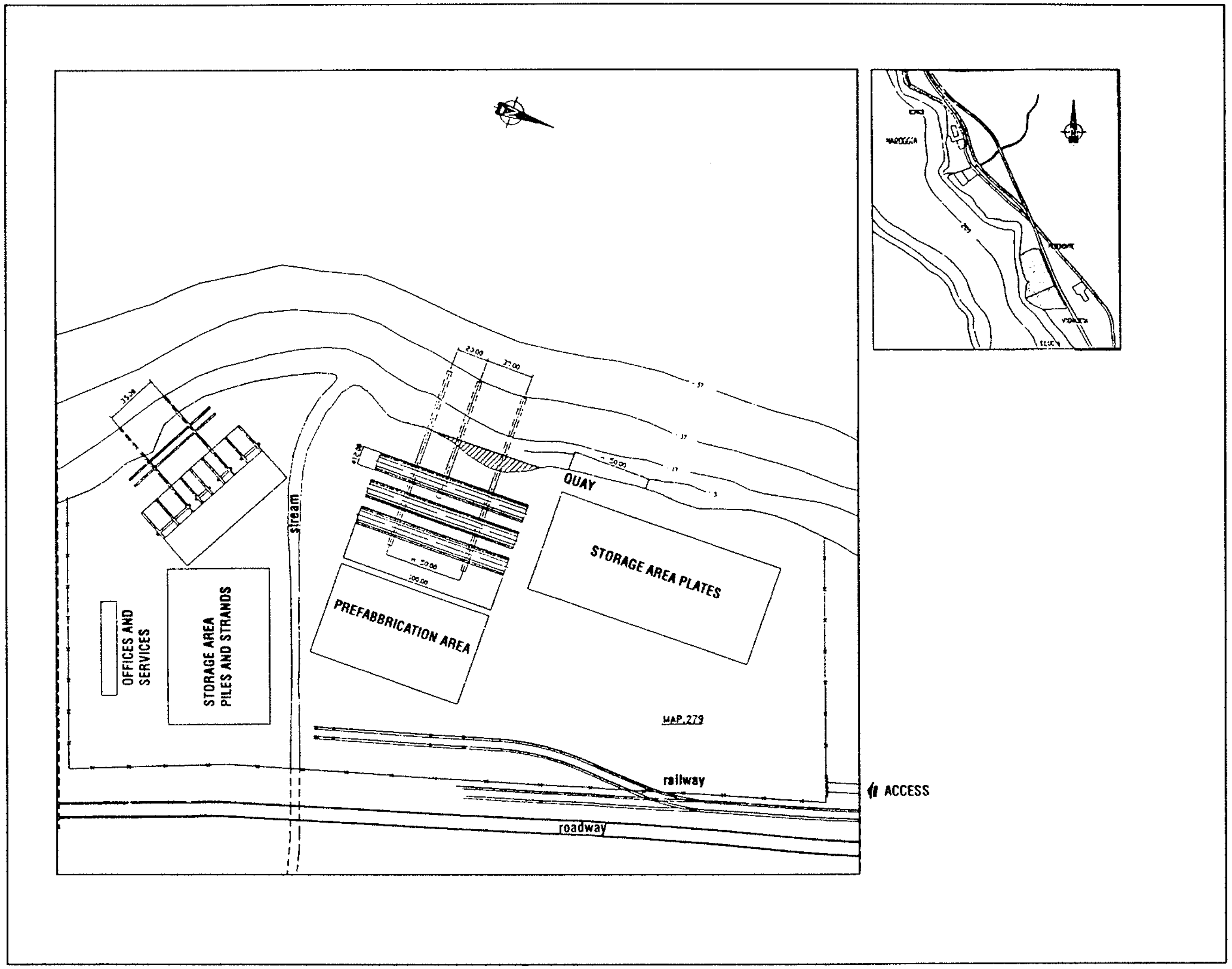


Fig.29 - INSTALLATION PROCEDURE

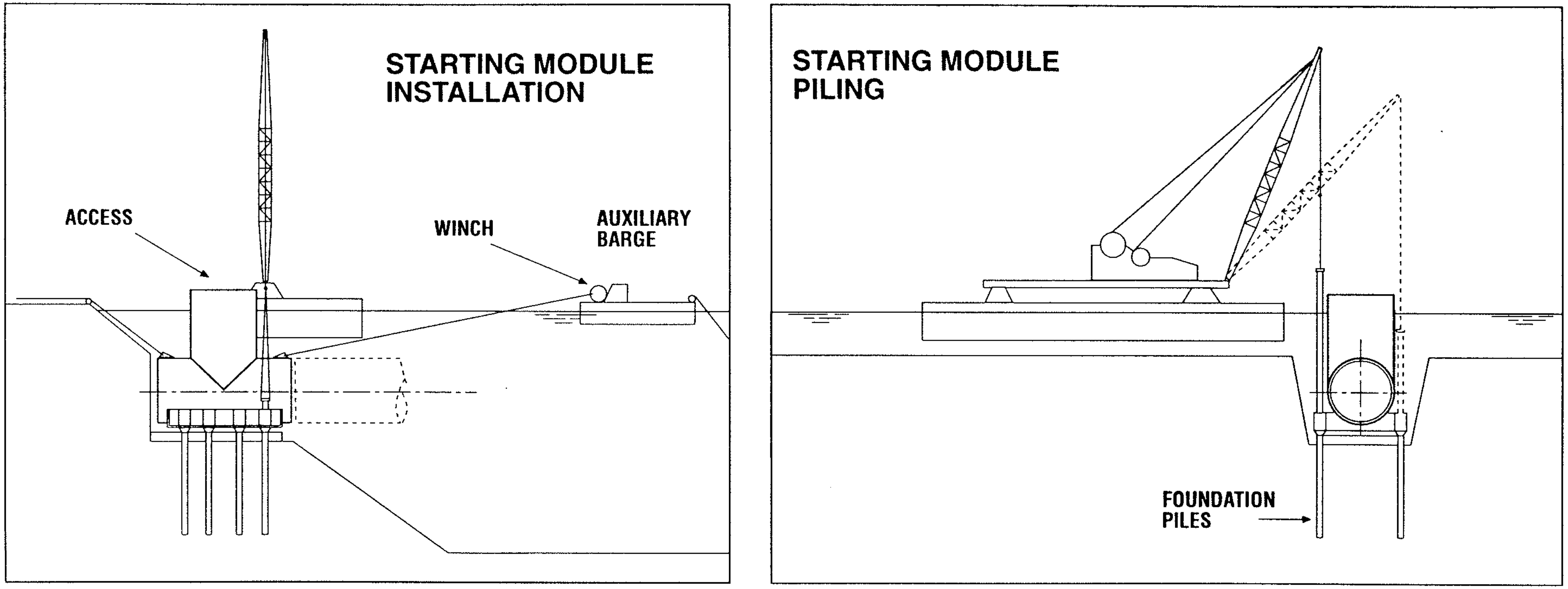


Fig. 30 - INSTALLATION MODULES

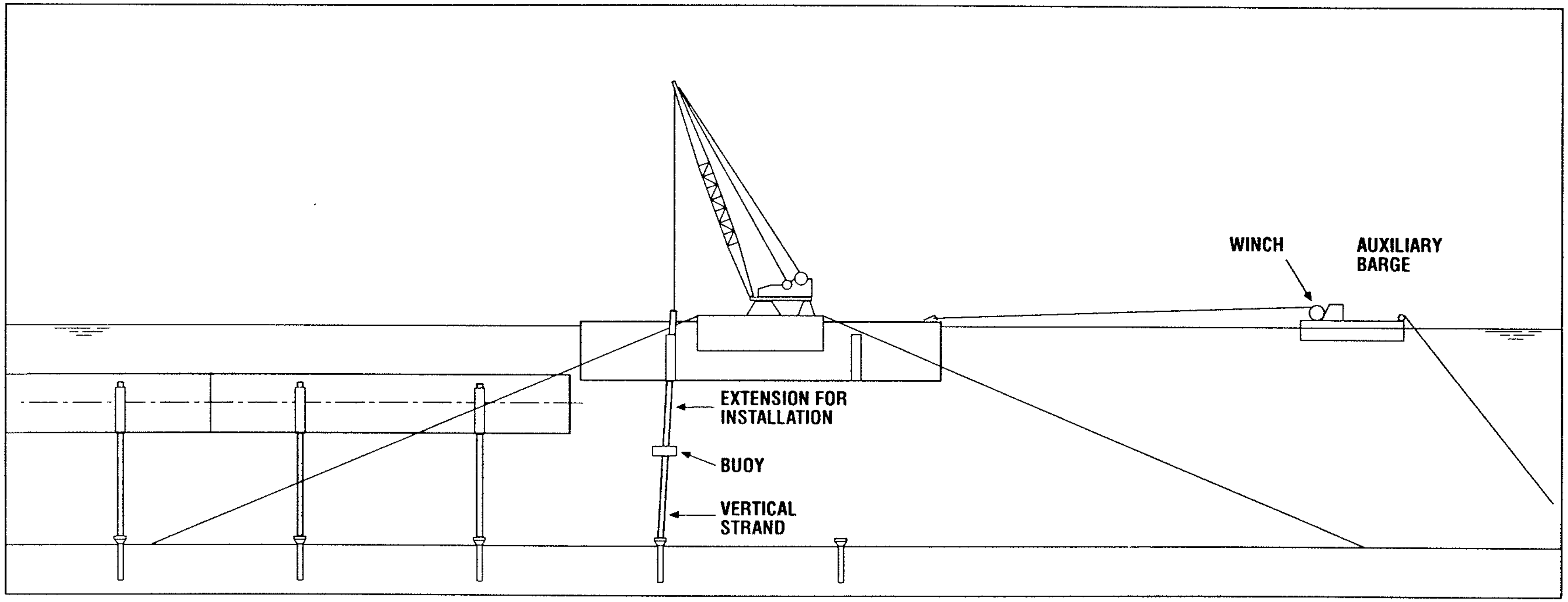


Fig.31 - POTENTIAL ENVIRONMENTAL IMPACT MATRIX

	ATMOSPHERE	LAKE ENVIRONMENT			LAKE BOTTOM	SOIL	NOISE AND VIBRATIONS			LANDSCAPE	ECOSYSTEMS			
	QUALITY	QUALITY	USE	HYDRODYNAMICS	QUALITY	USE	IN LAND NOISE	IN WATER NOISE	VIBRATIONS	PROTECTION	COSTS	PLANCTON	BENTHOS	FISHING
Construction phase														
Prefabbrication yard preparation	■					■	■		■	■	■			
Prefabbrication yard	■						■		■					
Traffic to the yard	■						■		■		■			
Trenching for interface modules	■	■	■		■		■	■	■	■		■	■	■
Removal of trenched materials	■					■	■							
Foundation installations	■	■	■				■	■		■		■	■	■
Modules and strands installation	■		■				■	■		■		■		■
Grouting	■	■	■		■		■	■	■	■				
Restauring of prefabbrication yard	■	■					■	■						
Operating phase														
Presence of the tunnel		■		■									■	
Railway traffic								■						■

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

4. Messina Crossing

4.4 Proposed "Design guidelines for submerged floating tunnels"

R. Robino
Rina Industry

Certification of Submerged Floating Tunnels

↓

Reference tool needed for such a 'new' design

↓

Development of the <u>Guidelines Proposed for design of Submerged Floating Tunnels</u>

Design Guidelines for the Certification of Submerged Floating Tunnels

- **Scope and Application**

to provide the requirements, reccomendations and criteria to be used as reference during the certification of structures and equipments relevant to underwater floating tunnels

- **Table of Contents**

Design Guidelines for the Certification of Submerged Floating Tunnels

STRUCTURAL SAFETY CRITERIA

Guidelines based on a

SEMI-PROBABILISTIC FORMAT

The safety target is implicit in the partial safety factors, combination factors and return periods to be used in the different design checks (limit states)

RELIABILITY BASED CALIBRATION PROCEDURE

1. selection of the safety target
2. application to the different limit states
3. calibration of the partial safety factors

Design Guidelines for the Certification of Submerged Floating Tunnels

Section on STRUCTURES

- assessment for the design life
- classification of primary, secondary and special structural components
- definition of design scenario (persistent, transient and accidental situations)
- modelling of actions (operational, environmental and accidental loads)
- partial safety factors and combination factors for the different limit states:
  - serviceability
  - ultimate
  - progressive
  - fatigue
- verification requirements for the different limit states
- conditions of tethers to prevent progressive collapse
- classification of structural components for the fatigue limit state (as a function of inspections)

Design Guidelines for the Certification of Submerged Floating Tunnels

Section on FOUNDATIONS

- general criteria relevant to the safety and load carrying capacity of the foundations
- design recommendations relevant to the different limit states:
  - serviceability
  - ultimate
  - progressive

  during persistent, transient and accidental situations
- criteria relevant to the soil behaviour and the design of foundations for the several actions transmitted by the structural components

Design Guidelines for the Certification of Submerged Floating Tunnels

Section on <u>EQUIPMENT</u>

- general criteria for the protection of life and property transported (for both road and railway traffic) and of the environment against accidental events due to human factors, such as fire, explosions, leakages, collision, vehicle impacts

- recommendations for the design of equipment used during normal operations, including criteria for traffic control and control of hazardous materials

- recommendations for the design of equipment during emergency conditions, including transmission of alarms, rescue and evacuation operations and protection of critical installations

- recommendations for the application of quantitative risk assessment and simulation techniques, including targets for components and system failure rates and for consequence analyses due to traffic operations

Design Guidelines for the Certification of Submerged Floating Tunnels

Section on <u>CONSTRUCTION AND INSTALLATION</u>

- recommendations relevant to the different temporary phases, in order to show that operations will fulfill the intended design for purpose and that there is a reasonable level of strength of the structure and of safety for the personnel on-board and afloat
- criteria for the definition of the design environmental conditions and resultant loads on the structure and the mooring and seafastening system, during transportation, installation and construction
- recommendations on navigation, towing routes and strategy for adverse weather conditions, with reference to existing rules about the safety of navigation

Design Guidelines for the Certification of Submerged Floating Tunnels

Section on INSPECTION, REPAIR AND MAINTENANCE

- recommendations about the inspections to be carried out during the fabrication, installation and offshore construction
- considerations relevant to the planning of periodical surveys and in-service monitoring of structures and equipment, in order to have a continuous detection of the structures and equipment during the design life
- recommendations on planning maintenance and repair, to ensure adequate safety and fitness for purpose of the structure and equipment during the design life
- considerations about the application of reliability based methods for the optimization of inspection and maintenance schemes

GENERAL CONCLUSIONS FOR THE CERTIFICATION OF A SFT

International Conference on Submerged Floating Tunnels

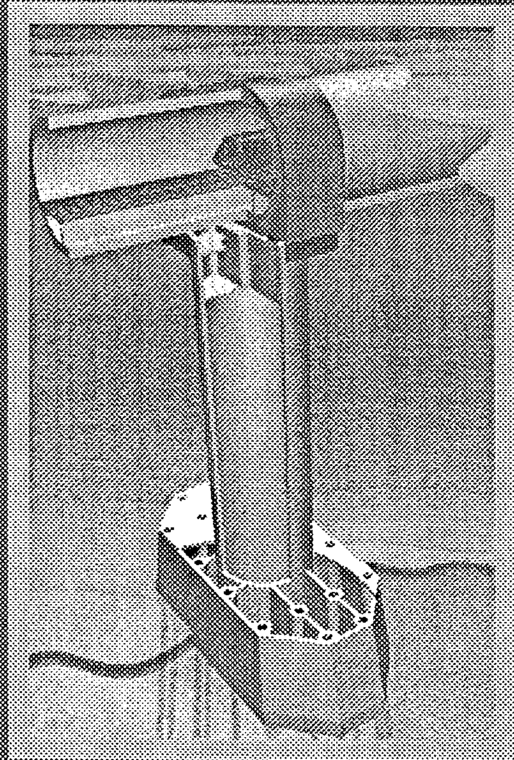
Sandnes, Norway, 29 - 30 May, 1996

5. Lugano Crossing

T. O. Olsen
Dr. techn. Olav Olsen AS

INTERNATIONAL CONFERENCE ON SUBMERGED FLOATING TUNNELS

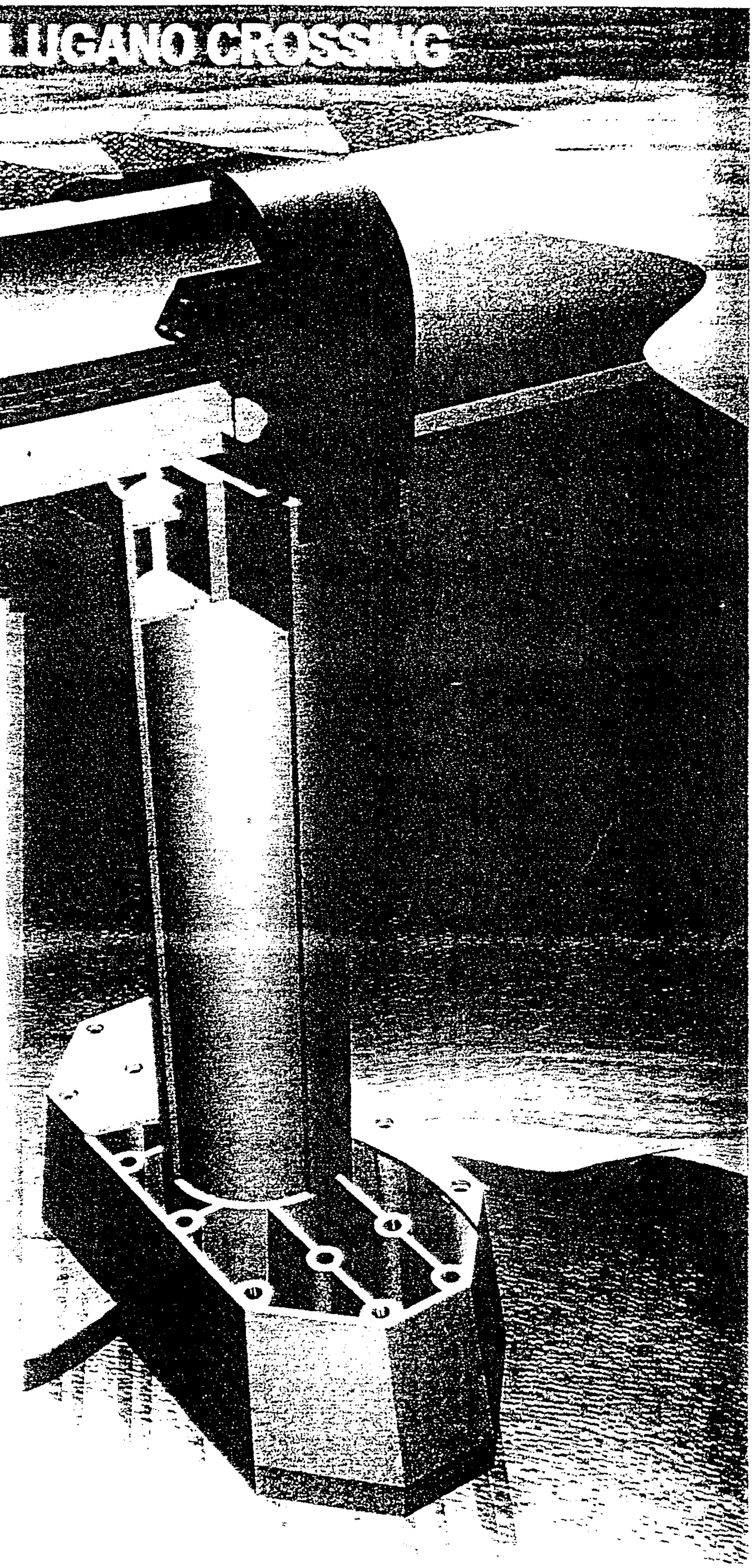
Sandnes, Norway
May 1996



LUGANO CROSSING

Tor Ole Olsen

THE LAKE LUGANO CROSSING



AlpTransit

Studio Grignoli & Muttoni Lugano has, together with Dr. techn. Olav Olsen a.s, developed a new and unique concept for a railway crossing of the Lake Lugano in Switzerland.

The crossing will be a part of the planned ALPTRANSIT railway line that extends from Basel towards the Italian border.

Conventional bridge crossings would cause a considerable intrusion on the scenery in and around the lake.
The apparent solution was therefore a submerged crossing with construction methods that minimises the environmental impact.

Conventional technology characterises all aspects of the concept. Design, construction and installation methods are, as well as foundation principles based on traditional and well proven technology. Emphasis is put on safety robustness and constructability in addition to cost. The concept can, due to it's robustness and flexibility, easily be adopted and modified for use at other sites.

CONCEPT DESCRIPTION

The submerged tunnel is designed to accommodate a double track railway line carrying high speed trains at 230 km/h. Maximum traffic load on the railway is will be 320 trains/day.

Both the sub and the superstructure are made of reinforced prestressed high strength concrete (ND60) with a total volume of 63 000 m^3. Nearly 7 000 tons of ordinary reinforcement and 2 500 tons of prestressing tendons are used in the various structural parts.

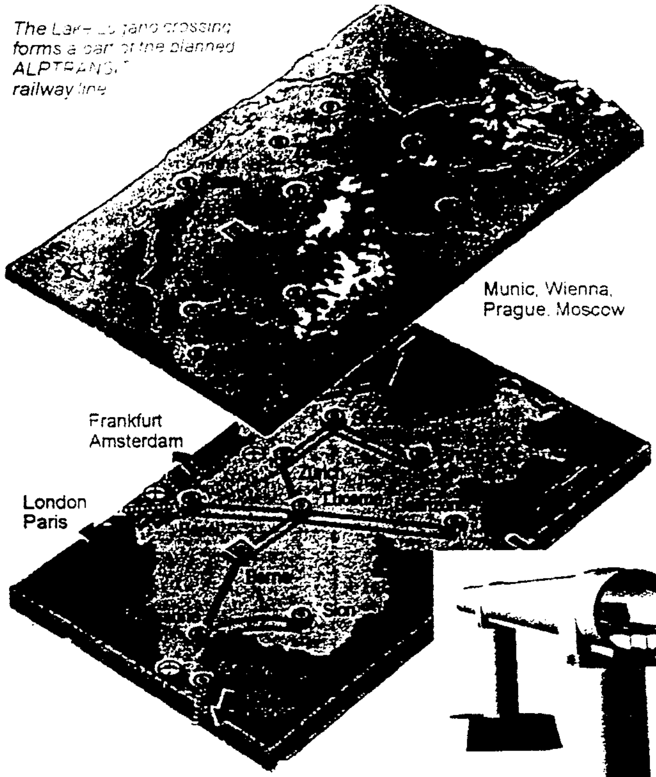
Superstructure

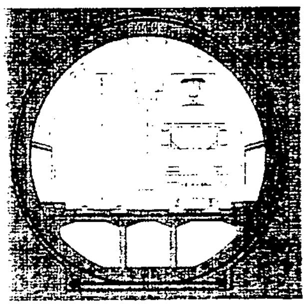
The submerged buoyant tunnel crosses the Lake Lugano with an overall length of 930 m between the landfalls on each side. The alignment is straight both horizontally and vertically. Both ends of the tunnel is fixed at the landfall structures. Fixed ends require a minimum of maintenance and inspection, and a high degree of durability is obtained. Between the landfalls, the tunnel is supported by four intermediate piers, subdividing the tunnel into 5 spans of equal length. The tunnel is localised 6 m below the mean water level, in order not to interfere with the marine traffic in the lake. Cross section of the tunnel is mainly governed by the section required to accommodate the double track railway line together with necessary outfitting. The quasi-circular tunnel has an internal diameter of 10.6 m and a wall thickness of 0.85 m which is increased over the supports.

Substructure

The Substructure consists of a 10.5 m high octagonal caisson and a circular column with an outer radius of 5.7 m. Sixteen open ended steel piles driven 60 m into the seabed support the caisson. A 2.5 m deep skirt foundation secures stability in the temporary phases prior to and during piling.

The Lake Lugano crossing forms a part of the planned ALPTRANSIT railway line

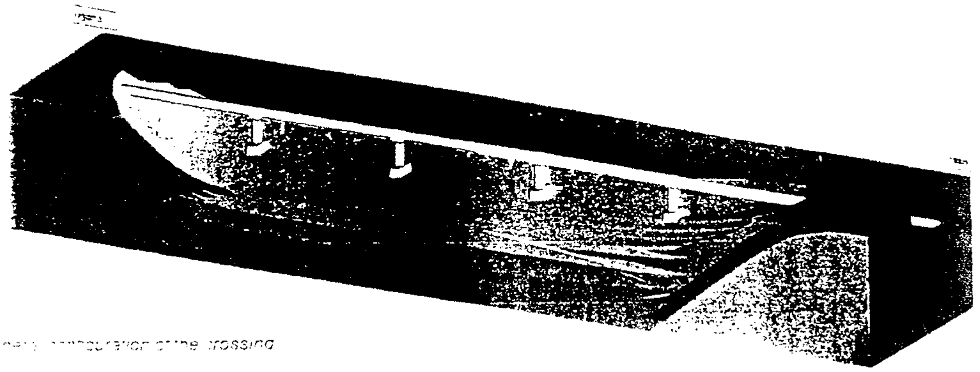




Cross section of the tunnel

Support system

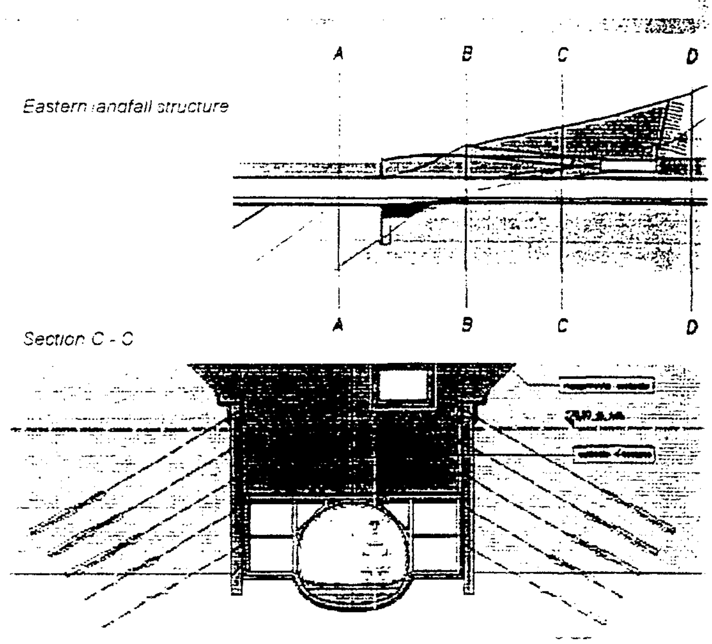
Two variants of tunnel / column intersections are developed. This is done to improve the concept's flexibility to accomodate changes of the geotechnical parameters on which the settlement calculations are based. An alternative with adjustable bearings that creates a pinned tunnel support is proposed if the settlements have such magnitude that they have to be counteracted by the bearings. The monolithic alternative that creates a fixed tunnel support is simpler and more cost efficient with respect to both construction and maintenance.



[illegible] configuration of the crossing

Landfalls

The landfall structures at both ends create a fixed end of the tunnel. Axial compressive forces in the tunnel due to temperature rise are transferred to the rock by cogging of the landfall structures.
Longitudinal walls anchored by rock anchors secures flexural resistance during operation and act as watertight plates for the dry dock during construction.
The landfalls are founded on a line of drilled concrete piles transverse to the tunnel at one end, and directly on the rock at the other end.
The tunnel is equipped with external access at each of the two landfalls.



CONSTRUCTION AND INSTALLATION

Both the tunnel and the foundations will be prefabricated in a dry dock integrated in the eastern landfall excavation.
Two separate dock gates in the dry dock allow a separate construction of the components in parallel.
The caisson with the skirts will be completed in dry dock prior to tow out.
The shaft will be slipformed while afloat (hooked up) by gradually increasing the draft.
Equipped with a steel cofferdam at top of the shaft, the piers will be water-ballasted, submerged to the seabed and piled by a number of steel piles running through sleeves in the caisson.
The tunnel is constructed in segments of 186 m, corresponding to the span length, by utilising the adjacent rock tunnel as additional dry dock length.
Prior to tow out of dock, the tunnel segments will in each end be equipped with a temporary watertight bulkhead and a steel shaft.
With a freeboard of about 2.5 m the tunnel segments will be towed to the site.
Using the shaft as access, the segment will be water-ballasted and submerged to the piers.
Installed on the piers, the tunnel segments will be monolithically connected in dry environment and finally water de-ballasted.
The dry environment will be secured by means of a temporary steel cofferdam.

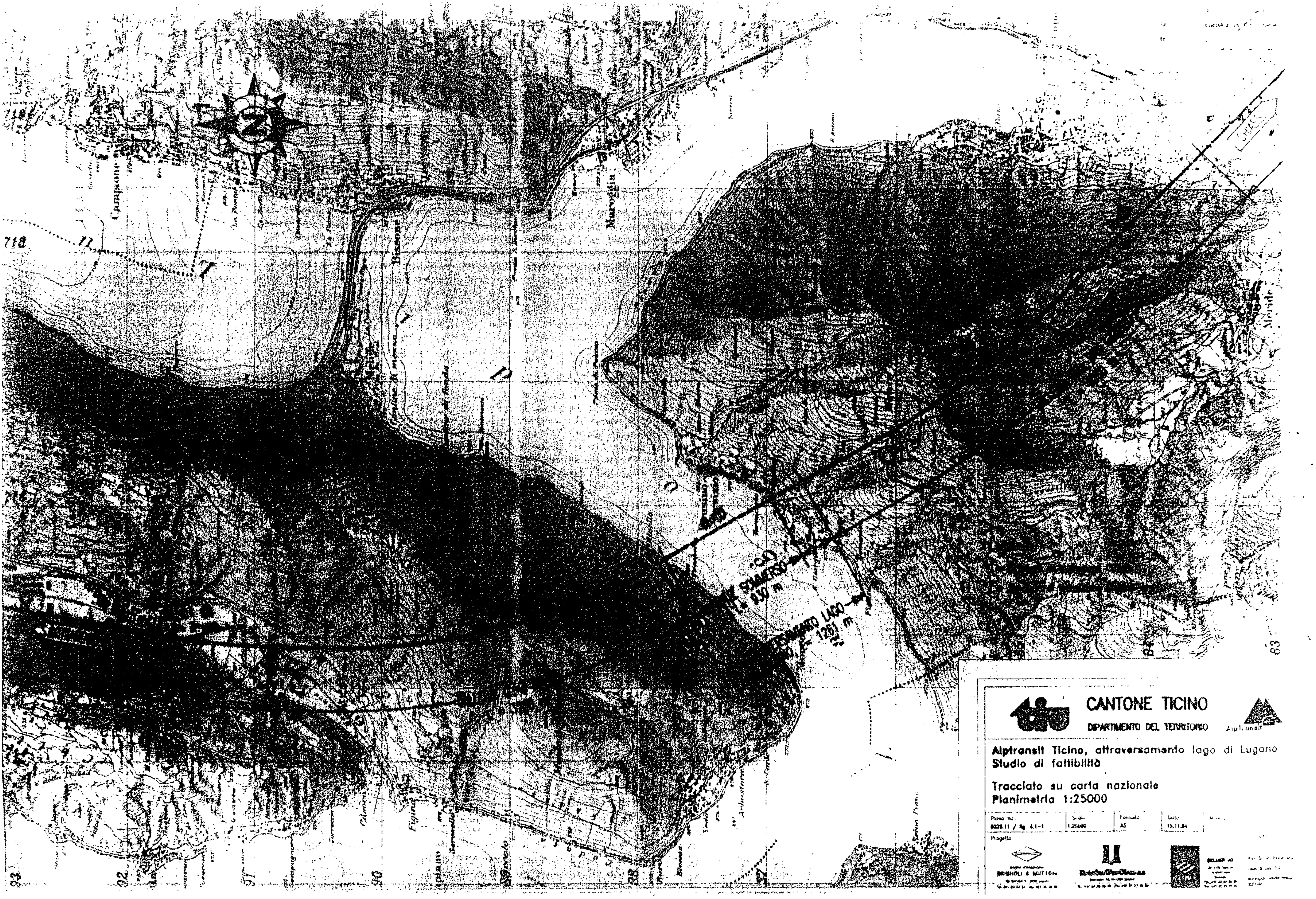
PROJECT ORGANIZATION:

Client	Dipartimento del Territorio, Cantone Ticino, Swizzera
Concept development and design	Studio Grignoli & Muttoni Lugano Dr. techn. Olav Olsen a.s, Oslo Selmer A.S., Oslo Prof. Dr. Bruno Thürlimann, Zürich
Soil Mechanics	Studio Leoni & Gysi S.A., Lugano Norges Geotekniske Institutt, Oslo

KEY FIGURES:

Length of crossing	1 260 m
Total length of tunnel	930 m
Number of spans	5
Horizontal alignment	R = oo
Vertical alignment	R = oo
Grade	0.0 ‰
Maximum water depth	74 m (M.W.L.)
Clearance ballast	6.0 m
Total concrete volume (ND60)	63 000 m^3
Total reinforcement	6 900 tons
Total prestressing	2 500 tons
Number and types of piles	64 Ø 1 100 x 25 mm open ended steel piles
Concrete ballast	1 360 m^3
Span length	186 m
Outer diameter of tunnel	12.3 m
Internal radius of tunnel	5.3 m
Heigth of caisson	10.5 m
Depth of skirts	2.5 m
Outer diameter of columns	11.4 m
Construction period	46 months

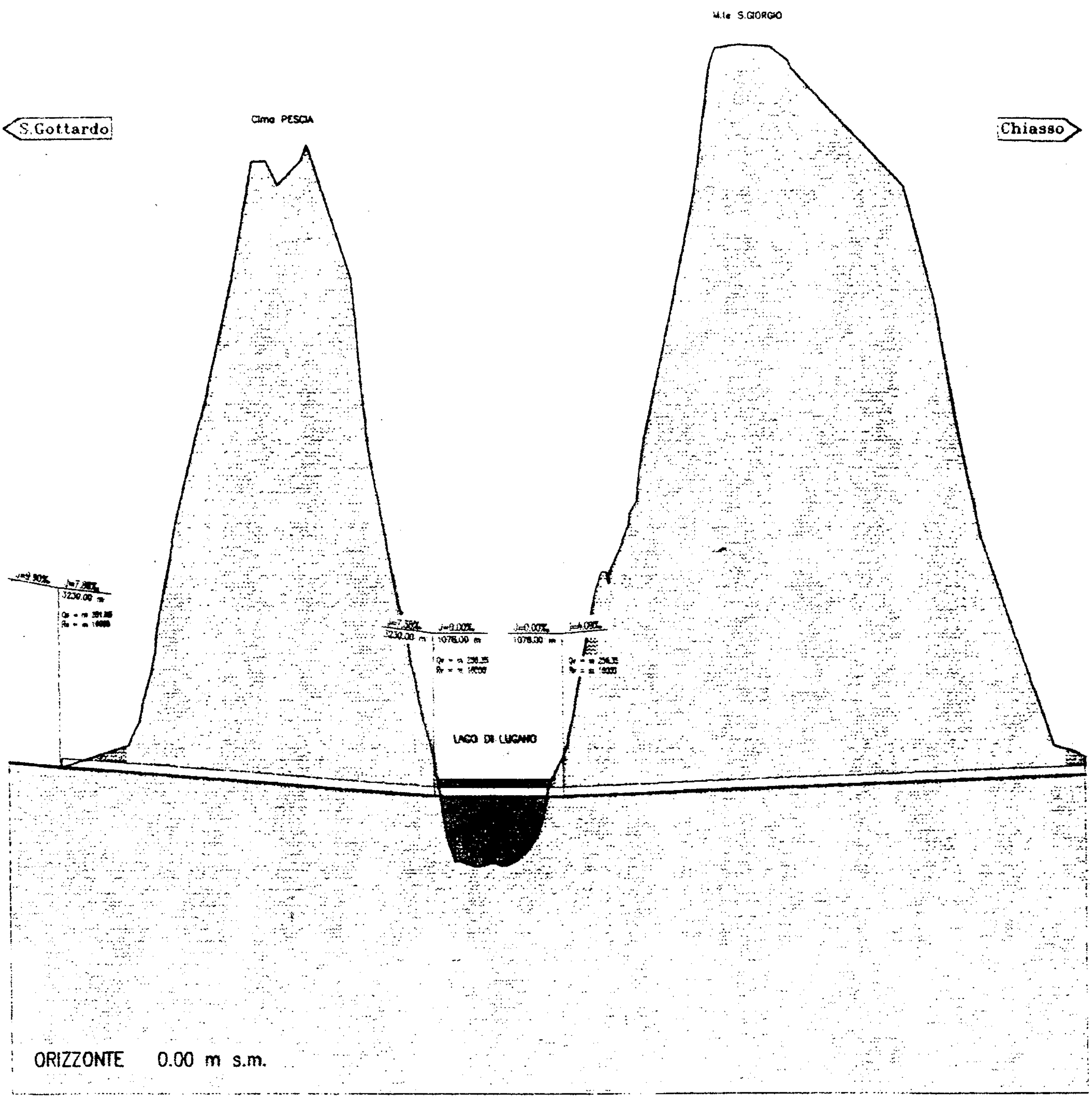
Dr. techn. Olav Olsen a.s
STRUCTURAL ENGINEERS - ONSHORE AND OFFSHORE
Dicks vei 10 - P.O.Box 139 - 1324 LYSAKER - NORWAY
☎ + 47 67 53 22 75 - Fax: + 47 67 53 49 89



CANTONE TICINO
DIPARTIMENTO DEL TERRITORIO
Alptransit
Alptransit Ticino, attraversamento lago di Lugano
Studio di fattibilità
Tracciato su carta nazionale
Planimetria 1:25000
Progetto
BRUGNOLI & MUTTONI
SOMMERSO L= 930 m
SPROFONDAMENTO LAGO L= 1261 m
Campione
Bissone
Maroggia
Melide
Figino
Arbostora
Meride
93
92
91
90
88
87
84
83
718

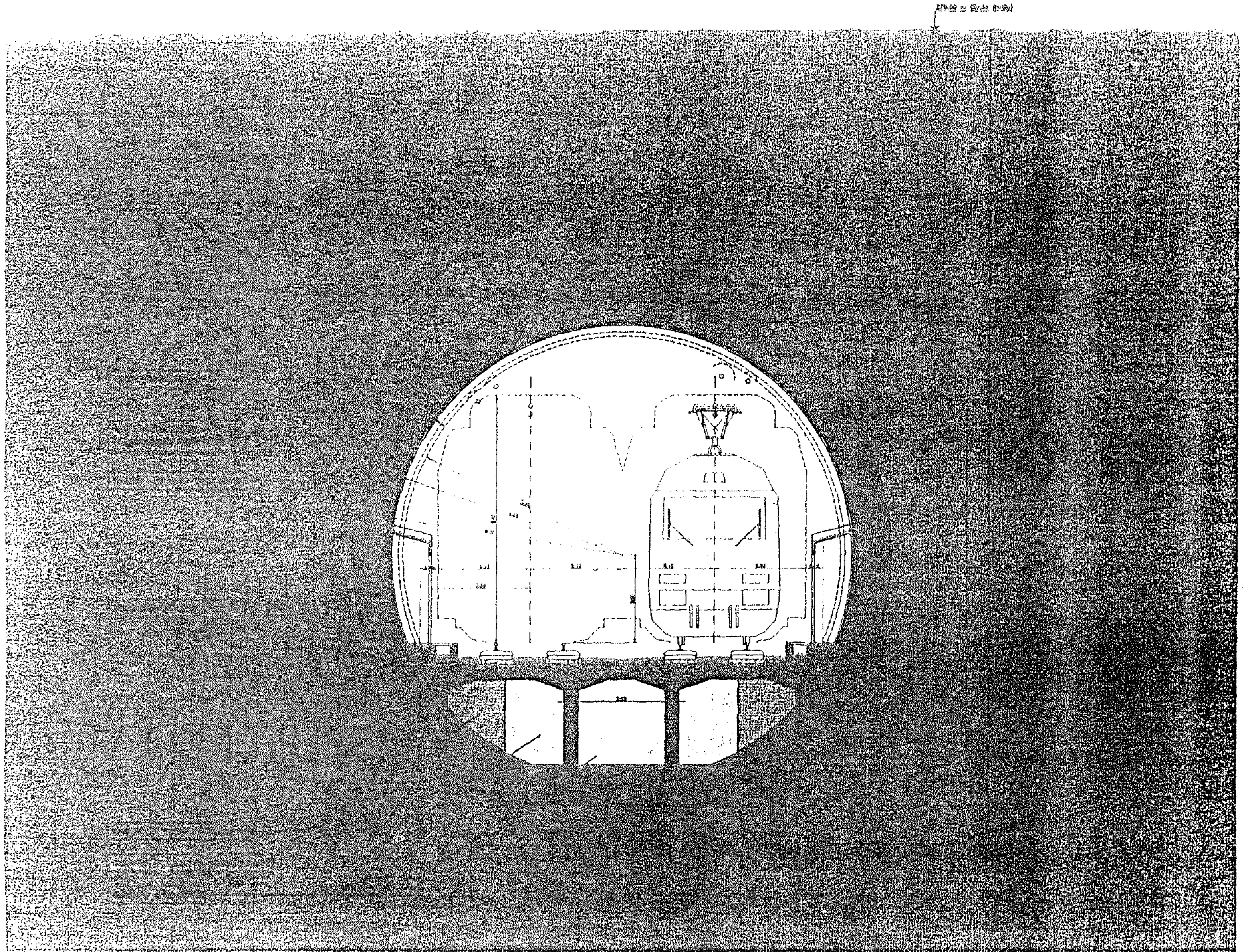
AlpTransit

Alp Transit Ticino



TRAVERSAMENTO

SEZIONE TRASVERSALE 1:100



CANTONE TICINO

DIPARTIMENTO DEL TERRITORIO

AlpTransit

Alptransit Ticino, attraversamento lago di Lugano
Studio di fattibilità

Sezione trasversale galleria

Piano no.:	Scala	Formato	Data
	1:100	A3	11.11.96

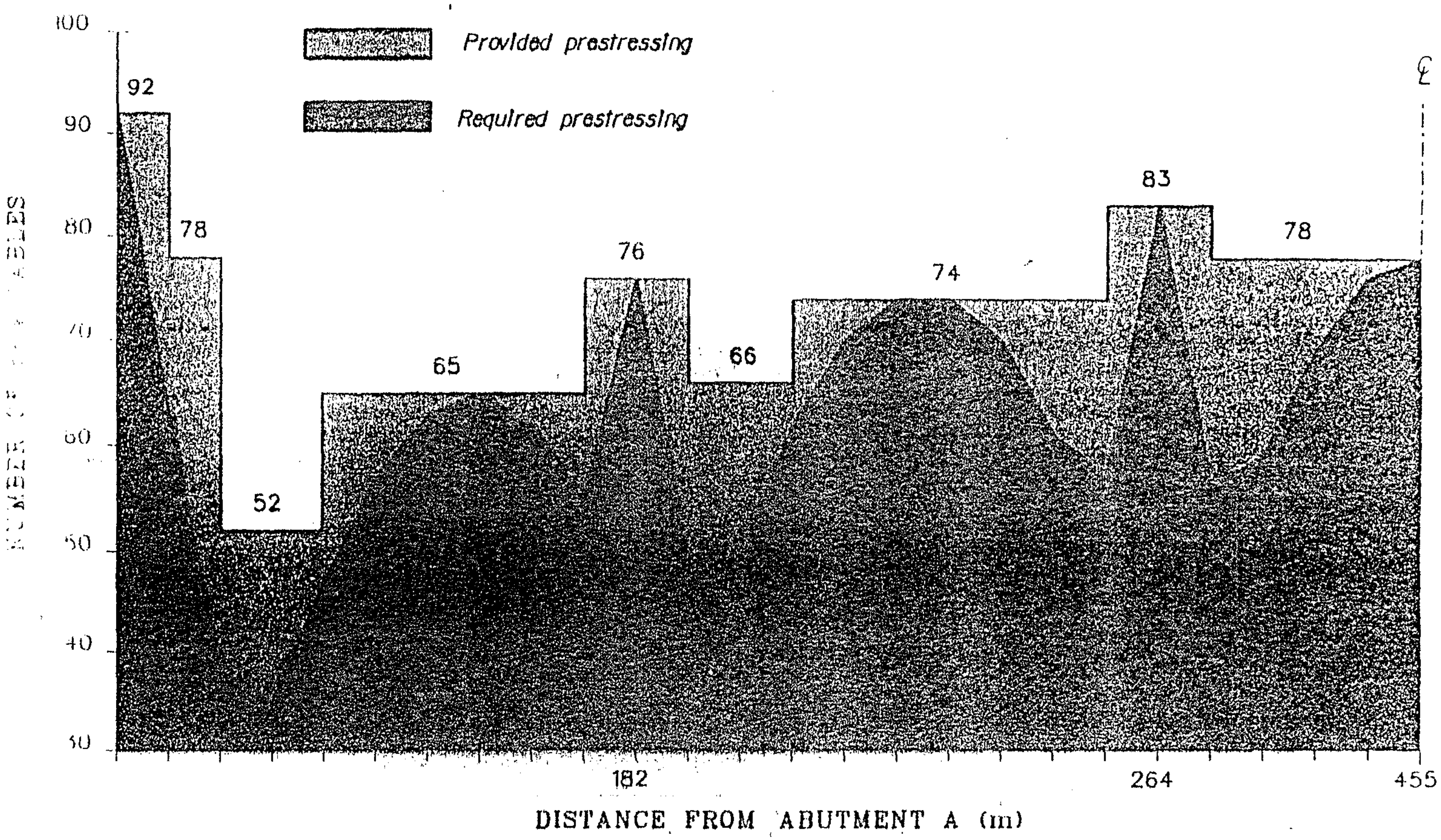
Progetto:

BRIGNOLI & MUTTONI

AlpTransit

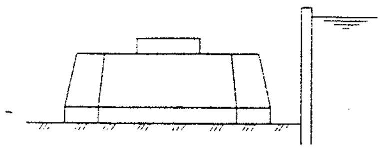
Alp Transit Ticino

Intensity of longitudinal prestressing



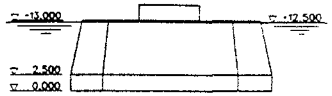
STUDIO D'INGEGNERIA
GRIGNOLI & MUTTONI

Dr.techn. Olav Olsen a.s



1. CONSTRUCTION IN DRY DOCK

Concrete volume	2050 m^3

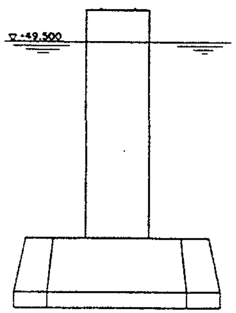


2. TOW OUT OF DRY DOCK

Draft	12.5 m
Displacement	7000 m^3
GM-value	1.5 m
Freeboard	3.5 m

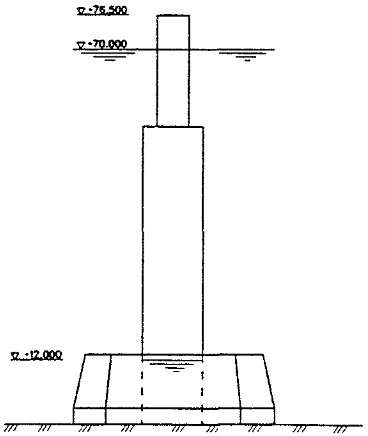
3. SUBMERGENCE OF CAISSON

Concrete volume	2050 m^3
Draft	13.0 m
Displacement	7200 m^3
GM-value	0.5 m
Freeboard	3.0 m



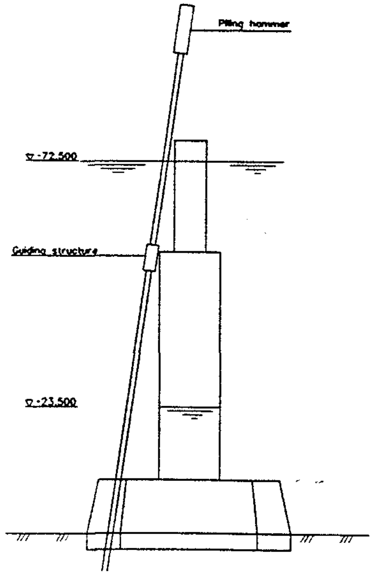
4. CONSTRUCTION OF COLUMN COMPLETED. CAISSON WATER FILLED

Concrete volume	2760 m^3
Draft	49.5 m
Displacement	11200 m^3
GM-value	7.7 m
Freeboard	6.0 m



5. TOUCH DOWN

GM-value	9.9 m
Water level in column	9.5 m



6. PILING

Submerged weight	9 MN
Water level in column	23.5 m

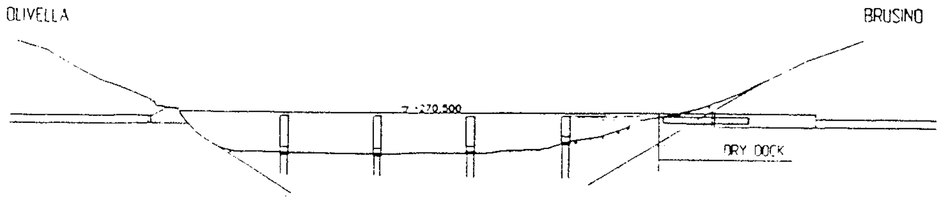
01 05.10.94 PRELIMINARY LLM

CANTONE TICINO
DIPARTIMENTO DEL TERRITORIO

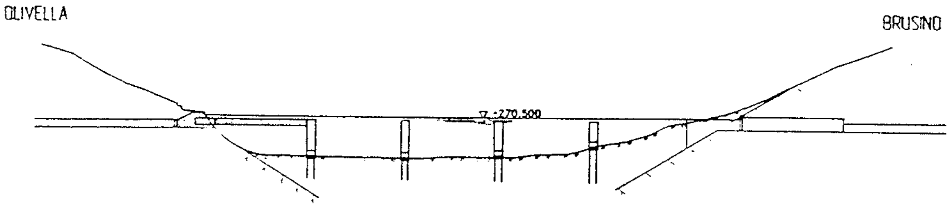
Consultants:
Dr techn Olav Olsen a s
6025

Scale 1:500
A3

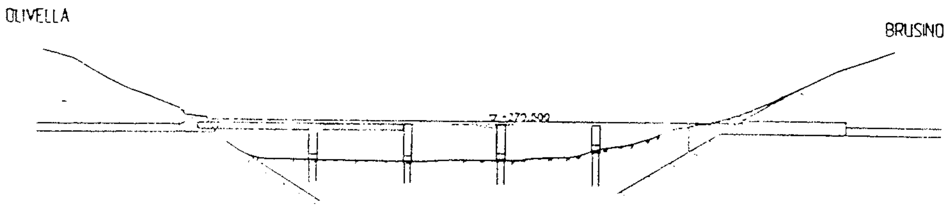
Drawing title:
CONSTRUCTION PHASES
OF PIERS



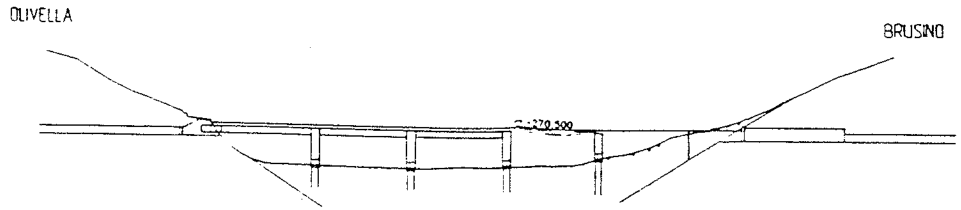
1. CONSTRUCTION SEQUENCE
CONSTRUCTION OF LANDFALL - OLIVELLA
+ GENERAL CONSTRUCTION OF SPAN ELEMENTS



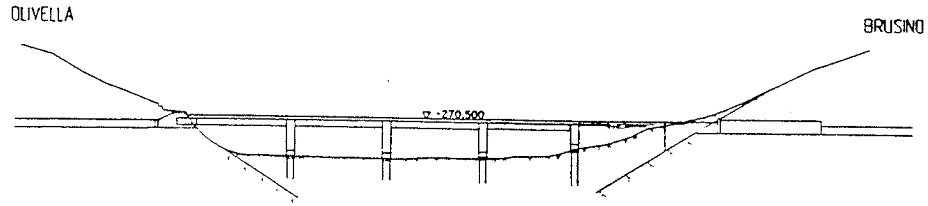
2. CONSTRUCTION SEQUENCE
PLACING OF FIRST SPAN



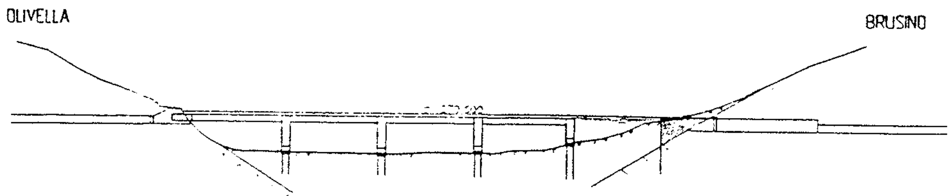
3. CONSTRUCTION SEQUENCE
PLACING OF SECOND SPAN



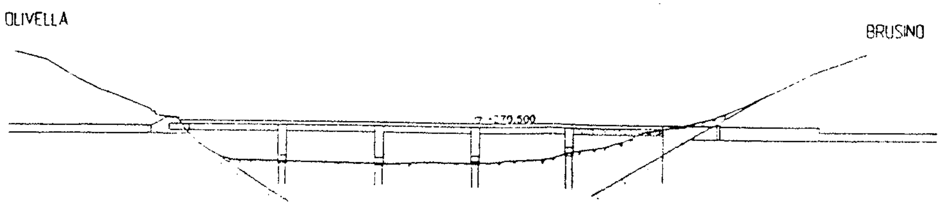
4. CONSTRUCTION SEQUENCE
PLACING OF THIRD SPAN



5. CONSTRUCTION SEQUENCE
PLACING OF FOURTH SPAN



6. CONSTRUCTION SEQUENCE
CONSTRUCTION OF LANDFALL - BRUSINO-ARSIZIO
5TH MODULE HOOKED UP



7. CONSTRUCTION SEQUENCE
PLACING OF FIFTH SPAN

01	05.10.94	ISSUED FOR REPORT	HSL

CANTONE TICINO

Dr techn Olav Olsen a s

6025

CONSTRUCTION PHASES
OF TUNNEL - SECTIONS

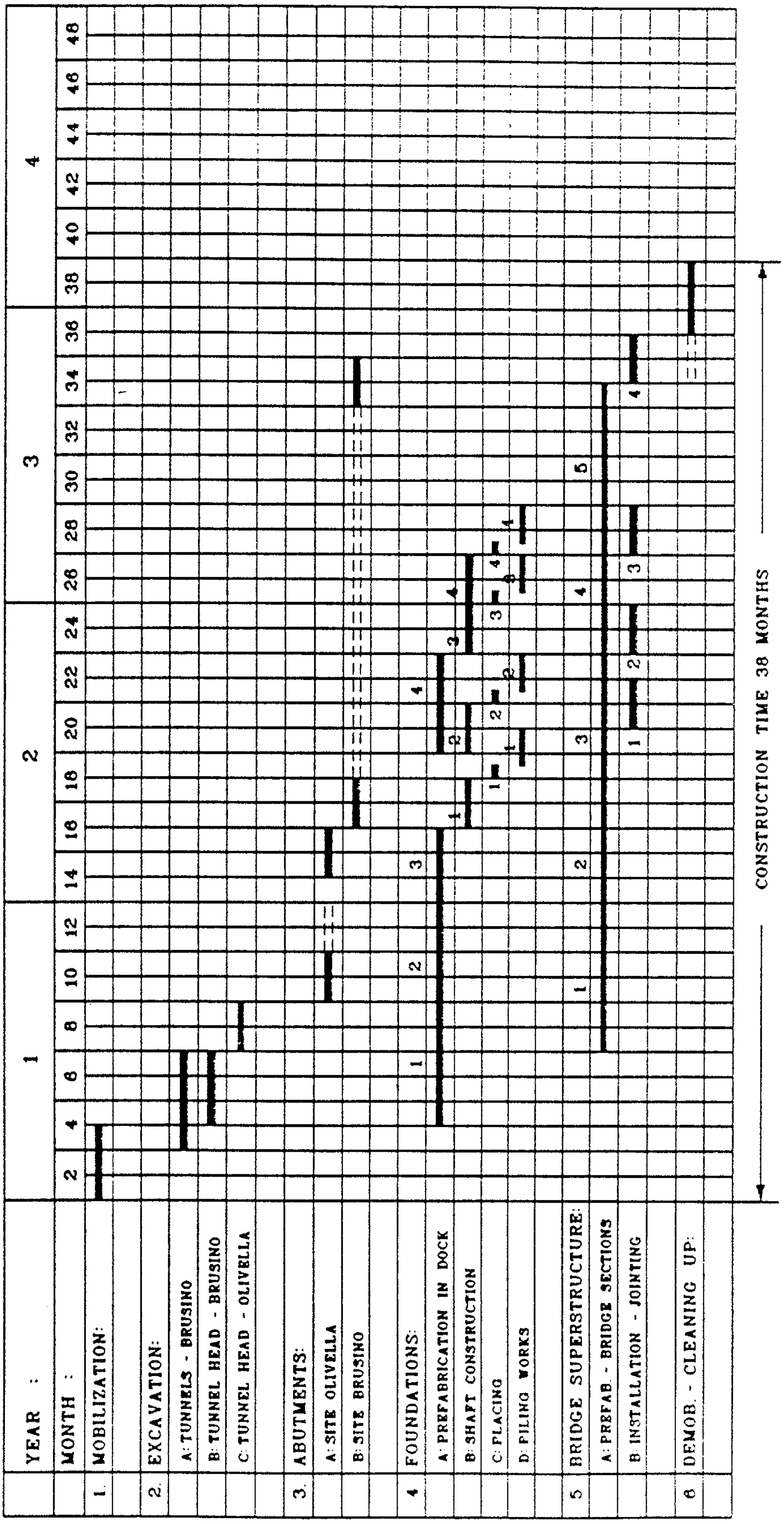
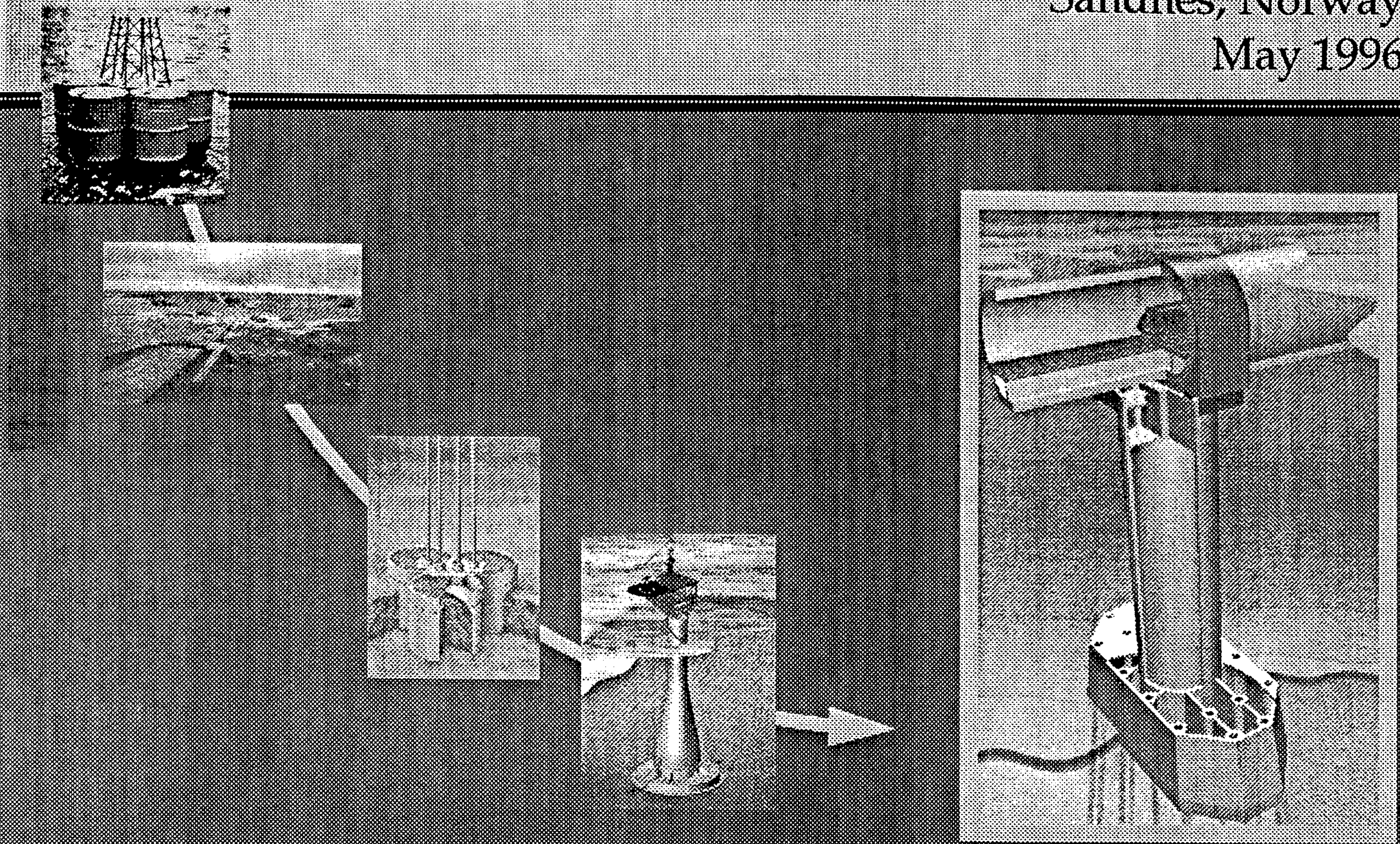


Figure 9.2-1: Time and work schedule.

INTERNATIONAL CONFERENCE ON SUBMERGED FLOATING TUNNELS

Sandnes, Norway
May 1996



LUGANO CROSSING

Tor Ole Olsen

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

6. Høgsfjord Crossing

1. H. Østlid
2. S. Johansen
3. A. Loen
4. T. Einstabland

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

6. Høgsfjord Crossing

6.1 Høgsfjord Crossing

H. Østlid
for Aker Norwegian Contractors

Aker Norwegian Contractors

This company was unforunately unable to present their concept. The reader is therefore referred to K.A. Nyhus' presentation of Tension Anchoring (presentation no. 12.2) where the concept may be seen.

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

6. Høgsfjord Crossing

6.2 Høgsfjord Crossing

S. Johansen
The Eeg-Henriksen Group

EEG-HENRIKSEN ANLEGG AS

INTERNATIONAL CONFERENCE ON
SUBMERGED FLOATING TUNNELS

IN SANDNES, NORWAY
MAY 29 - 30, 1996

HØGSFJORD CROSSING

THE EEG - HENRIKSEN GROUP CONCEPT

EEG-HENRIKSEN ANLEGG AS
A/S BETONG
HOLLANDISCHE BETON- EN WATERBOUW BV

Presented by: Siv.ing. Steinar Johansen
Eeg-Henriksen Anlegg AS

EEG-HENRIKSEN ANLEGG AS

INTRODUCTION

The Eeg-Henriksen group presented two alternative tube crossings in april1988. Alt. A, an octagonal tube supported on three intermediate piers to the seabed as well as the two abutments. A-02
A constant vertical curvature has been chosen. The horizontal alignment is straight. The Lauvik abutment provides the fixed support, whilst the Tube is free to slide on the Oanes side. The foundations are of gravity caisson type with penetrating skirts.

Alt. B, a circular tube supported on three floating pontoons together with two 01
abutments. The vertical curvature equals the horizontal curvature. The Tube is fixed in both ends. Both alternatives are based on continuos launching from the Lauvik side

In June 1994, Statens Vegvesen Rogaland (the owner) asked the prequalified contractors to review the cost estimate for the tube crossings presented in 1988. Eeg-Henriksen decided to concentrate the work on alt.B. The construction phase was further developed which led to some changes. These changes are incorporated in the following presentation of our alt. B concept.

THE STRUCTURE

General
The tube is 1500 m long with a constant curvature with a radius of 9500 m in both vertical- and horizontal direction. When the tube is completed, it will be fixed at both abutments with no allowance for sliding in the longitudinal direction.

In the fjord, the tube will be connected to three pontoons by a stiff steel structure. The pontoons acts like foundations regarding vertical forces on the tube. The pontoon size is determined such as when vertical forces are acting on the tube, the pontoons are heaving with small movements so that the forces on the tube are kept low.

In the construction phase, the pontoon is supported by tensioned cables to the shore. Analyses done in 1988 shows that the tube is able to withstand cantilevered spanlenghts of 320 -365m. When the tube is completed and fixed to the abutments on shore, the cables are removed. The tube will in the completed phase withstand all horizontal forces on the pontoons and tube by its arched curvature and bendingstiffnes.

The Lauvik abutment will be completely founded on solid rock. On the Oanes side, the abutment have to be founded on sand/gravel.

The tube
The inner diameter of the tube is 4.75 m all over. The wall thickness is 0.9 m, 02
chosen to give a balanced relationship between the need for concrete section and to achieve necessary buoyancy in the different phases.

The tube is made with two large haunches on each side (at 1500 and 2100 hrs.). These are made for supporting the tube during concreting and launching in the Lauvik side yard and below the pontoon-steeltowers.

The owner wants, during all permanent phases, the tube to have positive buoyancy. Estimated value is maximum 50 kN/m net upward force. This means that in all permanent phases, a varying compression force will act on the steeltowers connected to the pontoons. And again the pontoons will be heaved relative to its own natural floating position in accordance with that same variation.

In the construction phase it is necessary to achieve a tubeweight close to the buoyant force. Due to the demand for positive buoyancy in the permanent phase, it is necessary to place a severe amount of ballast weight temporarily in the construction phase. The ballast (aggregates with no fines) will be removed when the tube is fixed to the abutments.During the removal of ballast in the tube, the pontoons are ballasted to keep the tube in its neutral position.

Pontoon and connection to tube

The pontoons are shaped like cylinders with outside diameter d = 31m and a total height of 15m. 03

The structure is divided into several compartments so that the pontoons are able to withstand ship collision with collapse on two compartments.

The connection between the tube and the pontoon is a braced tower made of steelpipes. The tower, which is 11.5 m squared and approximately 27m high is fixed to the bottom side of the pontoon. In the lower part of the tower, two large steelrailings are fixed to the tower structure on both sides to coincide with the haunches on the concrete tube. This is the concrete tube support both during construction and in the permanent phase. Between the inside of the rail and the outside of the concrete haunch, there will be space while launching the tube. An arrangement for guiding and friction reduction between rail and haunch will be installed. In the permanent phase the support i.e. the space between the rail and the haunch will be grouted and hence, a fixed support will be achieved. 04

The tower gurts are 800 x 25 mm steelpipes and the bracings are 400 x 10 steelpipes. The dimensions are chosen to achieve a structure with submerged weight close to zero. The railing on each side of the tower, is shaped like a channel-profile with dimensions approximately 5.0 x 0.4 x 12m. The railing is welded to sliced halfpipe tower gurts. This makes the railing an integrated part of the connectiontower.

The tower is fixed to the pontoons by means of tensioned threadbars in each of the four gurts, anchored in the concrete pontoon structure.

Abutments

On the Lauvik side two portals mounted between the rockwalls on either side of the trench at a distance of 28m are required during construction / launching. The portals serve as a fixed end. 05

Topside of the tube is placed about 4m below the waterline on shore. By placing the tube at such a depth, direct ship collision is no problem.

On the Oanes side the topside of the tube is placed 4m above the sea on shore. This is done to reduce the total tube length and to reduce the trench length. To secure the tube against ship collision, earthdams are placed on each side of the tube from shore to where the height of water above the tube exceeds 4m. 06

The Oanes abutment is a 100m x 25m and 1m thick concrete plate placed on the existing soil. At each end of the plate, portals are placed. The portals are supported by vertical concreted steel pipes. Between the portals a rockfill will be placed to a height of 2m above top tube. The rockfill will ensure sufficient weight on the concrete plate to achieve sufficient frictionforces.

CONSTRUCTION

The tube

Tube fabrication is planned on an assembly line with continuos launching to achieve 25m sections per week. The assembly line, extension of the tube alignment, will be adjacent to the production yard on the Lauvik side. A workshop will be built to house the assembly line. The workshop floor will be concreted. In the middle of the floor, a longitudinal ditch is built. The bottom of the tube haunches will rest on the ditch edges. 08

The workshop contains prefabricated hydraulic operated concrete forms in two separate positions. One for the lower half tube and one for the upper half.

The lower part is produced in position 1. The lower outside concrete form is mounted in the longitudinal floor ditch and is hydraulic operated. The lower inside form is divided in two separated parts which makes dismantling easy. The form is operated by a traverse crane. Fig. A

The upper part is produced in position 3. The innerform is a shrinkable wheeled form with a «driving through» 3 x 2.5m opening. The outer concrete form will be divided in two wheeled parts. The form is connected in the top by threadbars and supported on the shop floor.

The reinforcement mesh will be prefabricated in jigs outside the forms and placed in the forms by traverse running cranes.

The road slab is prefabricated elements produced outside the tube and mounted in position 2.

Concrete pouring is done in fixed places by pumps directly from the mixing plant in yard.

Pontoons

The pontoons will be produced in a dry dock in the vicinity. 07

When floated, the pontoons will be towed to the site by tugs. The pontoons are assumed anchored to the shore by cables. Maximum spanlength is approximately 1 km. Due to dead weight, the cables will achieve a severe sag. To maintain the cable stiffnes determined by the steel area, it is necessary to mount «buoyant force bodies» to the cables. With a prestressing force of 2 - 4000 kN, it is necessary to mount «buoyant force bodies» centered 300m.

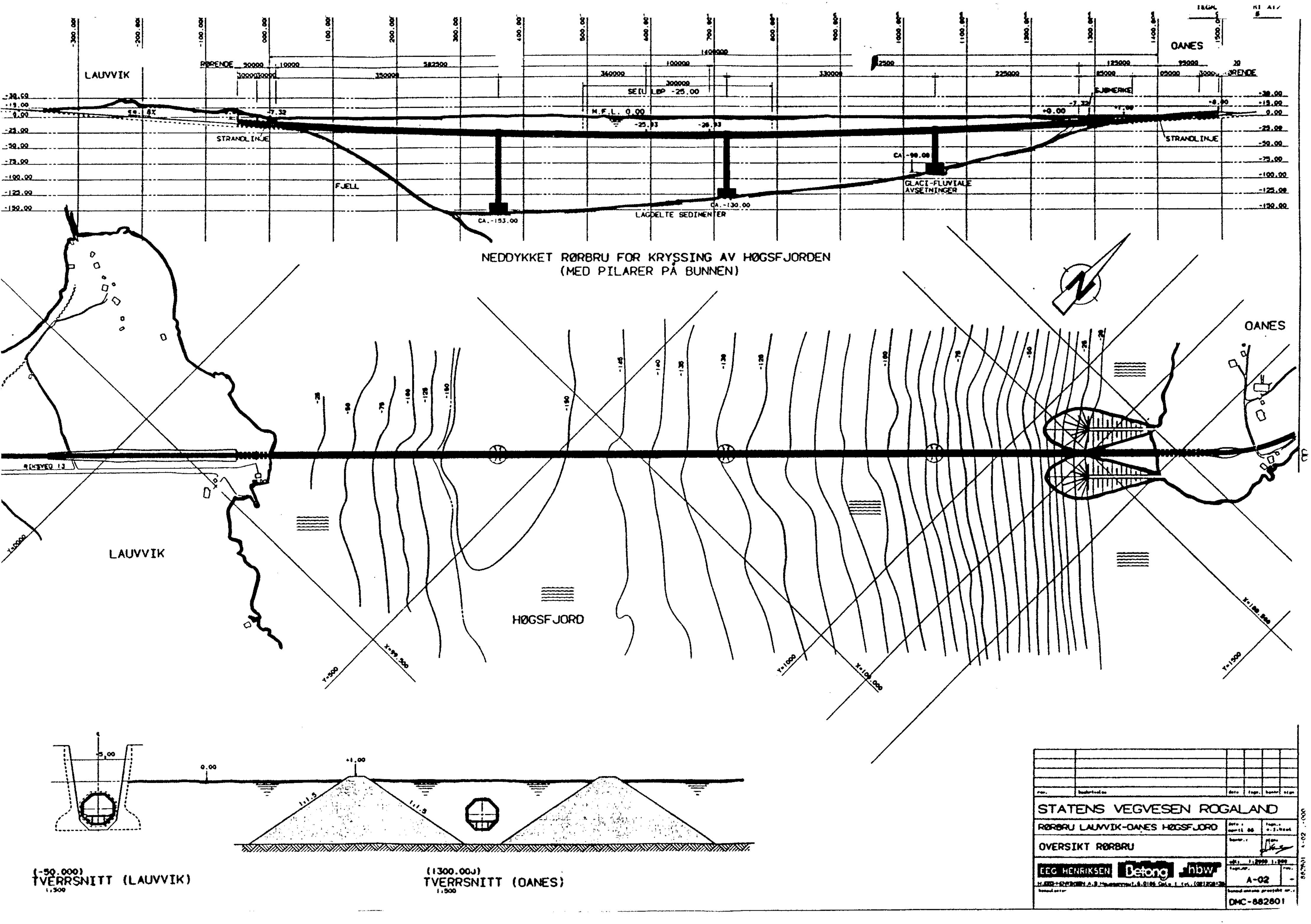
The connection towers are completed in the steelshop and transported by barge to the site and lifted into the sea by crane. Due to low submerged weight, the tower is easily handled in the sea.

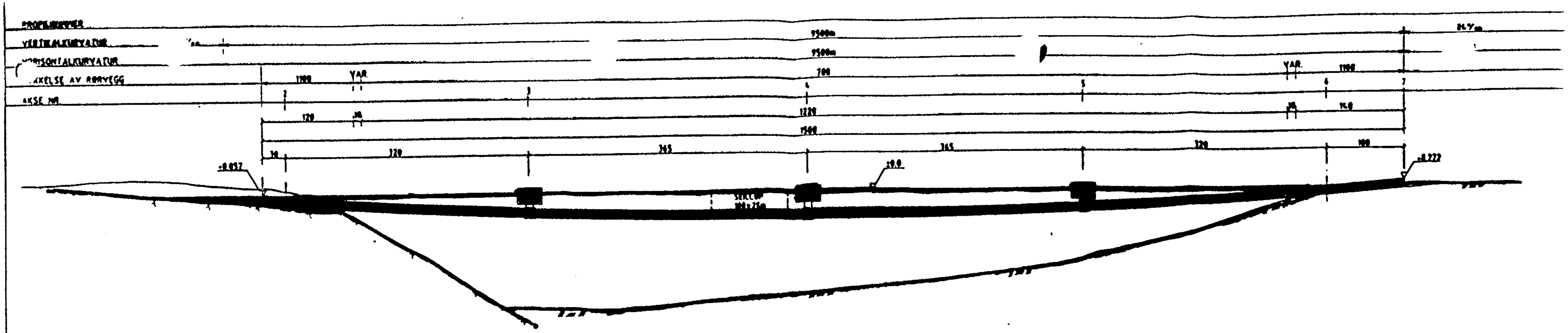
Launching

When the structure leaves the workshop, the tube will be ballasted according to production figures in order to achieve almost neutralized buoyancy. The tube will then have no need for any launching force, theoretically speaking. However, horizontal layers of water with different density results in a shifting buoyant force. Furthermore the tide will have an effect on the buoyant force acting on the tube. Hence, the launching forces must overcome the environmental forces as well as the friction forces at the upward sliding on the Oanes side. Fig. F

The launching system is a hydraulic devise mounted on a cantilevered concrete structure which is placed in pos. 4. The concrete structure consists of a fixed part and a sliding part made of steel. The sliding steel structure is connected to the fixed conrete structure by the main hydraulic cylinders. Both the sliding and the fixed structure are locked and relocked to the tube by radial hydraulic steel pins. Recesses are continuously made in the tube haunches to coincide with the steel pins. Fig. E

The main hydraulic cylinders are pushing the tube a length equal to the cylinder striking length (approx. 1.0m). The tube is then fixed before the cylinders are withdrawn to zero position before the next 1m push.


LAUVVIK
OANES
RØRENDE
SEIL LBP -25.00
M.F.L. 0.00
SJØMERKE
STRANDLINJE
FJELL
CA.-153.00
CA.-130.00
CA.-90.00
LAGDELTE SEDIMENTER
GLACI-FLUVIALE AVSETNINGER
NEDDYKKET RØRBRU FOR KRYSSING AV HØGSFJORDEN
(MED PILARER PÅ BUNNEN)
RIKSVEG 13
HØGSFJORD
(-50.000)
TVERRSNITT (LAUVVIK)
1:500
(1300.00)
TVERRSNITT (OANES)
1:500
STATENS VEGVESEN ROGALAND
RØRBRU LAUVVIK-OANES HØGSFJORD
OVERSIKT RØRBRU
EEG HENRIKSEN
Betong
hbw
A-02
DMC-882801



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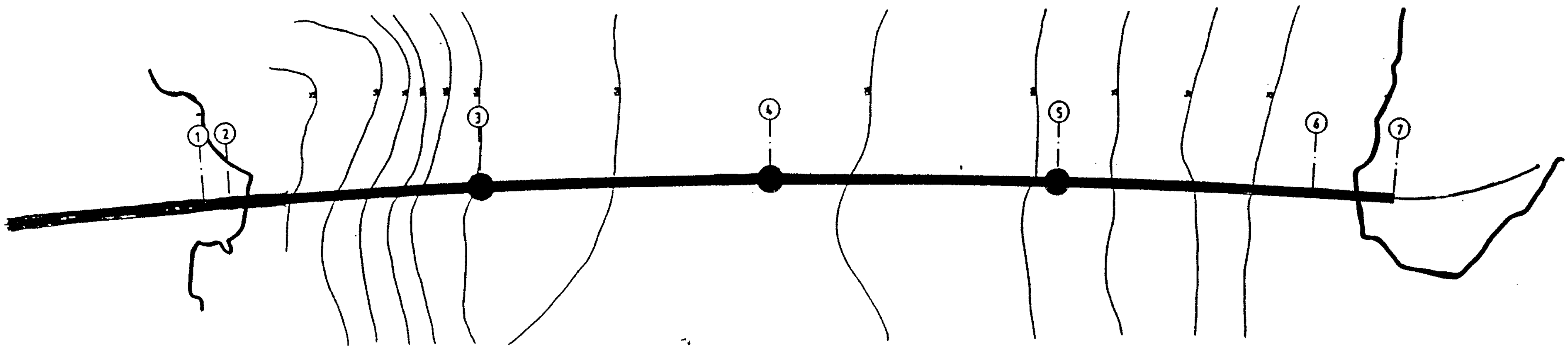

PLAN
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HENVISNINGER:

TEGN.	002	TYPISKE RØRTVERRSNITT
	003.	PONTONGER
	004	DETALJER FOR PONTONGER
	005	LANDFESTE LAUVIK
	006	LANDFESTE OANES
	007	KONSTRUKSJONS-SEKVENSER

STATENS VEGVESEN ROGALAND

RØRBRU LAUVVIK-OANES HØGSFJORD

ALTERNATIV MED PONTONGER
OVERSIKT

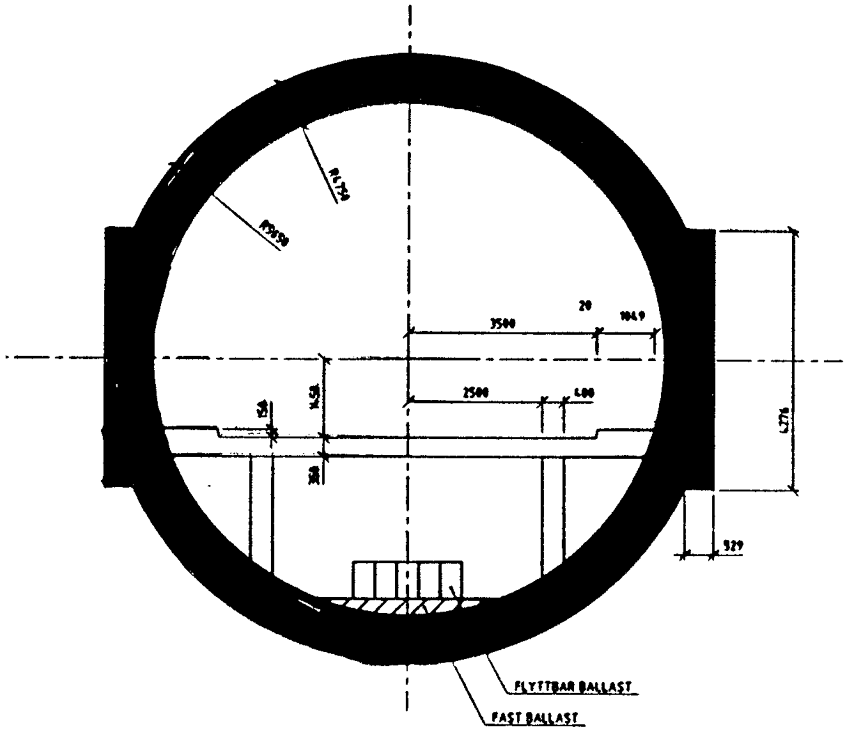
Dato 21.03.88

1:2500

01

KKG-HENRIKSEN · Betong · hbw

AAS-JAKOBSEN



TYPISK TVERRSNITT
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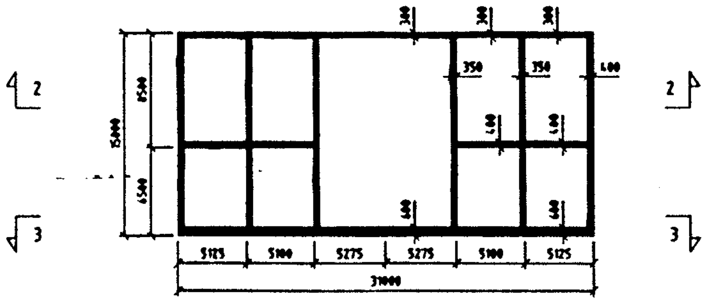
STATENSVEGVESEN ROGALAND

RØRBRU HØGSFJORD
TYPISK RØRTVERRSNITT

5457

[redacted]

02



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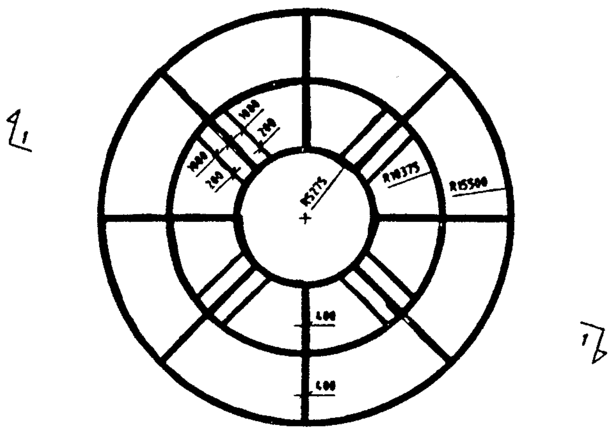

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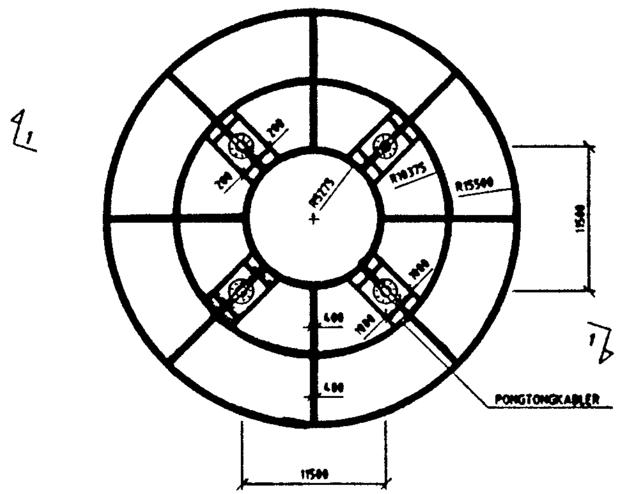

PONTONG SNITT 3-3
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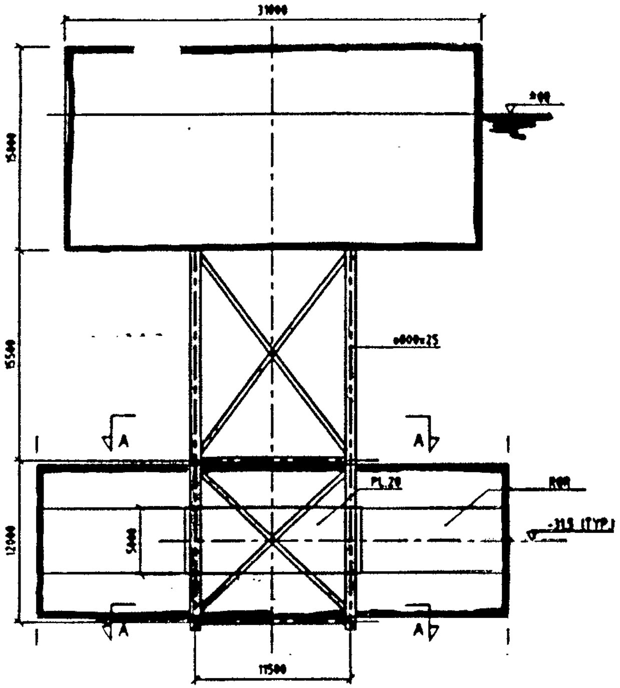
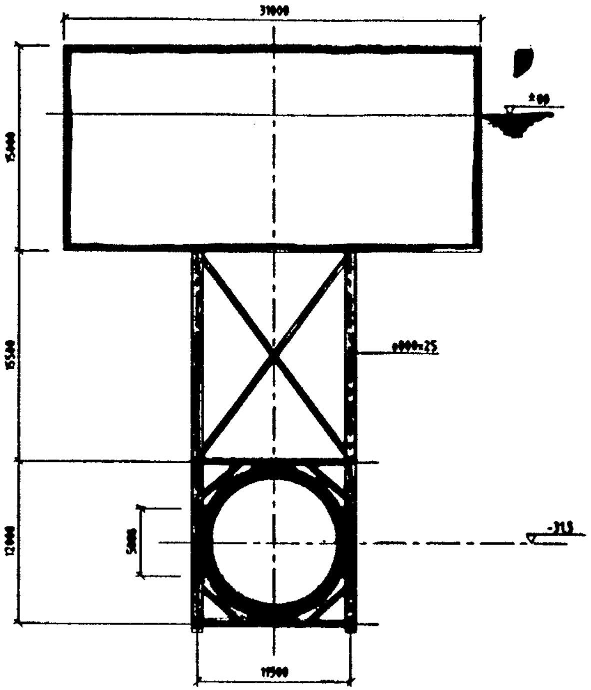
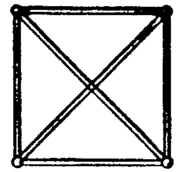
STATENSVEGVESEN ROGALAND

RØRBRU HØGSFJORD

PONGTONGER







(A) SNITT
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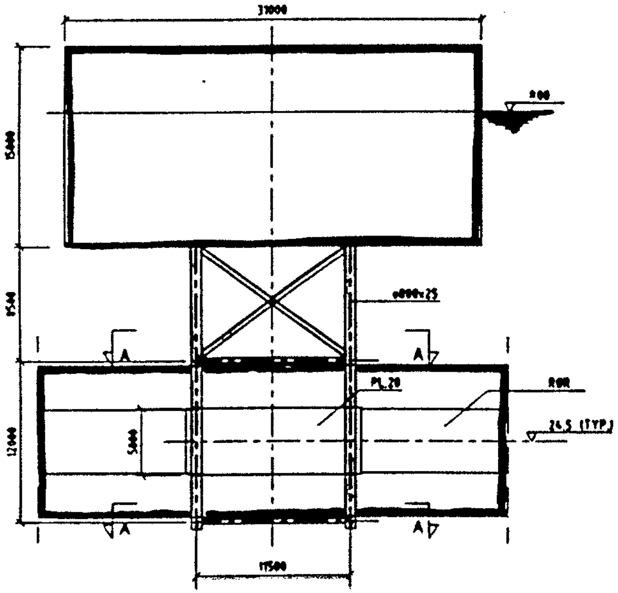
MIDTRE PONTONG

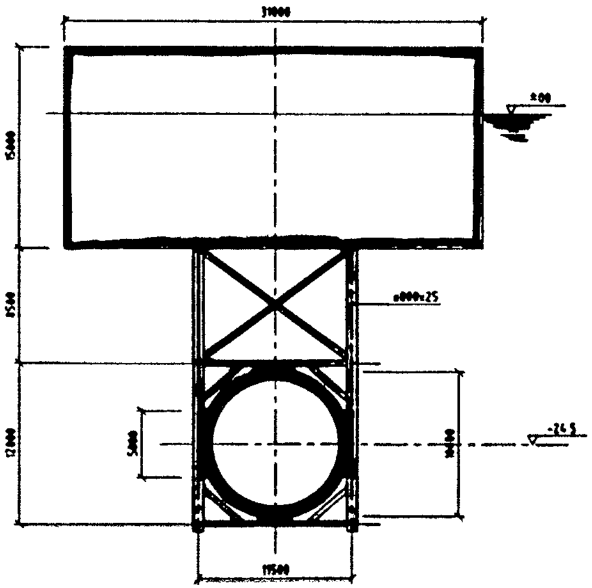
OPPRISS-LANGS RØR

OPPRISS-TVERRS RØR

BEMERKNINGER:

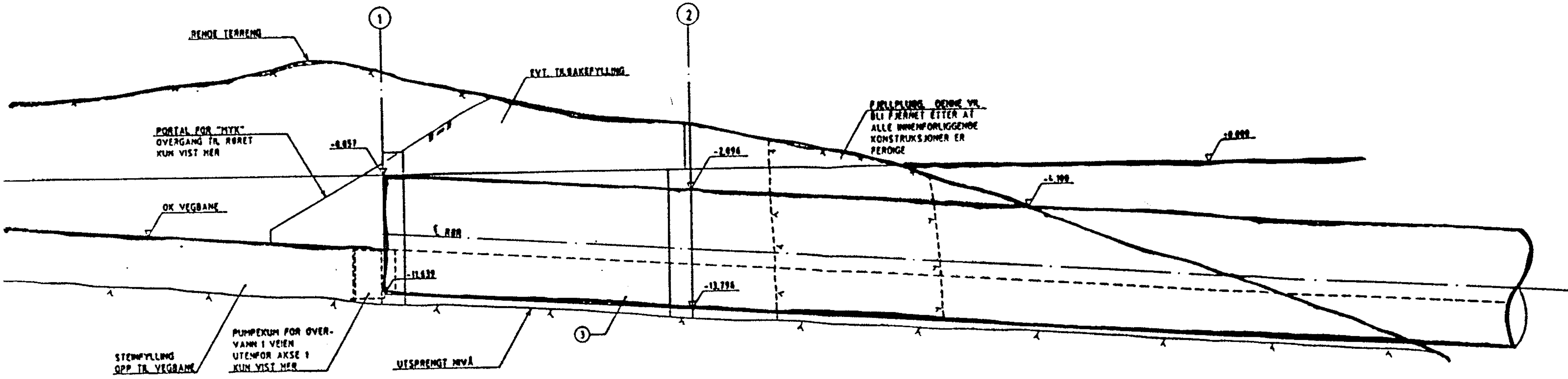
- ALLE RØR ER 400x10 HVIS IKKE ANNET ER ANGITT.





SIDE PONTONG (2 STK.)

STATENSVEGVESEN ROGALAND	
RØRBRU HØGSFJORD	
RØR PONGTONG FORBINDELSE	5457
	01



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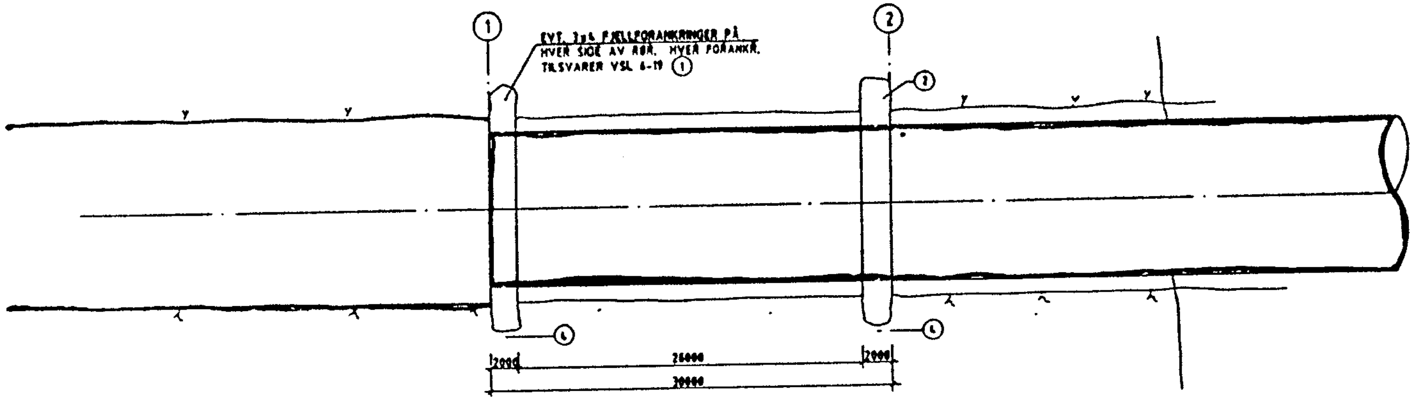


PLAN

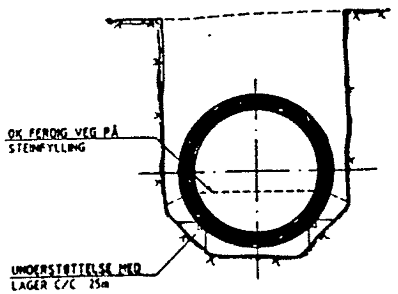
BEMERKNINGER:

(1) PORTALEN FORANKRES TIL FJELLET V.H.J.A. UTSTROSSING SOM VIST. DERSOM GEOLOGISKE UNDERSØKELSER VISER AT DETTE IKKE ER TILSTREKKELIG, VIL FJELLFORANKRINGER BLI BRUKT.

(2) AKSE 2 UTFØRES SOM AKSE 1, MEN DERSOM FJELLFORANKRINGER BLIR NØDVENDIGE, VIL MAKSIMALT 6 STK. FORANKRINGER TILSVARENDE VSL 6-19 BLI BRUKT.

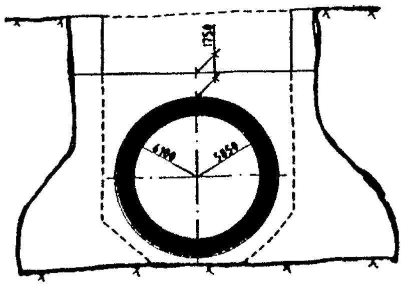
(3) UK RØR MÅ FORTANNES UTVENDIG OPP TIL ℄ RØR. MELLOM AKSE 1 OG 2 GYSES MED BETONG MELLOM FJELL OG RØR OPP TIL ℄ RØR.

(4) FOR Å FÅ EN TILNÆRMET VANNTETT FJELLSKJÆRING VIL FJELLET BLI TETNINGSINJISERT VINKELRETT PÅ BRUAKSEN I BÅDE AKSE 1 OG 2. I FJELLSKJÆRINGEN VIL FJELLET VÆRE UBEHANDLET.



TYPISK SNITT

LAGER UTENFOR AKSE 1 VED FREI



SNITT I PORTAL

STATENS VEGVESEN ROGALAND

RØRBRU LAUVVIK-OANES HØGSFJORD

ALTERNATIV MED PONTONGER
LANDFESTE LAUVVIK

21.03.88

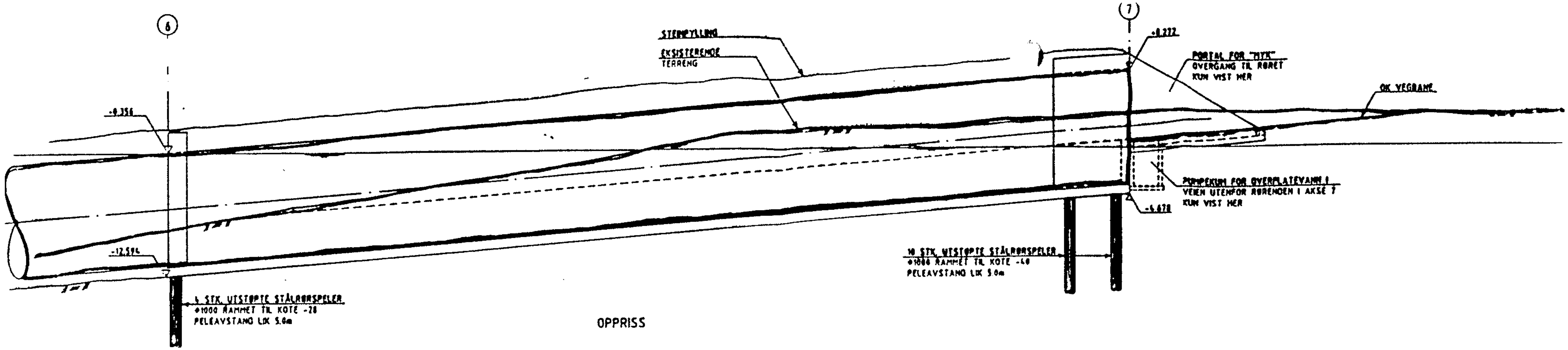
EEG-HENRIKSEN Betong hbw

AAS-JAKOBSEN

05

AAJ: 2925
CPG: 30299

13



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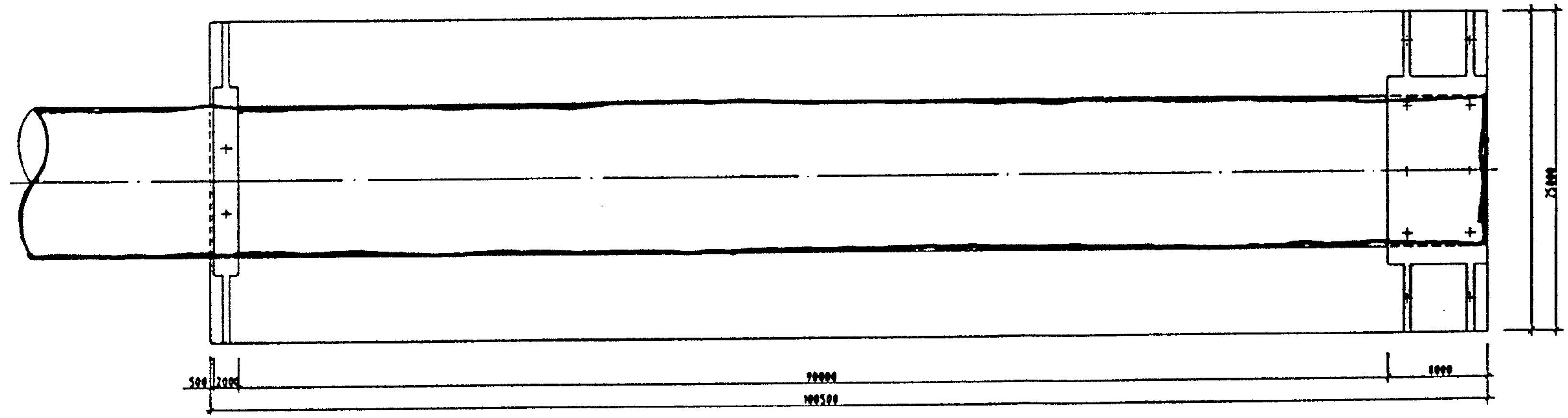
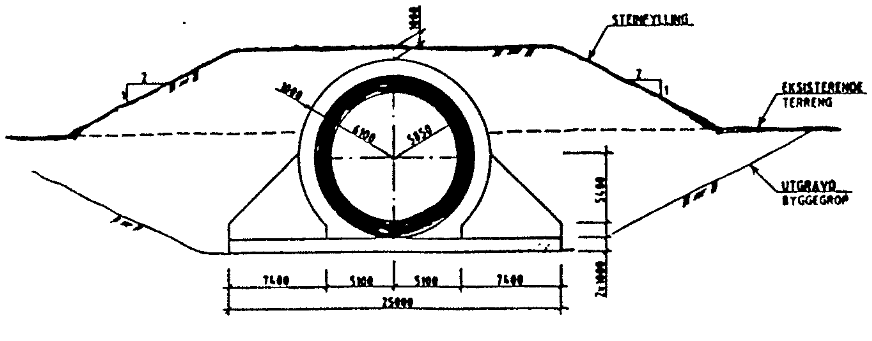


PLAN



TYPISK SNITT

STAT.VEG
LAST
UTVIKET
CTS I BUR
CSS I
LANDFESTE

STATENS VEGVESEN ROGALAND.

RØRBRU LAUVVIK-OANES HØGSFJORD

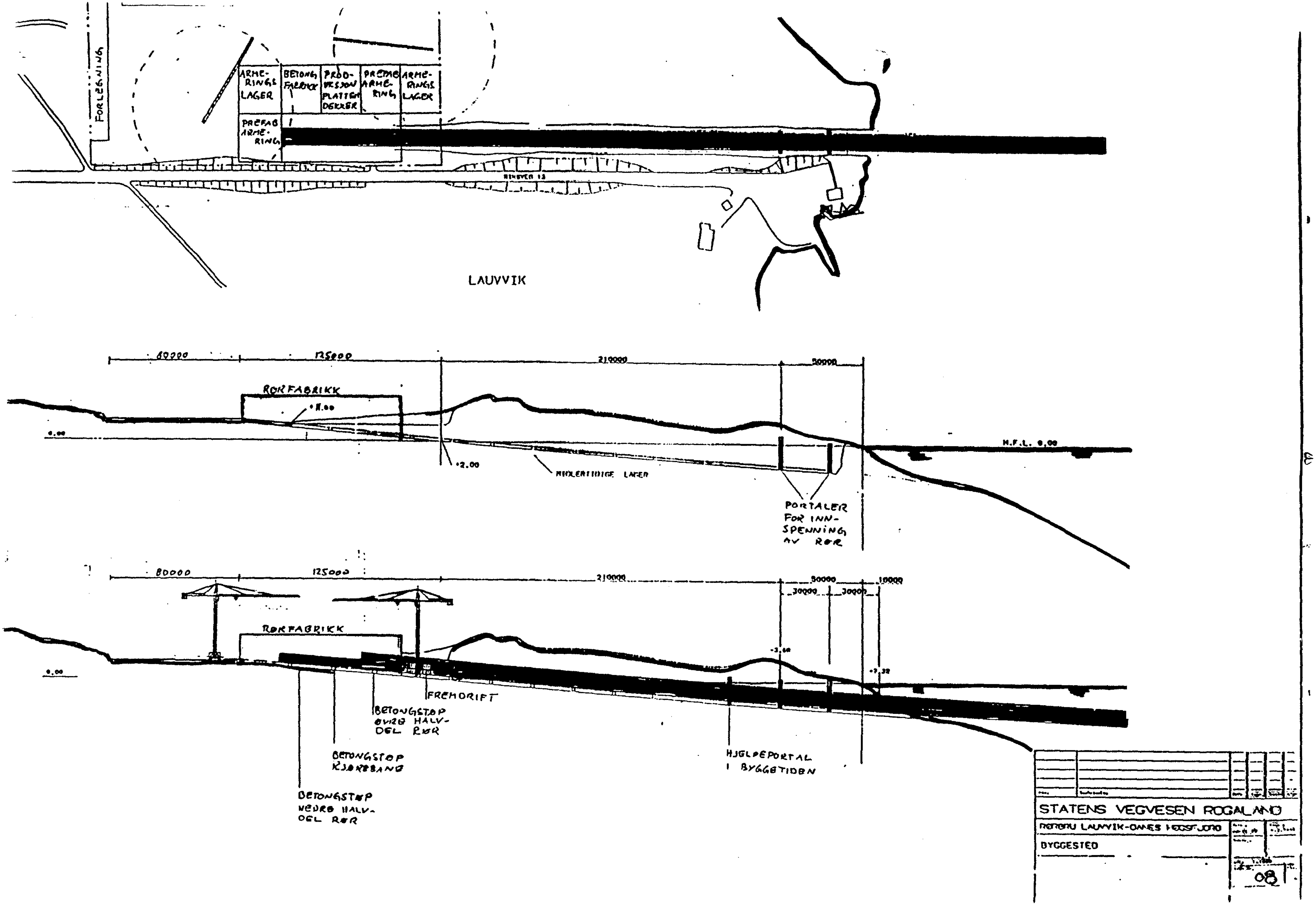
21.03.88

ALTERNATIV MED PONTONGER
LANDFESTE OANES

06

AAS-JAKOBSEN

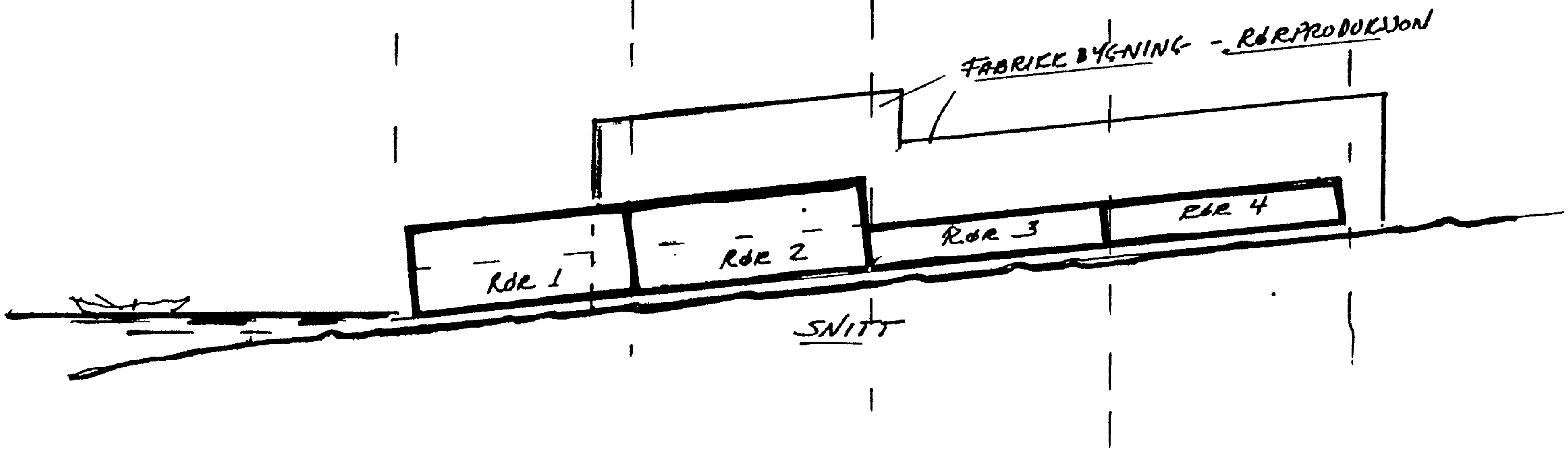
AAJ: 2985
CPS: 30109

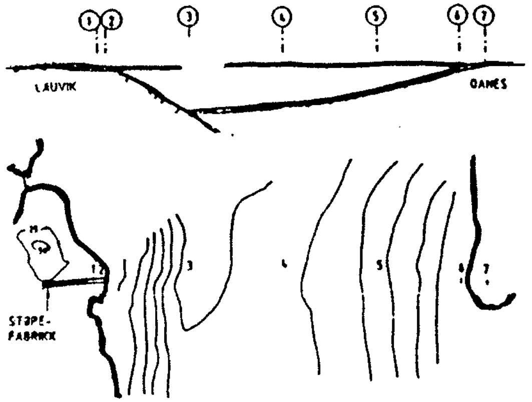

FORLEGNING
ARMERINGS LAGER
BETONG FABRIKK
PRODUKSJON PLATTEN DEKKER
PREFAB ARMERING
ARMERINGS LAGER
PREFAB ARMERING
LAUVVIK
80000
125000
210000
50000
RØRFABRIKK
H.F.L. 0,00
MIDLERTIDIGE LAGER
PORTALER FOR INNSPENNING AV RØR
80000
125000
210000
50000
10000
30000
30000
RØRFABRIKK
FREMDRIFT
BETONGSTØP ØVRE HALVDEL RØR
BETONGSTØP KJØREBANE
BETONGSTØP NEDRE HALVDEL RØR
HJELPEPORTAL I BYGGETIDEN
STATENS VEGVESEN ROGALAND
BYGGESTED
08

RØRBRU OVER HØGSFJORD – RØRFABRIKK – FIG. A

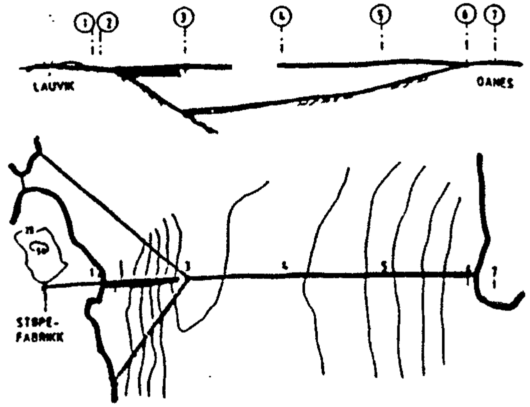
Pos. 4	Pos. 3	Pos. 2	Pos. 1	
Ferdig rør	Øvre halvdel Rør	Nedre halvdel Rør	Nedre halvdel Rør	
Klargjøring for sjøfasen Rør 1	Forskaling Armering Støping Rør 2	Montering og støp kjørebane og ballast Rør 3	Forskaling Armering Støping Rør 4	Prefab. slakk og spennarmering til nedre halvdel av rør.
	Prefab. slakk og spennarmering til øvre halvdel av rør	Prefab. ballastblokker og plattendekket til kjørebane	Betongfabrikk med betongpumpe og rørfordelings–system	

PRINSIPPSKISSE

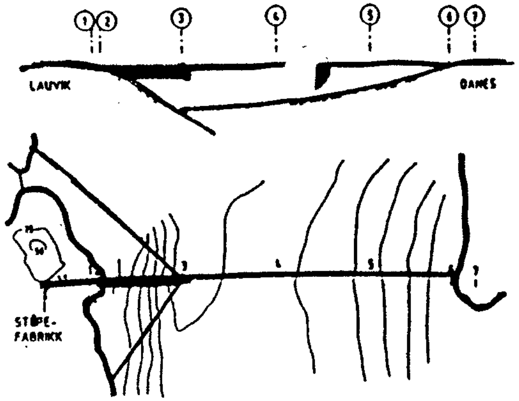




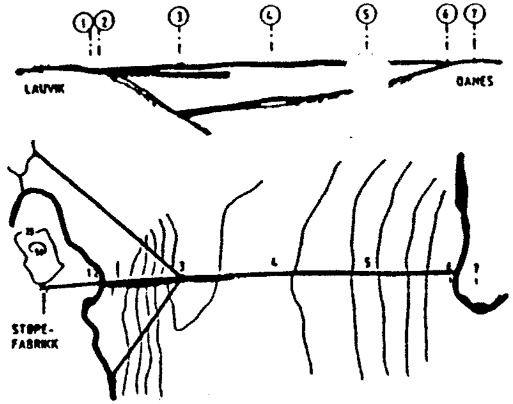
A. UTSPRENGNING AV FJELLSKJÆRING LAUVIK-SIDEN. PORTALER I AKSE 1 OG 2, GLIDELAGRE OG STØPEFABRIKK UTFØRES FØR FJELLPROPP MOT SJØEN FJERNES.



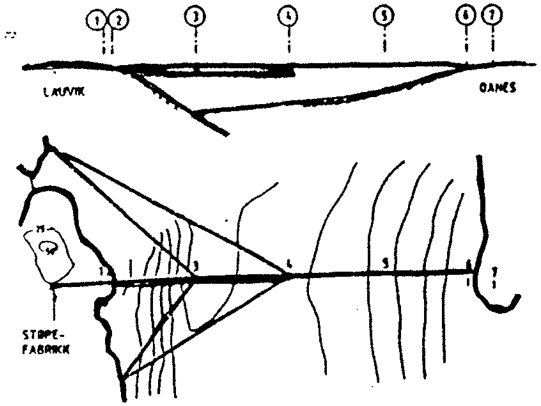
B. RØRET PRODUSERES I FABRIKK OG SKYVES ETAPPEVIS FRAM MOT POSISJONEN FOR FØRSTE PONTONG. 1'STE PONTONG FABRIKERES I TØRRDOKK MED RINGER. DEN TAUES TIL POSISJON I AKSE 3 OG AVSTAGES SIDEVEIS SOM VIST.



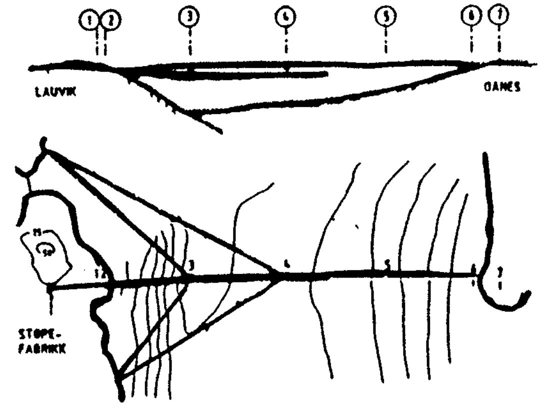
C. RØRENDEN ENTRES GJENNOM RINGEN UNDER PANTONG I AKSE 3.



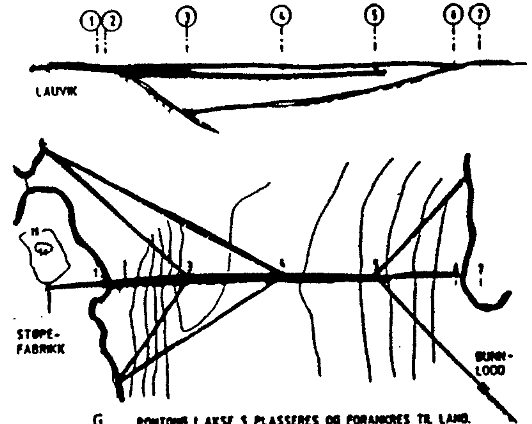
D. RØRET SKYVES FREM GJENNOM RING I AKSE 3.



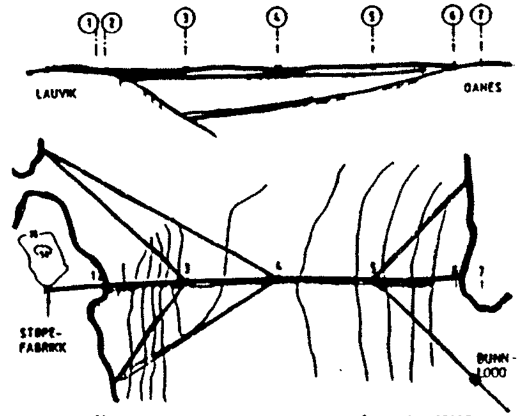
E. PONTONG I AKSE 4 PLASSERES OG FORANKRES TIL LAND. RØRENDEN ENTRES GJENNOM RINGEN UNDER PONTONGEN.



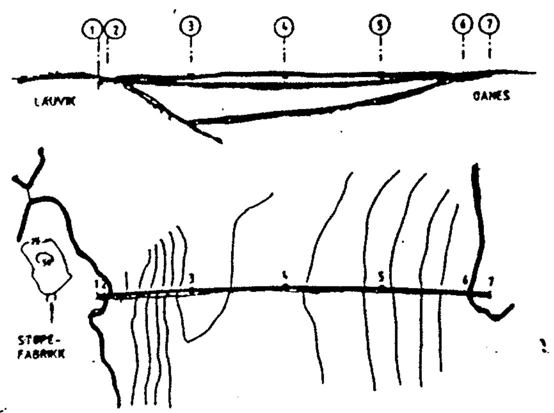
F. RØRET SKYVES FREM GJENNOM RING I AKSE 3 OG 4.



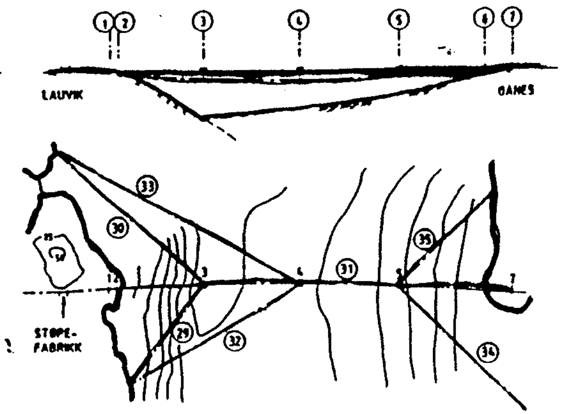
G. PONTONG I AKSE 5 PLASSERES OG FORANKRES TIL LAND. ET BUNNLODD BENYTTES PÅ DET ENE STAGET FOR Å SIKRE FRI DYBDE. LODDET LEGGES PÅ CA. 13m DYP. RØRENDEN ENTRES GJENNOM RINGEN UNDER PONTONGEN.



H. RØRET SKYVES VIDERE FRAM OG ER NÅ FERDIG UTSTØPT.



I. RØRET DRAS/SKYVES INN I LANDFESTE PÅ OANESIDEN OG STØPES INN I BEGGE LANDFESTER. SIDEFORANKRINGER FJERNES. BRUA KLARGJØRES FOR BRUK.



NOTASJON FOR FORANKRINGSKABLER

BEMERKNINGER :

1. OPPTREDENDE KRAFT (mm) I SIDEFORANKRINGENE ER VIST I TABELLEN I BRUKS- OG BRUDDGRENSETILSTANDEN.

EL. NR.	POSISJON I*		POSISJON G*		POSISJON F		POSISJON E*		POSISJON D		MAKSIMAL-VERDIER	
	BRUKS	BRUDD	BRUKS	BRUDD	BRUKS	BRUDD	BRUKS	BRUDD	BRUKS	BRUDD	BRUKS	BRUDD
29	±1.52	±1.60	±1.72	±1.95	±1.46	±1.89	±1.58	±2.00	±1.44	±1.65	±1.72	±2.00
30	±1.01	±1.10	±1.11	±1.20	±1.08	±1.17	±0.36	±1.06	±0.92	±1.00	±1.11	±1.84
31	±1.62	±2.60	±1.08	±1.41	±0.90	±1.21		±0.63		±0.46	±1.62	±2.60
32	±1.25	±1.69	±3.60	±4.33	±2.12	±2.71					±3.60	±4.33
33	±1.26	±1.36	±2.96	±3.27	±1.91	±2.06					±2.96	±3.27
34	±2.09	±4.42									±2.09	±4.42
35	±2.11	±2.50									±2.11	±2.50

* I DISSE POSISJONER ER IKKE RING/PORTAL I HENHOLDSVIS AKSE 6,5 OG 4 ENTRET.

HVER KABEL MÅ FORSPENNES SLIK AT TRYKK IKKE OPPSTÅR I BRUDDGRENSE-TILSTAND.

-STATENS VEGVESEN ROGALAND

RØRBRU LAUVVIK-OANES HØGSFJORD

ALTERNATIV MED PONTONGER
KONSTRUKSJONER

23.03.88

1

07

EEG-HENRIKSEN | Betong | hbw

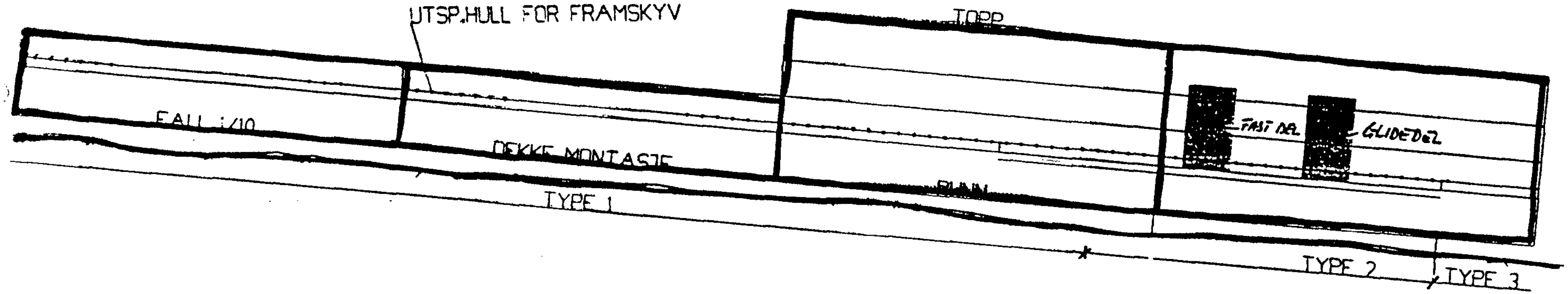
AAS-JAKOBSEN

AAS : 2975

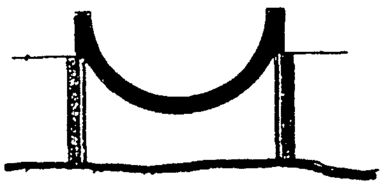
Fu F

RØRBRU OVER HØGSFJORDEN

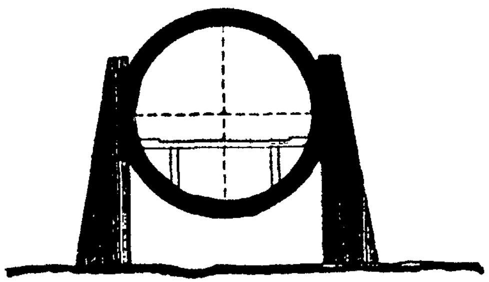
PRINSIPPSKISSE FRAMSKYVARRANGEMENT



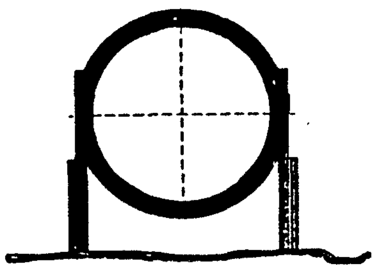
VANGE FOR OPPLEGG/ GLIDEFLATER
TYPE 1



VANGE FOR OPPLEGG/ GLIDEFLATER
TYPE2



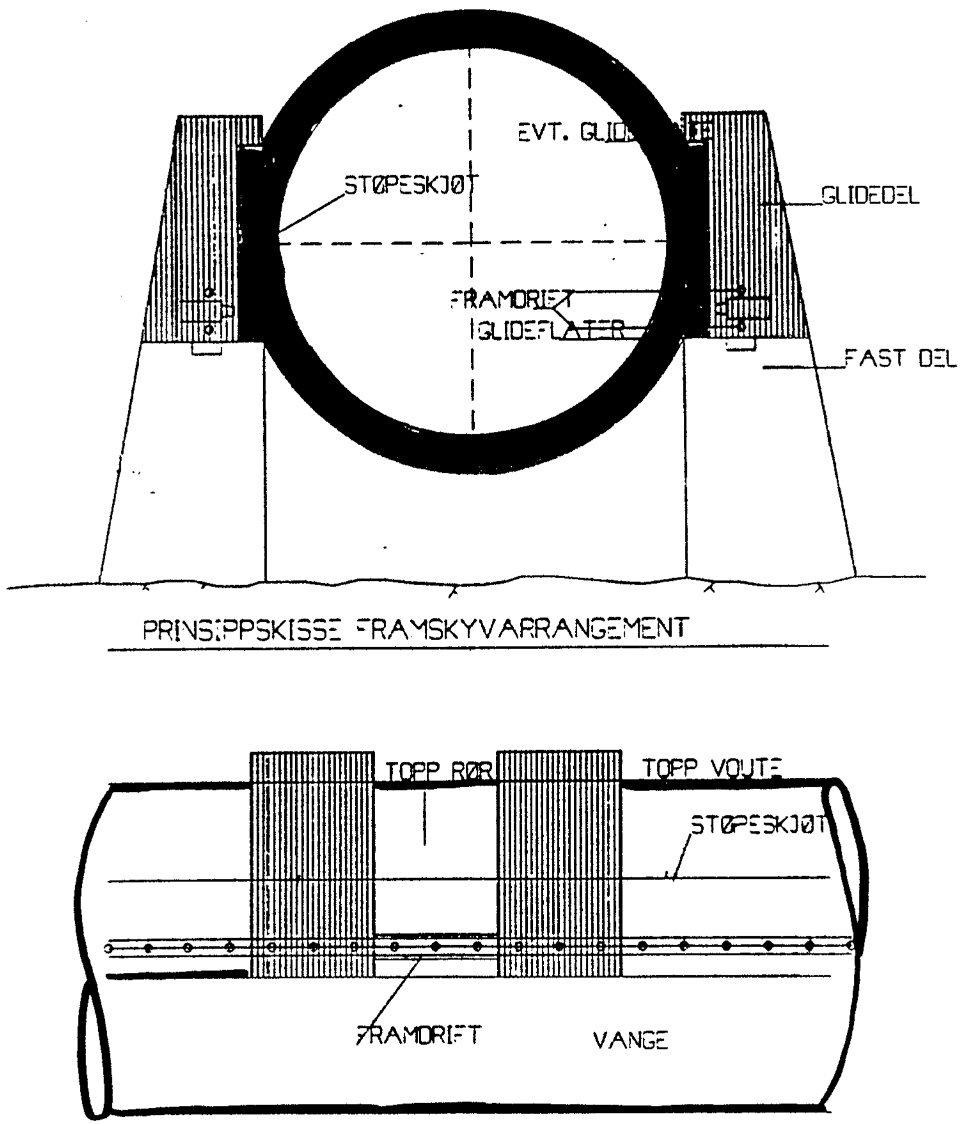
TYPE 3



EEG-HENRIKSEN ANLEGG AS

Fig. E

RØRBRU OVER HØGSFJORDEN



PRINSIPPSKISSE FRAMSKYVARRANGEMENT

EEG-HENRIKSEN ANLEGG A.S.

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

6. Høgsfjord Crossing

6.3 Submerged Turbular Bridge

A. Loen
Kværner Rosenberg a.s

SUBMERGED TUBULAR BRIDGE

1. INTRODUCTION

Presented by Arnljot Loen, Kværner Rosenberg a.s (KR), Stavanger.
The following is a presentation of Kværner Rosenberg's submerged steel tubular bridge which is the only steel consept developed for crossing Høgsfjorden.

Initially you will find a brief presentation of Kværner Rosenberg.
Then you are given an overview and status of the tubular bridge that KR believes is a competetive solution. Main design criteria for the steel consept are described along with some of the studies that have been performed. Finally, conseptual benefits are summarized.

2. KVÆRNER ROSENBERG A.S

Kværner Rosenberg Offshore Yards comprise Rosenberg Yard in Stavanger and Egersund Yard in Egersund, some 70 km south of Stavanger. Our primary market is the Norwegian oil and gas industry.

We have since 1970 been a member of the Kværner Group which today is a world-wide engineering and industry company with nearly 60.000 employees.

Rosenberg Yard was founded in 1896, hence celebrating our 100th anniversary this year. For the last 18 years we have maily constructed oil platforms for the offshore industry.

The Kværner Rosenberg yards employ 2100 individuals, whereof 250 engineers performing design and fabrication engineering. Through the last 10 years we have completed several turn-key (EPC) contracts which have included conseptual studies and design engineering followed by detail engineering, procurement and construction.

At present KR is engaged with several projects comprising floating structures which, similar to a submerged tubular bridge project imply heavy steel work and marine operations:

- Bingo - a multi purpose semi submersible platform
- Visund hull - the hull structure for the Visund floater
- Sea Launch - A floating rocket base to be used for launching commersial sattelites

SUBMERGED TUBULAR BRIDGE

OTHER EXPERIENCE

In addition to the wide range of modules for the oil and gas industry, Kværner Rosenberg has developed a floating bridge consept.

The floating bridge consept was extensively developed through technical and commersial bid phases for the two floating bridges which where built in Norway recently.

The basis of the consept is a steel beam with a rectangular hollow section with curved side panels. The bridge beam is supported on partly submerged concrete pontoons.

3. STATUS OF KR'S SUBMERGED TUBULAR BRIDGE

STATIC/ DYNAMIC SYSTEM:

- Horizontal bow 150 m wide
- Vertical bow of 35 m height
- Vertical support at shore, and by waterplane stiffness and weight of pontoons
- Horizontally the bridge need a degree of fixity at the end suppoerts in order to reduce loadeffects at the midsections. Two alternatives exist:
  a) end fixation ($r_z = 0$)
  b) pinned support + tension cables

CROSSECTIONS:

- Bridge tubular and pontoons by steel, NS grade St52 i.e yield stress 335-355 N/mm^2
- Shell plate with variable thickness up to 70 mm in areas where ship collision may occure
- Ring stiffeners
- Internal side panels by concrete to smooth the side walls for safety of personnel and to protect ring stiffeners from damage
- Rock ballast
- Drain system
- Corrosion protection by paint systems and catodic protection or by metalisation

PONTOONS

- Permanent buoyancy in tubular for safety (governmental requirement)
- Water ballast in pontoons to counteract permanent buoyancy
- Failure control preventing damage in main tube

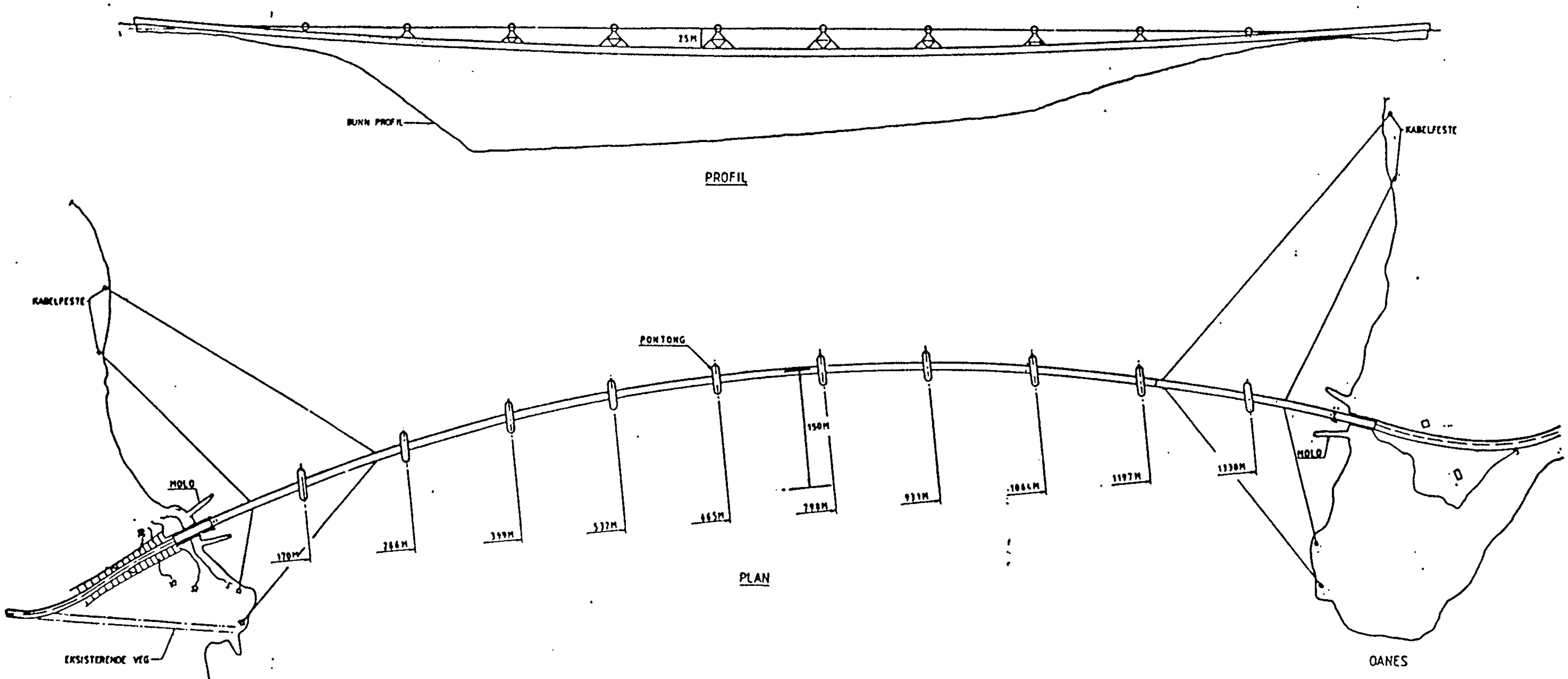
KVÆRNER ROSENBERG a.s

- Yards in Stavanger and Egersund
- 2100 employees
- 250 engineers
- Offshore market
- Turn-key delivery contracts
- Turnover in 1995: US$ 313 mill.

- Kværner Group
- Industry, construction and engineering
- 60.000 employees
- 10.000 employees in Norway

Kværner Rosenberg a.s

SUBMERGED TUBULAR BRIDGE



PLAN AND PROFILE

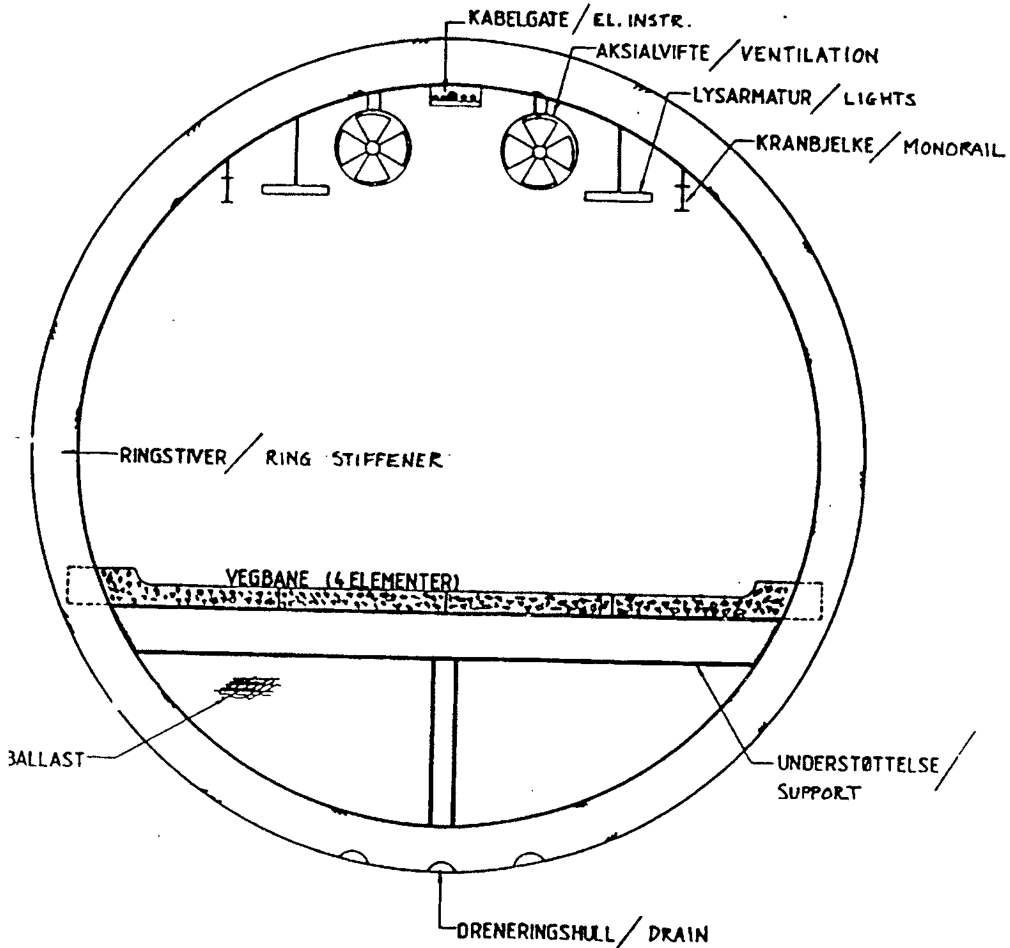
LAUVVIK

KVÆRNER

Kværner Rosenberg a.s

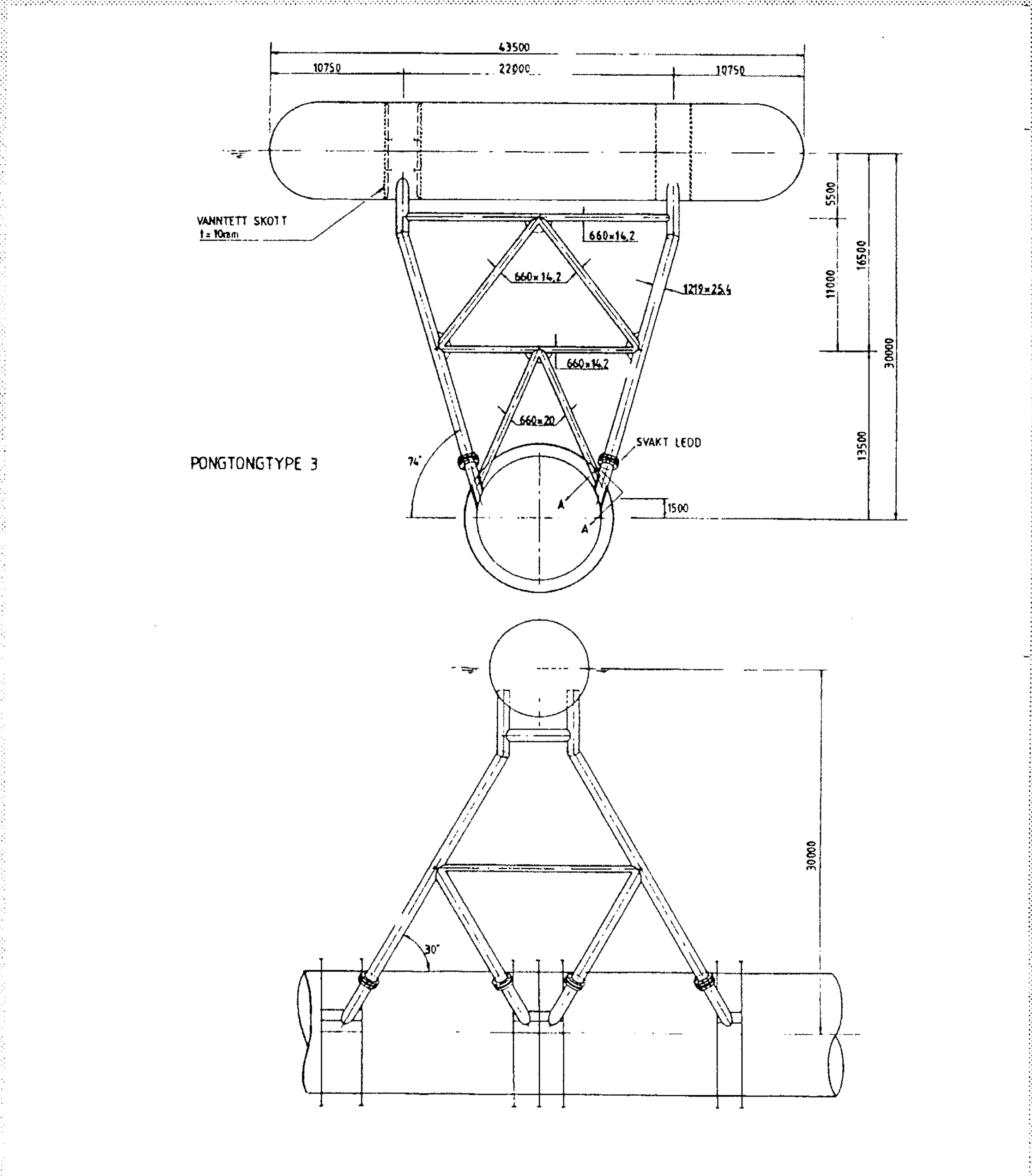
SUBMERGED TUBULAR BRIDGE

CROSSECTION



SUBMERGED TUBULAR BRIDGE

PONTOONS AND CONNECTIONS



SUBMERGED TUBULAR BRIDGE

CONCEPTUAL ADVANTAGES

- No seabed foundations required
- Water tight material
- Well known and predictable material properties and behavior
- Control of corrosion problems
- Easy detectable damages
- Offshore experience and quality used in design and fabrication
- Possible parallel production ⟶ shorter fabrication
- Easy inspection during fabrication and in operation
- Easy maintenance

HIGH SAFETY
AND
LOW LIFE CYCLE COSTS

SUBMERGED TUBULAR BRIDGE

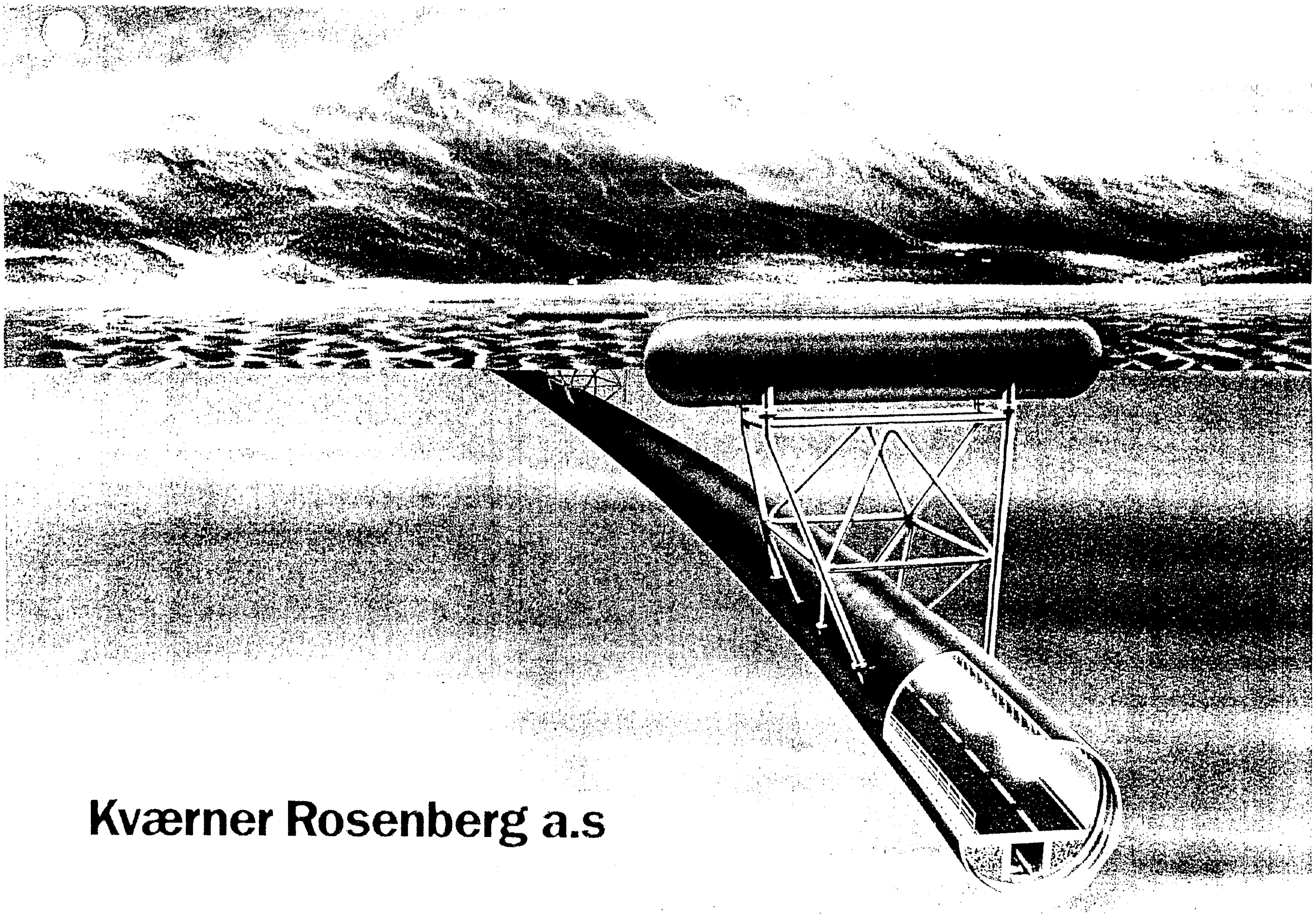
MAIN DESIGN PARAMETERS

1. Outer diameter
2. Wave action
3. Ship collision
4. Variable functional load
5. Permanent net buoyancy
6. Lifetime 100 years

SUBMERGED TUBULAR BRIDGE

PERFORMED EXTERNAL STUDIES

1.	NHL-SINTEF	:	Wave and current forces
2.	SINTEF	:	Non-linear response analysis
3.	VERITEC	:	Dynamic analysis (sink-source)
4.	VERITEC	:	Ship collision calculation
5.	VERITEC	:	Fire action calculation
6.	JOTUN	:	Corrosion protection



Kværner Rosenberg a.s

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

6. Høgsfjord Crossing

6.4 Høgsfjord Crossing

T. Einstabland
Selmer ASA

International Conference on Submerged Floating Tunnels
Sandnes, Norway

HØGSFJORD CROSSING

Proposal by **Selmer ASA**

Presented by Tomas Einstabland

INTRODUCTION

This paper deals with a consept for fjord crossing, a submerged floating tunnel. The consept was first presented on an International Symposium of Strait Crossing in Stavanger, Norway, Oct. 7-9, 1986. The consept was further developed in "Konseptkonkurransen" initiated by Vegdirektoratet, Norway, in 1988, and updated further, espesially with respect to cost in 1994. This presentation to-day will consentrate on the construction of the tunnel.

THE CONSEPT

The tube bridge concept consists of a submerged tubular concrete arch with circular cross-seciton. The arch is kept in vertical position by means of surface-piercing pontoons. The pontoons act as anchoring to the surface thus avoiding a mooring system to the bottom.

The tube has been given a circular arch form in order to minimize current loadings. Further, the tube is assumed to be ballasted to have a net positive buoyancy. Consequently the pontoons must be ballasted to excert a downward force to keep the tube in submerged position. The net effect will be reduced moments in the tube due to traffic loads and added safety in case one of the pontoons is damaged by ship collision.

In order to minimize the built-in end moments in the tube an optimalization of the number and placement of the pontoons is needed in each particular case. More pontoons add vertical stiffness to the structural system and thus reduces the effects of traffic loads and water density fluctuations. But increased stiffness on the other hand increases the built-in end moments due to tidal variation.

Ship collisions represent a problem. One way to deal with this problem is to design the pontoons with a deliberate weak spot. Thus, if a pontoon is subjected to a collision impact above a certain prescribed level the pontoon breaks off without damaging the rest of the structure. As a further safety requirement, the structure will be designed to function satisfactory with one pontoon removed.

CONSTUCTION

The Høgsfjord Crossing consists of the following:

- Submerged tube, Lauvvik to Oanes
- Rocktunnel/culvert at Lauvvik
- Rocktunnel at Oanes

The submerged tube consists of the following:

- The Tube
- Towers from the tube to the surface
- Pontoons fixed to the towers
- Anchoring structures at landfalls

The tube has been given a horisontal curve of 1,500 m. The waterdepth over the tube at mid-span is 26.5 m. The vertical curve is some 12,000 m. The internal diameter is 9.6 m. The wallthickness is 0.9 m at mid-span and 1.45 m at landfalls.

The tube is equipped with 6 towers, and the pontoons are fixed in the upper part of the towers. Both the towers and the pontoons are made of steel. The towers are waterfilled in operation. The tube is designed to provide an uppward force of 10 kN/m.

KEY DATA

Width of fjord	1,513	m
Length of ~~fjord~~ tube	1,585.5	m
Length tunnel/culvert Lauvvik	approx. 145	m
Length tunnel/culvert Oanes	approx. 270	m
Total length	**approx. 2,730**	**m**
Clearance over tube at midspan	26.5	m
Clearance over tube at landfalls	0	m
Tube, inner diameter	9.6	m
Tube, min. wallthickness	0.9	m
Tube, max. wallthickness	1.45	m
Tower	6	pcs.
Concrete quality	C35-C75	
Concrete quantity	approx. 67,000	m^3
Concrete quantity culverts etc.	approx. 9,000	m^3

LANDFALL LAUVVIK

The access to the submerged tunnel will be provided through a rock-tunnel and a concrete culvert.

The anchoring structure consists of a caisson, as an integrated part of the tube. This structure will be placed on prepared foundations on excavated rock. The caisson will be ballasted by rockfill and anchored to rock.

LANDFALL OANES

The landfall at Oanes has been placed on rock and the anchoring structure is similar to the structure at Lauvvik. From the concrete culvert, the road will pass through a rock tunnel. This rock tunnel will be used as dry dock for the construction of the tube.

CONSTRUCTION SEQUENCES

The sequences will be as follows:

Phace 1 7 tube sections each 226 m long, with roadway and some ballast will be casted in the dock tunnel at Oanes.

Phase 2 The sections will be towed to the jointing site in Skreddersundet.

Phase 3 Section 1 will be moored at the jointing site.

Phase 4 Connection of section 2 and 3 after ballasting with concrete.

Phase 5 Rearrangement of moorings.

Phase 6 Connection of sections 4, 5, 6 and 7 after ballasting with concrete.

Phase 7 Installation of tower and pontoons. Connection of tugs.

Phase 8 Towing to the site, ballasting with water in tower, pontoons and ballast containers.

Phase 9 Ballast containers will be filled with water to touch-down onto the foundations.

Phase 10 Ballast with rock-fill.

Phase 11 Concreting between ballast containers and foundations.

Phase 12 Installation of rock anchors at each end.

Phase 13 Concreting between tube ends and culverts.

Phase 14 Removal of bulkhead walls.

Phase 15 Installation of fans and drainage arrangement. Asphalt pavement.

HIGH PERFORMANCE CONCRETE

The application of using high performance concrete is a must in a structure like a submerged floating bridge.

Experiance gained in Norway through the last decades proved to be very usefull in "estimating" the life time of a structure.

As a referance project we would like to mention a project, constructed by Selmer in 1982, the "Shore Approach", a submerged concrete tunnel.

This structure has been inspected several times after the completion. The most remarkable results are those connected with the resistance to cloride ingress. In the case of a concrete cover of 50 mm the calculated "service life" is 220 years, fig. 11.

CONCLUSION

Based upon the studies performed and on our general experiance, we are confident that a submerged floating tunnel, as presented, is a feasible and safe solution, both from a construction and operation point of view.

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M.MAAGE, S.HELLAND, J.E.CARLSEN "Chloride Penetreation in High Performance Concrete Exposed to Marine Environment", 3d International Symposium on the Utilization of HSC, Lillehammer, Norway, June 1993
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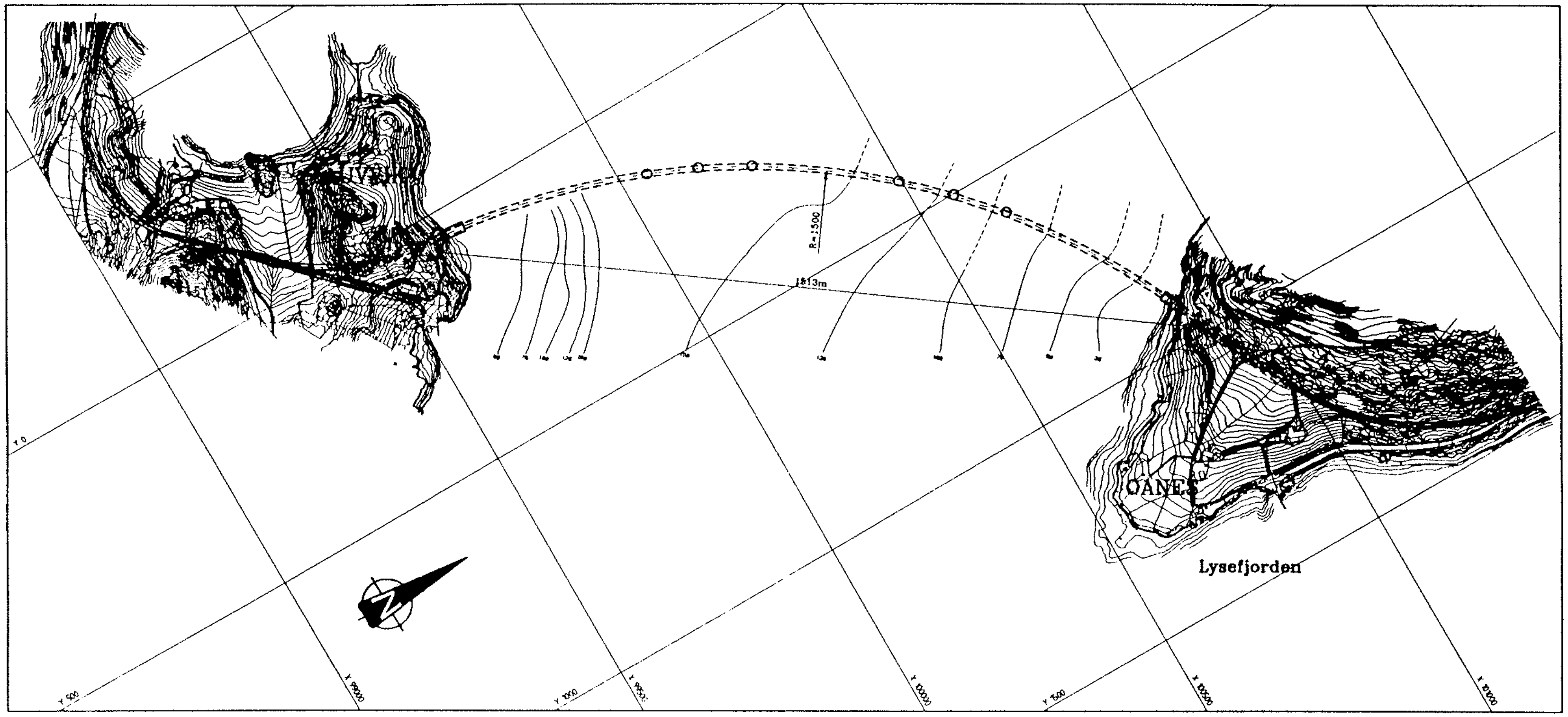
FIGURES

Fig. 1 to fig. 11.



Kartgrunnlag : Statens vegvesen, Rogaland

PLAN
1:5000

FIG. 1

METER
0 200 400 600 800

A	15.12.94	KOSTNADSOVERSLAG	Tør	T.L.	T.E.

STATENS VEGVESEN

RØRBRU HØGSFJORDEN - KONSEPT 94

OVERSIKT

1:5000

Selmer A.S

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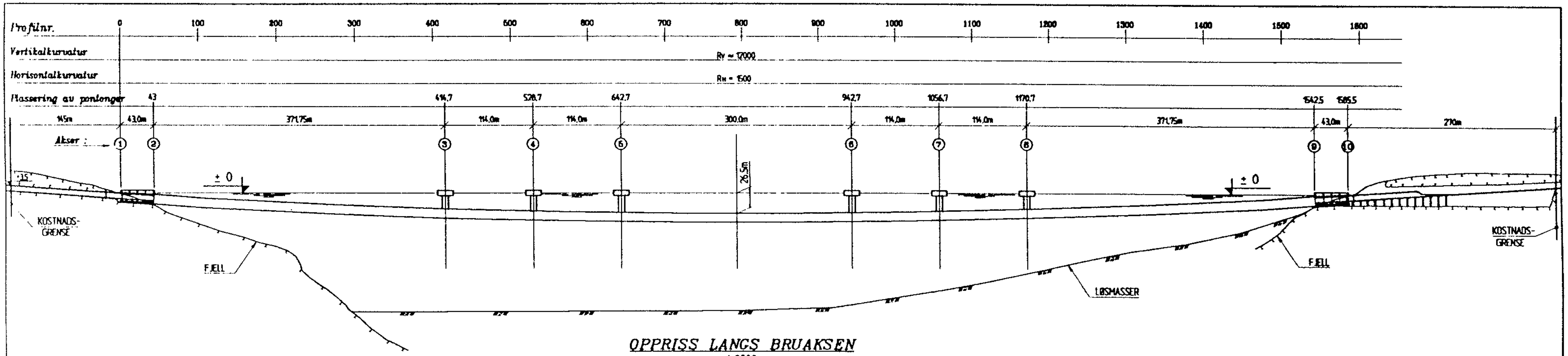
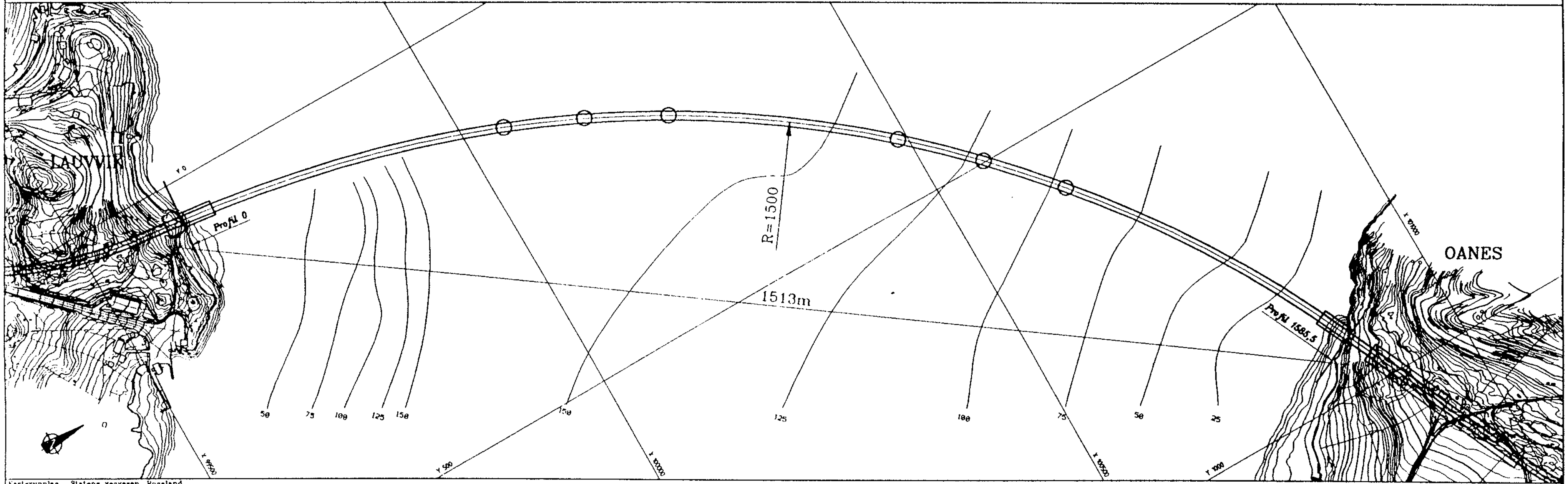


OPPRISS LANGS BRUAKSEN
1:2500



Kartgrunnlag Statens vegvesen, Rogaland

PLAN
1:2500

METER
0 25 50 75 100 125 250 375

FIG. 2

A	15.12.94	KOSTNADSOVERSLAG	Ter.	T.L.	T.E.
Rev	Dato	Revisjon			

STATENS VEGVESEN

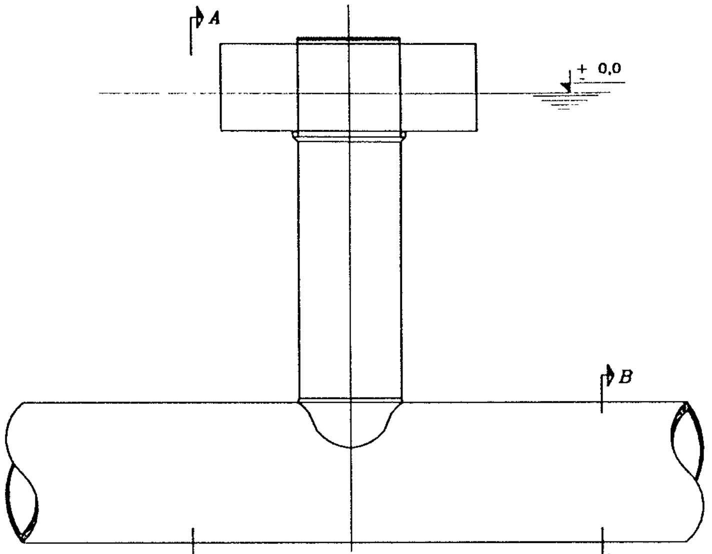
RØRBRU HØGSFJORDEN - KONSEPT 94

PLAN OG OPPRISS

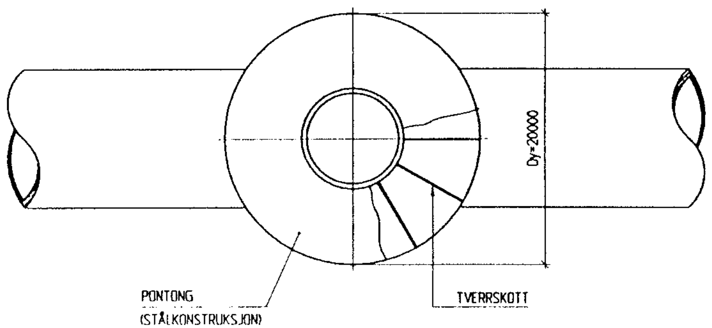
Skala 1:2500

Selmer A.S

94075-002 A

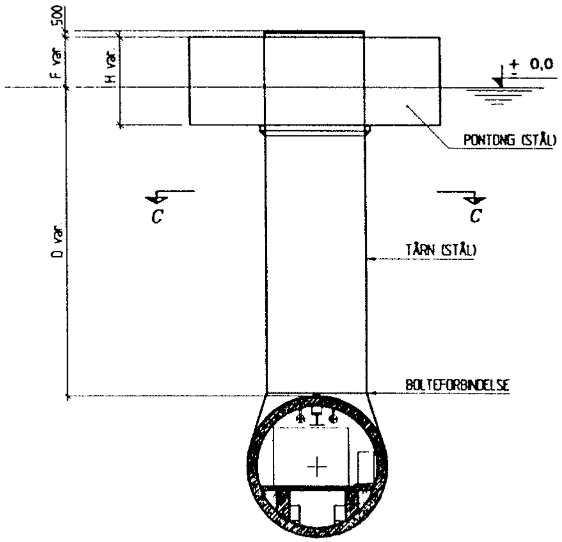


OPPRISS TÅRN M. PONTONG
(DEL AV FERDIG BRU)
1:200



PLAN
(DEL AV FERDIG BRU)
1:200

METER
0 2 4 6 8 10 20 30



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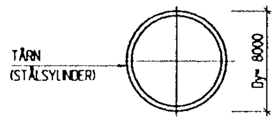
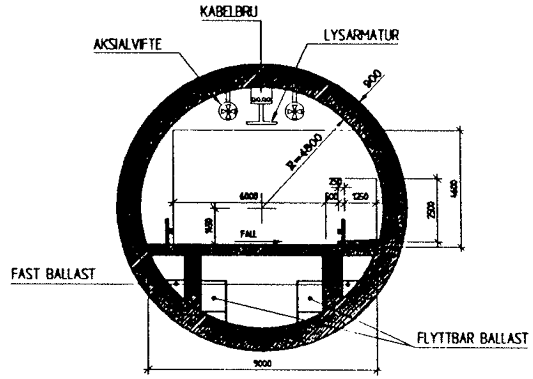
1:200



SNITT C–C
1:200

TÅRN I AKSE	3 og 8	4 og 7	5 og 6
D (m)	19	23	25
H (m)	5,0	6,0	7,0
F (m)	3,0	3,0	4,0



Typisk SNITT B–B
1:100

METER
0 1 2 3 4 5 10 15

FIG. 3

A	15.12.94	KOSTNADSOVERSLAG	Ter.	T.L.	T.E.

STATENS VEGVESEN

RØRBRU HØGSFJORDEN - KONSEPT 94

TÅRN/ RØR-GEOMETRI

1:100
1:200

Selmer A.S

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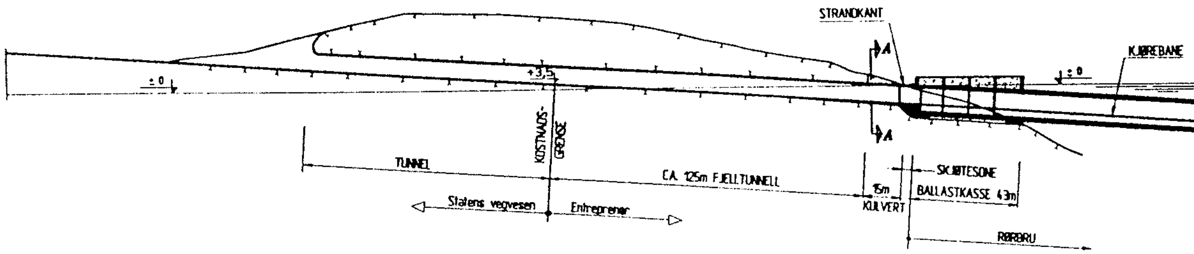
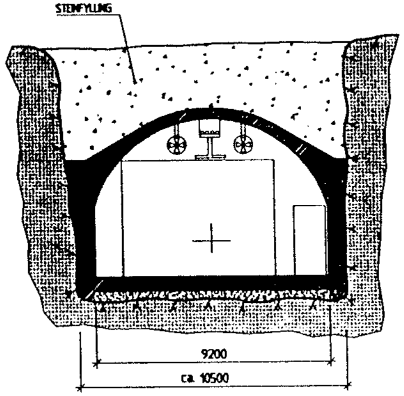
K-7520



LENGDESNITT, LANDFESTE
1:1000



SNITT A–A
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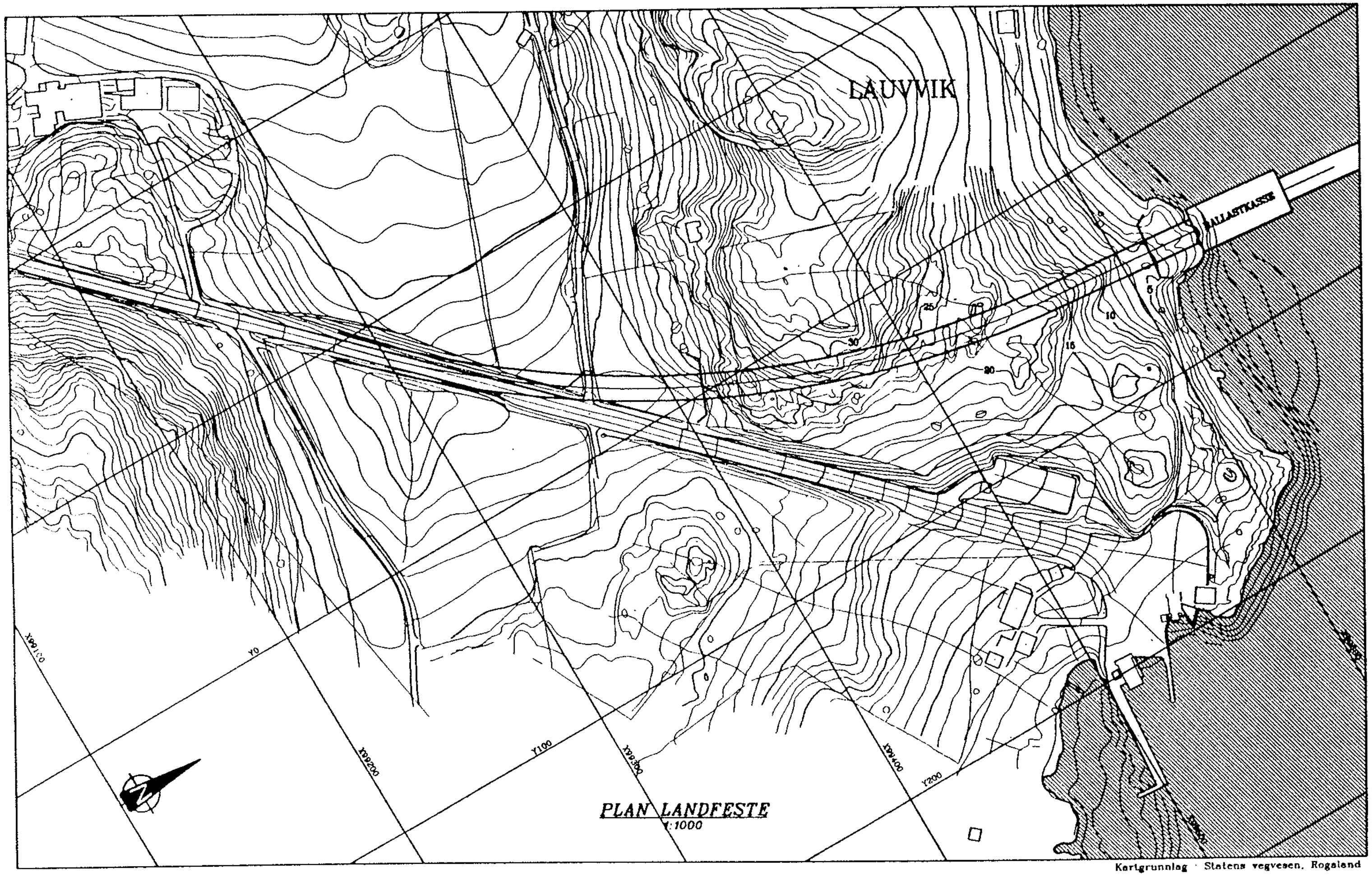

Kartgrunnlag : Statens vegvesen, Rogaland

FIG. 4

A	15.12.94	KOSTNADSOVERSLAG	Ter.	T.L.	T.E.

STATENS VEGVESEN

RØRBRU HØGSFJORDEN - KONSEPT 94

LANDFESTE VED LAUVVIK

1:1000

Selmer A.S

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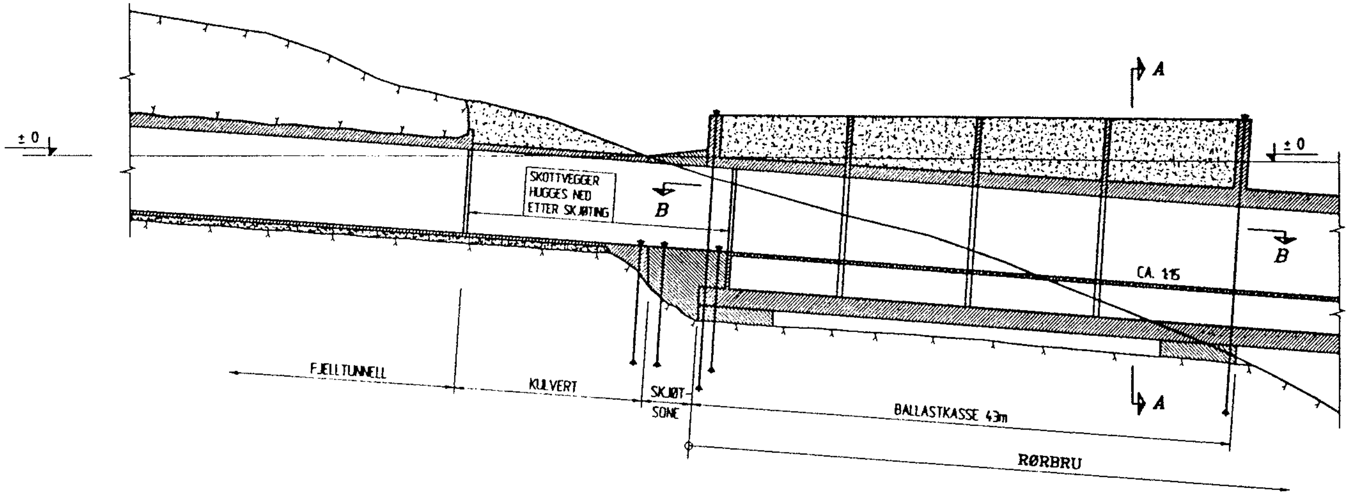
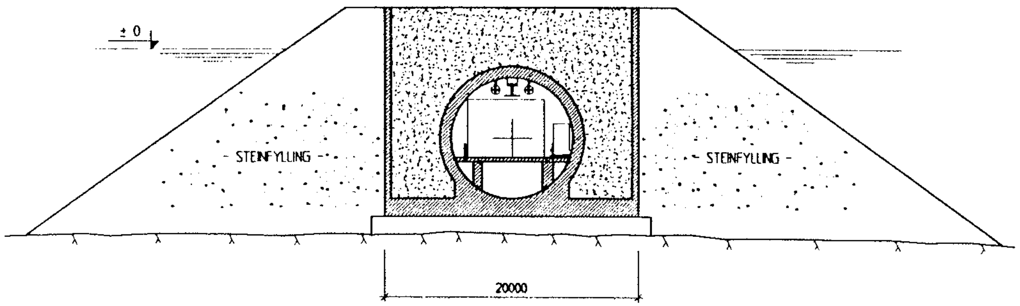


DETALJ LANDFESTE
1:200



SNITT A–A
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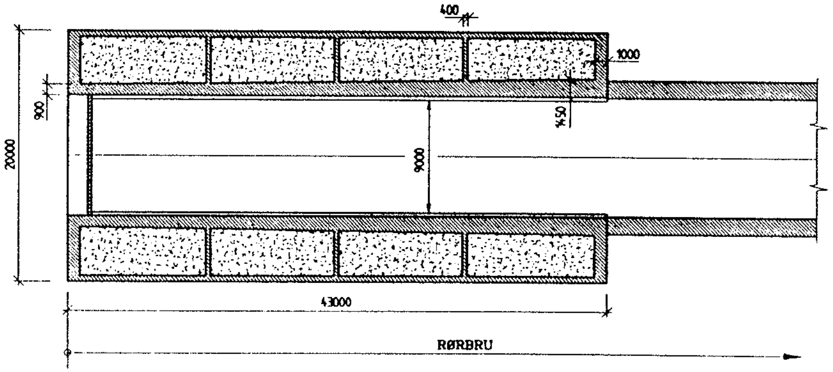


HOR.SNITT B–B, BALLASTKASSE
1:200

FIG. 5

A	15.12.94	KOSTNADSOVERSLAG	Ter.	T.L.	T.E

STATENS VEGVESEN

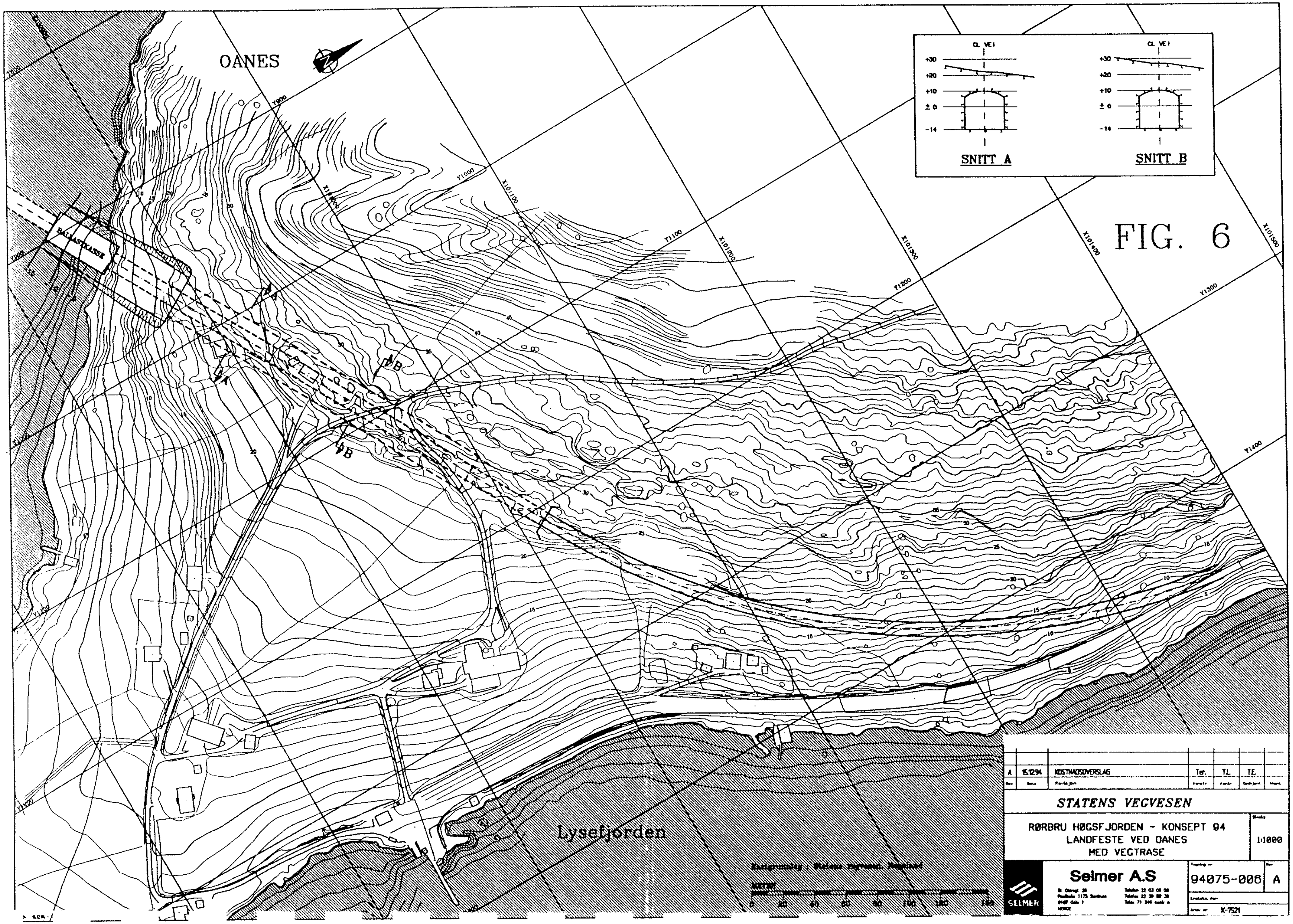
RØRBRU HØGSFJORDEN - KONSEPT 94
BALLASTKASSE VED LAUVVIK
DETALJER OG SNITT

1:200

Selmer A.S

94075-004 A

K-7528


OANES
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+30
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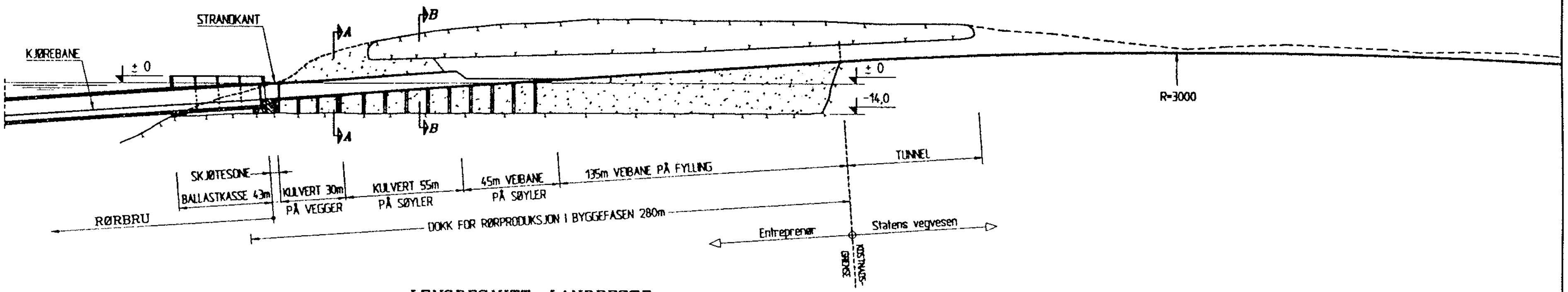
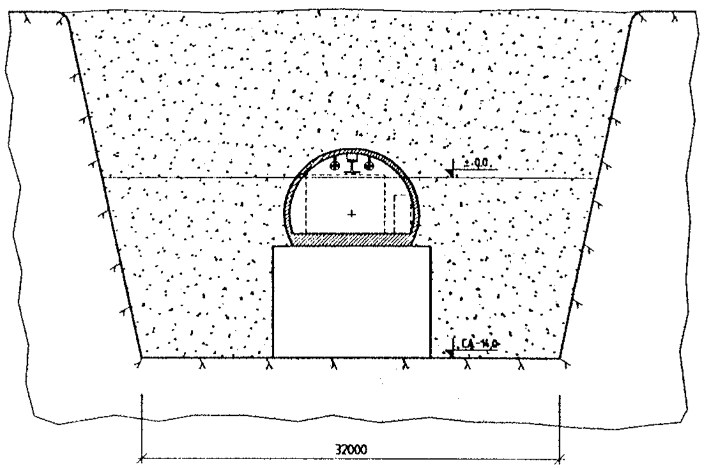
-14
SNITT A
SNITT B
Lysefjorden
A
B
A 15.12.94 KOSTNADSOVERSLAG
STATENS VEGVESEN
RØRBRU HØGSFJORDEN - KONSEPT 94
LANDFESTE VED OANES
MED VEGTRASE
1:1000
Selmer A.S
94075-006
A
K-7521

FIG. 6



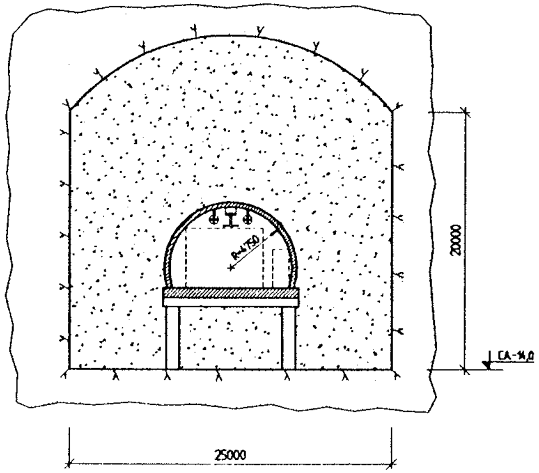
LENGDESNITT, LANDFESTE
1:1000

METER 0 20 40 60 80 100 120 160



SNITT A–A
1:200

METER 0 5 10 15 20 25 30



SNITT B–B
1:200

FIG. 7

A	15.12.94	KOSTNADSOVERSLAG	Ter.	T.L.	T.E.

STATENS VEGVESEN

RØRBRU HØGSFJORDEN – KONSEPT 94
LANDFESTE VED OANES
SNITT

1:1000
1:200

Selmer A.S

94075-007 A

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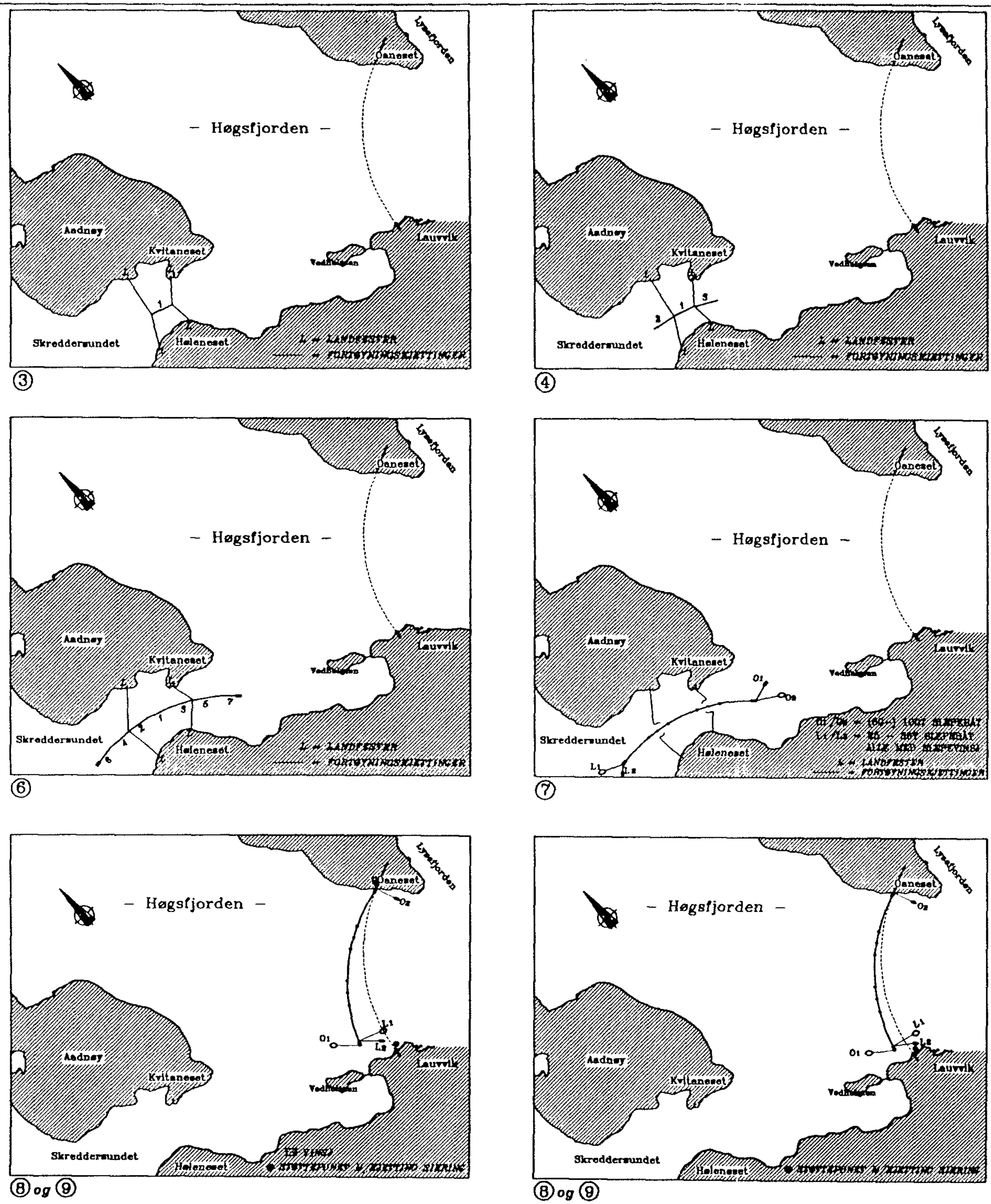


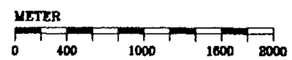


FIG. 8

A	15.12.94	KOSTNADSOVERSLAG	Tegn.	T.L.	T.E.

STATENS VEGVESEN

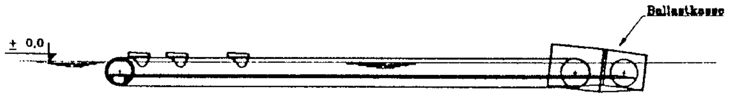
RØRBRU HØGSFJORDEN - KONSEPT 94
BYGGEPROSESSEN
SAMMENKOBLING RØRSEKSJONER
BUKSERING OG INSTALLASJON

1:30000

Selmer A.S

94075-011 | A

K-7523



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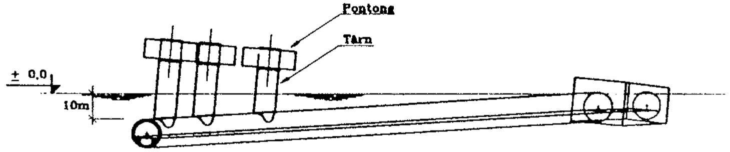
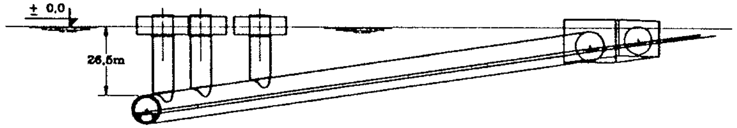
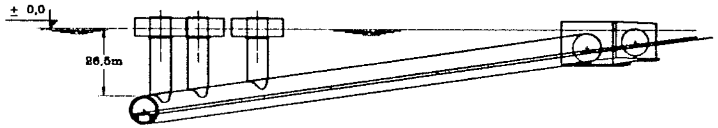


Snitt A - fase ⑦
1:1000

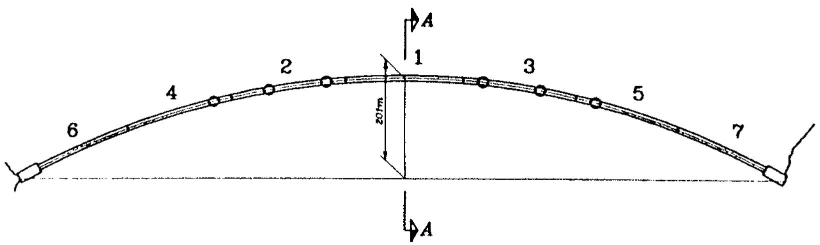


Snitt A - fase ⑧
1:1000

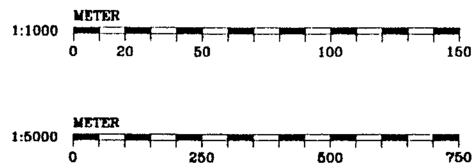


Snitt A - fase ⑨/⑩
1:1000

FIG. 9



PLAN RØRBRU MED RØRSEKSJONER
1:6000



A 15.12.94 KOSTNADSOVERSLAG Ter. TL TE

STATENS VEGVESEN

RØRBRU HØGSFJORDEN - KONSEPT 94
BYGGEPROSESSEN
SAMMENKOBLET RØRBRU/ FLYTESTILLINGER

1:1000
1:5000

Selmer A.S

94075-012 A

K-7561

F. SELMER

SHORE APPROACH

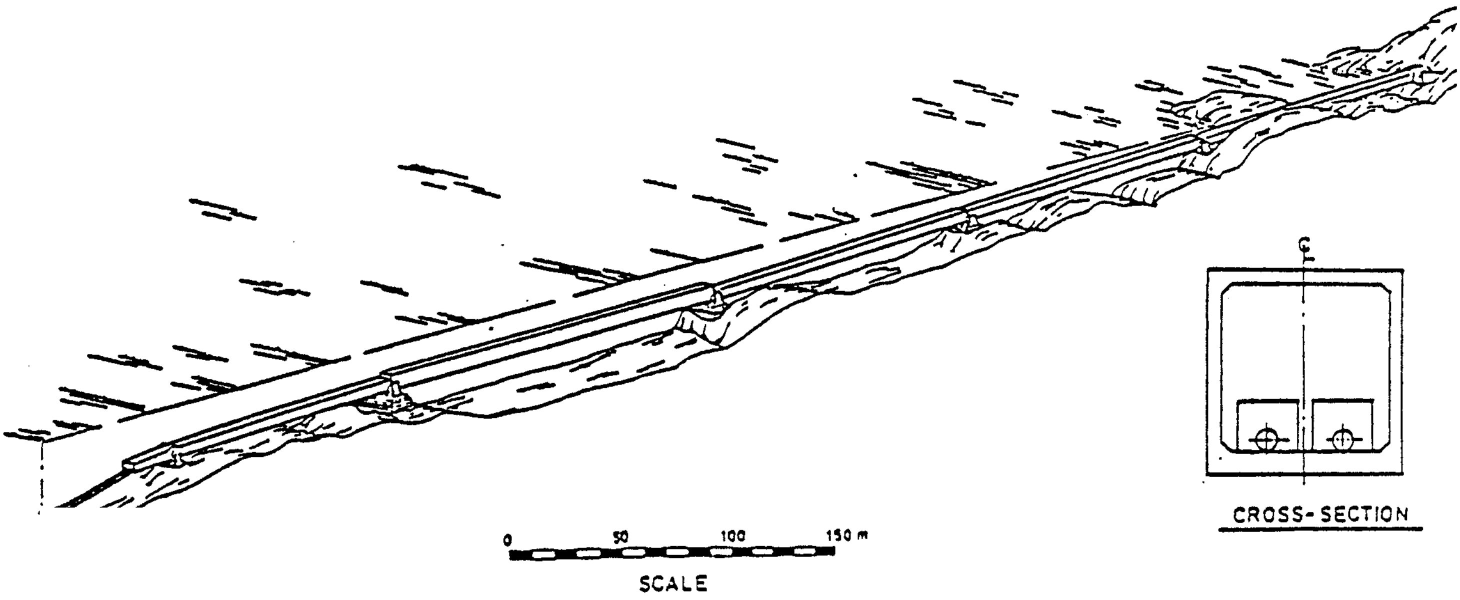


Fig. 10

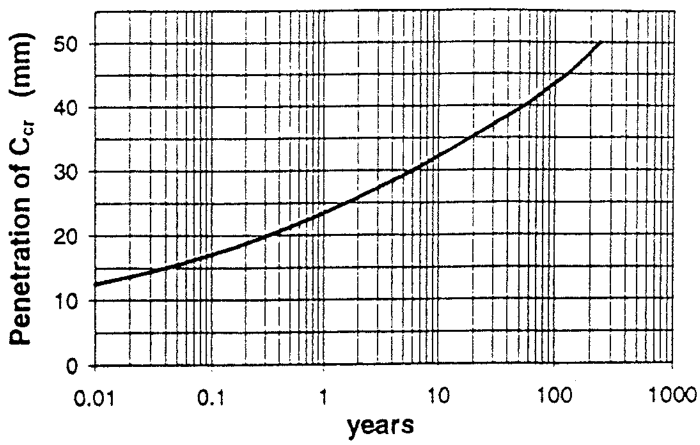


Fig 11 "Shore Approach" - Penetration of the threshold chloride content, C_{cr} = 0.06% of concrete, into the structure

Fig. 11

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

7. European Union - SFT Analysis Project

G. Ferro

INTERNATIONAL CONFERENCE ON SUBMERGED FLOATING TUNNELS

PREPRINT

EUROPEAN UNION SFT ANALYSIS PROJECT

by
Giovanni Ferro
(Italy)

Sandnes, Norway
May 29-30, 1996

EUROPEAN UNION
SFT ANALYSIS PROJECT

by
Giovanni Ferro
(Project Manager - Italy)

1 - INTRODUCTION

All the activities in the area of research and technological development by the European Commission, for the period 1994-1998, are being developing according to the Forth Framework Programme, adopted by the European Parliament and Council on 26 April 1994. In this framework, the Council, on 1 December 1994, adopted a specific research, technological development and demonstration programme in the field of transport, for the period 1994-98.

This programme is targeted to support the development and implementation of a common transport policy, with the general aim of achieving "efficient and cost-effective transport networks for goods and passengers under the best possible environmental, social and energy consumption conditions". Among the more specific aims of the programme the following items are included:

- "to develop more efficient, safer and environmentally friendly transport systems for passengers and goods";
- "to promote the design and management of infrastructures with a view to reducing the damage to the environment and improving the quality/price ratio".

Additionally, "the general objective of the research is to arrive at prenormative or prelegislative conclusions making it possible to incorporate into the transport sphere new policy options and facilitate the implementation of new generic technologies".

Under such premises the Danish Road Institute, on the behalf of the Forum of European National Highway Research Laboratories (FEHRL), submitted to the European Commission the application for a grant to support an analysis on Submerged Floating Tunnels (SFT), to be developed jointly with the Norwegian Road Research Laboratory and the Italian Company Ponte di

Archimede S.p.a., with a view:

- to produce reliable documentation about SFT as a concept, with special emphasis on safety, economy and environmental impact;
- to identify and describe procedures involved in planning, design, construction and operation of SFT;
- to identify and describe future research project related to SFT, including cost estimate and time schedule.

The Analysis Project was approved by the Commission and the grant awarded, to cover half of the project budget, the remaining being covered by the three Project partners. The contract was signed in March 1996 and the Project will be completed by October 1996.

This paper briefly illustrates the Project development and findings, up to now. To this aim, it is organized as follows:

- Project organization (Section 2);
- preliminary Project outcomes (Section 3);
- foreseen future research development (Section 4).

There is no need to stress that, as the Project being in progress, all the results and recommendations presented in the following are, by far, incomplete and preliminary, subjected to revision and addition, as the Project progresses.

2 - PROJECT ORGANIZATION

The Project organization was set up by the three partners:

- Danish Road Institute;
- Norwegian Road Research Laboratory;
- Ponte di Archimede S.p.a.;

according to some basic concepts, they intended to characterize the Project, such as:

- each topic should be addressed from a really multinational point of view, so enabling a deep exchange among different experiences, which turned out to be extremely fruitful;
- the Project should include a balanced mix of high level scientific know-how and practical experience on technological issues and application problems;

- the Project should benefit from the contributions from different engineering fields, such as transportation engineering, offshore engineering, naval architecture, etc..

On such bases, the Project organization sketched in Figure 1 was set up, which implies that:

- the Project is coordinated, at a general level, by a Steering Committee, where the three partners are represented, and which is in charge of: fixing the basic lines of the Project, taking care of the relationships with EU, defining the Project budget, approving intermediate and final documents;
- a Project Manager, reporting to the Steering Committee, was put in charge of the Project activities, with the specific duties of coordinating the Working Groups and producing intermediate and final Project documents;
- six Working Groups were established, each one composed by four to seven members, from the three countries, in charge of carrying out the Project work in each specific field.

The Working Groups (WG) deal with the following topics, respectively:

- WG1: basis of the SFT technology, state of the art, review the existing projects;
- WG2: SFT merit, potential applications, environmental impact, cost estimates;
- WG3: rules and regulations for SFT design, construction, operation and maintenance;
- WG4: dynamic actions, responses and interaction;
- WG5: anchoring, connections and construction;
- WG6: safety, vulnerability, instrumentation, operation, maintenance and repair.

WG members were carefully selected, in order to incorporate the best available expertise not only by the three partners, but also from universities, research institutes, regulatory bodies, contractors and consultants.

In order to integrate the WG work and avoid overlap or missing items, a general mid-term Project meeting was fixed, where the intermediate results by all WG has been extensively discussed, collectively by most of the WG members. The contents of the subsequent sections reflect the outcomes of this meeting.

3 - PROJECT OUTCOMES

In order to fulfill the targets set in the application for the EU grant, the following specific aims have been identified for the Project:

- discuss the merit of the SFT technology, identifying main advantages and potentially critical issues and make a review of possible applications in the EU countries;
- evaluate the state of the art of rules and regulations in the EU, applicable for design, construction and operation of SFT;
- review the procedures involved in design, construction and operation of SFT, identifying the most critical items, requiring further research efforts;
- suggest a program of research and development activities, suitable to foster SFT applications in the EU and to improve the competitiveness of the European industry in this market.

The most relevant (intermediate) Project outcomes for the first three above mentioned aims are briefly presented in the following subsections, while research and development issues are preliminary discussed in Section 4.

The final results of the Project will be presented in a report, which will be submitted to EU, and is expected to be an update reference document on SFT technology.

<u>SFT Merit</u>

SFT merit should be evaluated with respect to its possible alternatives, which basically are: bridges (supported, when the water depth is low, or suspended, cable stayed, floating, etc.), immersed (or underground) tunnels, and 'circum-water' roads and rails.

With respect to any above-ground solution, the main advantage of SFT is related to the environmental impact, which is, almost in all cases, much less. Reduced environmental impact is mainly related to the conservation of the landscape and the segregation of the traffic, with obvious reduction of noise, air pollution (vehicle emissions, being collected, can be easily subjected to further clean-up treatment) and other undesired effects. Additionally, SFT do not pose any constraint to the use of the ground and water surface, where most of the human, economic and biological activities occur. In a medium term perspective, when the SFT technology matures (as discussed later), SFT will be also less costly than bridges, in many cases, especially as the length increases (the cost of bridges increases approximately with the cube of the span, while SFT cost is expected to increase linearly with the length). As a further consideration, SFT provide better serviceability than bridges, since less subjected to atmospheric limitations.

With respect to immersed and underground tunnels, which share with SFT most of the above mentioned features, the main advantage of SFT is economic, not only for a minor construction cost (as the water depth exceeds a few dozens of meters), but also for a significantly less energy consumption by traffic, as the total difference in depth, as well as the total length, are much lower in crossing SFT than immersed or underground tunnels, which are located at greater depth.

Obviously, the Project team does not think that SFT is a kind of 'miraculous technology', appropriate to replace, in all cases, any other water-crossing solution. However, it can be a really competitive option, in many cases, appropriate to cover, in the future, a significant share of water-crossing connections. SFT technology appears to be extremely competitive, in particular, when:

- water depth is larger than 50 meters and up to several hundreds meters (the upper limit can increase as the technology progresses);
- the crossings is greater than one thousand meters;
- sea currents are reasonably limited (at least in the first applications).

On the other hand, some critical issues exist, which require particular care in considering SFT applications, even when this technology matures. One of the most critical issue is the interaction with submarine traffic. In principle, SFT could even be designed to withstand submarine impacts and/or be protected against them; as a matter of the fact, SFT should not become a constraint to the submarine operations by his own Navy or allied forces, especially in war conditions. Therefore SFT crossings should be carefully considered in area with potential submarine operations, for which final decision should incorporate proper consideration of military needs. Additionally, special care should be used in shallow water, when the SFT depth is a large fraction of the water depth, so that SFT construction can cause modifications to the environment. In the short term, until SFT crossings become widespread practice, particular care should be devoted to public acceptance, with respect to the psychological attitude of the users, who are not used to travel surrounded by the water, and so can feel not comfortable in.

As far as the review of possible SFT locations in the EU is concerned, a questionnaire has been submitted to all FEHRL members; most of the answers have not been arrived yet. The results of the inquiry will be presented in the final report.

Rules and Regulations

SFT are not covered, at present, by any kind of official provisions; it means that their procedures for design, construction and operation are not supported by given criteria, which are assessed at international level and, therefore, codified in a general way.

SFT projects recently carried out (Hogsfjord, Messina Straits) have been based on the combined application of existing regulations for onshore (mainly tunnels and bridges) and offshore structures, supplemented by a set of guidelines, developed ad hoc. However, special care should be put in extending the application of existing regulations to SFT, since design bases can be quite different (for instance: onshore structures do not experience significant degradation phenomena, such as marine growth and corrosion, as occurring in a metal structure surrounded by water; while offshore structures are typically designed for a much shorter lifetime, such as 15-25 years, versus 50-100 years required for SFT).

Ad hoc guidelines, developed for specific projects (by Norwegian Public Road Administration for Hogsfjord and by Italian Register of Shipping for the Messina Straits), tried to overcome these problems, appropriately defining a set of design criteria. However, they are, on one hand, site specific, and, on the other hand, mostly limited to general principles, still far from a codified design procedure, as, for instance, typical of Eurocodes.

The development of an appropriate set of regulations, codified at international level, according to generally accepted standards, appears to be a key issue for the expansion of SFT technology. This turns out to be even more important, if one considers the problems related to the psychological attitude of the potential users, who need to really trust SFT safety criteria, in order to give general acceptance to the technology.

Design, Construction and Operation Procedures

When discussing about design, construction and operation procedures, the Projects refers to the ones peculiar to SFT technology, many others being basically analogous to those well established in other applications (mainly for immersed tunnels). However, special care is required in extending procedures from other types of structures, both for the reasons already illustrated above, with reference to regulations, and for the specific operational and economic constraints SFT technology can impose (some examples will be presented in this subsection).

At least seven areas imply procedures peculiar, to a major or minor extent, to SFT technology. They are:

- hydrodynamic behavior;
- seismic behavior;
- tethering and anchoring;
- shore connections;
- construction methods;
- equipment and instrumentation;
- safety and vulnerability.

A comprehensive review of these procedures will be presented in the final report. In this paper, just some issues will be mentioned, which either typically arose from the multinational nature of the Project, or are especially relevant for the foreseen research and development activities.

As most of the hydrodynamic background for SFT analyses comes from the offshore engineering, some features, specific for internal waters, can be not well modelled. For instance, hydrodynamic response to very long period water level variations in lakes (seiches) may require further investigations, as well as wave spectra for not fully developed conditions (typical of coastal regions and internal waters, to the extent wave loads can still be relevant for SFT design in these locations).

Seismic loads play a crucial role for SFT conceptual design, so SFT structural schemes may be quite different for high seismicity regions (such as most of Southern Europe) with respect to low seismicity regions (such as most of Northern Europe). In seismic areas, high transverse stiffness is of major importance and it can become the basic design constraint. In almost aseismic areas, especially if meteo-marine conditions are not challenging, transverse stiffness is much less relevant and it can even be provided just by the shape of the tunnel axis.

SFT tether and foundation technology has sound bases in the experiences for tension leg platforms, which have been moored to more than one thousand meters depth. However significant differences exist, such as:

- unit cost of tethers and foundations can play a much different role with respect to the economics of the entire project, as we move from some dozens of tethers and a few foundations (typical for TLP) to hundreds of tethers and foundations (typical for SFT);
- installation procedures can be quite different, when inclined tethers are used (typically for SFT, when high transverse stiffness is required), with respect to vertical tethers for TLP;
- very big cranes and barges, which are mostly used during operations for installations at sea, cannot be used as well, due to size limitations (even for ground transpot), in some internal waters, where many SFT applications are expected.

Safety against flooding, especially with reference to accidental (or voluntary) events, deserves for SFT a special attention, which is comparable only to ship design, although ships are not open to uncontrolled public access. The possible ingress of water in any traffic way not only directly threatens user safety, but also jeopardizes the entire structure, due to both uncontrolled flooding progress and loss of buoyancy. For this reason, SFT safety, especially against flooding, has been subjected to extensive investigations for all the conceived applications. However, uniform safety levels and safety criteria are still to be established, as well as an optimal balance between passive and active safety measures (including limitations to SFT use) is still to be reached. Additionally, attention to vulnerability problems (in particular suicide car bomb attacks) has been quite different in different countries, from being a major design issue in Italy to being considered a marginal risk in Norway.

4 - SUGGESTED RESEARCH AND DEVELOPMENT PROGRAMS

As it turns out from the brief discussion in the previous section, SFT technology has reached an application level, but it is still far from being established.

While no doubt can exist any more on the possibility of building a first SFT, especially in favourable environmental conditions, as the progress of the Hogsfjord project is also proving, a large number of questions still are to be answered, before SFT technology can have extensive applications. Leaving a more comprehensive discussion to the final Project report, a number of issues to be faced in the near future to improve SFT expansion are:

- development of uniform international complete regulations for design, construction, operation and maintenance of SFT;
- enhancement of technological confidence, towards applications in more adverse environment, also on the basis of the outcomes of the first SFT applications;
- improvement of design and construction procedures, in order to: optimize SFT technology, reduce costs, cope with peculiar problems, standardize SFT schemes;
- dissemination of SFT know-how, towards: decision makers, technical institutions, engineering profession, as well as the public opinion.

This implies that, rather than basic research, applied research is presently mostly needed for SFT. In this perspective, one should not forget that, at the present stage, not a single 'SFT concept' exists, but many 'SFT applications', depending on the specific features of each location.

Keeping in mind what above, the following basic research and development needs can be identified, which will be better analyzed in the final Project report:

- develop a provision package for design, construction and operation of SFT, in the framework of Eurocodes;
- assess uniform and appropriate safety and vulnerability criteria and standards;
- analyze a number of application cases, where design and construction procedures can be refined, in face of

specific problems and towards the minimum cost, as well as the above provisions and criteria can be calibrated;
- disseminate SFT technology.

In order to reach such research and development targets, a multinational effort, coordinated by the European Commission, is required, with the participation of the interested European industrial companies, so enlarging the present Project group.

Since SFT technology has reached, as discussed above, an application level, demonstration of the technology can start, in parallel with above defined research and development. Obviously, the Hogsfjord project, which presently is the most advanced application in the EU, can be used as the first demonstration. However, other alternatives are possible, such as small scale pilot projects.

5 - CONCLUSIONS

In this paper the present progress of the SFT Analysis Project has been briefly described. The Project is presently undergoing, by Danish Road Institute, Norwegian Research Road Laboratory and Ponte di Archimede S.p.a., with the financial support by the European Union.

Although many studies and specific projects have been developed in recent years with reference to SFT, this Project is the first attempt to put SFT technology in a strategic perspective in the development of EU transportation policy. Additionally, this is the first effort based on a real multinational cooperation, which has already been proved to be extremely fruitful, allowing a deep exchange of significantly different experiences.

Aside from defining an updated situation of SFT technology and applications in the EU, the Project is aimed to identify a suitable research program, to foster SFT applications in the EU and to improve the ability of the European companies to compete in this market.

The final results of the Project will be available by October 1996.

ACKNOWLEDGEMENTS

The definition of Project has really benefited from extremely fruitful comments and suggestions by many officials from DG VII of the European Commission, especially by Mr. Gonzales Finat and by Mr. Leonardi.

The Project would not had ever taken place without the enthusiastic commitment by Mr. Elio Matacena (Ponte di Archimede S.p.a.), who first conceived it, by Mr. Ivar Schacke (Danish Road Institute), who promoted it, especially to FEHRL, and by Mr. Kaare Flaate (Norwegian Road Research Institute), who is also chairing the Steering Committee. Basic support to the Project starting and set up also came from FEHRL and from the Italian Trade Office in Bruxelles.

Fruitful results of the Project came out only due to the valuable contributions by all WG members and especially by WG coordinators (Mr. Ostlid, Prof. Mazzolani, Dr. Jacobsen, Mr. Nyhus, Prof. Fiorentino).

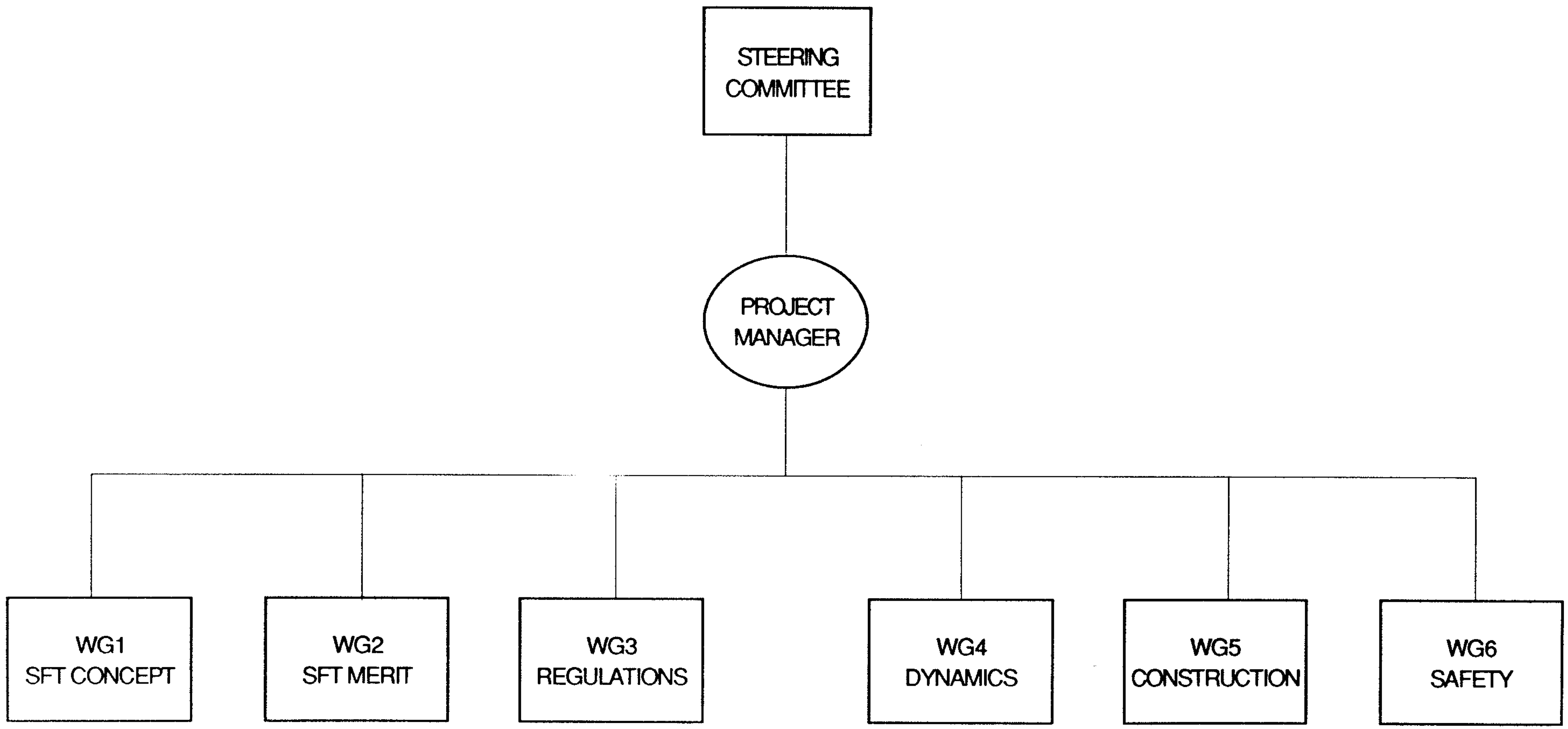


FIGURE 1: Scheme of the Project Organization

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

8. Possible sites for SFT around the world

K. Flaate
Norwegian Public Roads Administration

International Conference on Submerged Floating Tunnels, Sandnes, Norway, 29-30 May, 1996

KF/BN

Possible sites for SFT around the world

Kaare Flaate
Norwegian Public Roads Administration

This presentation of possible sites for SFT around the world is neither complete nor correct. The requirements for an SFT crossing are different from one place to another. Transport policy and financial situation are important elements. The physical environment and the available technological solutions influence the final decisions.

What I am presenting is, to a large extent the result of looking at a map. We have done this several times and many of these sites are included in the ITA report on Tunneling and Underground Space Technology, Chapter 6 on Submerged floating tunnels, dated 1993. The only thing that is certain is that the list is by no means complete.

As for the Norwegian sites, we made a new inventory only a month ago. Knowledge about possible sites is important for us in our evaluation what to do in Høgsfjord. Having several crossings in the future could influence the prosess of financing and building the first project.

The possible sites are listed on the following pages and presented on three maps. One map of the world outside Europe, one map for Europe excluding Norway, and one map for Norway. These should only be considered as examples. We all know that there are other ways to establish permanent crossing, which also have to be taken into account.

However, one thing I do expect to happen with this list of possible sites, is that over the next few years it will grow. Grow because people will become aware of this alternative crossing method and begin to see possibilities for its construction in their locality.

Statens
vegvesen

Possible SFT sites in Norway

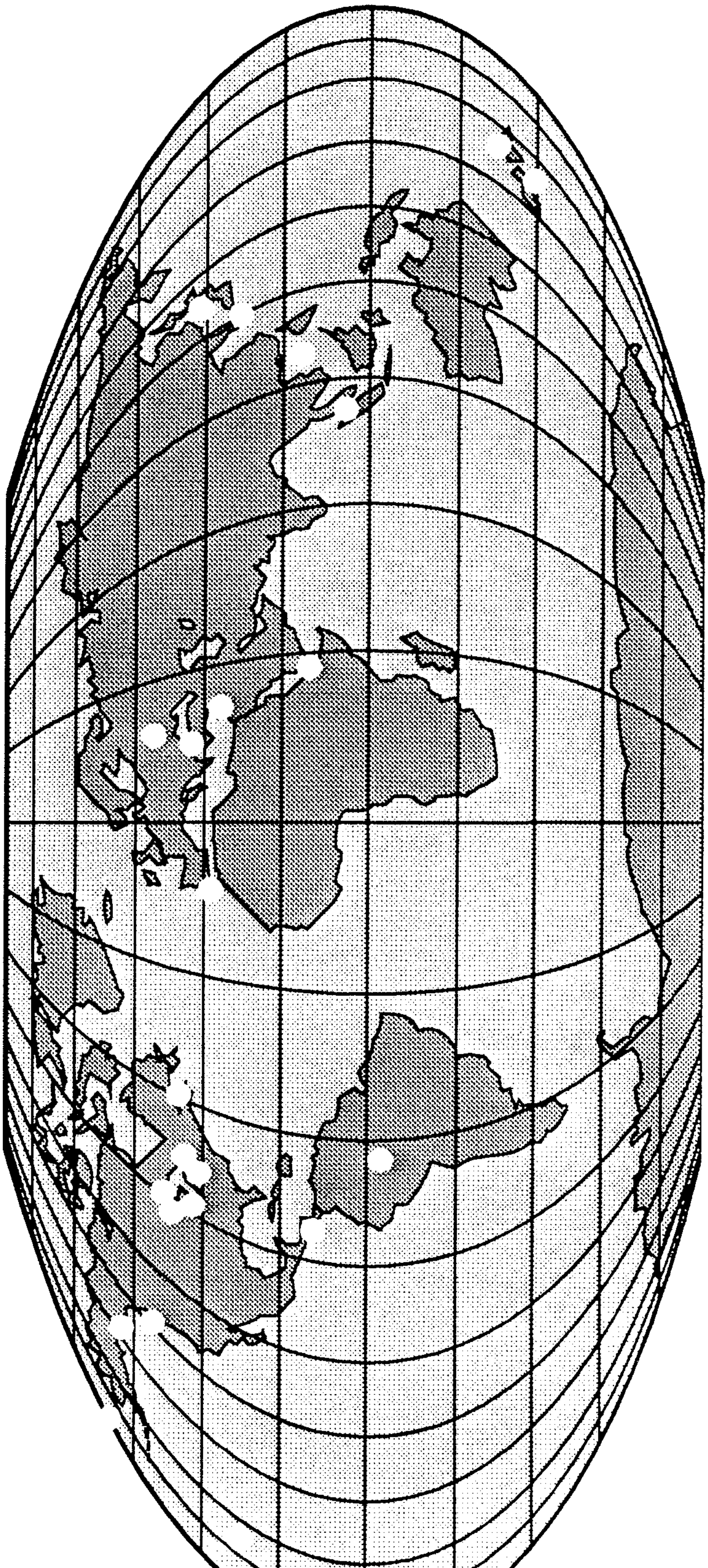
Possible locations of SFTs

Table 1: Sites in Europe

Country	Locations
France	Gironde
France, Switzerland	Geneva/Leman
Germany, Austria, Switzerland	Bodensee/Constance
Greece	Mainland to islands
Italy	Strait of Messina
Italy	Como/Lecco
Italy	Maggiore
Italy	Lugano
Italy	Iseo
Italy	Garda
Norway	Many fjords
Portugal	Rio Tejo
Spain/Morocco	Strait of Gibraltar
Sweden	Vättern
Switzerland	Neuchâtel
Switzerland	Vierwaldstettersee/Lake of Lucerne
Switzerland	Zürichsee
Turkey	Between continents; between mainland and islands

Table 2: Sites in the World

Country	Location
Alaska	Bering Strait
Canada, U.S.A.	Fjords on west coast
Canada, U.S.A.	Superior
Canada, U.S.A	Huron
Canada, U.S.A.	Erie
Canada, U.S.A.	Ontario
Nicaragua	Managua
Peru, Bolivia	Titicaca
U.S.A.	Michigan
Coast of Southeast Asia	Mainland to islands
Israel, Jordan	Dead Sea
Japan	Between islands
Japan	Biwa Ko
New Zealand	Taupo
New Zealand	Wakatipu
South China Sea	Between islands
Ukraine	Azov



International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

9. Steel tunnels - 40 years of experience

W. C. Grantz
Chesapeake Bay Bridge and Tunnel District

PRESENTATION TO BE MADE AT THE INTERNATIONAL CONFERENCE ON SUBMERGED FLOATING TUNNELS IN SANDNES, NORWAY MAY 29-30, 1996

Steel Tunnels -- 40 Years of Experience

Walter C. Grantz* -- USA

INTRODUCTION

Fig.no.

I have been a bit uncomfortable with the title of this talk as it is a little misleading. 1
Steel immersed tunnels have been around a lot longer than 40 years. I, personally, have had 40 years of experience with immersed tunnels but "entered the trade" as it were when it was already at a fairly mature stage. At that time, some thirteen rail and road tunnels and ten service tunnels had been constructed by the immersed method. I can still remember my reaction as a junior design engineer with Parsons Brinckerhoff when I was first introduced to the concept by Don Tanner who had by then designed a couple of these tunnels. I could not believe that you could build something so big and connect it underwater. Don assured me not to worry, that we would put the 23 elements of the tunnel together and meet in the middle of Hampton Roads with only inches to spare! We did it too!

WHAT TO COVER?

In considering what should be covered in this talk I found that my hope would be to capture the interest of as broad a range of engineers at this conference as I can. I decided to try the following outline:

* I decided that this presentation should not to be a dry compilation of facts and figures on tunnels built in the last four decades since such information can be readily obtained from the ITA State of the Art Report published in April 1993. I will, however, narrate some of the early history of the immersed method which you may find a bit surprising.

* It is hoped that the presentation will bring to my counterparts from other parts of the world a clearer understanding of how a typical steel shell tunnel is constructed in the United States. It should be also kept in mind that steel shell tunnels of various types have also been constructed in Hong Kong and Japan.

* The talk should relate some of the innovations and developments which have occurred in the past half century or so.

* In a number of cases I will try to point out a direct relationship to the subject at hand -- the SFT.

* Finally, I thought that it would be interesting to mention some of the rather surprising occurrences -- some near disasters -- during construction of some of these tunnels and some of the lessons learned by them. Some of these I

*Chief Engineer - Chesapeake Bay Bridge and Tunnel District
Cape Charles, Virginia, USA

Fig.no.

experienced personally and others were on other projects . To avoid distress
to anyone, I may not identify the project involved. Each case is, however,
completely factual and illustrates that "if anything can go wrong it will go
wrong." These close calls might well be kept in mind when designing and
building a SFT which is a particularly critical operation somewhat akin to
building a "house of cards."

HISTORY

First a bit of history. It is not widely known, but the concept of the immersed tunnel 2
was originally proposed by a very ingenious British Engineer in 1810 - a whole century
before the first rail immersed tunnel was to be completed in Detroit. In a competition 3
to cross the Thames, Charles Wyatt proposed to build a tunnel using 50 foot long
closed brick cylinders, sink them in a dredged trench and backfill them. Brick domes
at each end of each element would then be removed from inside to make a continuous
passage. Mr. Wyatt was wise enough to propose a shallow water test. Two 25 foot long
cylinders with an internal diameter of 9 ft. were used. The test was carried out with
great care by another fine British engineer, John Isaac Hawkins, and was considered
a success. Unfortunately costs far exceeded estimates and the project was dropped
much to the disappointment of Wyatt and Hawkins. The immersed tunnel idea did
not make its appearance again until 1893 with the construction of three sewer
ines in Boston, U.S.A. These lines were six-feet in diameter and were known as the
Shirley Gut Siphon, the Nut Island Outfall and the Deer Island Outlet.

What followed gradually led to the construction, in 1910, of the first full scale
immersedtransport tunnel. A series of single and double steel shell sewer syphons
had been constructed in various European countries including Denmark, France, and
Germany in the early 1900's. In a technique perhaps drawn from the Boston sewer . 4
tunnels combined with these experiences in Europe the two-track Detroit River Railway
Tunnel was designed as a double shell immersed tunnel. In this first method that was
used, the concrete lining was not installed until after the tunnel elements were on the
bottom. Bulkheads were provided, but the interior of the element was partially flooded
to allow it to sink. Once the element was inplace on the bottom, the exterior area was
enclosed in tremie concrete and with that completed, water was pumped out, the
bulkheads were removed and the interior was lined with concrete much as a mined 5
tunnel would be. This method was used only one other time with the construction of
the four tube Harlem River subway tunnel in New York City in 1914. 6

In between, in 1912, a single shell concrete-lined tunnel was constructed in Chicago.
This was the first tunnel to be constructed as a single steel shell and then lined with 7
concrete while in flotation. It was a 85 m. long single-element tunnel. 8

The Posey Tunnel between Oakland and Alemeda, California came next in 1928. 9
It was the first-ever vehicular immersed tunnel. It was designed as a cylindrical
concrete tunnel cast in a drydock. This was also a first. It is a bit surprising to 10
realize that the concrete immersed tunnel was born in the United States and the
shell steel tunnel concepts were probably born in Europe with those early sewer
tunnels. Apparently the State of California was satisfied with the Posey Tunnel as

Fig.no.

they constructed a virtual carbon copy 34 years later in 1962. It was called the Webster Street tunnel.

The next tunnel was to be the vehicular tunnel connecting Detroit, USA and Windsor, Canada. This was really the first tunnel in a direct chain of evolution toward the modern day steel shell tunnel. Before then, all joints between the steel plates of the shell had been riveted shiplap joints. In the Detroit Windsor Tunnel for the first time the longitudinal joints were welded. The circumferential joints were still riveted. This was not a bad decision for the time since the circumferential welds are, even now, the most difficult to make. The design utilized a special method to tie together the concrete and the steel shell of the element, a design that was followed through several subsequent projects including the Baltimore Harbor Tunnel. This was done by providing double layers of reinforcing steel tied together - one layer positioned near the interior and the other near the exterior face of the concrete. The exterior layer was threaded through angles welded to the shell. This, no doubt, was a rather awkward arrangement. In later tunnels the arrangement was eventually replaced with a single layer of reinforcing at the inside face tied back to the steel shell with stud welded "J" hooks. By this means the interior concrete and the exterior structural steel are considered to act together compositely. Ballast was installed in pockets formed between the exterior diaphragms and external wood forming. Timber bulkheads were used at the ends of the element. The Detroit-Windsor Tunnel was completed in 1930, about the time I was born. 11

Following the Detroit Windsor Tunnel came the Bankhead Tunnel in Mobile, Alabama completed in 1940 and the State Street Tunnel in Chicago completed in 1942. Bankhead followed much of the Detroit Windsor design. State Street was a dual tube subway tunnel of steel shell construction with ballast pockets. 12 13

At this point World War II had started. In the Netherlands, the Maas Tunnel was being constructed and an entirely new immersed tunnel concept was being born, a concept that has been used in Europe exclusively ever since. This was the construction of multi-lane immersed tunnels as concrete boxes constructed in casting basins as a series of completed units. This tunnel was started as war broke out. During the invasion of the Netherlands, one of the elements was nearly sunk by gunfire. This tunnel was opened to traffic twice, once during the war and the second time at the end of the occupation after removing explosives that had been planted in it. It was a groundbreaker for immersed tunnels in Europe. Dr. Nestor Rasmussen will describe the concrete immersed concept in his paper at this conference. 14

In the United States, there was a brief departure from what was becoming an "off-the-shelf" design for single tube tunnels. The Baytown Tunnel was designed as a cylindrical tunnel comprised of a single steel shell with a concrete lining. It had an unusually thick shell plate -- 25 mm. The concrete lining was configured to lower the center of gravity for launch and tow. Today a variation on this section might lend itself to a design for a SFT. 15

The Baltimore Harbor Tunnel, completed in 1957, was the first four lane steel shell immersed tunnel. The tunnel essentially melded two single tube steel tunnels together into a single welded structure. In this case the shell was protected with a thin 50 mm 16

Fig.no.

exterior layer of reinforced concrete. A concrete keel was provided and ballast was installed between the tubes.The Baltimore tunnel continued what had become standard practice for joining the elements, the tremie type of joint.

During the operation of placing the elements, they were first set on guide plates on the previous element, aligned horizontally, and pinned. At this point, steel sheet piles (or just a curved plate) was used to enclose the space between the elements. Tremie concrete then was placed into this space to form the joint. It was a method which provided an
exceptionally strong joint and which allowed considerable flexibility in horizontal and 17
vertical alignment. However, it was expensive, tended to leak water or, worse, concrete grout into the joint, and it sometimes involved blocking the navigational channel for
several days. A great advance was made when the rubber gasket was introduced, first in 18
Europe and later in the USA in 1970 with its adoption for the 57 element Cross Bay 19
(BART) Tunnel in San Francisco.

This gasket, as used in the USA, consists of double gaskets around each tube, the 20
outermost being equipped with a cantilever flap to produce an initial seal. The final 21
closure across the joint between the elements generally took the form of a welded 22
closure plate. More recently the Omega gasket commonly used in European tunnels 23
has been used for the final closure in the United States. 24

The BART Tunnel was a groundbreaking tunnel in other ways as well. It still holds . 25
the record as being the longest immersed tunnel and it was the first tunnel ever to
incorporate a triaxial siesmic joint. It used a single shell element of two tubes with . 26
a subway track in each tube. Ballast boxes were installed on top of the tunnel for the containment of stone ballast.

Thirteen years earlier in 1957 when the first Hampton Roads tunnel was constructed, it broke ground in another way. It was the first immersed tunnel to be constructed between man-made islands. This accomplishment made possible the construction of the Chesapeake Bay Bridge Tunnel with its two tunnels and 20 kilometers of trestles virtually constructed out in the Atlantic Ocean.

Development of the steel shell tunnel both single and double shell, monocular and . 27
binocular -- as the single tube and double tube versions are termed -- continued to 28
slowly evolve to the present. It is a field in which each project has its own site and operational requirements to accommodate and these are often more important to the designer in terms of schedule and budget than are efforts to change and improve constructability. Further, it is rare that an engineer has the opportunity to work on more than one or two of these projects. Consequently, innovation and progress has been slow.

Nevertheless, new technology has done a great deal to help improve things. Higher strength concrete combined with better methods of analysis such as the finite element method have increased the effeciency and reliability of the cross section. Improvements in survey equipment and data handling have assisted greatly in the three dimensional aspects of placing and aligning these huge elements. Automatic fabrication devices and computer-aided design and drafting allow great economy of scale for these seemingly

Fig.no.

complex but basically simple, repetitious steel assemblies.

My entry into the immersed tunnel field began with the design of the first Hampton Roads bridge tunnel across the entryway to the harbors of Norfolk and Newport News, Virginia. This was 1954 and in those days -- not really that long ago -- it was all drafted by hand and designed with a slide rule! I prepared a lot of the drawings and learned a great deal but never had the opportunity to work in the field on the project. I was very lucky to apprentice under Donald N. Tanner, who by that time was an old hand. Don later was one of the coauthors of the Tunnel Handbook. Don passed away in 1984 at his desk busy designing his ninth immersed tunnel. 29

I did make a couple of field trips both to the site of the tunnel and to the locomotive works near Philadelphia where the elements were being fabricated. It was during my visit to the fabrication plant that I first came face to face with one of the elements. I was astounded! I had been used to picturing the tunnel structure about 15 cm. high on the plans and suddenly it grew to a structure nearly as tall as a four-story building. This is something to keep in the back of one's mind when designing the SFT. It may look easy on the drawings but these are ***huge*** structures!

My career with PB took me away from immersed tunnels for a while and I worked on ports and harbors, bridges, and highways. I followed the construction of the Hampton Roads Bridge Tunnel, the BART Tunnel and the Second Elizabeth River Tunnel with interest but never figured I would get another chance to work on one of these projects. Then suddenly I was offered a job as Assistant Resident Engineer on the Second Hampton Roads Bridge Tunnel. Not only that, but I was to work with one of the most respected engineers in the field -- Roger B. Stevenson, who had built the BART Tunnel, the First Hampton Roads Tunnel and a number of other major projects.

The project was an order of magnitude larger than anything I had ever built. Roger and I worked very well together and I found myself involved in every aspect of the construction. I was given the responsibility of working alongside of the Contractor in the placing of the 21 elements. Out of this experience came the realization that not all the problems of building this type of tunnel had been solved. There were a lot of "old wives tales" that rightly or wrongly dictated how things had to be done. In the years that followed with Parsons Brinckerhoff, I found myself in situations where I could help introduce improvements in design and procedure.

Unlike other countries, there is generally a rather sharp division within the consulting engineering field in the USA between those who work in the office as designers and those who work in the field as construction managers. I was fortunate to be able to straddle this fence and do both design and construction. The down-side of this was that in the construction office I was considered a "dumb designer" and in the design office I was considered "just a field man!" On the other hand, I was in a position where I could look at a problem in the field, read the designer's mind, and then find a way to get the problem corrected.

An example of this was a serious problem we had in getting the Hampton Roads elements to align satisfactorily. At the time this was only our second experience

Fig.no.

with the gasketted joint that is closed by water pressure. When the joints were
closed they compressed the gaskets and made thousand ton metal to metal contact
on a steel bearing bar that ran all around the tube in between the gaskets. The
result was to fix the horizontal alignment of an element rigidly "like a flanged pipe
connection." For the HRT elements the plan aspect ratio was over 10:1. This meant
that for every error of 1 mm across the mating faces, there was 10+ mm of horizontal
alignment error at the outboard end of the element being placed. The method for
correcting this was to repressurize the joint area, pull the elements apart, send a diver
down with steel shims to place on the bearing surfaces to cause a separation of a
known amount, depressurize the joint again and hope that the horizontal alignment
would then fall within tolerance. It took hours each time this was done. The fallacy
was that because of fabrication tolerances and vertical alignment error combined with
horizontal error, one could never really know how much shimming was required. In
one case it took us many tries and two days to get an element into acceptable alignment.
From this experience we developed a concept for a system of two wide, slender wedges
with a machined taper located on the axis of the element, which could be readjusted to 30
realign the elements. This method was used on all subsequent immersed tunnels built
in the USA. Horizontal alignment within 5 mm can usually be obtained in the first 31
adjustment using this device.

Another thing we learned from the Hampton Roads project was derived from the need
to align all the internal features of the tunnel to disguise the fact that the tunnel is
composed of discrete elements. By this means, sidewalks, handrailings, and lights are all 32
made to look perfectly smooth. By the same token the roadway paving is made to be
smooth riding. In working with the as-built tunnel surveys for this alignment work, we
learned that due to residual weak clay zones under the two nearly parallel tunnels, the
first Hampton Roads Tunnel and the second, both had experienced as much as 40 cm of
differential settlement after backfilling. While some concrete cracks had appeared at the
joints there was no leakage. This was a fine testament to the ductility of the steel shell
design. Birger Schmidt and I later wrote a paper on tunnel settlements for the American
Society of Civil Engineers based on this experience. Nestor Rasmussen subsequently
wrote a similar paper concerning the settlements of concrete tunnels.

After a couple of intervening projects I was assigned to the Fort McHenry Tunnel Project. 33
This was a two kilometer eight-lane tunnel constructed as two separate double
tube tunnels in a single trench less that three meters apart. To complicate this, the two
tunnels, in order to skirt around the Fort McHenry National Shrine (where the National
Anthem of the United States was written), were all on a long horizontal curve. 32 34
elements were required, 16 for each tunnel. The sophistication of the survey work was
unique. A screeded bedding was chosen, which has been the more commonly used 35
practice for supporting the tunnels in the USA. We had a great deal of difficulty with
siltation over the bedding. In some cases we were able to air lift or flush it out prior to 36
final placement. In others we were required to remove the element and repeat the
screeding process. There have been a number of similar problems in Europe using sand
flow methods in rivers which carry a heavy bed load of silt. I am not sure which method
would have worked best in the case of the Fort McHenry Tunnel. Both methods were
provided for in the bidding documents as alternatives.

Fig.no.

During outfitting operations, we had always had a problem and the Fort McHenry tunnel was no exception. All the elements had access hatches which were only 2.4 x 4.2 m. in size. This small opening had apparently become a standand in the industry. Since all the elements had their interior concrete installed after the end bulkheads were in place (in flotation) this meant that all the formwork, form traveler equipment and the like, had to be lowered through these small holes and assembled for use, then disassembled and lifted back out -- a very slow, labor intensive and costly operation. Later, for the Boston Harbor Tunnel, we were able to get a large opening provided at one end of each of the two tubes on each element. Seeing this, when the project was awarded, the Contractor chose to add the same openings at the other ends of the tubes at his own cost so that the formwork could flow in from one end of the element and out of the other in each tube. The method worked very well and should be incorporated in future tunnels.

One of the "old wives tales" was that the elements, once backfilled, would never afterward experience a significant thermal expansion and contraction. During the construction of the Second Hampton Roads Tunnel we discovered that we were experiencing considerable thermal movement in the joint areas, as much as 10 mm in aggregate at a single joint. Since the standard detail then in use of a welded, grouted joint was not designed to take this thermally induced stress or movement, the concrete in the joint area was cracking and tile finish was being knocked off the wall. The effect eventually diminished in magnitude, but never completely went away. To compensate for this effect, on the Fort McHenry tunnel the joint area was provided with additional reinforcing and a control joint. The cracking persisted but was not as widespread or as pronounced. During the design of the Boston Tunnel, we heard that the Third Hampton Roads Tunnel (Monitor/Merrimac) had the same problem. So, for the Boston Harbor Tunnel we incorporated a true expansion joint which in effect acted as a "bell and spigot" and used an Omega gasket connection between tubes for a final watertight barrier.

Interestingly, on both the Fort McHenry Tunnel and the Boston Harbor Tunnel, we had end elements with approximately 15 m of backfill placed over them. In each case, the geotechnical engineers had confidently predicted a maximum of 5 cm of differential settlement under these huge loads -- in actuality we got 15 cm! Again, while we saw signs of concrete cracking at some joints, the ductility of the steel shell saved the day. This is something to keep in mind for the SFT. Steel is forgiving while concrete cracks! Combined, it is not a problem, but concrete by itself could be risky.

We learned a new lesson on the Boston Project. By substituting the Omega Gasket for the steel liner plate, we lost an important feature. The feature lost was the early structural connection that had been provided by the liner plates that formerly had been welded around the interior periphery of each tube. While the inboard end of the element *being placed* has large "horns" which align the mating faces and are designed to take large vertical loads and transfer them to the tunnel already in place, the previously placed element is free to move downward if backfill is placed over it before continuity is established within the joint area by means of the concrete lining. In Boston, this meant that backfilling of previous elements could only be done following completion of the joint. This was sometimes a constraint to the critical path of the construction The next tunnel should be designed so that the horn beams act to take load in both vertical directions: up or down.

Fig.no.

STATE OF THE ART

The present state of the art design for a steel shell tunnel now normally incorporates the following basic features:

* The composite steel/concrete design whereby the element is designed as a repetition of unit sections which include one flanged diaphragm welded to the shell, a shell plate of say 8 to 10 mm in thickness. The shell plate is tied to the 37
interior concrete reinforcing layer by means of stud-welded ties. Only one layer of concrete reinforcing steel is used with the loaded bars running 38
circumferentially and temperature/shrinkage steel running longitudinally. The composite ring resisting external loads is formed this way and its strength is adjusted to external loadings by either increasing the size/thickness of the diaphragm flanges and/or decreasing the spacing of the concrete reinforcing 39.
Where very heavy fills require it, the diaphragm spacing may be reduced (usually by half to maintain the same modular repetition).

* Gasketted Joints. Rarely are tremie joints used anymore except for mid river closure joints. Practice has been to use the double gasket system which works very well and appears to provide a double barrier. It is very expensive however and in the Boston Tunnel we allowed in the specifications the use of either the double gasket system, or the single GINA gasket commonly used in Europe and other parts of the world. The latter is likely to be less costly. Also it has been my experience in many cases that the external cantilever exterior gasket of the American system was the only one of the two that was doing the job. The 40
interior gasket was seen not to be compressed at certain locations around the joint due to variations in element alignment or in the flatness of the bearing surfaces.

* The use of the alignment wedges. These have been used on the Fort McHenry Tunnel, The Second Downtown Tunnel, in Norfolk, Virginia, The Monitor-Merrimac Tunnel in Norfolk and the Ted Williams Tunnel in Boston. They allow extremely accurate adjustment of the element in an operation which takes about thirty minutes. These might be useful for the SFT if it is constructed as separate elements.

* The design of steel elements normally allows for either side or end launching. Some of these elements have been constructed in drydocks but usually they 42
are built on shipways and launched. Outfitting with interior concrete is then 43
usually done while the element is in flotation at a dock. 44

* Ballasting of double shell elemnts is usually done entirely with tremie concrete 45
fill. For the case of single shell tunnels such as the BART Cross Bay Tunnel and the 63rd Street Tunnel in New York City, ballasting has been done by placing stone or concrete slabs on top of the element.

* Placement of elements and initial and final ballast is generally done from an 46
assembly of two catamaran barges linked together with two carrying beams from

Fig.no.

which four lowering falls are operated.

* The use of guide beams for horizontal and vertical alignment of the mating faces 47
of the two elements being joined. 48

* Foundation bedding is usually done by the screeded method. While provisions 49
were recently written into several federally funded projects for sand flow foundation as an alternate, the American contractors have all opted for the method that they are used to, the screeded method. Only one tunnel in the United States has used the sand flow method in recent years. That was the Interstate 10 Tunnel in Mobile, Alabama in 1973.

Joint closure. While the steel liner welded across the joint has obvious appeal since it connects the steel shell from one element to the other it has the problems I mentioned before. We used the Omega gasket closure for the first time in the design of the Boston Tunnel. This design provided for a true free expansion/ contraction of the elements at the joint, for the main immersion gasket the doub gasket style was used mainly because of a very tight schedule and the Contractor's concern that there might be delays in getting the GINA detail approved. The GINA could have provided a substantial saving if it have been the bidder's choice. In the case of the steel shell SFT, a full-strength continuous welded connection would be the proper choice.

* Depth of cover has varied but generally consists of 1 to 2 meters of sand topped with granular fill or stone.

MURPHY'S LAW AT WORK

There is a saying about flying: "hours of boredom punctuated by moments of shear terror!". Much the same can be said about placing an immersed element.

Picture this scene. It is the wee hours of the morning, the crew in the control cab of 50
the laying barge is tired. Since previous morning they all have concentrated on moving the element into position and sinking it. Outside,only the area of the catamaran barges used for placement are floodlighted and it is dark inside the control room except for the glow of the instrument displays and the computer screen. The tube is now on the bottom. Five story high snorkles, later to be used for lowering material, are filled with water to eliminate their buoyancy. The surveyors have just started down the personnel stairway into the element to do some checks. Everything is peaceful while we await the results; some people are dozing on their arms. Suddenly word bellows on the speaker box: radioed from the surveyors, "Water is pouring into both tubes!!" We are jarred from our reverie. The water in the snorkles is gushing through what was supposed to be seal-welded hatches and running down the roadway to the other end of the element! With the tide rising and no way to add concrete ballast to the element until the next day, the increase in buoyancy of the snorkles could pick one end of the element up while the other end was being overloaded and possibly cause serious damage. With the water in the element, it could not be lifted again. The Contractor kept his head, and managed to add enough weight to the light end so that the problem was averted. The heavy leakage was later

Fig.no.

traced to stiffeners on the hatches at the bottom of the snorkles which had been stitch-welded instead of seal-welded. The error was not caught because there were a series of shop drawing changes made at the last minute!

* I know of at least three other immersed highway tunnels that have been seriously flooded during construction. The first Elizabeth River Tunnel was provided with flood gates. These were in place, tested and operable when a hurricane hit the Norfolk area. The only problem was that the Contractor had installed temporary pipes down through the area where the gates were supposed to seal. The result was that the gates could not be closed, the water overtopped the approach and ran down into the tunnel and flooded it. During the construction of the Fort McHenry Tunnel, a water main broke and partially flooded the tunnel at the low point. During the construction of the Monitor-Merrimac Tunnel a similar thing happened. Cold weather caused a water main to break. Note that none of these occurences involved a breach in the tunnel wall but nevertheless would have been a disaster if it had happened during or after construction of an SFT. I believe that, while it is likely to be impractical to design an SFT to float while entirely full of water, it is feasible to provide sufficient automatically actuated pumping capacity to handle a water main break in the tunnel. This should be considered in preparing the design and specifications. 51

* A near flooding of a tunnel occurred when a dewatering valve on a joint was knocked off by a cable. The leak was eventually stopped by a diver who drove a wood plug into the pipe. It was a nervous moment however since the bulkheads had openings in them by then.

* Another interesting occurence included an instance of poor detailing which led to an element being fabricated almost a meter too long. This required fast action since the section was the next to the last to be fabricated. The last element had to be shortened. This was not easy to do on such short notice.

* During a storm on Chesapeake Bay, a similar problem was faced after wave action caused an element to slide down the foundation some five meters after it had been fully ballasted and could not be moved back into position. Fortunately it stayed relatively straight during its uncontrolled slide so that the next element could be shortened and joined to it. Even so, there is still a visible kink in that tunnel where that happened.

* Once an element under tow into Hampton Roads broke free and went drifting out to sea. The Engineer and the Contractor argued nose to nose, "It's *your* tube." "NO, it's *your* tube!" 52

* During concrete outfitting at a dock, a water hose was inadvertently left running into a hatch over a weekend. Since the element was nearly completed, it was very low in the water and the water that ran into it sank it at the dock.

* Perhaps one of the worst events was when one of the four lowering falls broke its connection during a placement. This caused the other three connections to fail

Fig.no.

progressively allowing the element to quickly sink to the bottom. Again the ductility of the steel shell saved the day and this near disaster did not result in major damage to the shell of the element.

CONCLUSION

I hope that the foregoing has introduced you to the world of steel shell tunnels. They are a very versatile, "forgiving" alternative for immersed tunnel construction. In addition to the ductility and inherent watertightness of the steel shell, this technique has another great advantage and that is that elements can be assembled thousands of kilometers from 53
the job site and towed at shallow draft to an outfitting pier nearby to where they will be placed. This opens the fabrication to a much wider competitive market. Further, the entire process of fabrication, outfitting, and placing is a continuous steady stream and easy to schedule reliably. This results in a very efficient meshing of dredging, placing, and backfilling into a continuous operation.

In designing the SFT, the watertightness and ductility of the steel shell should be considered. Also in a structure of this sort, really an underwater suspension bridge, analysis of risk during construction is very important. Inadverent flooding from the inside is a real danger, but unlike regular immersed tunnels founded on the ground where damage would generally be cosmetic, flooding of an SFT could lead to complete collapse and loss of the entire tunnel.

Finally, I would like to leave you with a thought which occured to me and that you might consider interesting: ***All immersed tunnels are briefly SFT's while they are being lowered into position.***

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

10. Concrete tunnels - 40 years of experience

N. Rasmussen
Christiani & Nielsen AS

CHRISTIANI & NIELSEN

CONCRETE TUNNELS

40 YEARS OF EXPERIENCE

Paper read before
INTERNATIONAL CONFERENCE ON SUBMERGED FLOATING TUNNELS
in Sandnes, Norway
30 May 1996

CONTENTS

1. INTRODUCTION

Mr Chairman

Ladies and Gentlemen

I thank you for having invited me to talk to you about immersed tube concrete tunnels - and I shall do so as I see these works in the light of my 40 years of experience with design and construction of such tunnels - sometimes on the Consulting Engineers' team - but mostly having been involved on the Contractor's side and then, in particular, with the transportation, sinking and founding of such tunnels.

I have throughout my professional life worked for the Christiani & Nielsen Group, until recently Danish owned and managed from Copenhagen. Today the Group is Thai owned with the parent company operating out of Bangkok.

Because Christiani & Nielsen from its start in 1904 were among the pioneers in design and application of reinforced concrete - for marine structures in particular - the company also pioneered the use of reinforced concrete for immersed tube tunnels - I here first of all refer to the Maas Tunnel in Rotterdam which was designed and built from 1937-1943 by a joint venture of Dutch contractors, including the Christiani & Nielsen subsidiary in the Hague, the design and planning of the immersed tunnel section being carried out at the Christiani & Nielsen Head Office in Copenhagen. This tunnel was the immersed tunnel No 9 in the world, the second concrete tunnel and the very first concrete road tunnel with rectangular cross section.

The experience gained on the Maas Tunnel placed Christiani & Nielsen in a strong position the following fourty years in a relatively limited market for immersed tunnels. Sometimes we acted as consulting engineers, generally to Highway Authorities - sometimes as contractors and in that case often in joint ventures with local construction companies.

The submersion technique - or as I normally call it - the Immersed Tube Technique - is, as you already know, used for construction of two types of tunnels

Traffic tunnels and
Service tunnels

Considering the audience present and the general scope of this conference, I shall mainly deal with traffic tunnels. Some interesting and innovative techniques have, however, been used in connection with the construction of service tunnels, for which reason I shall also tuch upon some of these.

2. CONCRETE TUNNELS VERSUS STEEL TUNNELS

When you look at the existing traffic tunnels worldwide, some 106 of which 16 are under construction,

36 are steel tunnels and
70 are concrete tunnels

ie the ratio is 1 to 2. Here the concrete tunnels are defined as tunnels for which reinforced concrete provides the basic load carrying structure.

While the steel tunnels historically and traditionally have been preferred in the United States of America, concrete tunnels have been chosen in Europe and in Japan - however, with some exceptions.

In spite of having been brought up with concrete tunnels I have - from a technical point of view - an open mind to concrete tunnels versus steel tunnels, and I think that we all should have that. Although tradition and conservatism play a role when the type of tunnel is to be chosen - a cost comparison between the two alternatives often is decisive at last.

When a fixed crossing of a waterway is under consideration the first major decision to be made is whether the crossing should be a bridge or a tunnel, secondly whether a steel or a reinforced concrete solution should be chosen. In this context an evaluation of maintenance cost and life time of the fixed crossing must be made - and here it is important to recognise that a tunnel normally will be much less exposed to the environment than a bridge, burried as the tunnel is in the ground under the sea or river bottom, subject to moderate temperature variations, protected against wind and sunshine with no oxygen at the outside. On the other hand there will normally be no access to the outside of the tunnel structure, it being for inspection or maintenance works. Experience from bridges is therefore not directly applicable for tunnels.

As far as foundation methods for immersed tunnels are concerned, there has historically been a tendency to associate

steel tunnels with gravel bed foundation
and
concrete tunnels with injected sand foundation

This association is, however, not fully valid. The foundation method to be used has in each particular case to be chosen with due consideration first of all to the subsoil conditions and the degree to which the tunnel will be subject to dynamic loadings - earthquake loadings in particular.

Pile foundations have been used for a few tunnels, some five, all concrete tunnels by the way.

Common for the two types of tunnels is that the main difficulties to oversome are

(i) to excavate a tunnel trench to specifications and to keep it free of such siltation which may be detrimental to the permanent foundation until this foundation has been constructed and the tunnel has been brought to rest on it

(ii) to construct watertight and durable tunnel elements

(iii) to install the tunnel elements in the tunnel trench

(iv) to construct watertight and durable joints between the tunnel elements

(v) to construction durable foundation for the tunnel

My good professional friend, Walter Grantz this morning told you about steel tunnels. I shall now tell you a little about concrete tunnels. Both types have of course to meet certain common requirements as providing space for the traffic envelope, ventilation ducts, service ducts etc.

I shall try to concentrate on what is particular for the concrete tunnels.

3. TUNNEL TRENCH

The dredging works required in connection with the construction of an immersed tunnel may comprise:

1. Dredging of a casting or launching basin.

2. Dredging of test pit(s) in the waterway for evaluation of siltation of tunnel trench.

3. Widening of the existing navigation channel in order to provide temporary navigation channels outside the marine working area.

4. Deepening of the existing waterway in order to compensate for the reduction of the waterway cross section caused by the permanent tunnel works, and thereby avoiding changes in the hydrographical and biological condition in the waterway.

5. Dredging for adjoining in-situ built tunnel sections

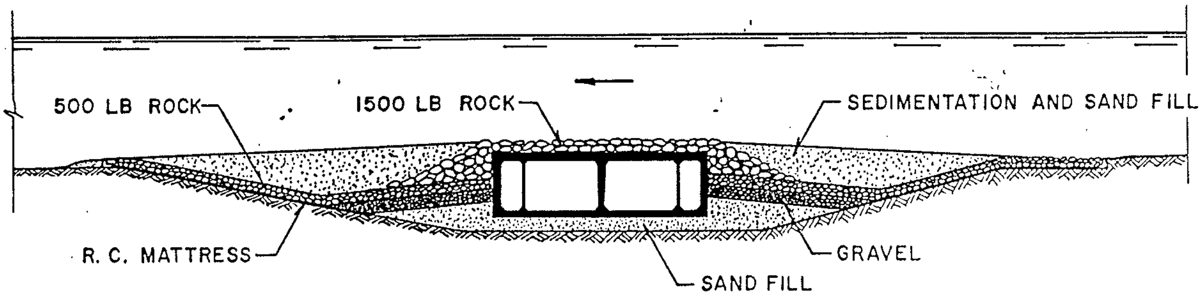
6. Dredging of the tunnel trench for the immersed tunnel section

7. Dredging of an access channel between the casting/launching basin and the tunnel trench

8. Maintenance dredging

the tunnel trench being the main dredging item on an immersed tunnel project.

(Show overhead shown below)



The volume is typically in the order of 1 million cu.m per km for a typical 4-lane motorway tunnel.

Also from a technical point of view the tunnel trench is the most important dredging item.

The purpose of the excavation is obviously to provide space for

(i) the prefabricated tunnel body

(ii) the sand or gravel foundation under the body

(iii) the protective backfill on the sides and on the top of the tunnel

As the top of the backfill has to be kept below the existing, or future, navigation channel profile, a trench bottom level at between 25 and 30 m below Low Water level is quite common. This has, at least in the past, often been beyond the reach of standard, high production dredgers, and a lengthening of the ladders has therefore been required.

Immersed tunnels in deep or open sea will of course require specially built dredgers.

Except for cases where very soft subsoil, deemed unsuitable for support of the tunnel, has to be removed and replaced by suitable materials, the general requirements to the dredging of the trench bottom are

(i) a clean, even surface, as close as possible to the upper acceptable limit in order to avoid the economic consequences of having to fill overdredged areas,

(ii) a minimum disturbance of the remaining, exposed upper soil layers in the trench bottom in order to limit the changes in the geotechnical characteristics of the subsoil.

The possible physical disturbance and softening of the exposed soil layers in the trench bottom, during the main dredging operation <u>and</u> during the maintenance period, does not affect the dredged volumes and the associated cost, but can, in particular in cohesive subsoils, have a considerable influence on the geotechnical behaviour of these soil layers later and hence on the quality of the tunnel support as a whole. This in turn influences the design of the structural tunnel body and thus eventually the overall economy.

The above-mentioned technical requirements are met by

(i) using the right type of dredger(s)

(ii) applying careful control of the position of the cutting tool, bearing in mind that the dredging normally has to be done in tidal waters and sometimes in waters subject to swell and waves

(iii) careful planning of the dredging operation in order to avoid undesirable failures of the slopes,

(iv) appropriate timing of the dredging operation in order to limit the time that the trench bottom is exposed and at the same time limit the sedimentation caused by late, nearby dredging.

4. CONSTRUCTION OF TUNNEL ELEMENTS

Casting Basin

(show overhead shown below)
T16

The tunnel elements, normally between 100 m and 150 m long, can either be prefabricated in a casting basin or in a drydock.

The element construction can take place next to the tunnel line or far away from this.

The casting basin can, depending on the availability of basins and land around the tunnel works and the construction time available, be laid-out in a number of ways.

The general requirement to the basin / yard is that it must

- provide room for preferably all tunnel elements at a time, room for access to the tunnel elements and room for bringing the element out after flooding of basin or for launching

- room for rational construction of the tunnel element and use of associated equipment

- for basins, depth which will provide a clearance of minimum 30 cm between basin floor and tunnel element soffit during float-out of tunnel elements

- for basins, be kept dry, normally by means of cut-off walls and groundwater lowering while the tunnel elements are constructed

- for basins, provide a drained bottom, which for the flooded basin limits the pressure head difference between pore water below element and outside element.

The types of casting yards / basins are

1. Existing drydocks / graving yards

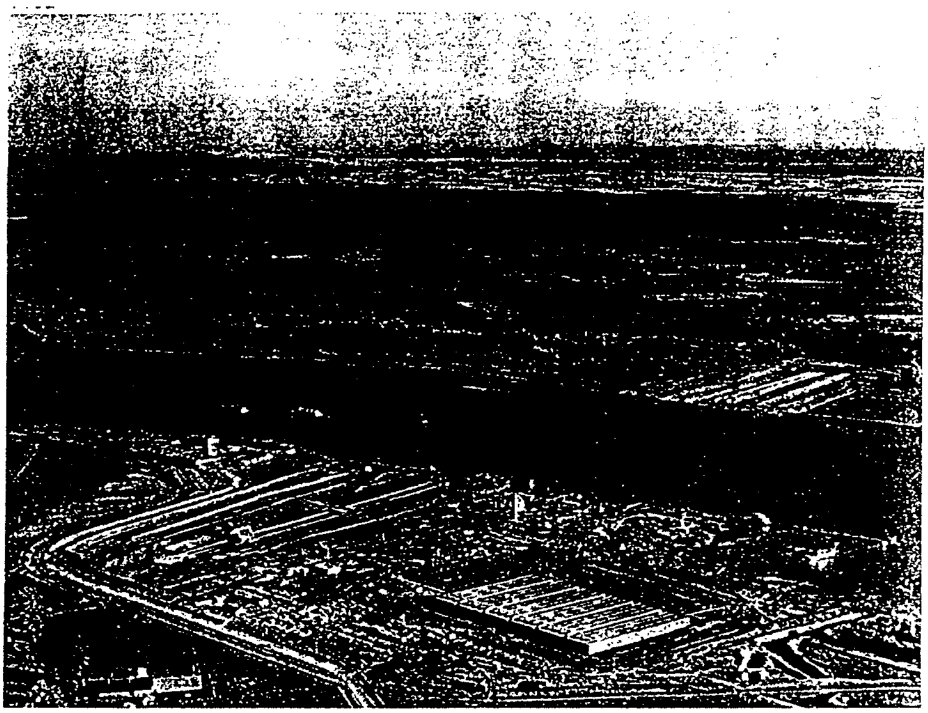
2. Closed existing harbour basins

3. Purpose-built casting basins

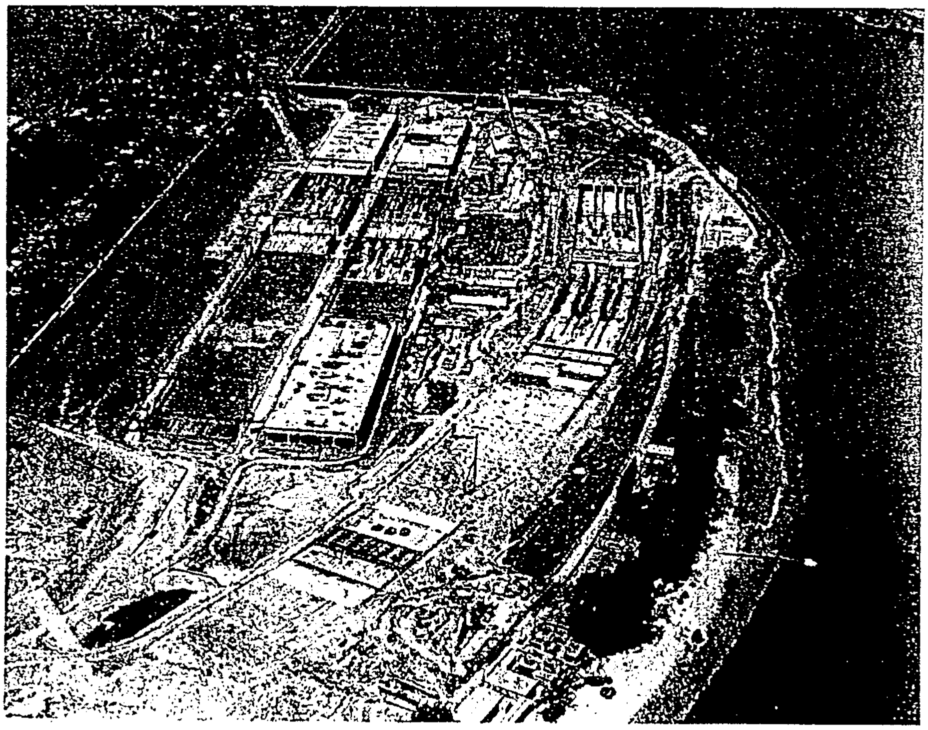
4. Approach ramps / cut-and-cover tunnel pits

5. Yard at ground level (and subsequent launching),
   for steel shell tunnels only

(show overheads on next page)



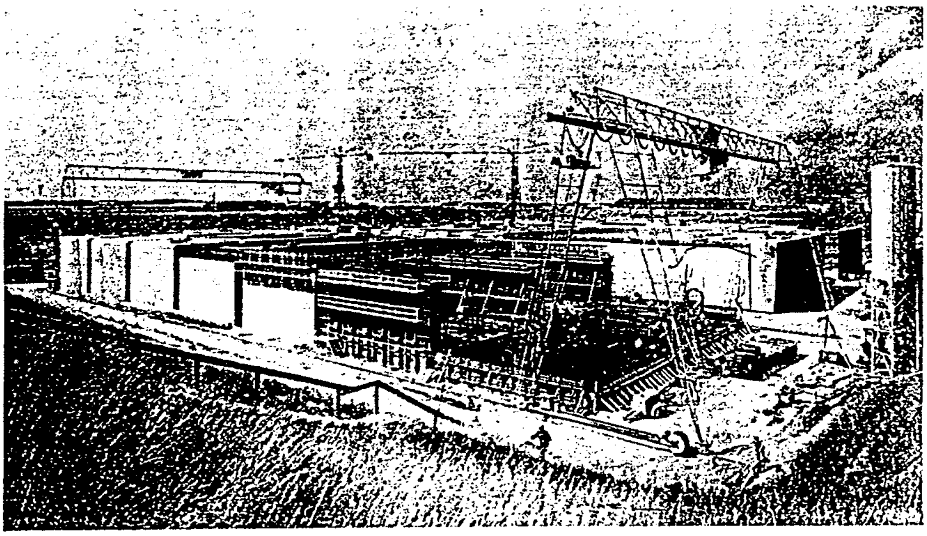
T17



T18

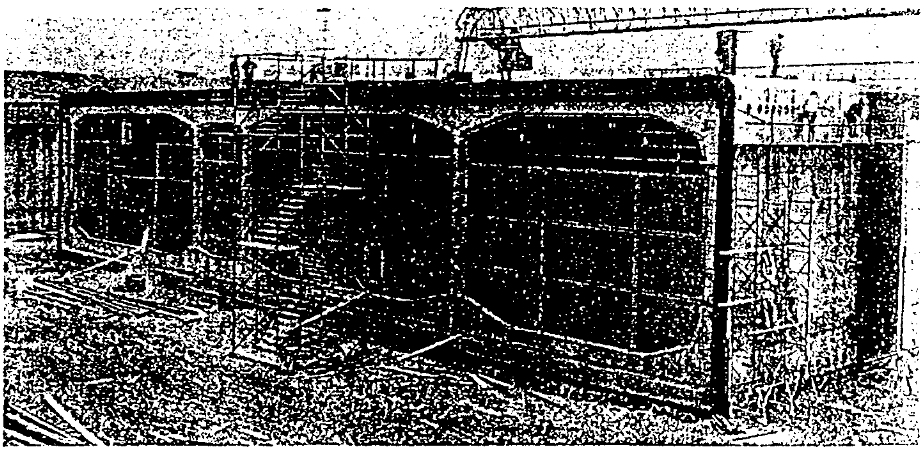
Tunnel Elements

(show overhead shown below)



The advantage of the prefabrication of the structural tunnel body, on shore, in daylight, is mainly that the fabrication takes place under favourable and well controlled conditions.

(Show overhead shown below)



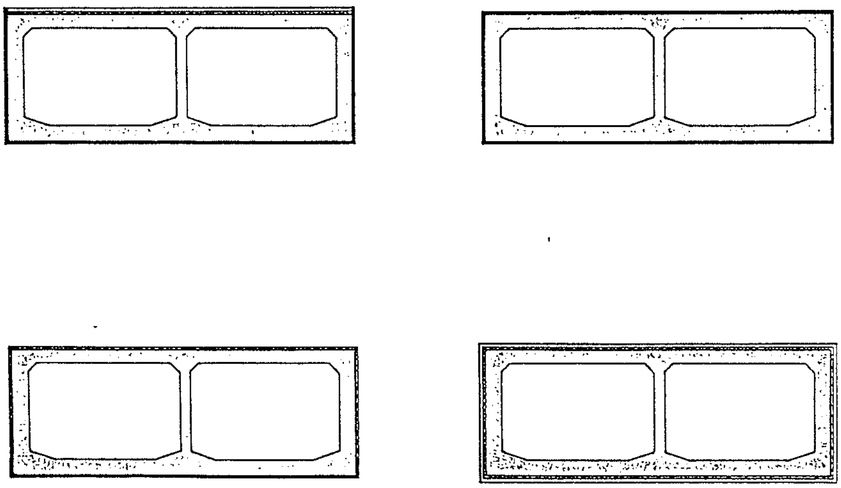
The tunnel elements are made buoyant, fully or partially, by means of temporary bulkheads installed at the element ends.

Apart from providing proper structural strength and controlling the weight of the element, the main design and construction problem for the reinforced concrete tunnel is to provide a watertight structure.

(Show overhead shown below)
T21

For many years the answer was to wrap the tunnel element in a watertight membrane. Steel on the bottom, steel on the outer walls and even steel on the roof, often, however, bituminous membrane on outer walls and roof.

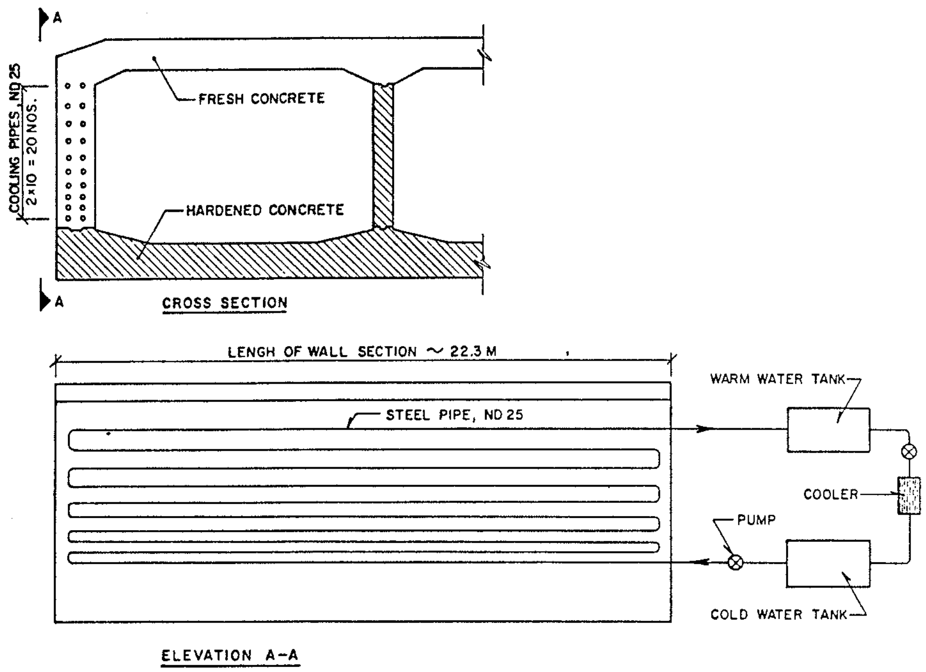
In some cases a Butyl membrane has been used throughout.



WATERTIGHT MEMBRANES

In recent years clients, mainly the Dutch Ministry of Public Works, have accepted reinforced concrete tunnels without a membrane at all, however, with some longitudinal posttensioning of the tunnel elements and sophisticated control of concrete temperature during hardening.

(Show overhead below)



In order to reduce the development of cracks during hardening, primarily in the walls when cast after the bottom slab, cooling of the lower part of the walls has, however, for many years been the practice.

Also insulation of the formwork and careful sequence of stripping of the forms are means used to control the concrete temperature.

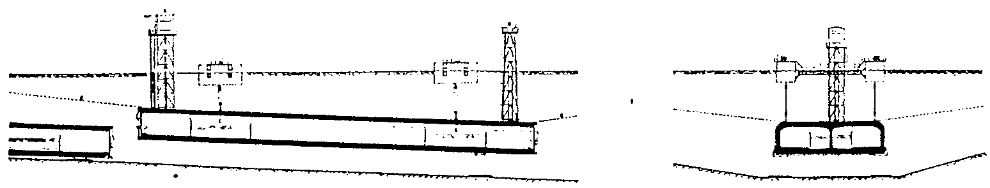
Improved field concrete technology aiming at minimising the development of cracks during hardening combined with moderate prestressing seems to be the course to follow rather than developing membranes which will remain difficult to build properly under site conditions.

For roadway and railroad tunnels, typically consisting of 5 to 8 tunnel elements, the elements will normally be cast in one batch in a casting basin. The building of the tunnel elements is relatively straight forward, however great care is required in order to meet durability requirements and avoid cracks in the concrete, and that a comprehensive programme for control of concrete density and concrete dimensions is required in order to control the weight and displacement of the tunnel elements.

The casting sequence is bottom/walls/roof, sometimes all in one go, in 15-20 m sections and the tunnel elements can be monolithic with just a water stop in the joints or be provided with flexible joints between tunnel elements, the latter to minimise longitudinal bending moments caused by compression of the subsoil in the permanent stage.

(Show overhead shown below)

Normally the tunnel elements will be buoyant for which reason they need be ballasted prior to flooding of the casting basin in order to make sure that they remain "parked" until they are to be brought to the sinking location. This ballasting is normally done with water contained in purpose built ballast tanks inside the tunnel element. Pumps and associated pipelines allow charging and removal of the ballast.



Apart from the ballast tank installation already mentioned, a number of lifting eyes and bollards will have to be provided in the element roof. Watertight, temporary bulkheads are, as already explained, installed at the ends of the element.

Rubber gaskets are mounted around the periphery of the one end of the tunnel element while a plane steel plate is provided at the opposite end. This gasket provides later, when the tunnel element is joined to the previously placed tunnel element, a watertight seal between the two tunnel elements.

As the casting basin is flooded or as the tunnel is launched from the dock, the tunnel element is checked for watertightness, the attention being directed principally towards the temporary bulkheads and pipe let-ins.

Installation of Tunnel Elements in Trench

incl. joining of tunnel elements

(show overhead on next page) T 27

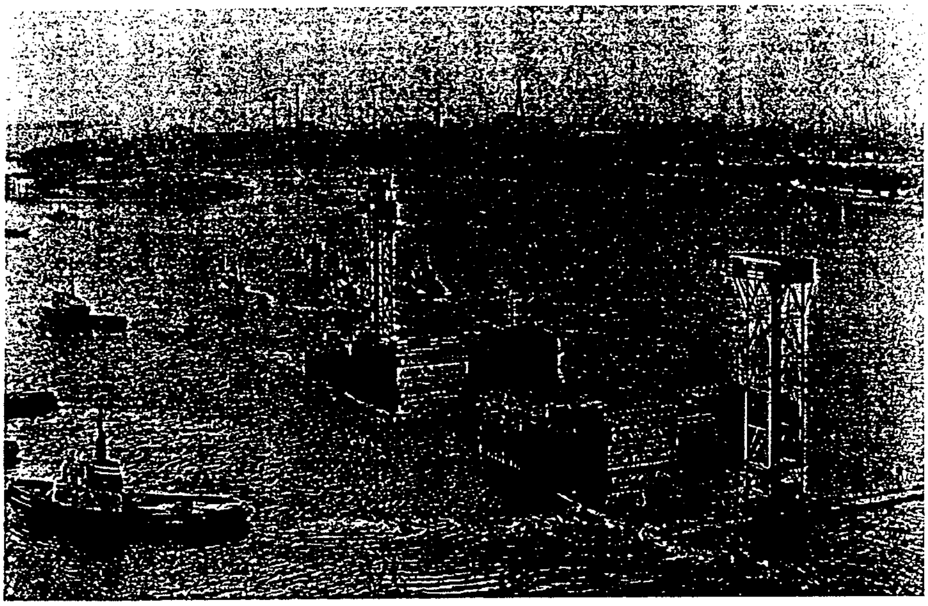
While the construction of the tunnel elements is a fairly straight forward operation, the transportation of the tunnel elements to the sinking place and the sinking and joining operations are more specialised. Furthermore tunnels are often to be transported and installed in fast running rivers and in open sea subject to wave loadings. When the operations are planned and directed properly, and staff and operators are disciplined, heavy marine construction organisations can, however, cope with these operations perfectly well.

For transportation of the element from the flooded casting basin or dock to the tunnel trench, conventional towage is normally used. Where room is insufficient for operating tugs to control the element, the tunnel elements are warped to the sinking position or to a position where tugs can take over.

In tidal rivers, the transportation and sinking will be carried out during periods with slack water but for safety reasons the mooring systems will be designed for holding the tunnel elements in position at all stages of the tide. Where the hydrological conditions are particular complex and previous experience is deemed to be insufficient, it can be necessary to carry out hydraulic model tests in order to establish the design loadings on sinking tackles, moorings and temporary supports for tunnel element when sunk.

The warping which ends up with having the tunnel element moored for sinking will normally be carried out by the contractor's organisation responsible for the subsequent sinking and joining while towing normally will be done by experienced towage companies.

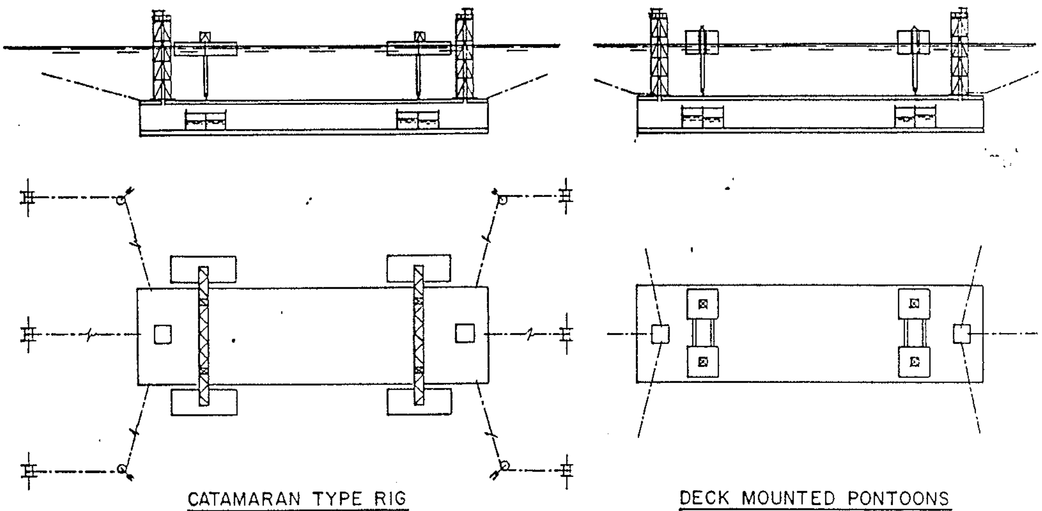
(show overhead below)



(show overhead below) T 24

The equipment and ancillary works required for the sinking are basically

- sinking rigs
- alignment / survey towers and access shafts, mounted at each end of the tunnel element
- mooring systems incl anchors
- water ballasting systems in tunnel elements, as mentioned before
- temporary support systems in tunnel elements and on trench bottom



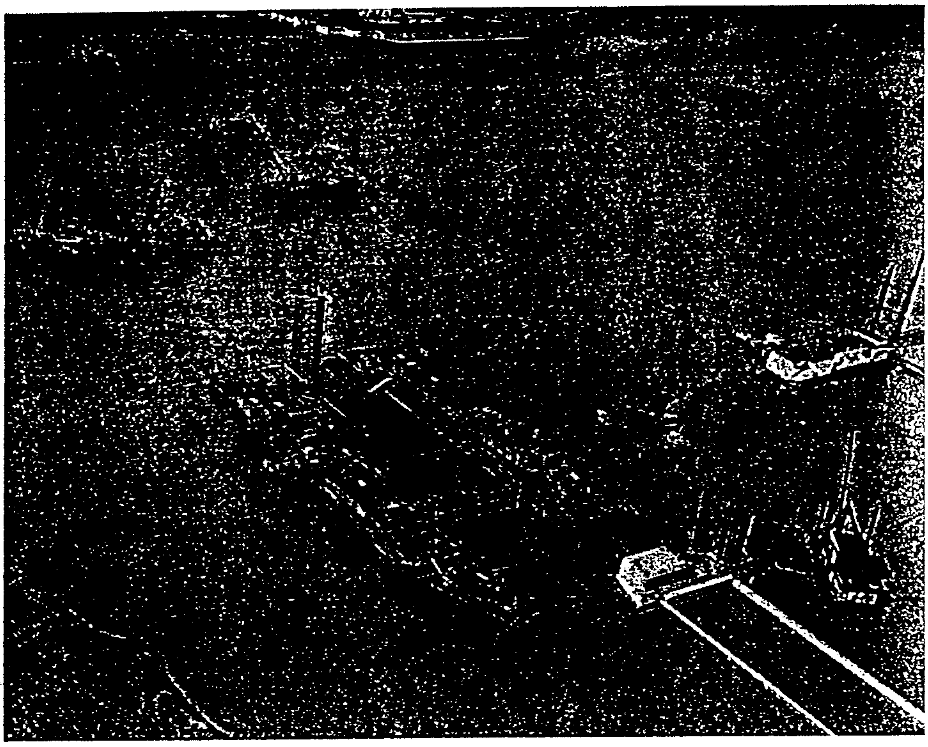
SINKING RIGS

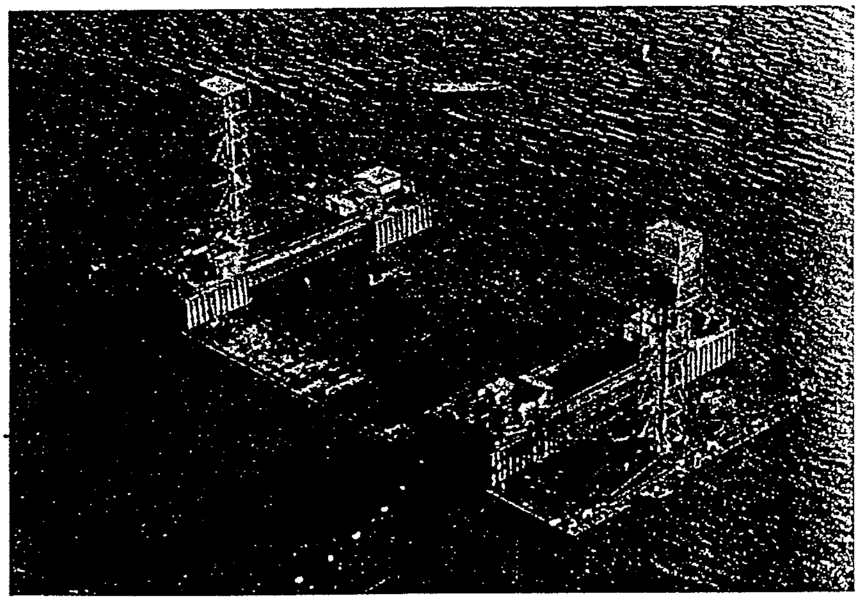
Looking at the overhead:

The Catamaran Type Rig is the most universally used but it requires heavy trusses or beams to transfer the load in the sinking tackles to the pontoons or barges. If the tunnel elements are not buoyant the dock gate width required will be governed by the width of the rig.

For the Deck Mounted Pontoon solution the tunnel element must be buoyant and this solution is basically confined to tidal waters where the pontoons can be floated in on top of the deck at High Water. Dock gates need just provide width for the tunnel element to pass.

(Show the overheads below)





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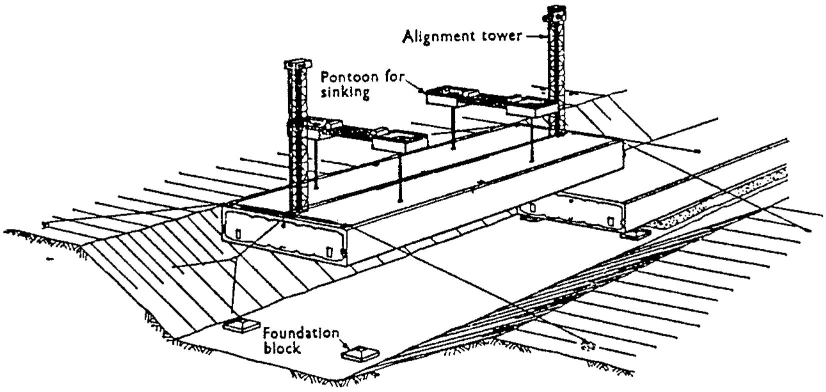


The sinking of the tunnel element is carried out once the tunnel element has been moored and the element has been ballasted, as required, to provide adequate loads in the sinking tackles.

If the tunnel element is going to be founded on a screeded gravel bed, this gravel bed has, of course, been prepared beforehand and the element is lowered to a position just touching this bed and being so close to the previously placed element that the elements can be joined during the subsequent step. Quite often temporary guides ensure mating.

If the tunnel is going to be founded on a jetted sand foundation, the element will be landed on temporary foundation blocks and close to the previously placed tunnel element, these foundation blocks normally having been placed beforehand on screeded gravel beds.

Four temporary foundation blocks and corresponding heavy steel rams activated by hydraulic cylinders from inside the tunnel element is the original standard solution but later three blocks have been used and further the front pair substituted by brackets (one or two) on the previously placed tunnel element, as you can see on the overhead (T 28)

In short -

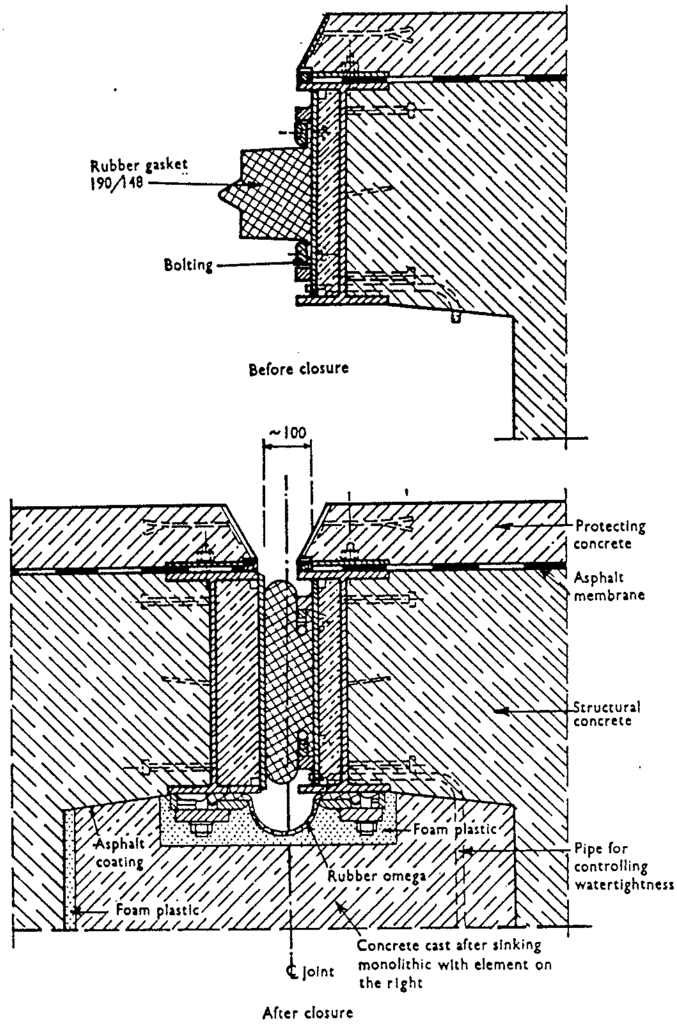
Once the tunnel element is moored and the conditions for sinking are met, the tunnel element is lowered stepwise onto the temporary supports as shown on the overhead.

The surveyor controls continuously the position of the element relative to the brackets and temporary foundation blocks, the deviation of the element from the tunnel axis, the distance to the previously sunk element and the transverse inclination.

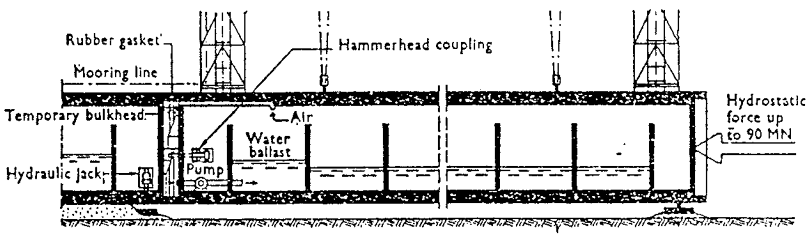
This illustration is for the post sinking constructed injected sand foundation. In case that a gravel bed is already constructed the tunnel element is landed directly on this bed.

(Show overhead below) T 29

I continue with the joining solution widely used in Europe and shall explain an important feature of this operation.



(Show overhead below) T 30



Once landed the elements are joined first by bringing the joint rubber gasket in contact with the steel face of the previously placed tunnel element, secondly by draining the joint chamber, thereby mobilising the full hydrostatic water pressure on the cross section.

By opening doors in the bulkheads at the joint, access is gained to the joint chamber for inspection of this and to the just placed tunnel element. The precise position of this element can now be determined and corrections made prior to lowering of element onto a screeded bed or prior to start of the construction of the sand foundation.

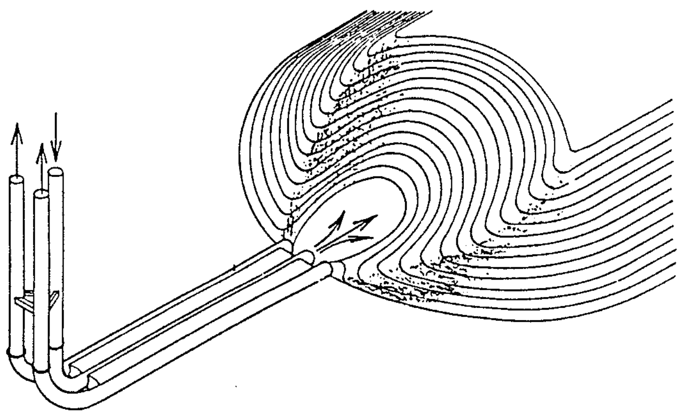
Founding of Tunnel Elements

(show the overhead below) T 31

The construction of gravel beds is a conventional operation, it requires - in this context - considerably sized screeding frames and a lot of diving assistance is necessary. This founding methods is not suitable in areas where considerable bed load movement or siltation is encountered.

The construction of a jetted sand foundation under rectangular shaped tunnels was, ~~as I told you before~~, developed by Christiani & Nielsen, Copenhagen in 1935-36 in connection with the tender for, and subsequent construction of, the Maas Tunnel at Rotterdam, Holland. This method has since then been further developed and applied successfully for the construction of 17 tunnels.

(show the overhead shown below) T 32



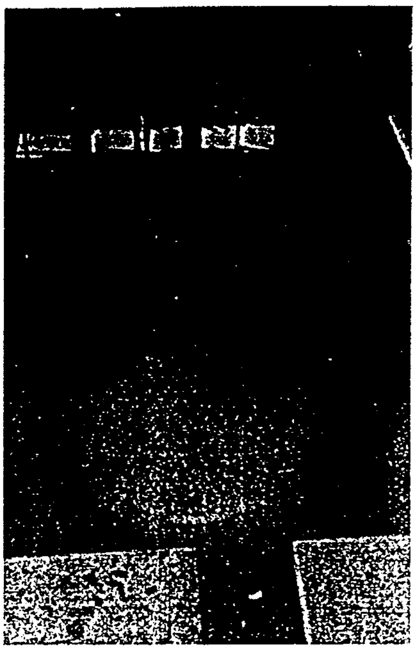
The sandjetting method consists basically in jetting a sand/water mixture into the space between the bottom of the structure and the trench floor while simultaneously sucking back a corresponding amount of water from the jetting area.

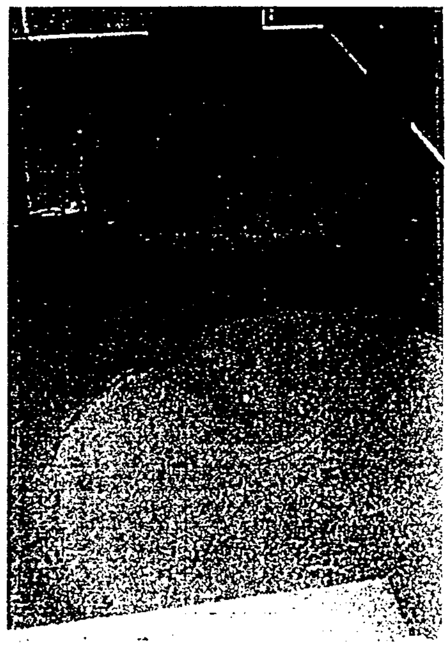
By continuous observation of the amount of sand contained in the return water, means are provided for remote control of the jetting process.

Extensive model tests, and observations made during jetting in practice, prove that when a sand/water mixture is jetted horizontally into a space of confined height under a structure with a plane bottom, an almost circular-shaped deposit of sand is created, a deposit which fills the space right up to the bottom of the structure.

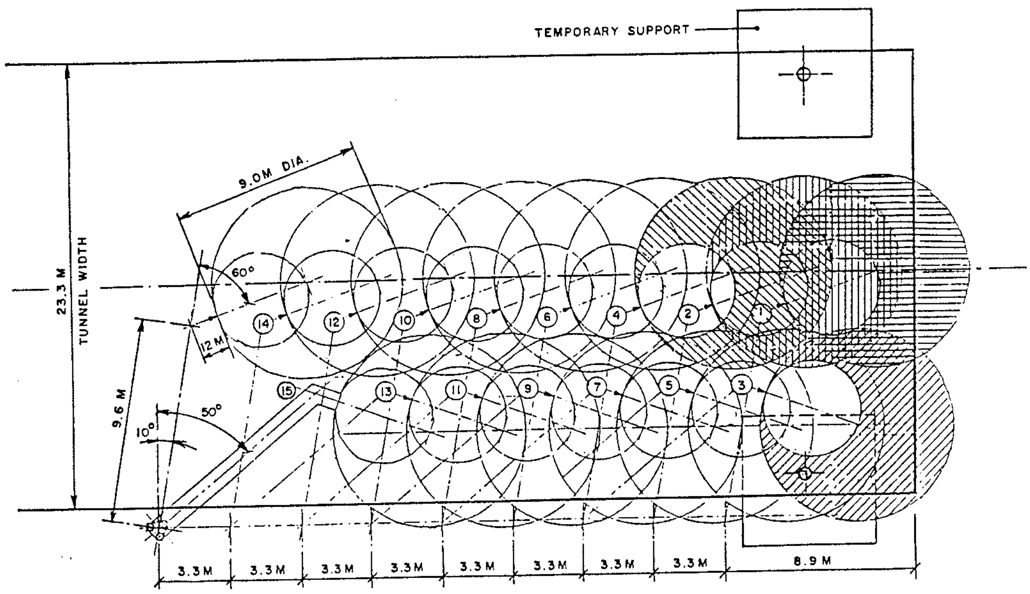
(Show overhead below) T 33

The photos shown on this overhead were taken in connection with supplementary model tests carried out by Christiani & Nielsen in Holland in the Sixties, after draining of the model tank and removal of the structure model, shown such typical individual deposits.





(Show overhead shown below) T 34



By proper selection and balancing of mixture discharge, suction discharge, pressure, nozzle diameter and deposit pattern the space below the foundation base can be filled completely.

It should be noted that the material to be used is a plain coarse sand, no chemical admixtures are required - thus there is no contamination of the environment.

The method has given satisfactory results even in river crossings where the siltation was extremely heavy. In one instance, the tunnel trench was filled with stiff mud to tunnel roof level in a couple of days. This mud was agitated with water jets and sucked away within 10 m wide strips which immediately thereafter were filled in with sand.

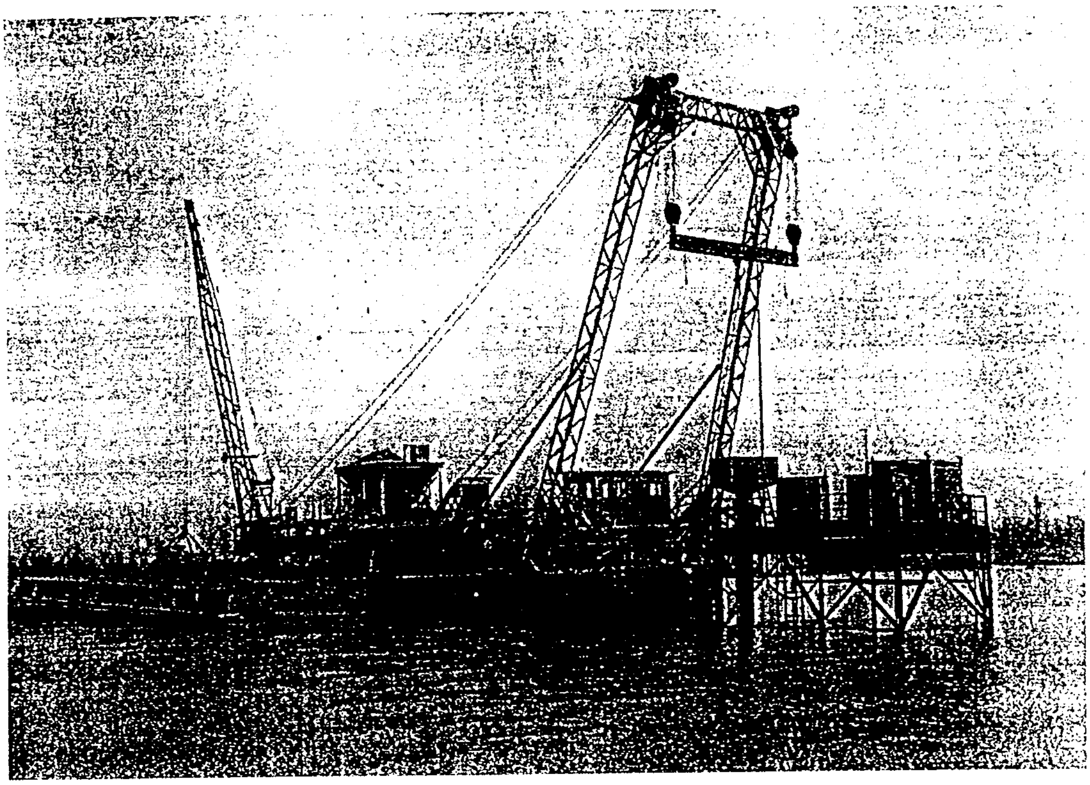
No preparation, for instance levelling of the trench bottom, is required prior to start of sandjetting, only removal of mud and silt. The tunnel bottom acts as upper shuttering for the sand foundation which is tailor-made to the element and the trench, so to speak.

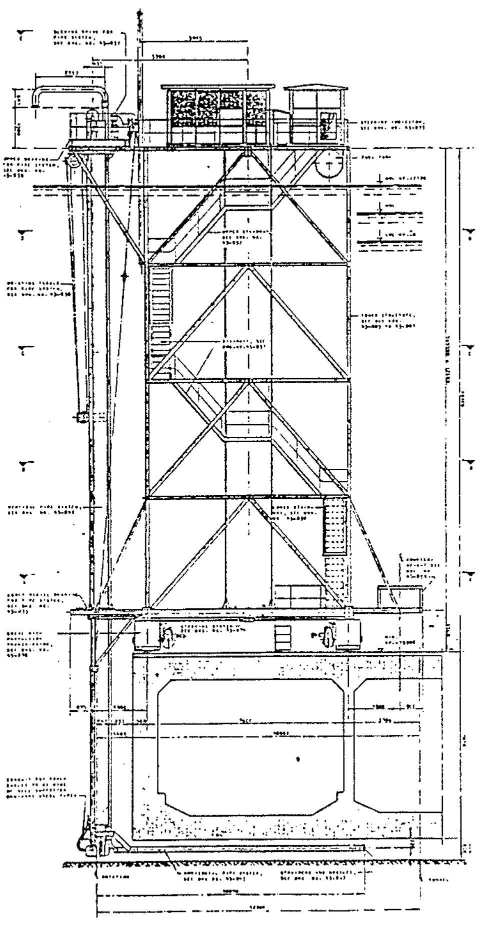
(Show overhead shown on next page) T 35

For the construction of the jetted sand foundation is needed

- a pipe system carrying apparatus running on the sunk tunnel
- a sand/water mixture supplying installation often on a moored steel barge
- sand supply barges
- floating crane for mounting/removal of sandjetting apparatus

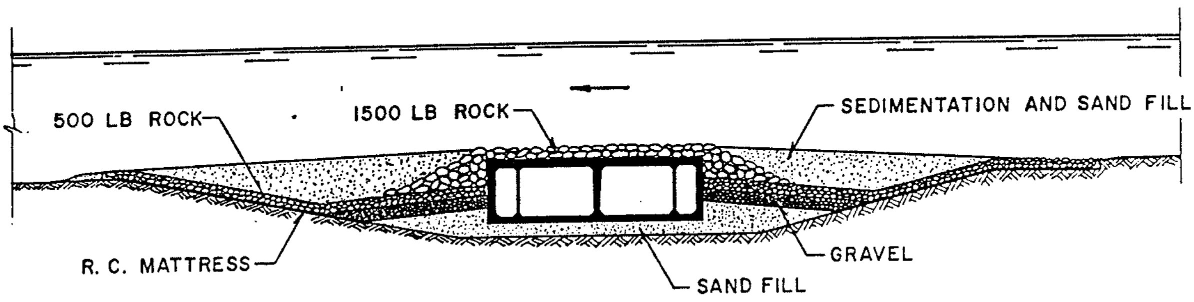
(show lower overhead shown on next page) T 36





Backfilling of Tunnel Trench

(show overhead below) T 37



The complementary works comprise

- backfilling of tunnel element
- placement of protecting mattresses or membranes
- placement of rock fill (armour rock)

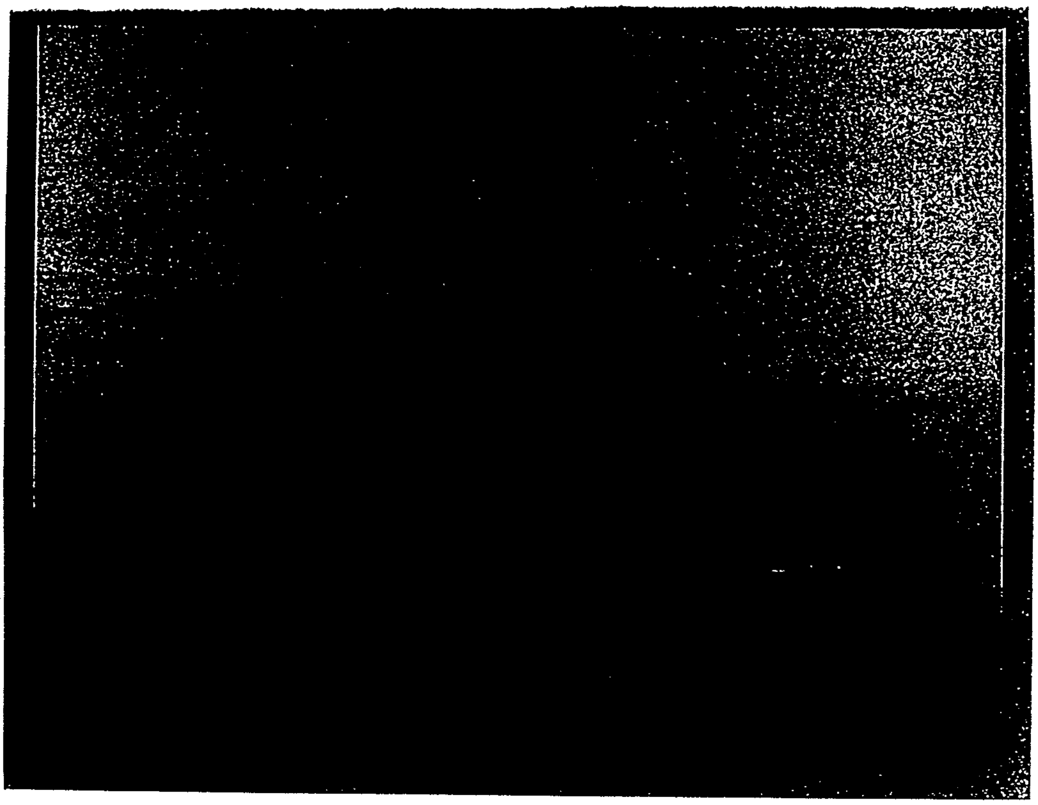
the purpose being to

- protect the permanent tunnel foundation from scour
- provide horizontal support of the tunnel
- protect potable water resources under the tunnel from contamination from waterway
- protect the backfill from scour
- protect tunnel from falling objects, eg ship anchors

Full filling of the tunnel trench to original waterway bed level will normally not be required; this will be left to be done by sedimentation and bed load movement.

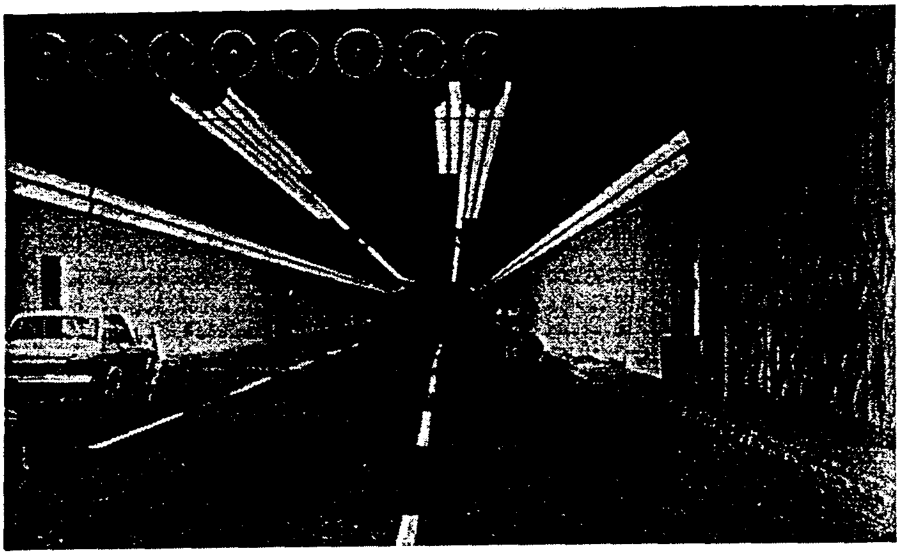
(Show overhead on next page) T 38

The backfill material is normally sand, and it can often be part of the excavated material from the tunnel trench. The backfill can normally be placed by pumping using a small suction dredger, otherwise it is placed by grab.



Complementary Works in Tunnel

(show overhead shown below) T 39



As soon as the tunnel elements have been brought to rest on the permanent foundation and been ballasted as required to satisfy the safety against uplift criteria the complementary works inside the tunnel can start.

These works comprise (show overhead below) T 40

Civil works

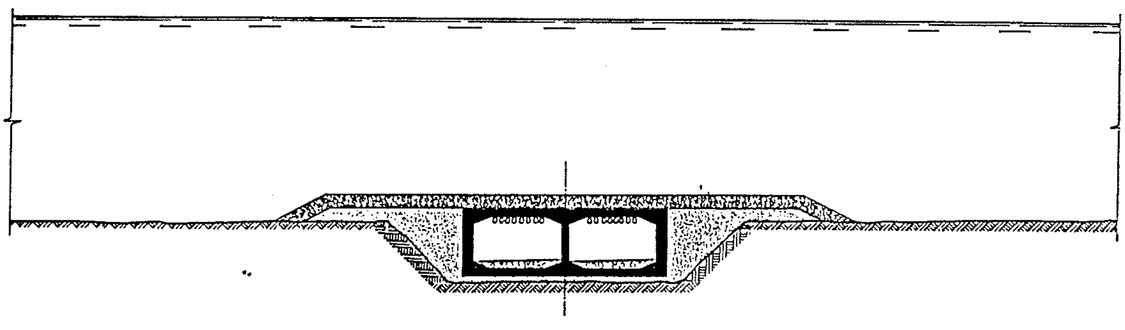
- removal of hydraulic jacks at temporary supporting rams, permanent sealing around rams and concreting box outs
- removal of end-bulkheads
- casting of ballast concrete and subsequently removal of ballast water, slurry pumps and ballast tanks. In this context provision of gutters and ducts for permanent draining system
- installation of remaining seals at element joints and construction of remaining joint structure
- surface treatment of tunnel walls (painting, mounting of tiles or panels) and tunnel ceiling for fire-protection and/or noise reduction
- black-topping of roadway or installation of rails on sleepers on ballast

M&E works

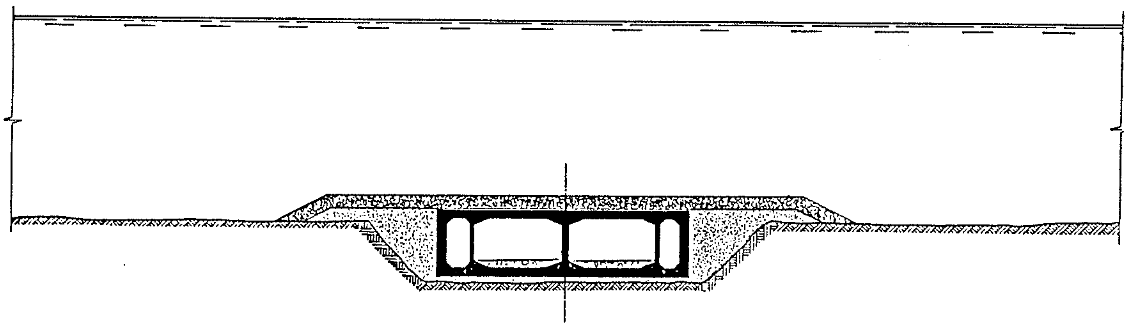
- removal of temporary lighting and power installations
- installation of permanent
  - lighting system
  - ventilation system
  - draining system
  - fire-protection system
  - traffic control system

Looking at the overall time-schedule it must be realised that only very little can be accomplished in(side) the tunnel element before the end bulkheads have been removed.

(Show the overhead shown below) T 41



LONGITUDINAL VENTILATION



SEMI-TRANSVERSAL VENTILATION

I shall like to say a few words about the ventilation of roadway tunnels, as the need for ventilation has diminished during the last decades and made tunnels more attractive from an economic point of view.

The public demand for cleaner exhaust from petrol cars - which originated in the United States - has spread all over the world. In consequence hereof the PIARC recommendation has from the 1975 edition to the 1983 edition halved the maximum content of Carbon-monoxide and other toxic gases to be assumed when designing tunnel ventilation. This has resulted in the ventilation requirements today being generally governed by the demand for visibility.

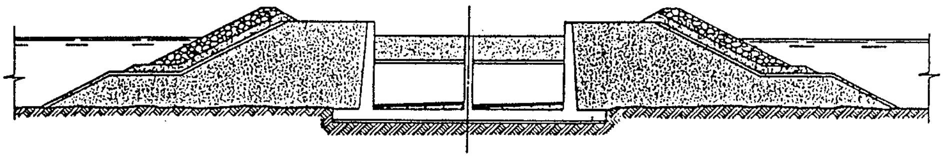
Depending on the gradients, a 3 km long tunnel can today be built with longitudinal ventilation without exceeding an air speed in the traffic duct of 10 m/sec, which is considered to be the acceptable limit.

The length of 3 km from portal to portal meets generally the demand of unhindered passage for shipping, even in open sea.

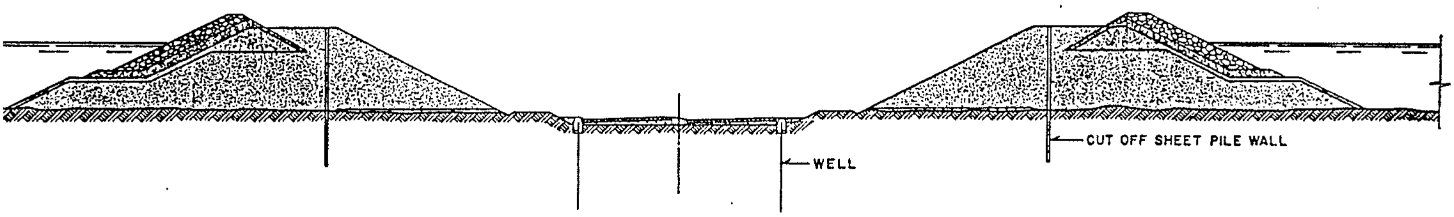
Compared with the more traditional transversal or semi-transversal ventilation, there is a marked saving in cost for both civil works and the M&E installations by using longitudinal ventilation.

Ramps

(show overhead below) T 42



GRAVITY RAMP



DRAINED RAMP

For Roadway and Railroad tunnels ramps are required to bring the traffic to and from the tunnel. We have in several cases found that a considerable saving can be obtained by constructing the ramps with natural earth slopes - using a permanent groundwater lowering system - instead of the conventional concrete gravity ramps, which have to be kept stable against buoyancy forces. It requires, however, that impermeable soil layers are found not too deep into the natural ground.

Although the size of the approach with natural earth slopes is much larger than the gravity ramps structure, it consists mainly of inexpensive fill materials. Furthermore, in order to be able to build a gravity ramp, a temporary working area very similar in size to the drained approach ramp would be needed. Consequently, by adopting the drained ramp solution, a temporary condition is being upgraded into a permanent condition.

5. IMMERSED SERVICE TUNNELS

(show overhead below) T 43

SERVICE TUNNELS

Conveyor belts

High voltage and Low voltage cables

Utility pipelines

Cooling water intakes and outfalls

Sewer culverts

Syphons

General

The construction of immersed service tunnels has of course a lot in common with construction of traffic tunnels. I shall therefore limit this presentation to what is characteristic of service tunnels -

first of all the tunnel elements are much lighter - in the order of ten times lighter

this makes the handling much easier.

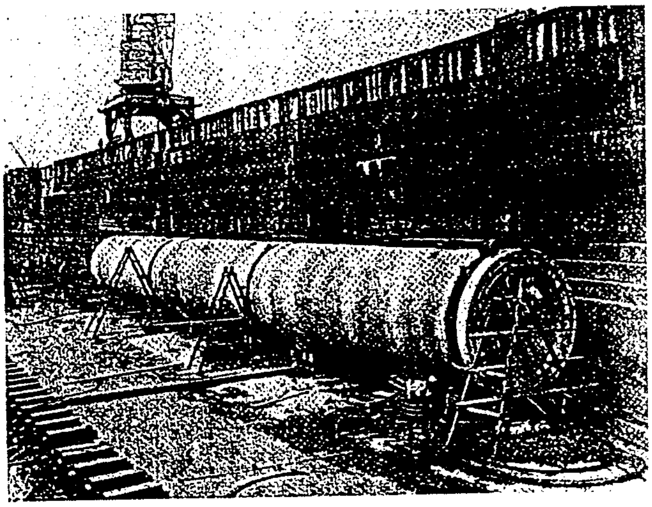
And as history has shown open up for a number of interesting and simplified methods for construction of the tunnel elements and for installation of the tunnel elements. Some of these methods are interesting, innovative and can, properly adjusted, also be used for traffic tunnels.

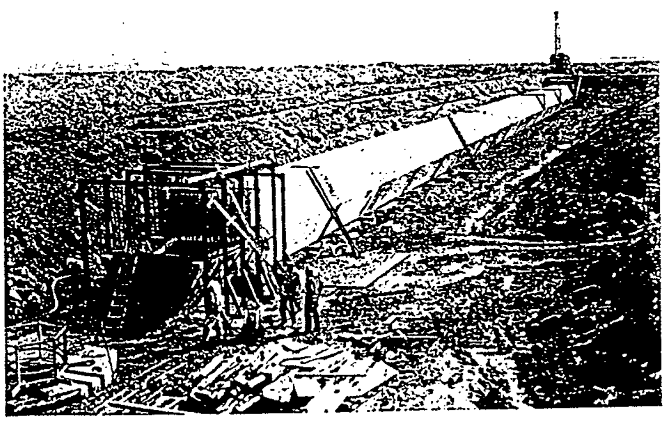
Casting Basin / Yard

(show overhead shown below) T 44

1. Existing drydocks / graving yards
2. Closed existing harbour basins
3. Purpose-built casting basins
4. Approach ramps / cut-and-cover tunnel pits
5. Yard at ground level (and subsequent launching on
   for steel shell tunnels only
6. Yard at ground level and launching by use of marine lift
7. Yard at ground level and launching by use of heavy floating cranes
8. Floating dock

(show overheads shown below) T 45 + T 46

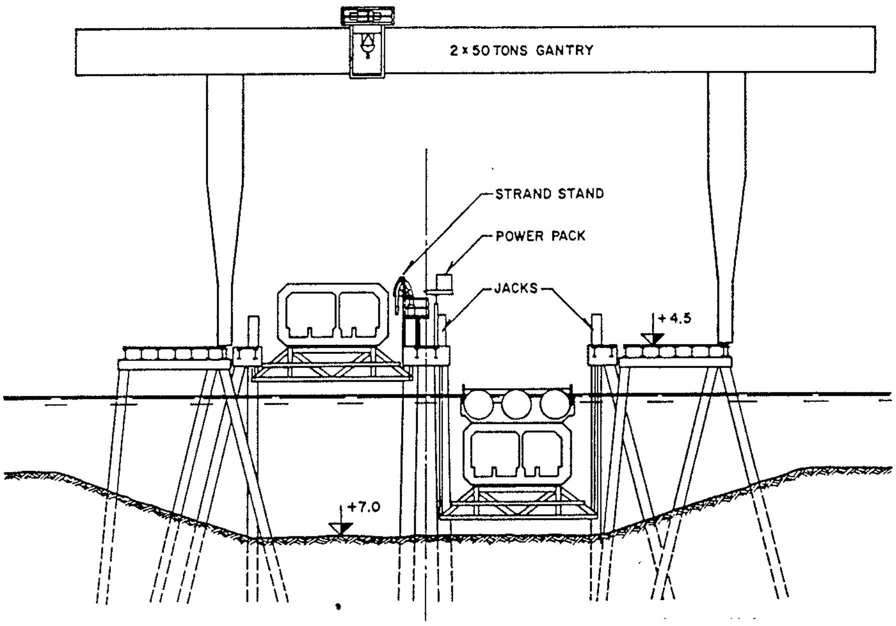


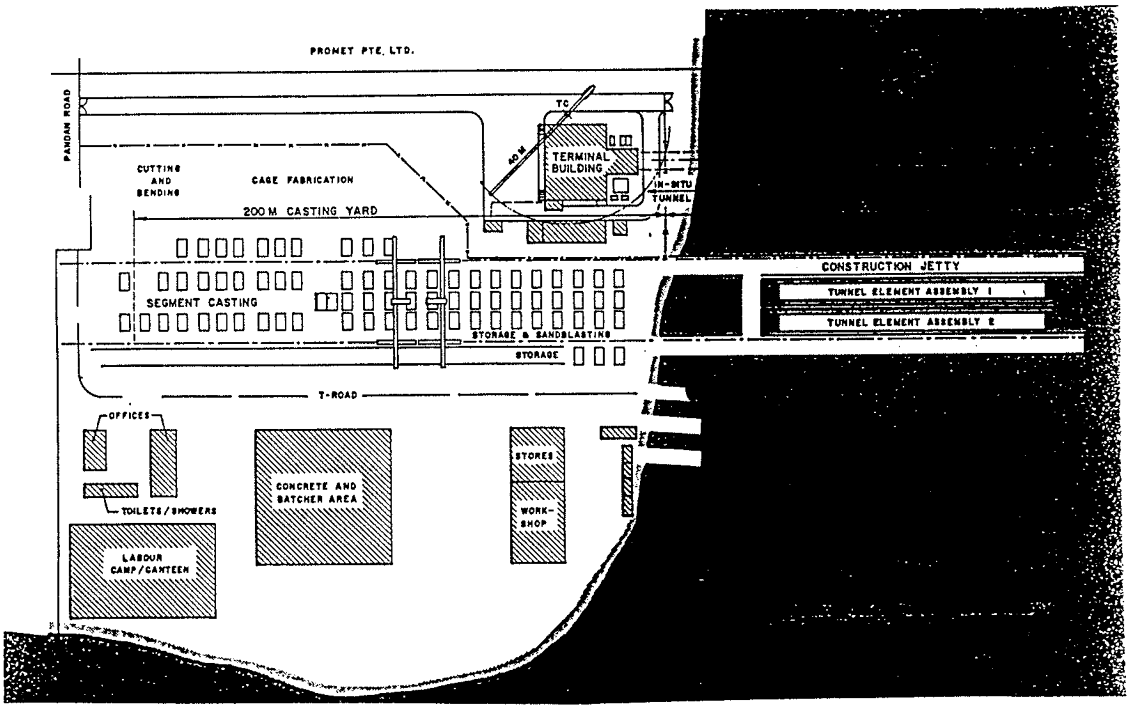


Apart from the existing docks / graving yards and purpose-built casting basins which we know are used for traffic tunnels - the following possibilities exist for service tunnels:

6. Yard at ground level and launching by use of marine lift

(show overheads shown below) T 47 + T 48





7. Yard at ground level and launching by use of heavy floating cranes

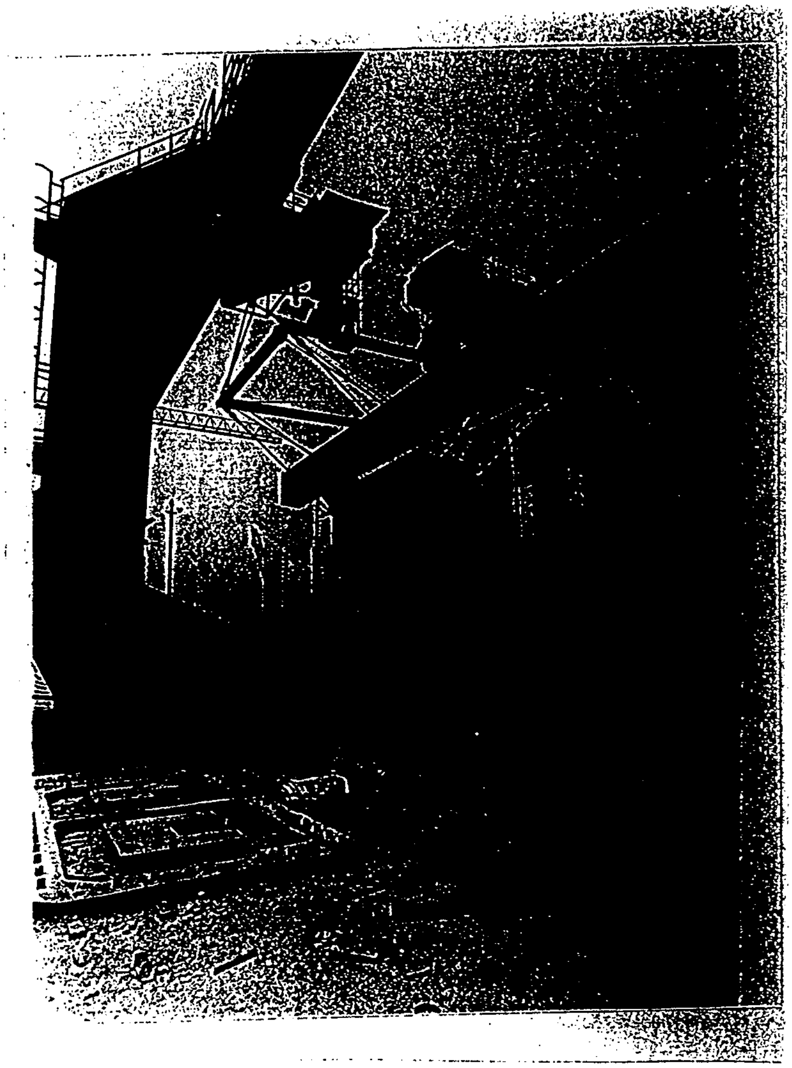
8. Floating dock

Tunnel Elements

The tunnel elements normally up to 100 m long are often built by joining a number of relative short precast segments which can be handled by means of standard lifting equipment - and when we are talking of many tunnel elements mass production methods are used, as for the Cable Tunnel between Pulau Seraya and Mainland Singapore in 1985-86.

The tunnel segments are often cast in vertical position - in one go - and this reduces the risk for development of cracks in the walls during hardening of the concrete. By combining this procedure with longitudinal prestressing of the tunnel elements watertightness has been provided without a waterproofing membrane. Because of the concrete cross sectional area is relatively small for the service tunnels, the prestressing can be provided at an acceptable cost.

(Show overheads below) T 49, T 50 + T 51

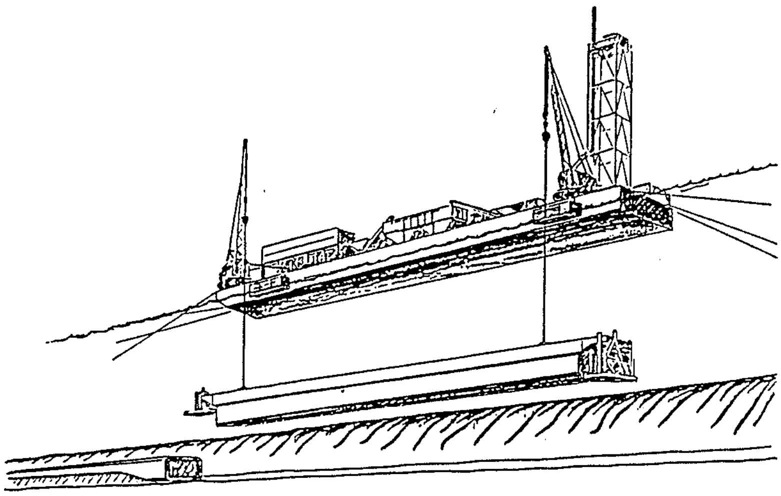


The precast segment scheme involves that the marine lift or the dock where the segments are joined to form tunnel elements need only be occupied for a relatively short period.

In some cases, as was the case in Singapore, the tunnel elements are "born" with the permanent overweight. In that case no water ballast and associated tanks, pumps and piping will be required in the tunnel element and the tunnel element can be made complete with up to some 80 per cent of the permanent mechanical and electrical installations <u>in</u> before the tunnel element is launched, this possibility being of particular interest for long tunnels. On the other hand, for such tunnel elements, a buoyancy aid will be required, ie buoyancy tanks attached to the tunnel element or catamaran type sinking rigs capable of lifting the tunnel element off the dock bottom.

Sinking and Joining of Tunnel elements

(show overhead shown below) T 52



SINKING OF TUNNEL ELEMENT

For the sinking and joining of the tunnel elements the same principles and methods, as used for traffic tunnels, are applicable.

Again because of the smaller masses and forces involved, lighter equipment will do, and some simplification can be made when careful planning and execution is done. The methods developed by Christiani & Nielsen Copenhagen for the Cable Tunnel in Singapore is an example hereof - admittedly justified by the length of this service tunnel, 2.6 km long, composed of 26 Nos 100 m long tunnel elements, and favoured by the tidal conditions, for instance

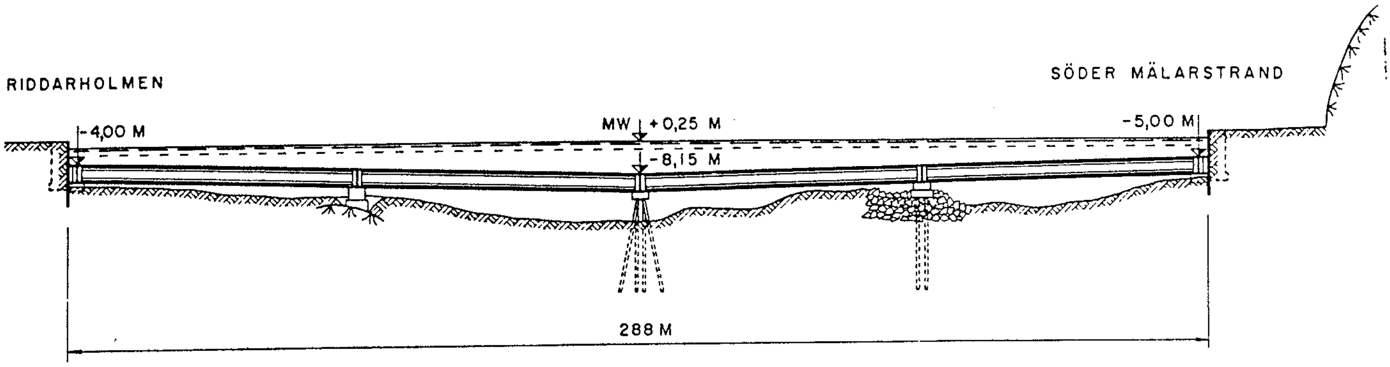
- no ballast tanks - it is non-buoyant tunnel elements
- no horizontal mooring system for sinking
- no temporary foundation pads and blocks in tunnel trench

Founding of Tunnel Elements

The permanent foundation for a service tunnel can typically be either

piles

(show the overhead below) T 53

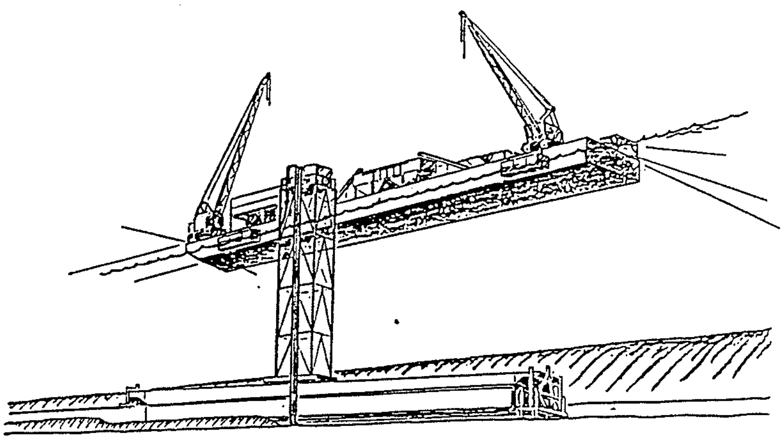


ELEVATION

where the tunnel is supported like an underwater bridge or

a sand foundation

(show overhead below) T 54



SANDJETTING OF TUNNEL ELEMENT

For tunnels with circular cross section conventional dumping of sand through "elephant trunks" will suffice.

For tunnels with rectangular cross section the permanent solution can be and has been constructed by use of the Christiani & Nielsen Sandjetting Method.

Backfilling of Tunnel Trench

In waterways where currents can cause scour of a hydraulically placed sand foundation or of the trench bottom around piles, part of the backfill, the locking fill shall be placed soonest possible after installation of the tunnel elements, while the remaining backfilling can be postponed - and normally will be - in order not to block the waterway with too much floating equipment at a time.

Complementary Works in Tunnel

Compared to traffic tunnels the complementary works in a service tunnel are generally reduced.

The works may comprise (show overhead below) T 55

Civil works

- removal of hydraulic jacks at temporary supporting rams, permanent sealing around rams and concreting box outs

- removal of end-bulkheads

- casting of ballast concrete and subsequently removal of ballast water, slurry pumps and ballast tanks. In this context provision of gutters and ducts for permanent draining system

- installation of remaining seals at element joints and construction of remaining joint structure

- surface treatment of tunnel walls and tunnel ceiling for fire-protection

M&E works

- removal of temporary lighting and power installations

- installation of permanent
  - lighting system
  - ventilation system
  - draining system
  - fire-protection system

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

11. Model testing, static / dynamic analysis

1. Japan	H. Kunishu
2. Italy	R. Carpaneto
3. Norway	T. Søreide

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11. Model testing, static / dynamic analysis

11.1 Submerged Floating Tunnels in Japan - Emperiments and Numerial Study

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Submerged Floating Tunnels in Japan
- Experiments and Numerical Study -

Abstract

In this paper, dynamic response characteristics of submerged floating tunnels due to wave force or earthquake were presented. The dynamic motions of SFT were computed by a numerical analysis method for a two-dimensional model based on kinematics equation.

The wave forces acting on the SFT were evaluated by the Morison's equation or potential theory. The computed results are compared with the experimental results.

We presents an analytical method to estimate response of SFT to vertical seismic excitations. Based on the one-dimensional duplicate reflection theory, the response of the sea bottom and water to the input earthquake motion is calculated.

1. Marine condition

In the case of Funka Bay in Japan, it faces the Pacific Ocean on the southeast at Hokkaido. Maximum water depth of this site is 100m. Significant wave height with a return period of 100 years ,which was equal to 9.3m, was evaluated by the Weibull distribution with k=1.0. Moreover, significant wave period was calculated by using the wave records 13.0sec which was corresponds to 99.9% probability of nonexceedance. It was reported that tsunami height in Muroran Port which is located at east part of Funka Bay was not more than 0.5m according to the Chile Tsunami struck Japan on May 24, 1960. The maximun tidal current in the Funka Bay was observed 1.0cm/sec at the Sahara coast.

2. Structural conditions

Two different diameter of SFT were proposed 11.4m(2 car lane) and 23.0m(2 car & 2 railway lane). The water clearance from the sea surface is determined in consideration for wave force reduction and the safety of navigation.

The hydrodynamic forces due to wave, tsunami and tidal current were evaluated to the Morison's equation. The wave force was the predominant as hydrodynamic force acting on the tunnel. Therefore, detailed experimental investigation and numerical calculation of dynamic response of tunnel were required.

3. Wave force and displacement of tunnel

Two-dimensional large wave tanks in Hokkaido Development Ministry were used in this study. Model experiments were carried out with a scale of 1/62.16 according to Frude law. Both regular and irregular waves were used as the wave conditions. Bretschneider-Mituyasu type spectrum was adopted for the irregular wave.

From the result of the regular wave experiment, the drag force and inertia force coefficient was calculated based on the least squares method. The result was showed that the drag force coefficients scatters when Keulegan-Carpenter number was small. On the other hand the inertia force coefficients drop within relatively small rang from 1.5 to 2.0. The drag force coefficient of 1.0 and the inertia force coefficient of 2.0 were generally used for the column structure. As the result of this experiments, it was proved that these number were adequate in this type of SFT. Comparison of the drag force with the inertia force, the drag force was corresponded to approximately 20% of the inertia force. Two-dimensional analytical wave force computed analytically using the Boundary element method(BEM) based on the potential theory and Morison's equation. The numerical method was applicable to evaluation of force for this type of tunnel.

Several type of mooring were used to the fixed method of tunnel. Type-A mooring was used vertical tension leg. In this type of mooring, it is easy to move the transverse direction because of no restraint. Type-B mooring was used slant tension leg, to control the transverse displacement of tunnel. Therefor slack of leg was observed in this type of mooring when wave height became large.

The tension acting on a mooring line of SFT consists of a initial tension due to a buoyancy of SFT and fluctuating component due to a exciting wave force. The slack of tension leg was observed when minimum fluctuating tension force of leg is equal to a buoyancy of SFT.

4. Earthquake response of SFT

It is found that the vertical motion of the sea bottom motion originating from an earthquake causes an exciting force which acts on the tunnel not only with the stress generating in the legs but P-waves that propagate in the water. We presents a two-dimensional analysis of responses to vertical motion of seismic excitation generating at the SFT. Concerning the incompressible fluid, this analysis uses the boundary integration method based on the potential theory to determine hydrodynamic properties such as added mass of tunnel. Through these analysis, we examine the characteristics of hydrodynamic properties and effect of P-wave on the response for the dimensions of the prototype tunnel. In the present analysis, fluid force properties are estimated applying a velocity potential theory to incompressible fluid, On the other hand, the one-dimensional duplicate reflection theory, we assume a multi reflection theory was employed to determine fluid motion properties. In the duplicate reflection theory, we assume a multi reflection of P-wave which exists between the sea bottom and the water surface, and with this theory, it was possible not only to take into consideration the conditions of the speed of P-wave in the water and ground, but also to approximate the water compressibility effect.

The coefficients of the added mass and exciting force remained almost constant within the range of the earthquake frequency band, the damping due to surfacewave is negligible. As for the sea water motion, the time history of acceleration response was more similar to that of an incident seismic wave, while the amplitude of the cable tension became much smaller than when the fluid motion was not considered. The stress wave propagation within a cable had almost no effect on the response with the depth of several handled meters. In this case, a cable can be deemed as an approximately linear spring.

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11. Model testing, static / dynamic analysis

11.2 The Dynamic Seismic Analysis of SFT

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D'Appolonia S.p.A.

The Dynamic Seismic Analysis of SFT

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D'Appolonia S.p.A., Genova, Italy

1.0 INTRODUCTION

This presentation of the dynamic seismic analysis of SFT is neither complete nor generally applicable, but gives some information to the reader about few typical aspects in this field. Three topics are discussed:

- o the Multiple Support Excitation Analysis (relevant to tethered SFT);
- o the Seaquake problem (more evident pontooned SFT);
- o the Seismic Damage Hazard Analysis of SFT.

2.0 THE MULTIPLE SUPPORT EXITATION ANALYSIS

It is generally agreed that sesmic ground motions are the result of interaction of several types of waves as well as of multiple reflections and refractions occurring at the interfaces between the subsoil formations.

The result of the above complex phenomena is that seismic motions at any two points are different both in severity (amplitude) of motion as well as in frequency content. Indeed, as the seismic waves propagate from the seismic source the ground motions along the propagation path will be out of phase. It is obvious that the longer the distance between the points the larger the difference, to the point that earthquake records at stations many tens of kilometers apart are totally uncorrelated.

Of course, the degree of difference of ground motions varies as a function which depends on many factors of which the most influential are the distance between the two points, the local soil and topographical conditions, and their location in relation to the earthquake source.

2.1 CONSIDERATIONS FROM EARTH CRUST MODELS

Three types of waves are possible in a medium with a free surface, i.e. a half-space: Dilatation waves (P-waves), Shear waves (S-waves), and Rayleigh waves which are generated when the previous two types encounter the free surface.

The earth crust is certainly a strongly nonhomogeneous and nonuniform inelastic medium. However, even if, for an instance, the assumption is made that it is a perfectly homogeneous and uniform elastic medium still ground motions at two points would be different. The reason is that the propagation velocities of Dilatation waves (P-waves) is higher than the Shear waves (S-waves) velocity and the Rayleigh waves velocity (typically about 90 per cent of the S-wave velocity). Therefore as the distance from the seismic source increases the times of arrival of these waves would be increasingly separated and the ground motions, namely the superposition of the motions due to these waves, would differ.

If the layering of the Earth crust is introduce, still retaining the assumptions of perfectly uniform elastic layers, the motions would be even more complex. This is due to three principal factors:

- o the dilatation waves (P-waves) and Shear waves (S-waves) will suffer multiple reflections and refractions;
- o Rayleigh waves are dispersive and therefore different frequency components propagate at different velocities (phase velocities). Detailed analysis of phase velocities can only be performed numerically because of the complexity of the formulations involved. However, it is generally agreed that the properties of the materials within about one wave length control the respective phase velocity;
- o other types of waves are generated because of the layering and interaction at the interfaces, like, as an example, love waves.

If now the real complex nature of the Earth crust is introduced, with discontinuities, fracture zones, inhomogeneity of layers, etc., the complex nature of observed ground motions becomes evident.

2.2 THE ENGINEERING APPROACH

Detailed and complete understanding of ground motions and identification of various wave components is beyond present State-of-the-Art. Nevertheless, the fact remains that ground motions differ from point to point and this gives rise to multiple support excitation which must be accounted for in the design of long structure like the tunnels under consideration for the Messina Strait. For this reason an engineering approach to this problem has been formulated and is currently applied based on conclusions and understanding drawn from both actual observations and theoretical studies. The main features and reasoning of the approach are briefly outlined below.

2.2.1 Shape and Frequency Content of Design Ground Motions

Analysis of records shows that a major contribution to motion difference between various points is due to surface waves (primarily Rayleigh and, also, Love waves). However, since identification of wave components is not practically feasible the analysis is performed with the ground motions actually observed. Thus actual earthquake records are used without any attempt to filter out particular frequencies.

2.2.2 Coherence of Seismic Input - Travelling Wave

Seismic input certainly varies along the propagation path. When the distance between support points is relatively small such variation is small to moderate. Because:

- o non-coherence is small for small distances;
- o non-coherence cannot be realistically estimated;
- o multiple supports for the Messina structure are ad distances which are small in seismological scale.

Therefore it is appropriate to use the same seismic ground motions at all support points and displaced in time by a time lag computed on the basis of the propagation velocity.

2.2.3 Apparent Propagation Velocity

The apparent propagation velocity of the seismic ground motions is one of the fundamental parameters in multiple support excitation analysis and has received considerable attention in relation to the analysis of underground tunnels (Constantopoulos et al., 1977 and 1980, Hall and Newmark, 1977). Very high velocity, at the limit, implies simultaneous seismic input at all points and elimination of the effects of the multiple support excitation.

If two points are considered, A and B, separated by a distance D the earthquake motions arrive (start shaking) at A at time t_1 and at B at time t_2. Then the apparent propagation velocity is defined as:

$$V = \frac{D}{t_2 - t_1} \tag{1}$$

The SFT structures are founded on the surficial layers. On the other hand it has already been mentioned above that from analysis of records it is concluded that a major contribution to motion difference between various points is due to surface waves, primarily Rayleigh waves, which propagate at approximately the velocity of shear waves. Thus, one might conclude to use the shear wave velocity of the surface soils. This is not correct. The following observations apply:

- Rayleigh waves are generated because of the existence of free surface but propagate at a velocity dictated by the properties of materials well below the ground surface;
- waves propagation in basement rocks arrive at the interface between surface soft soils and the rock and produce excitation which arrives at the ground surface. Certainly, the expression of this excitation propagates at an apparent velocity much higher than the shear wave velocity in the surface soils.

Regardless of the complex and largely presently unknown interactions of the waves at the site it is clear that the apparent propagation velocity is higher than the shear wave velocity of the surface soils. Detailed information from analysis is impossible. Therefore, use is made of available data from actual observations during earthquakes. Thus and based on the above we recommend to adopt and implement apparent propagation velocity values which take into account the following information.

Hall and Newmark (1977) observe that the propagation velocity is consistent with the properties of the materials at considerable depth and are not affected to a large extent by surface properties.

Tsushida and Kurata (1976) and Tamura et al. (1977) report and analyze data from four Japanese earthquakes with Magnitudes from 5.8 to 6.9 and epicentral distances 30 to 160 km from which apparent propagation velocities between, approximately, 2.5 and 5.5 km/sec are derived.

The analysis of the multiple recordings performed by the SMART 1 array (Bolt et al., 1982) lead to apparent wave velocities between, approximately, 2.5 and 5.5 km/sec. The Dilatation wave (P-wave) velocities of the alluvium are between 500 and 1000 m/sec while for the rock they are between 1800 and 2000 m/sec. These values correspond to Shear wave velocities of 200 to 400 m/sec for the alluvium and 950 to 1100 m/sec for the rock.

Finally, Hadjian and Hadley (1981) have found that the best estimate for the apparent (traveling) wave velocity is the shear (o Rayleigh) wave of the layer in which the energy is released (tipically a few kilometers).

3.0 THE SEAQUAKE

Seaquake is the propagation of the seismic motion of seabed through seawater normal to the mud line. In case of floating-type structures such as SFT, effects of vertical motion of seabed due to earthquake may be significant and seaquake load imposed on the tunnels can not be neglected.

This note presents the parameters that influence the magnitude of seaquake and defines the response of the submberged tunnel in order to evaluate the interaction existing between water and tunnel.

3.1 SEAQUAKE PARAMETERS AND EQUATIONS

The perturbation in seawater due to vertical motion of seabed is such that convective motions and viscous dissipation can be neglected, leading to the classical formulation of propagation of waves of volume in an elastic medium:

$$C^2 \nabla^2 \theta = \partial^2 \theta / \partial t^2 \tag{2}$$

where:

θ = volumetric deformation;

C = velocity of propagation in water = $\sqrt{k / \rho}$;

k = bulk modulus;

ρ = density of water;

t = time.

Free field vertical motion can be found developing equation (2) for the following cases:

- stress waves travelling vertically. Since no horizontal flow is induced, the compressibilities of water and sea-bed soil are taken into account;
- stress waves travelling through the soil in an oblique direction, with apparent horizontal velocity C_a. The soil surface motions produced by these travelling waves may induce horizontal flow in water. For the frequencies of interest, one can assume the water to be incompressible. If the apparent horizontal velocity is high, as in our case, we might fall in the previous assumption.

A finite element analysis is suitable for the first case. From (2), equation of waves in its monodimensional form can be also derived.

A model may be also developed by solving Laplace equation for the potential velocity function. This method is suitable for an analytical development, similar to those used by Airy in his formulation of linear waves.

By solving equation (2) the equation of motion, in terms of displacement, velocity and acceleration, for the water level of interest, corresponding to the free field at the position of the tunnel, is derived.

Seaquake motion and its effects, therefore, depend on the water depth, seismic intensity, predominant period and wave length of seabed in vibration.

However, the first case (vertically propagating body wave) may be conservatively retained since it yields generally higher output in the vertical direction.

3.2 EVALUATION OF STRUCTURAL RESPONSE

The evaluation of the motion of a column of water (free field) under seismic excitation at its seabed, is the first step to evaluate the global response of the system water-structure. The acceleration time history of water corresponding to the position of the submerged structure is conditioned by the presence of the structure itself, and this fact will be taken into account by the free field-near field concept. However the approximation in neglecting the effect of tunnel on water motion is generally negligible (Liou et al., 1985; Bruschi et al., 1990).

If a rigorous analysis is requested, the previous equation could be used with different boundary conditions on displacement and velocity.

3.2.1 Applicable Equation of Motions

Equations of motion are characterized by the combination of classical formulation of dynamic motion with the expression of acting loads due to Morison (1950).

The equation that describes the motion of the tunnel under seaquake may be expressed as follows:

$$M\ddot{u}_T(t)+C\dot{u}_T(t)+K u_T(t)=\rho_a A\ddot{u}_a(t)+C_m\rho_a\pi D^2/4(\ddot{u}_a(t)-\ddot{u}_T(t))+ \\ +\frac{1}{2}\rho_a Cd\,\pi D^2/4|\dot{u}_a(t)-\dot{u}_T(t)|(\dot{u}_a(t)-\dot{u}_T(t)) \qquad (3)$$

where:

M = mass of the tunnel, kg;

C = damping coefficient of the system water-tunnel, kg/s;

k = rigidity coefficient of the system water-tunnel, kg/s^2;

$u_T(t)$ = displacement of the tunnel, m;

q_a = density of water, kg/m^3;

A = cross sectional area of the tunnel, m^2;

D = diameter of the tunnel, m;

C_m, C_d = added mass and drag coefficients (see Paragraph 3.2);

$u_a(t)$ = displacement of water at contact with tunnel, m.

Two fundamental nondimensional numbers that describe the interaction between water flow and tunnel are *Re* (Reynolds number) and *KC* (Keulegan and Carpenter number) defined as:

$$Re = \frac{U_m D}{\nu} \tag{4}$$

$$KC = \frac{U_m T}{D} \tag{5}$$

where:

U_m = water particle velocity near the tunnel, m/s;

D = diameter of the tunnel, m;

ν = cinematic viscosity of water, m^2/s;

T = oscillation period of seaquake, sec.

Making reference to literature, for small *KC* and high *Re* numbers the flow of water is as nearly ideal with C_d very small and C_m approximately equal to 1.0 (McClelland et al., 1986; Sarpkaya et al., 1981).

3.2.1 Added Mass Coefficient

When a body is moving in water there is a resistance to this motion due to water particles that are displaced and accelerated in order for this motion to take place.

This effect is generally taken into account by considering the body to have an "added mass". The amount of added mass used depends upon the type of motion being considered.

The situation is the same for the submerged structure into a flow of water: the presence of the structure and its movement produce a resistance to water particle flow that can be expressed as an added mass to the mass of the same structure.

In Equation (3) added mass is computed as a fraction of the mass of the water displaced by the tunnel using a coefficient of proportionality named as "added mass coefficient", C_m.

The theoretical value for the added mass coefficient for a cylinder is $C_m = 1.0$, but large deviations from this value have been observed (Sarpkaya, 1981).

Various empirical diagrams relating C_m to KC, Re and roughness of the structure are given by literature (Sarpkaya, 1981), but no experiments were made for high Reinolds numbers and low Keulegan-Carpenter numbers, as found for the majority of SFTs.

4.0 SEISMIC DAMAGE HAZARD ANALYSIS (SDHA)

4.1 LIMITATIONS OF THE CONVENTIONAL APPROACH

The conventional seismic hazard analysis (SHA) is a valuable tool for assessing, for a given structure (e.g. SFT), the probability of experiencing severe earthquakes during its designed lifetime. More precisely, SHA provides an estimation of the probabilities that selected ground-motion intensity parameters (PGA - Peak Ground Acceleration - or S_a - Spectral Acceleration) will be exceeded at the site within a specified exposure time.

These results can be used to estimate the seismic loads having a certain likelihoods to hit the structure during its lifetime but a prediction of the damage the structure is likely to experience is not readily available in the conventional SHA. In fact PGA has long been recognized to have a very poor correlation with either actual observed or theoretically computed damage in structures. On the other hand S_a is able to capture only the peak elastic response of linear elastic single-degree-of-freedom (SDOF) systems at the frequency of vibration corresponding to the undamage state.

It is immediate to recognize that a sound prediction of damage in real structures cannot be based on this linear measures alone mainly because:

- the critical assumption of SHA, i.e., that a structure behaves as a linear elastic single-degree-of-freedom (SDOF) system during a severe ground shaking, is always violated;
- SHA fails to incorporate features of the structural response beyond the linear elastic regime when the damage induced by the seismic excitation changes its dynamic behavior.

The missing link is the definition of a simple but accurate characterization of the ground motion ability to induce damage in real structures which shall be included in a probabilistic seismic hazard framework. The SDHA presented hereafter makes this missing piece available and allows for the direct computation of the probability of structural damage and failure by performing seismic hazard analysis calculations coupled with a limited set of non-linear dynamic analyses of the structure.

4.2 SDHA METHODOLOGY

For probabilistic damage estimates the post-elastic response of the structure to severe seismic loadings has to be adequately taken into account in seismic hazard computations. To do so the damage potential of earthquakes must be captured in a simple but effective way.

The capability of specific ground motion records to cause a given level of post-elastic damage in the specified structure can be appropriately measured by a ground-motion- and structure-dependent random variable $F_{DM,l}$ (Bazzurro et al., 1994). The abbreviation DM refers to the damage measure (e.g., ductility ratio μ, normalized histeretic energy NHE, etc.) employed to gauge the damage at the location *l* of interest in the structure.

The factor F_{DM} can be defined as follows: $F_{DM,l=x}$ is the amount by which an earthquake record that causes incipient yielding in a system has to be scaled up to obtain a new record capable of inducing a non-linear damage level, *x*, associated with a specified damage measure, DM, at location *l* in the structure.

In mathematical form $F_{DM,l=x}$ is given by:

$$V = \frac{D}{t_2 - t_1} \tag{6}$$

where the numerator represents the rescaled spectral acceleration needed to cause the specified damage level, while the denominator is the spectral acceleration needed to cause the incipient yielding in the system. Incipient yielding is taken as the reference event for the computation of $F_{DM,l=x}$. The frequency, *f*, and the damping, ξ, of the system can be assumed to be approximately equal to the fundamental frequency of vibration and of the damping of the system when the reference event is about to be reached.

It follows from the previous definition that F_{DM} assumes greater values for less damaging ground motions, and smaller values for ground motions that are particularly severe for the structure. From this perspective, F_{DM} can be considered as an inverse measure of the damage potential of a ground motion for a given structure. The factor F_{DM} implicitly contains and displays any effect of the ground motion such as duration, degree of non-stationarity, etc, that may influence its damageability.

A possible realization of the random variable F_{DM} can be computed by applying a given earthquake ground-motion record to a non-linear finite element model appropriately describing the dynamic behavior of the SFT under a severe ground shaking. Given the non-linearities in the problem, dynamic analyses in the time domain are required for this purpose.

The systematic study of F_{DM} for non-linear SDOF systems and for some multi-degree-of-freedom (MDOF) structures has revealed that, as a statistical variable, F_{DM} has certain, consistently empirically observed properties that allow its use in the SHA methodology for the direct computation of the annual probability of structural damage (Bazzurro et al., 1994). In brief, these properties are insensitivity of F_{DM} (in the mean) to M and R, and a very moderate statistical variability relative to the variability of S_a. Together these two properties imply that only a very limited sample size of real records is sufficient to establish the mean value of F_{DM} for any given structure, and that this mean is the only structure-specific value needed to calculate the probability of structural damage.

Computationally the calculation of the annual probability of exceedance of a level, x, of a post-elastic damage, DM, at a location l in the structure involves the repetitive calculation of the following conditional probability:

$$P[DM,l>x|m,r]=P\left[S_a(f,\xi)>S_{a_{DM,l=x}}(f,\xi)\right]=P\left[S_a(f,\xi)>S_{a,yld}(f,\xi)\,F_{DM,l=x}|m,r\right] \quad (7)$$

where m and r are realizations of the random variables magnitude, M, and epicentral distance, R.

In light of the previous considerations, the seismic damage risk can be computed very simply by replacing $F_{DM,l=x}$ and $S_{a,yld}$ (f, ξ) by their unconditional means and inflating the original variability intrinsic in S_a in order to account for the comparatively small additional variability in $F_{DM,I=x}$ and $S_{a,yld}$ (f, ξ). It is important to recognize that the practical effectiveness of the method derives directly from the coupling of the ground motion attenuation and non-linear structural analysis. The method exploits the comparatively broad statistical variability in S_a (given M and R) to limit the number of costly non-linear dynamic analyses necessary for the determination of the structure-response-based factor F_{DM} and of the spectral acceleration $S_{a,yld}$. This improved SDHA methodology leads to the direct computation of seismic hazard curves for the structural damage DM of interest.

It is important to recognize that the probability of damage and, eventually, failure of a structure is directly computed without using any safety factor. Moreover, it has to be stressed also that this methodology is not tied to any particular type of structure but it can be applied to any structural engineering system.

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International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

11. Model testing, static / dynamic analysis

11.3 Review of SFT studies in Norway

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Reinertsen Engineering ANS

INTERNATIONAL CONFERENCE ON SUBMERGED FLOATING TUNNELS

SANDNES MAY 29-30 1996

REVIEW OF SFT STUDIES IN NORWAY

presented by

Prof. Tore H. Soreide

Reinertsen Engineering ANS
Trondheim, Norway

ABSTRACT

An effort has been made by the Norwegian Public Roads Administration to review previous work on dynamic response of the planned Høgsfjorden submerged floating tunnel. A specialist group was appointed, covering the major fields of dynamic load effects as well as the structure response characteristics. Emphasis has also been made to the implementation of model test results into design.

The group of experts is to come up with the evaluation of previous work and with recommendations for further studies. A report is to be completed at mid summer 1996.

INTRODUCTION

During the last decade, a substantial work on dynamics of SFT has been carried out for the Høgsfjorden project. Model tests on hydrodynamic load effects have gone in parallel with numerical studies. Different SFT concepts have been considered, both tension leg anchored and buoyant with pontoons.

The Norwegian Public Roads Administration initiated a project on quality check of the previous work. The major task was to make an evaluation of the studies on dynamic load effects, covering model tests, numerical analysis as well as the registration of environmental loading. In an effort to come up with recommendations for later design, activities were included on updating of guidelines and on analysis programs, respectively.

The document review was split into the following five activities:

- A01 Model tests
- A02 Environmental loads
- A03 Earthquake
- A04 Guidelines
- A05 Analysis programs

The activities A01, A02 and A03 handled the basic dynamic problem areas, while A04 and A05 were design oriented. Conclusions on the dynamic load effects were to be implemented in the guidelines. It also was essential that the analysis programs to be applied in design, handled all types of dynamic load effects.

The following group of experts was established:

- A01 Prof. Odd Faltinsen, NTNU
- A02 Prof. Torgeir Moan, NTNU
- A03 Prof. Svein Remseth, NTNU
- A04 Sen. Eng. Carl Hansvold, Johs. Holt AS
- A05 Sen. Eng. Einar Landet, DNV

The reference group has not yet completed its evaluation, however, some preliminary conclusions on the need for further work are listed in the subsequent chapters.

ACTIVITY A01: MODEL TESTS

Tests on section models have gone in parallel with measurements on full length flexible models, see Figs.1 and 2. The effects of tension legs and of pontoons are studied. From the tests there comes out that slowly varying response is present, in addition to the linear load effects.

The major recommendation from the reference group concerning model testing is that further effort is made to come up with analysis program on slowly varying response, which again is verified against a new set of model tests.

ACTIVITY A02: REGISTRATION OF ENVIRONMENTAL LOADS

An evaluation has been made on previous work covering load effects from wind, waves, internal waves, current, tide, water density and temperature. For design purpose, the correlation between wave and current has also been paied attention.

Based on the present document review the main conclusion is that further work is needed on the design wind velocities, where measurements on the near by Sola airport are considered. In parallel, the wind generated 100-year wave also should be updated.

ACTIVITY A03: EARTHQUAKE

The previous studies on earthquake for the Høgsfjorden SFT came up with response spectra, and analyses were performed, proving earthquake not to be governing in design. First order wave loading was found to be more critical.

The reference group recommends long periodic excitation to be taken into account, since both concepts for SFT show natural periods above 5 seconds. Further analyses are to be made in time domain, taking into account variation in excitations between basements as well as actual geotechnical data.

ACTIVITY A04: GUIDELINES

A proposal on Design Guidelines for the Høgsfjorden SFT was established in 1990, specifying loads as well as safety levels. There is a general need to update the document, especially on the consistency with the official authority regulations.

The socalled tolerance load in design, as a free load in direction and space, is to be considered in view of safety levels for weight and buoyancy. The weight increase due to water absorption is also to be included. Special care is to be made in the combination of environmental load effects, concerning correlation and whether superposition of base load cases can be used as a rational scheme for design.

ACTIVITY A05: ANALYSIS PROGRAMS

In the previous studies on SFT concepts, different analysis programs have been applied. There is now a need for establishing a design oriented program package, covering all phases of analyses, see Fig.3. It is also essential that in the run of practical design, the analysis program is available for the design engineers, not only for the specialists on dynamics. Output from the analysis should be in a form that easily can be taken into the capacity control.

In the design of a pioneering concept like the first SFT, there will be a special requirement on an independent verification, see Fig.4. For major stages in design, such as dynamic response analysis, the verification is to be based on independent calculations, possibly supplied by model tests.

The most rational analysis scheme is to let one program system handle all stages of design, namely fabrication and installation as well as completed system. The structure configuration and load pattern change during execution, making the need for a flexible program system. The reference group foresee problems with obtaining one general program handling all load effects. As discussed in the above chapters, special purpose programs are needed.

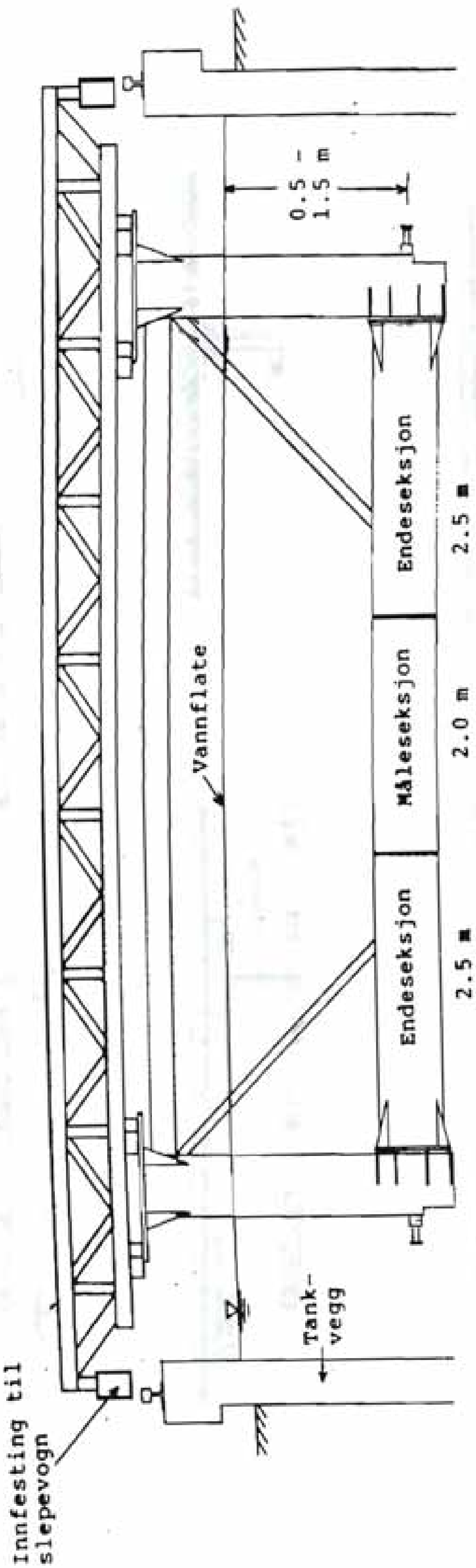

Figure 1: Section model

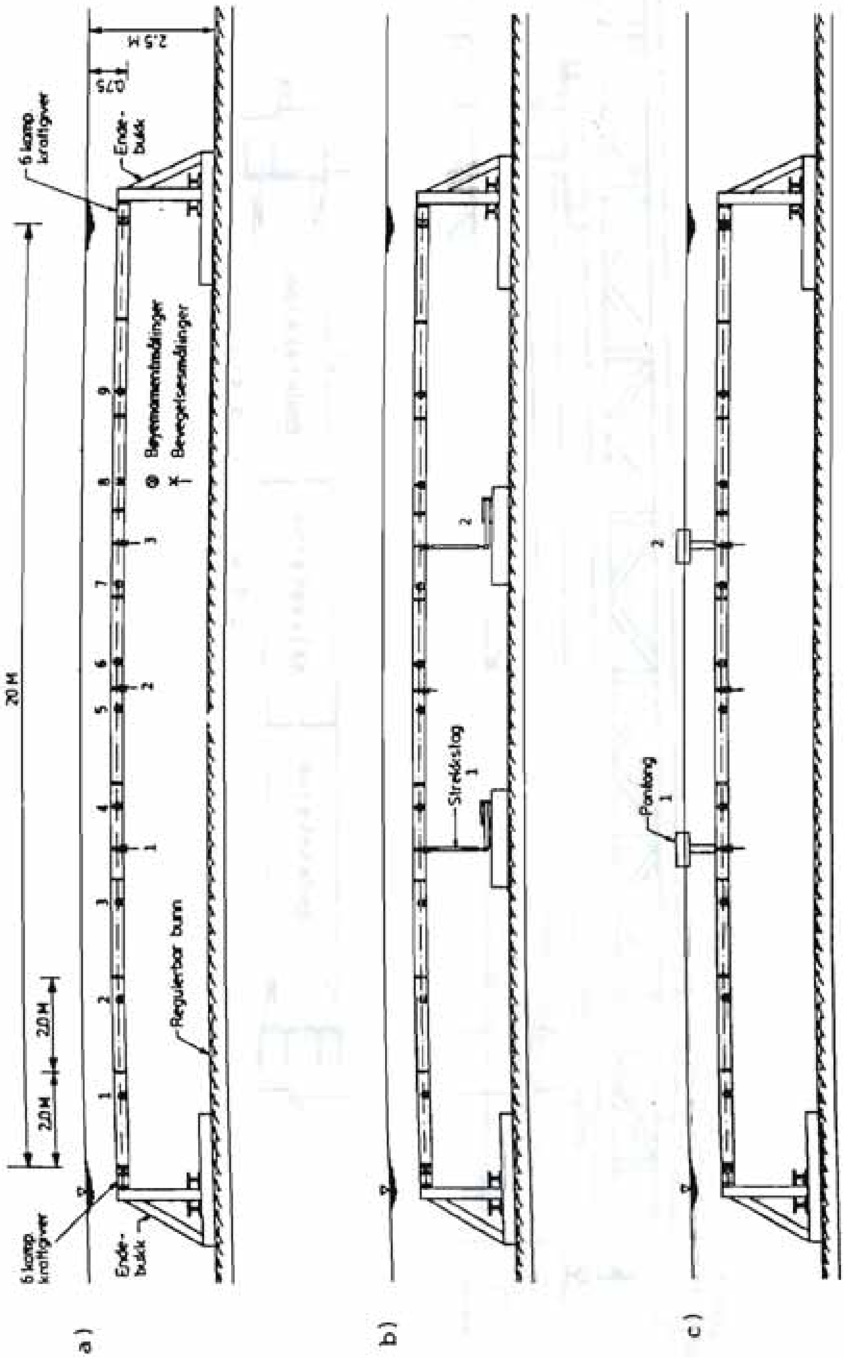

Figure 2: Flexible model

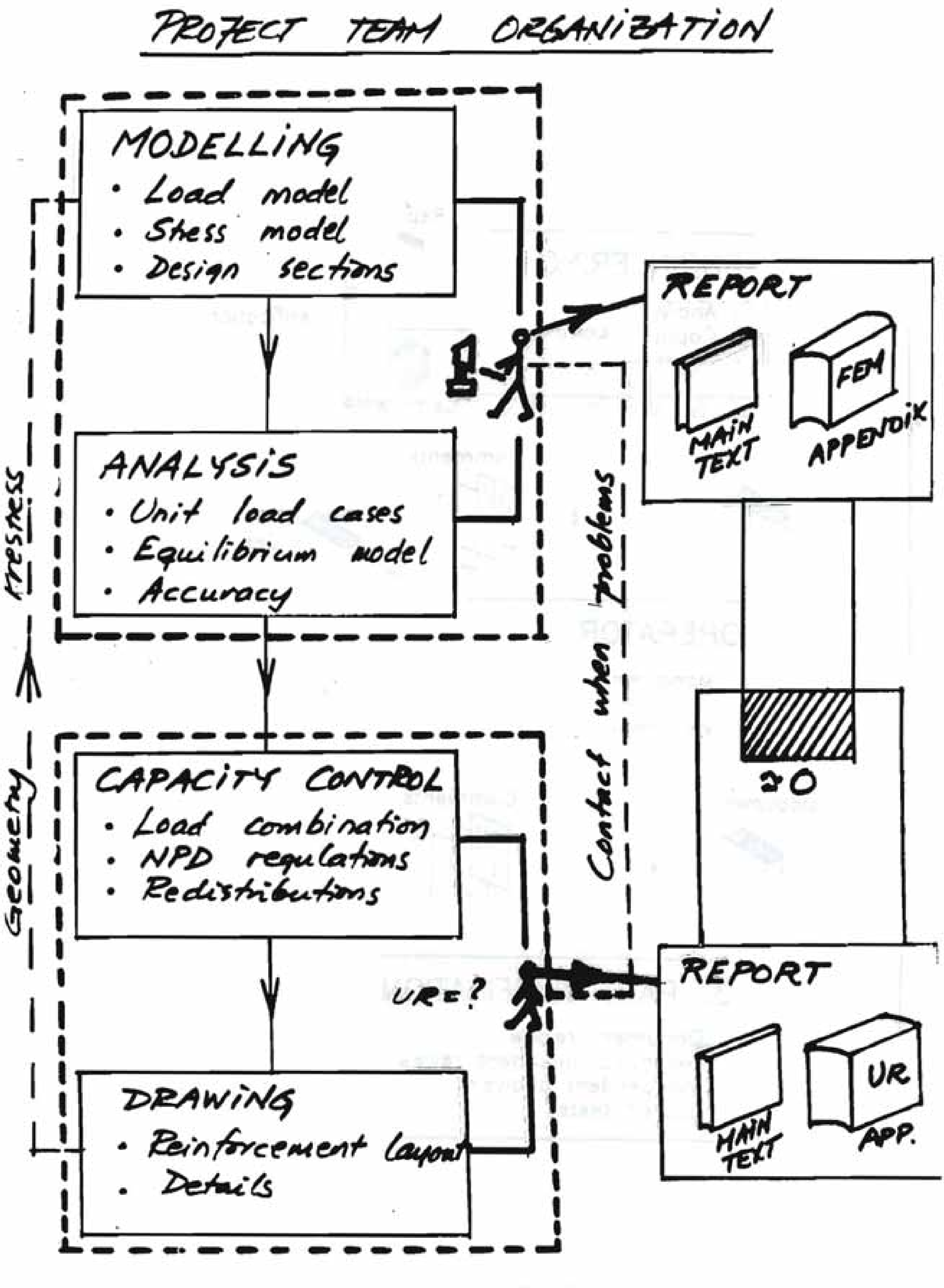

Figure 3: Design activities

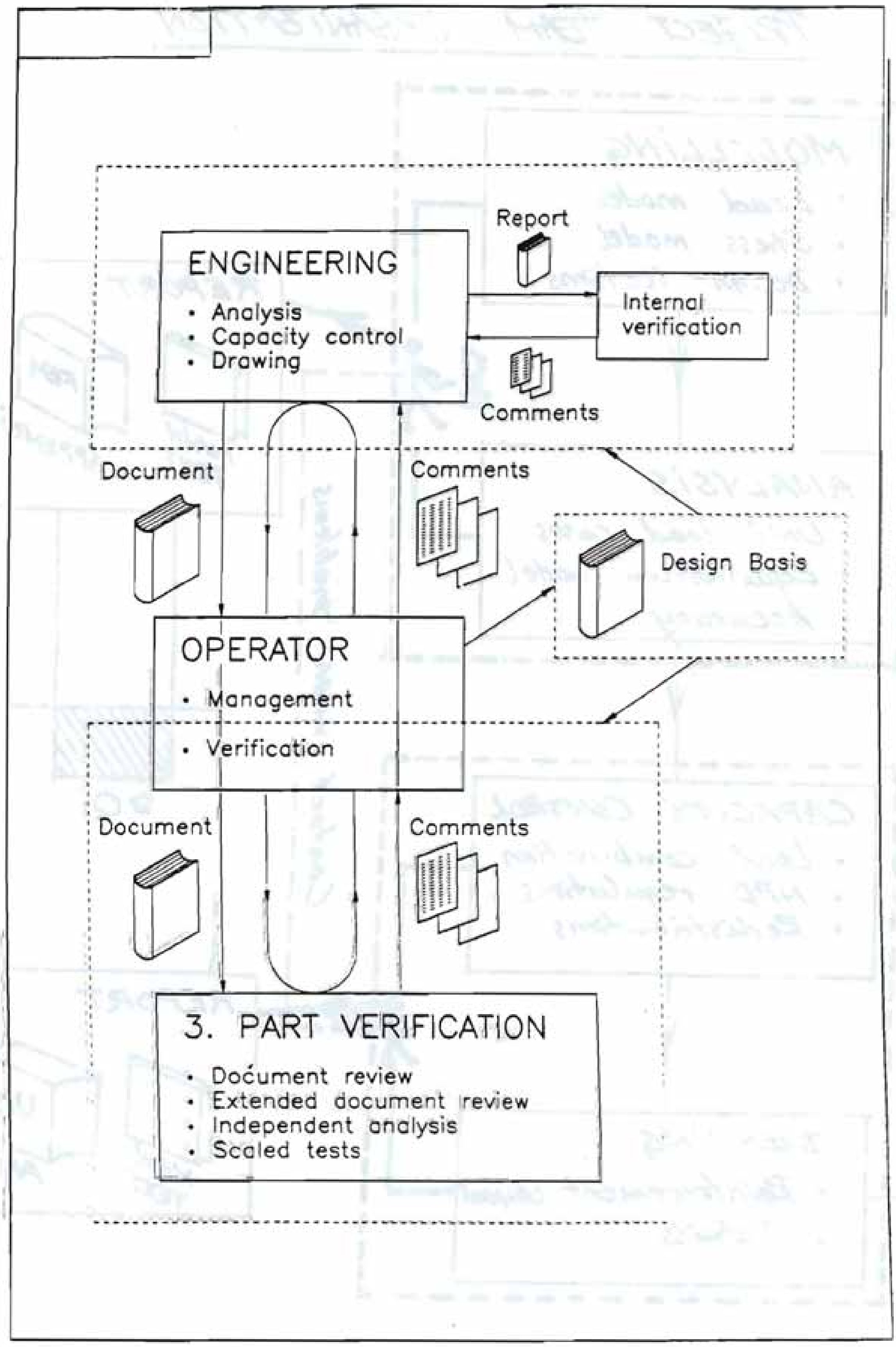

Figure 4: Engineering and verification

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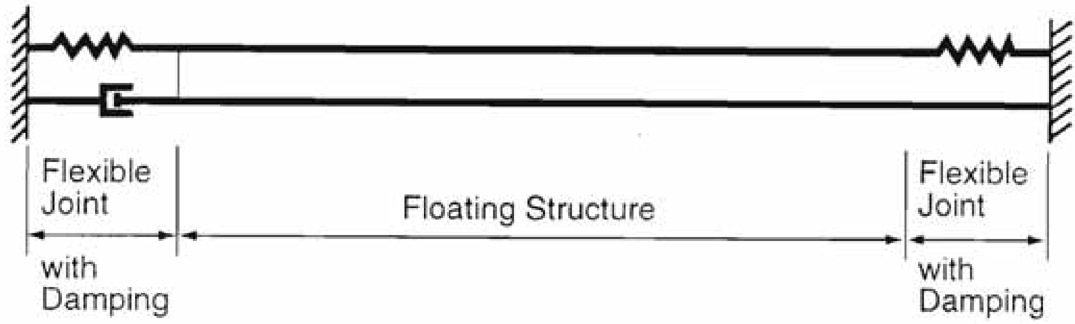
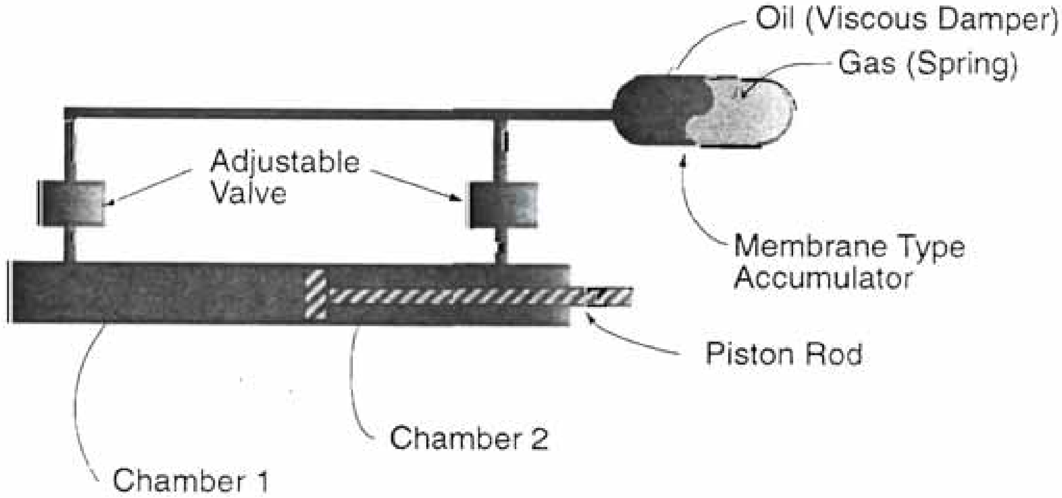
12. Constructional details



Idealized Model



Schematic Representation

Hydropneumatic Support System – (Figure 7)

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12. Constructional details

12.1 Land connections for Submerged Floating Tunnels

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SUBMERGED FLOATING TUNNELS

Sandnes, Norway

LAND CONNECTIONS FOR SUBMERGED FLOATING TUNNELS

by Ahmet Gursoy, Parsons Brinckerhoff International

Abstract

This paper examines design options available for the construction of transition and landfill tunnels connecting the submerged floating tunnels and the land facilities of a given deep-sea crossing. It examines the basic features of such transition tunnels and provides examples from the recent past.

General Considerations

Before entering into the design issues, I would like to repeat the theme of an article I read recently on the topic of outer space (interstellar) construction. The article states that construction in the outer space environment is less complicated than most people think. Because of the absence of gravity, friction and related tolerances, assembling items for construction in outer space is rather simple -- provided that the difficulties in exiting and re-entering the Earth's atmosphere for transporting the parts are resolved. I believe the same phenomena exists for the realization of submerged floating tunnel crossings. Once the logistics for the transition tunnels are resolved the basic design features of the submerged floating tunnels are rather straightforward.

Offshore construction in the North Sea has made significant contributions toward the advancement of deep-sea platform construction in Norway and other participating countries. In the USA the Auger tension leg platform in Gulf of Mexico by Shell-Oil Company of Houston, Texas received the most outstanding Civil Engineering Achievement Award of 1995, the third such award given to offshore platforms in the last twenty years by the American Society of Civil Engineers.

Recently there have been significant planning activities taking place in the field of floating tunnels in Norway, Italy and Japan. Starting in 1986 three strait crossings symposiums held in Norway (1986, 1990, 1994) organized by Public Road Administrations of Norway contained a number of publications related to submerged floating tunnels.

In 1993 ENI Consortium of Italy submitted three separate alignments of submerged floating tunnel options against the proposed bridge crossing for the Straits of Messina. The proposed tunnel crossing consists of a two-track, 5.2-km-long rail tunnel and two separate two-lane (5.8 km and 6.2 km long) roadway tunnels.

Earlier a preconceptual study by Parsons Brinckerhoff contained a proposed submerged floating tunnel and immersed tube tunnel combination for the Gibraltar Crossing totaling 22 km in length. This proposal, however, was rejected by the Spanish and Moroccan authorities in 1995.

Instead a 42.5-km-long bored tunnel rail crossing was announced by the Spanish-Moroccan authorities, serving both conventional rail and road traffic with roll-on roll-off shuttle trains. The tunnel portion of the proposed crossing is 39 km long with 28 km under the Straits of Gibraltar. The tunnel profile has average water depth of 300 meters with 100 meters cover.

Transition tunnels proposed for the Messina Strait and Gibraltar Crossings served important buffer zone functions between the submerged floating tunnels and the landfall tunnel. Design features of the transition tunnel elements will vary depending on the site conditions and the operating requirements. In the case of Messina Strait Crossing, design features include a combination of flexibility and restraint between the two different tunnel systems to satisfy the needs of normal operations as well as the site's unique environmental conditions, including the ability to withstand a 10,000-year earthquake event.

The site conditions and the inherently different constraints offered by the three differing types of tunnel construction for the Messina Strait Crossing dictated the utilization of two different transition tunnels, one between bored tunnels and immersed (landfall) tunnels and the other between floating and immersed tunnels. The total lengths of the landfall tunnels (transitions plus immersed tube tunnels) are 450 meters for the rail crossing and 400 to 600 meters for the two vehicular crossings.

The study performed by Parsons Brinckerhoff for the Gibraltar Crossing rail tunnel proposed two 5-km-long immersed tube tunnels at both ends of the 12-km-long submerged floating tunnels, totaling 22 km in length. For the vehicular crossing, transition tunnels were combined with the ventilation buildings located at the ends of submerged floating tunnel. Cross sections for the floating tunnels of the Gibraltar Crossing were composed of three circular shell plates. The outside shell plate had a concrete protection layer. The two inner plates contained 50-cm-thick reinforced concrete ring slabs. The three shell plates were stiffened by diaphragms and longitudinal stiffeners. The outer space between first and second plates contained compartments protecting against ship collision. The cross sections of immersed tubes for landfall tunnels were typical of double shell construction practiced in USA, with an octagonal outside form plate and a circular inner shell plate with reinforced concrete ring working compositely. The outer (form) plates and the inner shell plates were stiffened by transverse diaphragms and longitudinal stiffeners along the circumference of the shell plates. Pockets between form plates and inner shell plates were filled with tremie concrete as ballast.

In contrast the cross sections of the floating tunnels and the immersed landfall tunnels for Messina Strait Crossing are circular and very much alike, except that the landfall tunnels have a bottom template added to provide a foundation base for the circular cross section of tunnel elements to be supported on the gravel foundation. The foundation template also provides sleeves for pile connections to sustain the environmental and operational loads associated with currents, wave actions, earthquake, and longitudinal traction forces generated by the rail operation (see Figure 5). To overcome the varying nature of the loads mentioned above, two different flexible joints at two transition tunnel areas are proposed in the Messina Straits Crossing.

Parsons Brinckerhoff (PB) was responsible for the conceptual development of the design and for the construction staging and detailing of the joints, the transition tunnels, and landfall tunnels. PB also designed the bulkhead systems of the entire crossing.

In the following sections the basic engineering features of the landfall and transition tunnels including their one-of-a-kind flexible joints, are presented.

Special Features of the Messina Strait Crossing Transition Tunnels and Landfall Tunnels

The landfall tunnels at each end of the floating tunnel are comprised of three distinct tunnel elements:

- The Offshore Tunnel Interface Element (OTIE) is the first transition element connecting the landfall tunnels to the floating tunnels. It is 60 meters long and is supported by six pairs of piles. This element serves as an anchor to the series of landfall elements and functions as the end anchor of the floating tunnels. For this purpose, the OTIE contains a specially designed transition module to receive Flexible Joint "B," (described below) which also receives and retains the seismic loads.
- The Landfall Tunnel Interface Element (LTIE), the second transition element, connects the immersed tube tunnel with the land rock (bored) tunnel. The LTIE on the Calabria side contains the construction access shaft for the equipment to be used in interior work for the entire crossing. Both LTIEs are pile supported and are connected to the adjacent rock tunnel by means of a specially designed module called Flexible Joint "A."
- Intermediate Tunnel Elements (ITE) are the immersed tube tunnels located between the transition elements of OTIE and the LTIEs.

ITEs consist of three steel shell plates. The outer shell is 35 mm thick and stiffened by steel diaphragms and tee stiffeners. The diaphragms are 25-mm thick steel plates spaced at 3 meters on centers. The tee stiffeners are spaced at 0.75 meters on centers. The middle shell plate and the interior shell plate are 25 mm thick and stiffened by diaphragms and stiffeners and contain a 500-mm-thick reinforced concrete ring.

Flexible Joints

It is important to recognize that the fundamental difference in the support systems of the floating and landfall tunnels is that the two tunnel systems respond differently to seismic loads, ground motion, and to the loads associated with long-term operations. In addition, some of the loads acting on the two tunnel systems are different in nature. For example, the land tunnels are subjected to static and earthquake loads generated by dynamic soil pressures, while the floating tunnel is subjected to sea currents, earthquake motion transmitted from the ground through the tethers, and relatively large thermal expansion/contraction. Therefore, it is necessary to isolate the two tunnel systems from each other and from the rock tunnels by means of specially designed tunnel elements and joints that allow articulation between the connected segments in certain direction and transfer of load (reaction) in others.

For the Messina Strait Crossing, two such joints, Flexible Joint A and Flexible Joint B, are provided for this purpose. Both joints are designed to meet specific functional requirements of relative displacement and/or load transfer between the adjacent tunnel segments arising from normal service environmental conditions, as well as from a 10,000-year seismic event. For the Messian Strait project, the joints are required to have a service life of 200 years.

Flexible Joint A, in concept , is very similar to the seismic joint designed by PB in 1968 for the BART system in San Francisco. Joint A will not transmit any loads and can move longitudinally by 0.2 meters and transversely by 0.3 meters to satisfy environmental and operational conditions, including the 10,000-year earthquake event. The joint is designed to allow relative movement in six degrees of freedom (translation along and rotation about the longitudinal, vertical, and transverse horizontal axes). Joint A (Figures 1, 2 and 6) consists of a fixed ring and a moveable collar inserted between the intermediate tunnel and the transition tunnel element (LTIE) that connects to the rock tunnels.

The fixed ring and moveable collar provide the vertical surfaces on which the adjoining elements can move transversely. The fixed ring and collar are held together by means of wire ropes that allow the required transverse movements between adjoining elements. The ropes are tensioned to compress the gaskets between the two surfaces to the desired pressure. In addition, the LTIE slides longitudinally within the moveable collar. In order to inspect and maintain the wire ropes and allow for recompression of the vertical face gaskets, the fixed ring and moveable collar are made large enough to allow access through watertight manholes (see Figure 6). To ensure watertightness, a series of elastomeric gaskets will be installed around the circumference of the moving segment of the landfall tunnel element. Another set of gaskets is mounted on the vertical face of the moveable collar. The fixed ring and movable collar have outside diameters of 22.6 meters. An earth-retaining structure is built around the tunnel just behind the moveable collar to prevent backfill from exerting pressure onto the collar.

Flexible Joint B was developed to allow for longitudinal movement and angular changes along the axis of tunnel and to take loads resulting from operational and environmental conditions, including the 10,000-year earthquake event. The design had to satisfy the following conditions:

- In the axial (longitudinal) direction, the joint acts as a rigid connection for axial loads up to 60,000 kN (13,500 kips). For axial forces in excess of 60,000 kN and up to 160,000 kN (36,000 kips), the joint acts as a spring support with viscous damping. The amplitude of motion to be accommodated by the joint acting as a spring support is a total of $\pm$ 1.5 meters, which includes the effects of operational loads, environmental loads, and the 10,000-year seismic event.
- The joint is required to transfer shear between the floating and landfall sections of the tunnel as a result of weight, currents, and seismic events. The maximum shear for which the joint has been designed is 100,000 kN (22,500 kips), controlled by the 10,000-year seismic event.
- The joint is required to allow free relative rotation of the tunnel axis with a design value of up to 0.01 radian (0.6°). Torsional relative rotation between the floating and landfall tunnel sections is expected to be negligibly small.

Service conditions require that watertightness be maintained under all conditions for 200 years.

Flexible Joint B (Figures 3 and 4) is made up of a fixed collar attached to the landfall side, within which the tunnel section on the floating tunnel side (moving tunnel section) can slide longitudinally and rotate. The moving tunnel section contains the gasket assemblies and the shear transfer ring. These two elements are designed to move as a unit so that they are always at the center of rotation in any position. The shear transfer ring is a steel ring lined along its contact surface with an elastomeric pad. The face of the elastomeric pad is in contact with the

Teflon-coated inner surface of the fixed collar. Due to transverse shear from dead, live, current, and earthquake loads (100,000 kN max.), the shear transfer ring will bear against the collar and transmit the shear along a portion of the circumference between the two elements. The elastomeric pad provides the means to distribute a uniform load within the angular change of 0.01 radian when such loading occurs. The gaskets, made of natural rubber with aging and creep-retarding additives, are held in place by a series of ring plates. The inner face of the collar will be faced with Teflon to facilitate sliding of the gaskets. The joint will have an outside diameter of 20 meters. The gasket assembly of Joint B will be protected from external intrusions and aquatic life and human access by means of a box ring attached to the end of the collar, hugging the moving tunnel segment with an elastomeric pad. The ring is comprised of multiple sections that are removable for maintenance purposes.

Hydropneumatic Damping System of Joint B

An important feature of Flexible Joint B is its ability to perform specific functions under various conditions encountered during construction, normal operating, and earthquake conditions:

Installation. During construction, as the floating tunnel reaches completion, Joint B at the starting (Calabria) end will be required to retract in order to compensate for the net elongation caused by the removal of the water pressure from the end bulkhead. During the construction stage, a temporary restrainer system will be placed around the joint. The hydropneumatic system will relieve the axial stresses present in the temporary joint restrainer, transferring them to the tunnel element shell plates and the foundation as the construction is completed from the Calabria to the Sicily side. After removal of the restrainer, the system will be released to allow free movement at the joint so that the floating tunnel reaches its final operating position. Proper adjustment of tunnel length will be made at this stage by controlling the volume of oil in the hydropneumatic systems.

Normal Conditions. Under normal service conditions, the Joint B at both ends will provide the necessary restraint against traction, loads, current forces, and minor earthquakes, while accommodating thermal movements. During service, the hydropneumatic system is designed to keep the tunnel in normal position by receiving the axial loads due to environmental loads such as waves, currents, and tunnel elongation contraction due to temperature variations and service loads such as traction. Under quasi-static loads, such as thermal effects, Joint B acts as a spring support allowing axial displacement accompanied by a reaction in function of the displacement.

Earthquake. Finally, in the event of major earthquakes Joint B at both ends will function as dampened spring supports for the floating tunnels. The joints are designed to allow a maximum movement of $\pm$ 1.5 meters in the longitudinal direction. In response to dynamic loads of an oscillatory nature generated by earthquake ground motion, the hydropneumatic system, which contains a spring with viscous damping, will reduce the inertial force generated by the oscillating tunnel to an acceptable level by absorbing energy and allowing relative displacement.

In order to satisfy these requirements, each Joint B will be equipped with a hydropneumatic system consisting of a group of 20 hydraulic cylinders each connected with a membrane type oil/gas accumulator. This system will function as follows during the phases identified above.

For service and environmental loads, the tunnel will act as a fixed element at one end and a flexibly supported (free) element at the other. This will be achieved by setting the threshold of operation differently at each flexible joint of the tunnel. In case of an earthquake event, both joints will respond as flexible supports with viscous damping. The damping system will include an instrumentation system capable of providing remote monitoring of the position and the inclination of each flexible joint of the tunnel. This is accomplished by remote position indication of each cylinder of the damping systems. Each cylinder will be connected to a hydraulic accumulator designed for a maximum pressure of 485 bars. The hydropneumatic system also serves to install the tunnel to its initial positioning at the time of removal of the temporary restrainer. This is accomplished by pumping hydraulic fluid as required to the accumulator. The damping system comprises 20 double-acting cylinders with minimum strokes of $\pm$ 1.5 meters.

System Characteristics

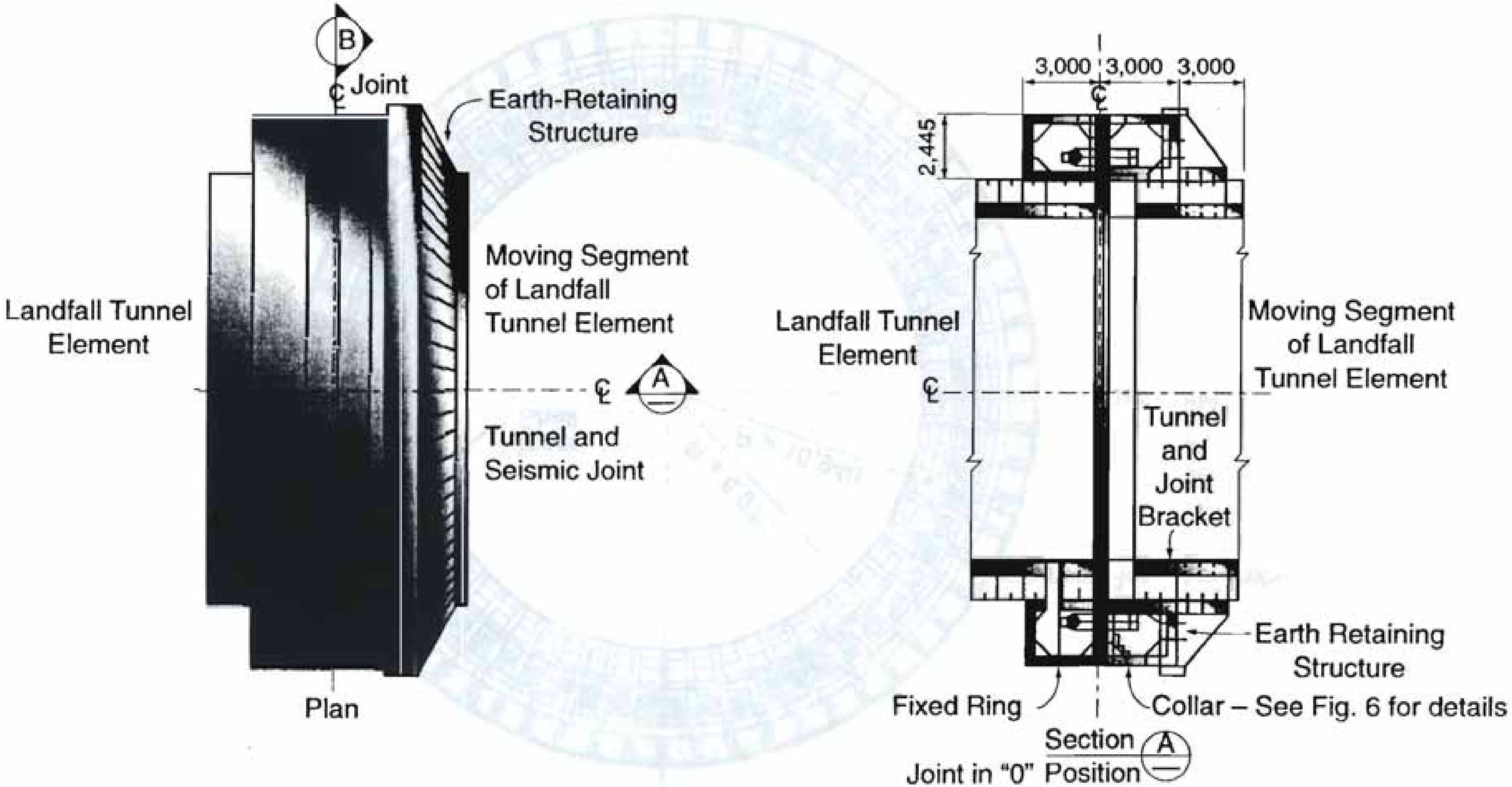
When functioning as a spring support, the hydropneumatic system exhibits two distinct characteristics depending upon the rate at which it is loaded.

Under quasi-static loads, such as thermal movements, currents, etc., the system responds essentially as a spring, reacting to the loading with a force that is solely a function of the displacement of the structure. This behavior is a result of the compression/expansion of the gas in the accumulator caused by the relative movement of the piston (connected to the floating tunnel) with respect to the cylinder (connected to the landfall tunnels). Since the flow rate of the hydraulic fluid is very low in this case, there is negligible viscous resistance or energy dissipation accompanying the spring force.

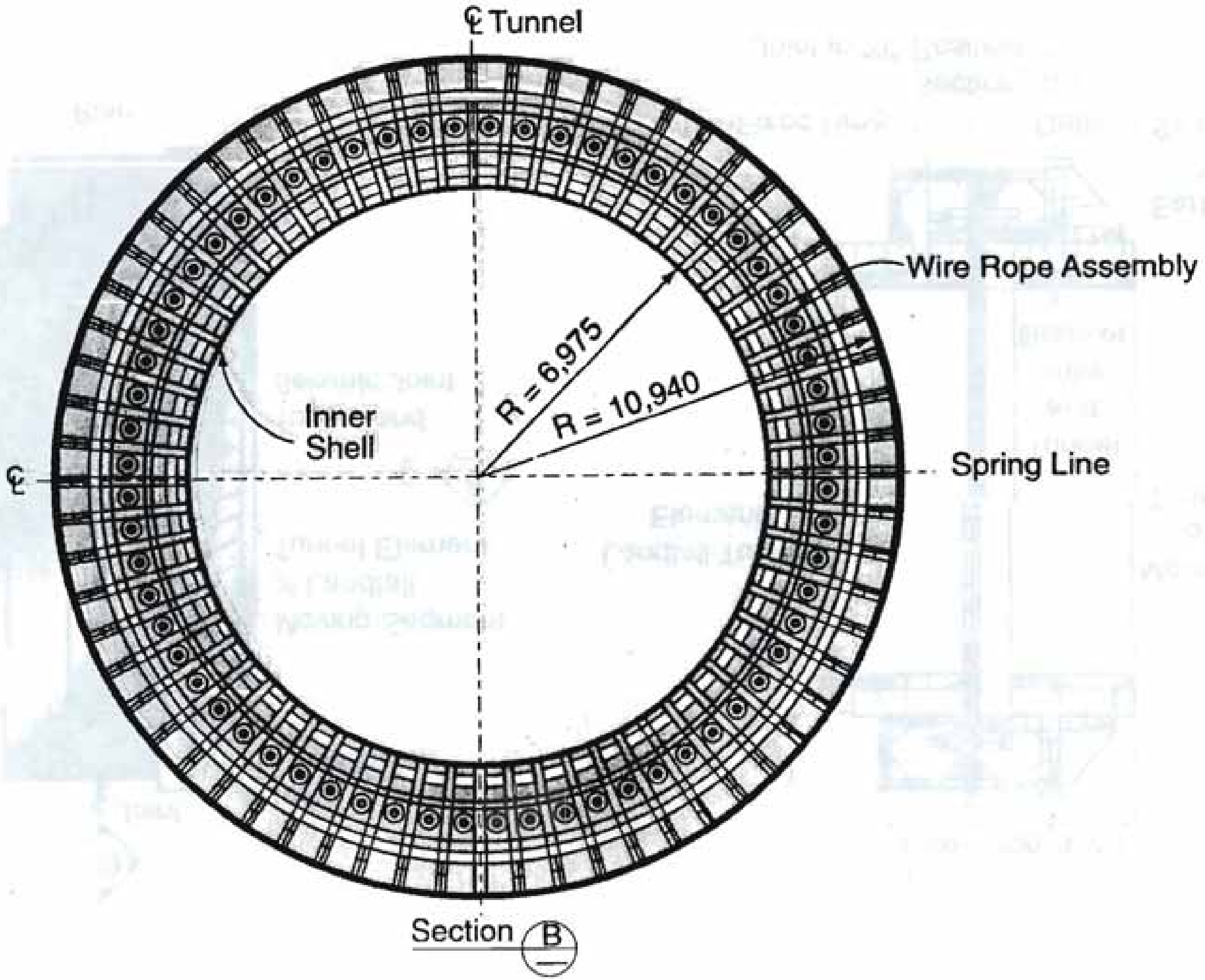
When subjected to dynamic loads, both the spring and viscous damper react to motion, the former as a function of displacement and the latter in proportion to the velocity. The overall response of the structure to the imposed dynamic loading is thus reduced due to the combined action of the spring, which consists of decoupling the structure from the ground, and the damper, which dissipates part of the energy transmitted to the structure.

Temporary Joint Restrainer Systems

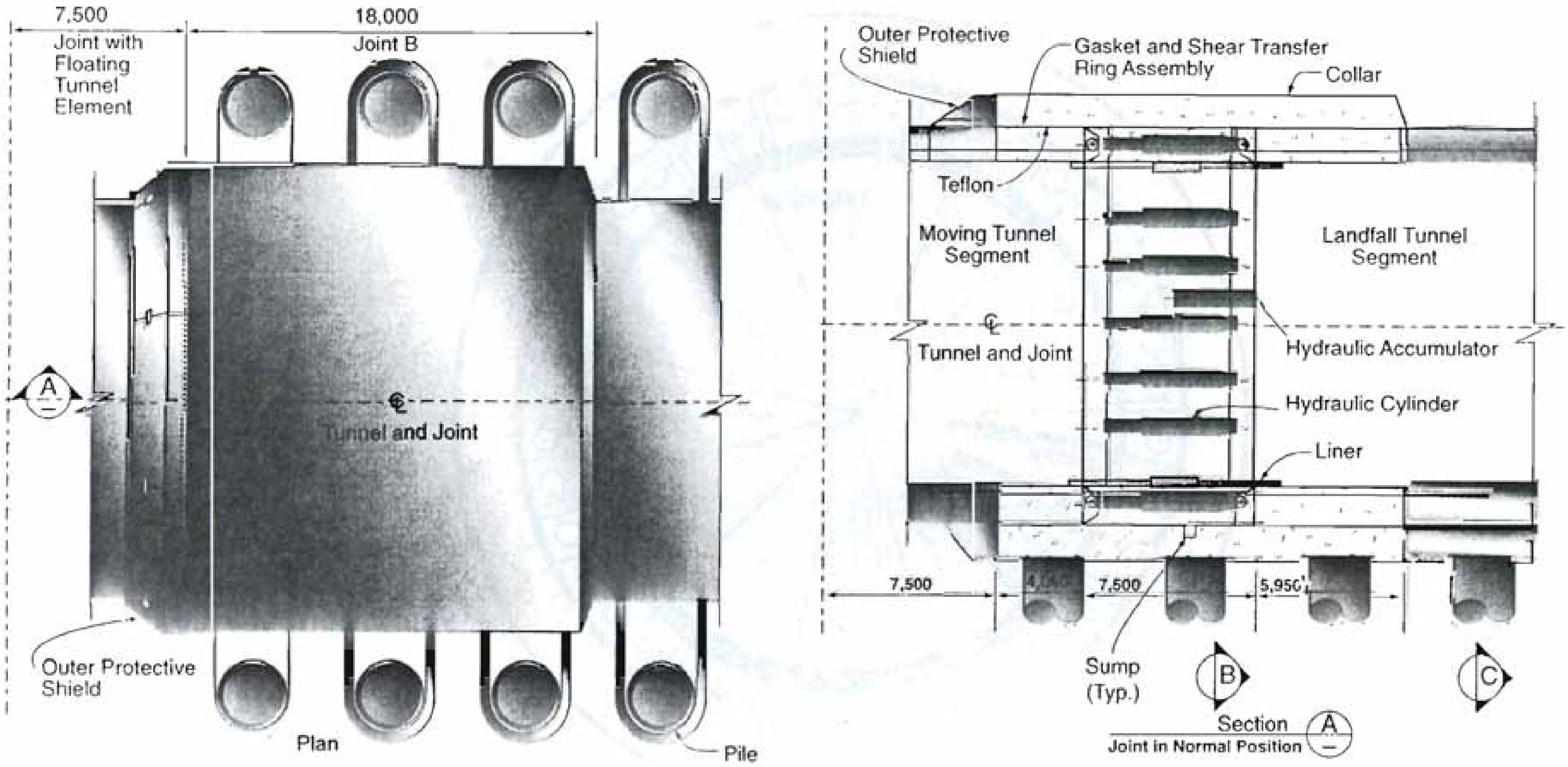
Flexible Joints A and B will be restrained during launching, installation, and joining of the tunnel elements by means of internal joint restrainer systems. The joint restrainer is designed to resist dead, hydrostatic, and other environmental loads to which the elements will be subjected during installation. The position in which the joints will be restrained will take into consideration the elongation of the tunnel at the time of removal of the end bulkhead and the ambient temperature at the time of installation. The restrainer consists of a cylindrical plate with longitudinal stiffening beams rigidly connecting the two sections of the joint.



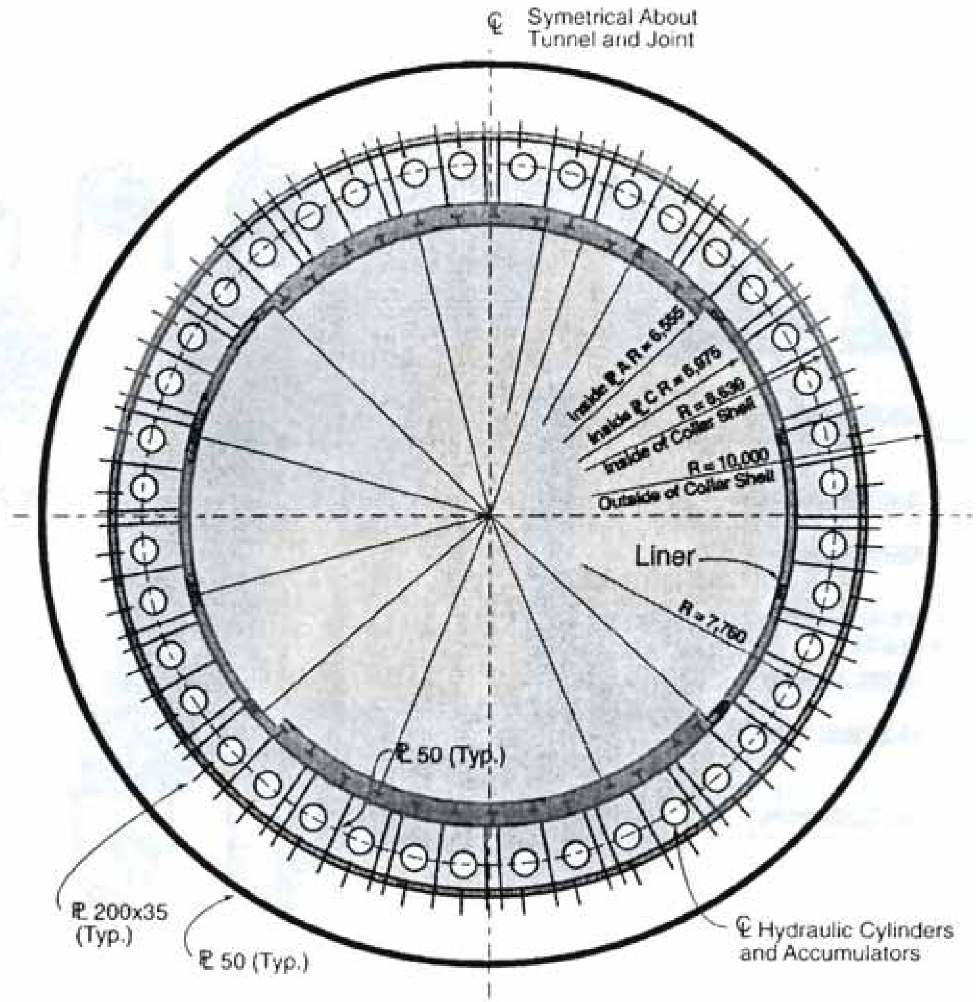
Joint A - Plan and Section (Figure 1)



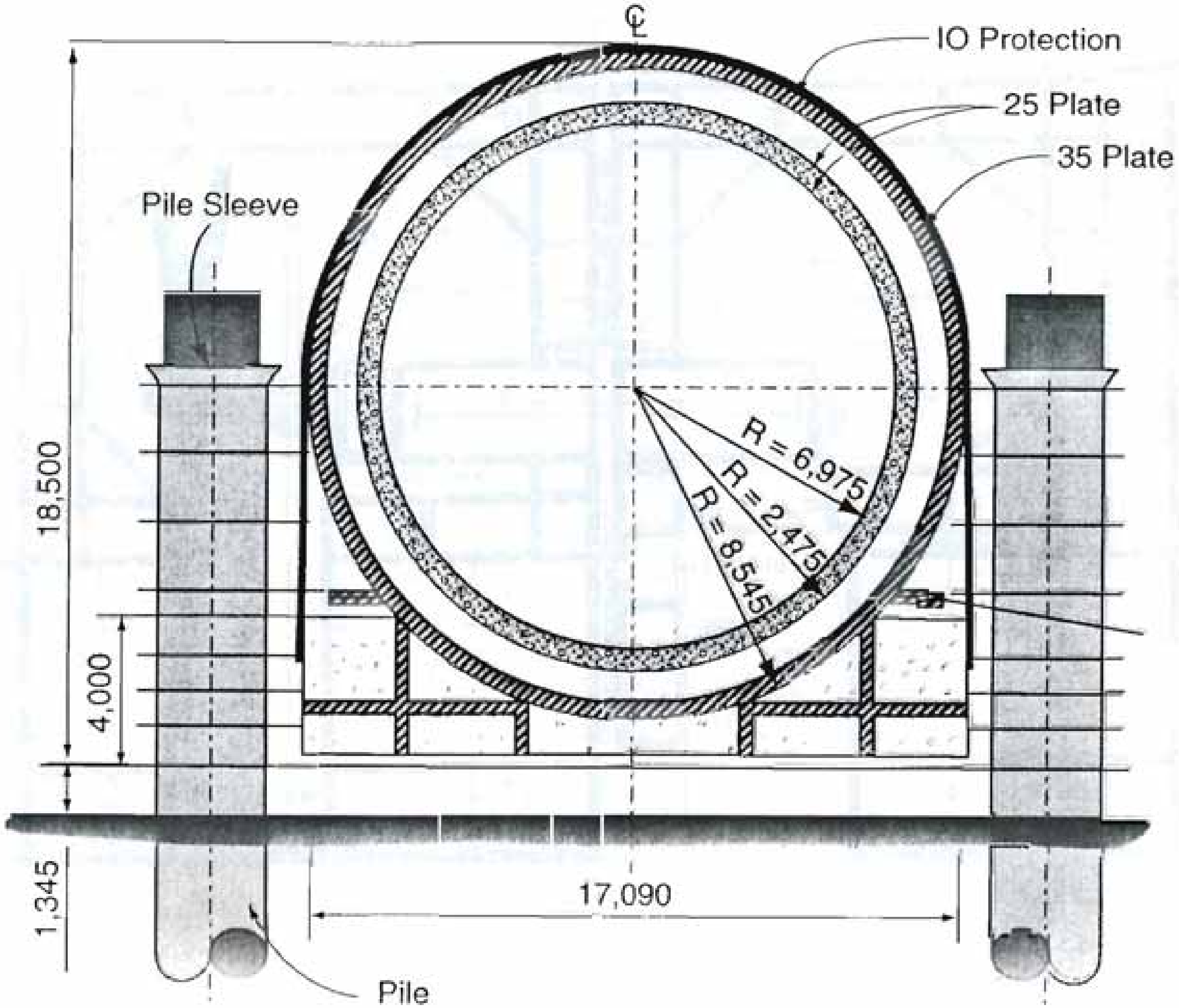
Joint A – Section B (Figure 2)



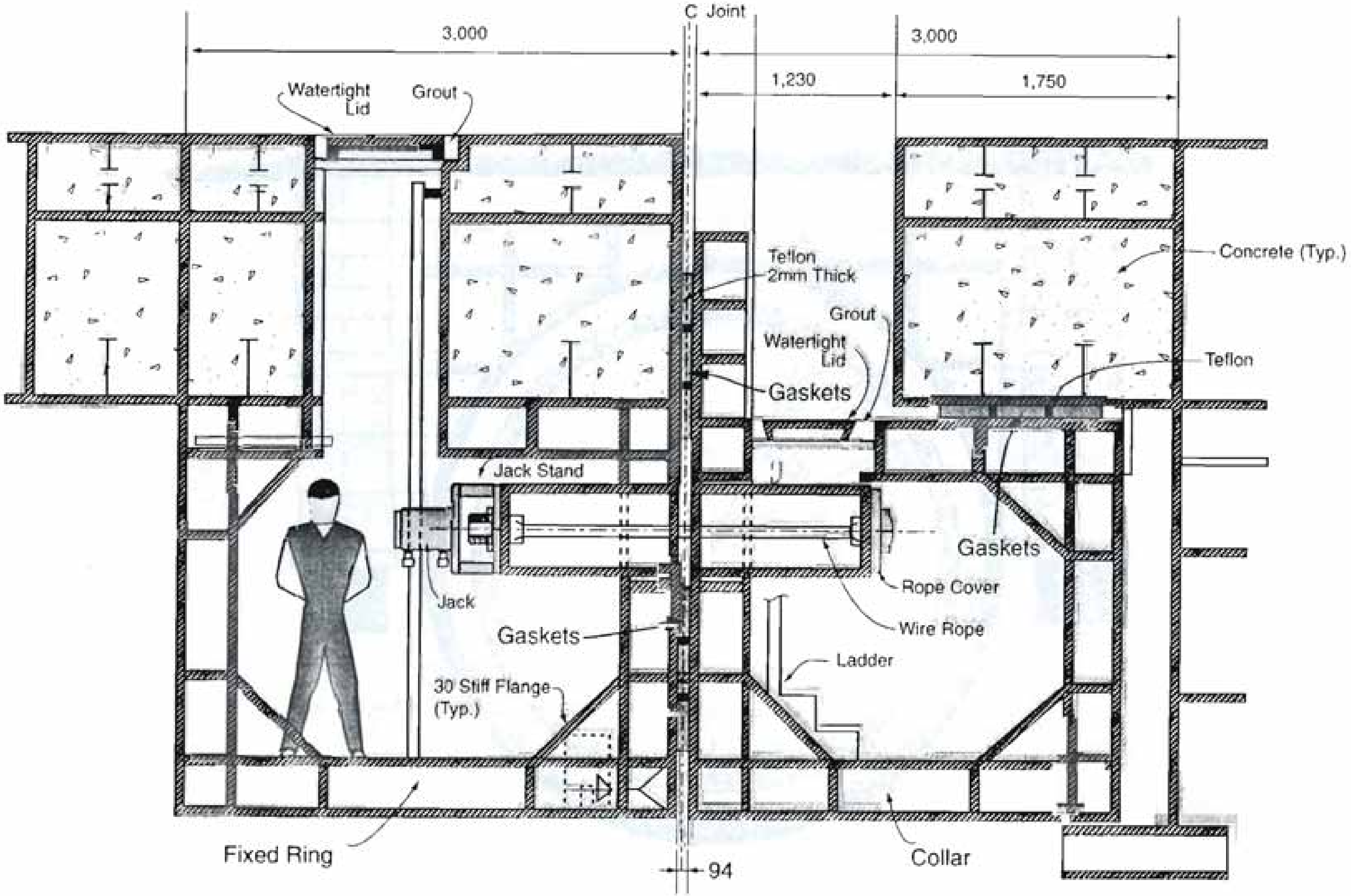
Joint B – Plan and Section (Figure 3)



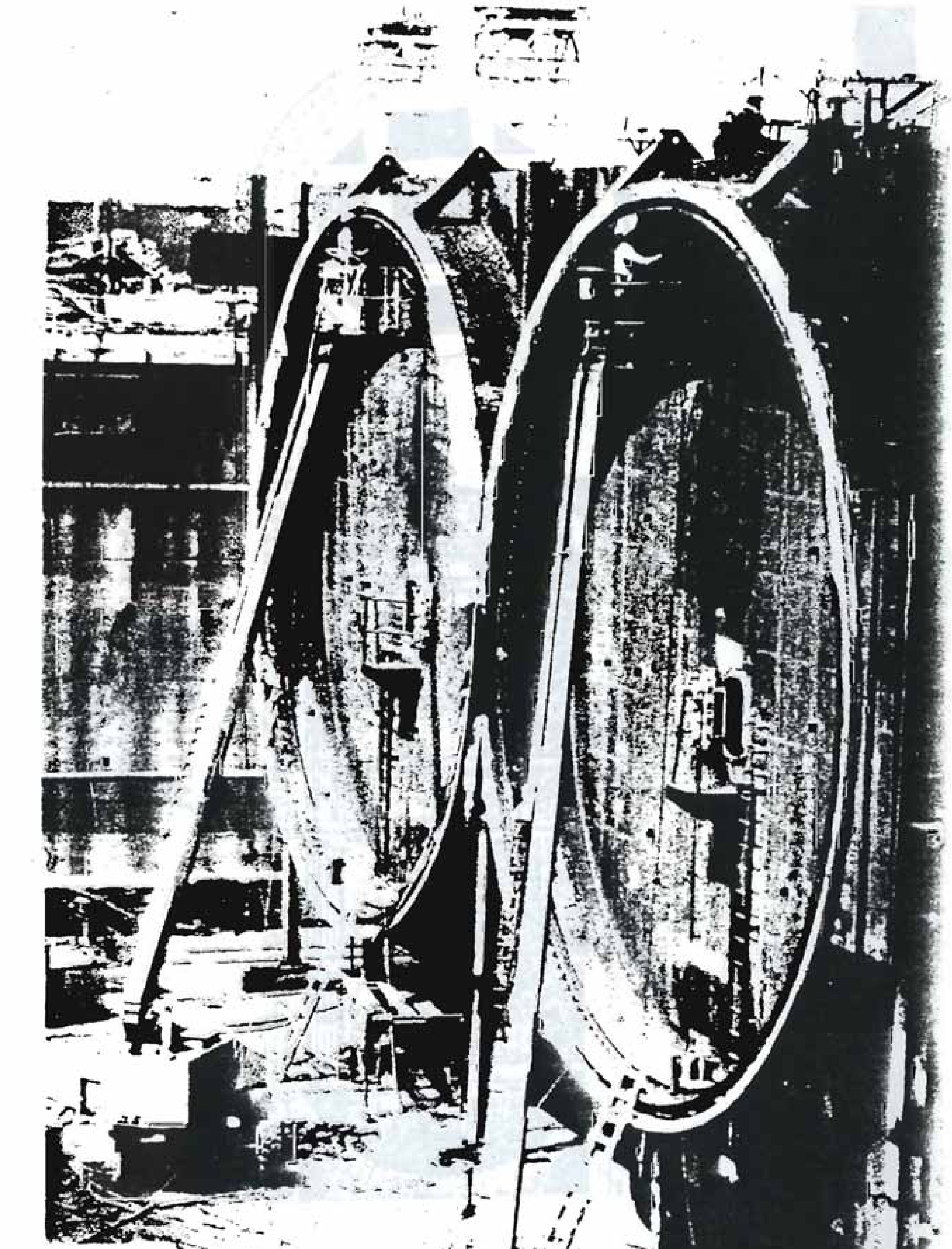
Joint B - Section B (Figure 4)

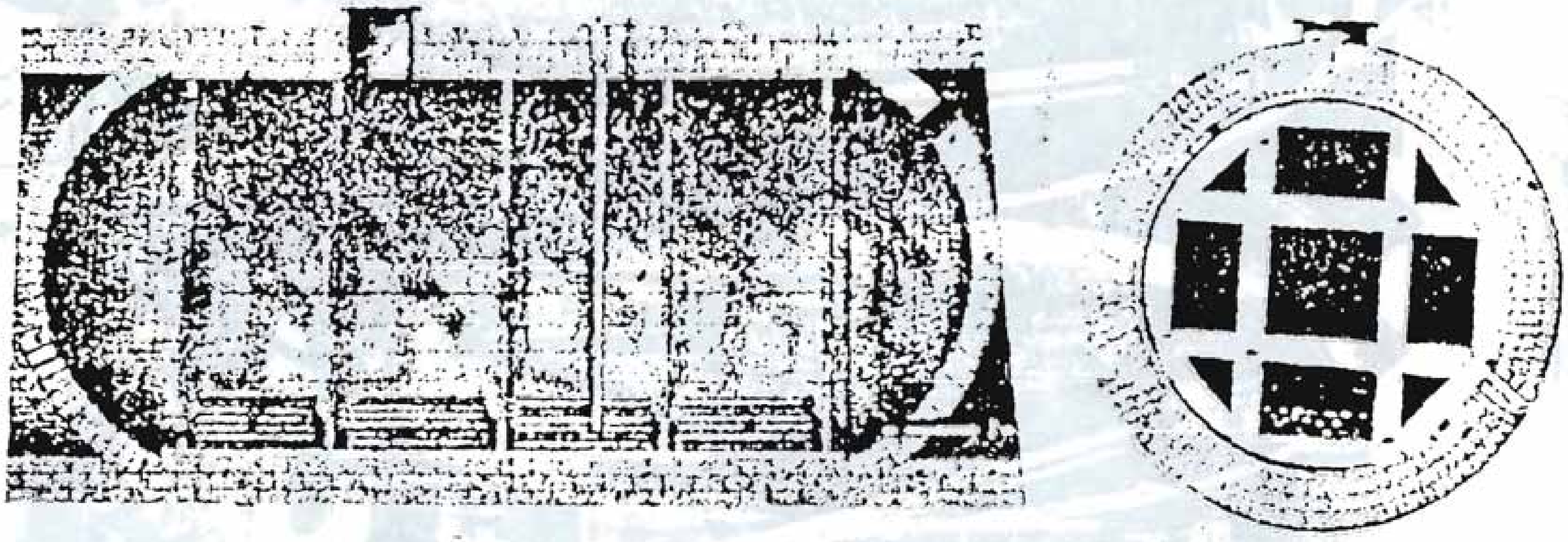


Section C (Figure 5)

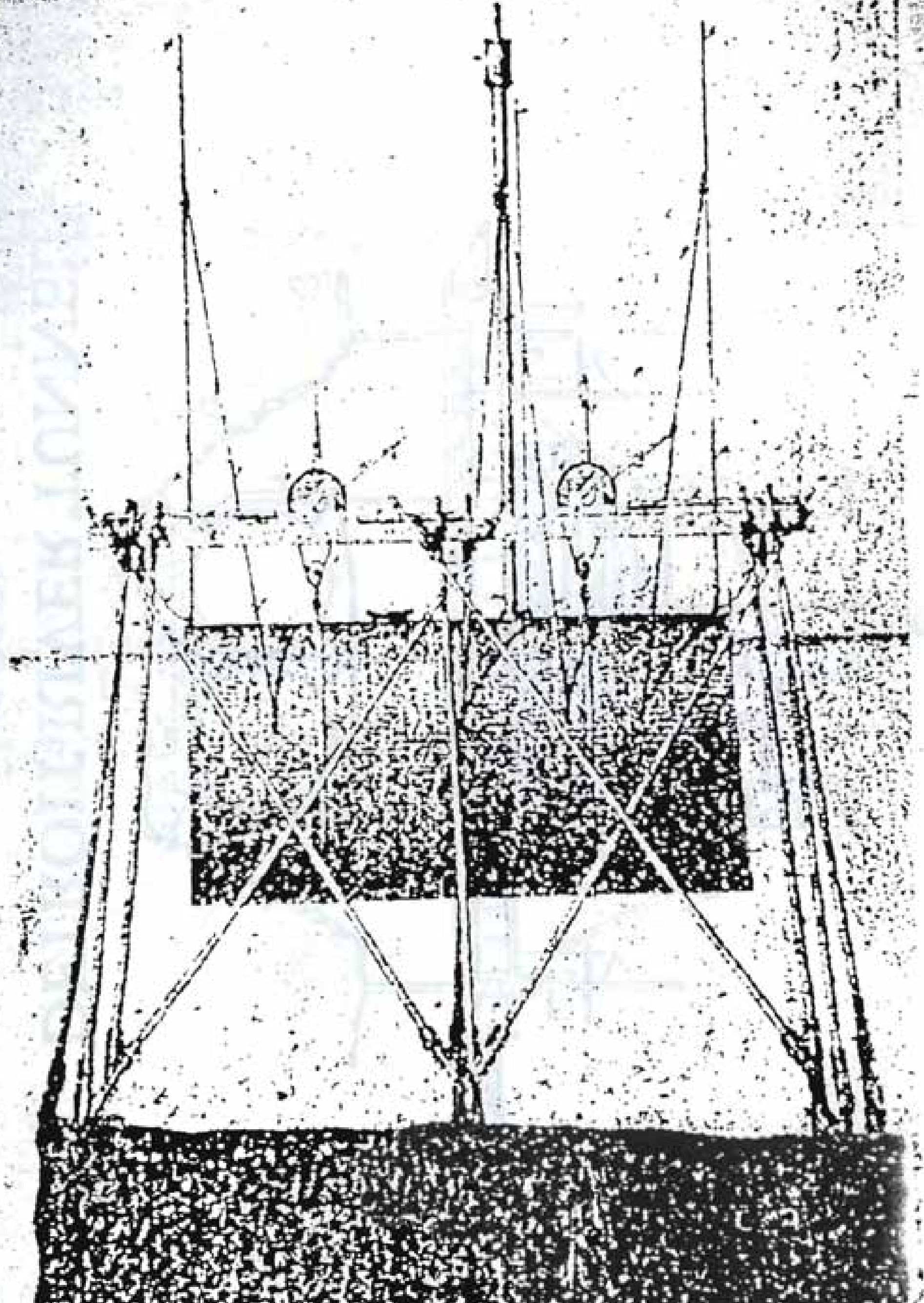


Wire Rope Assembly – Detail (Figure 6)

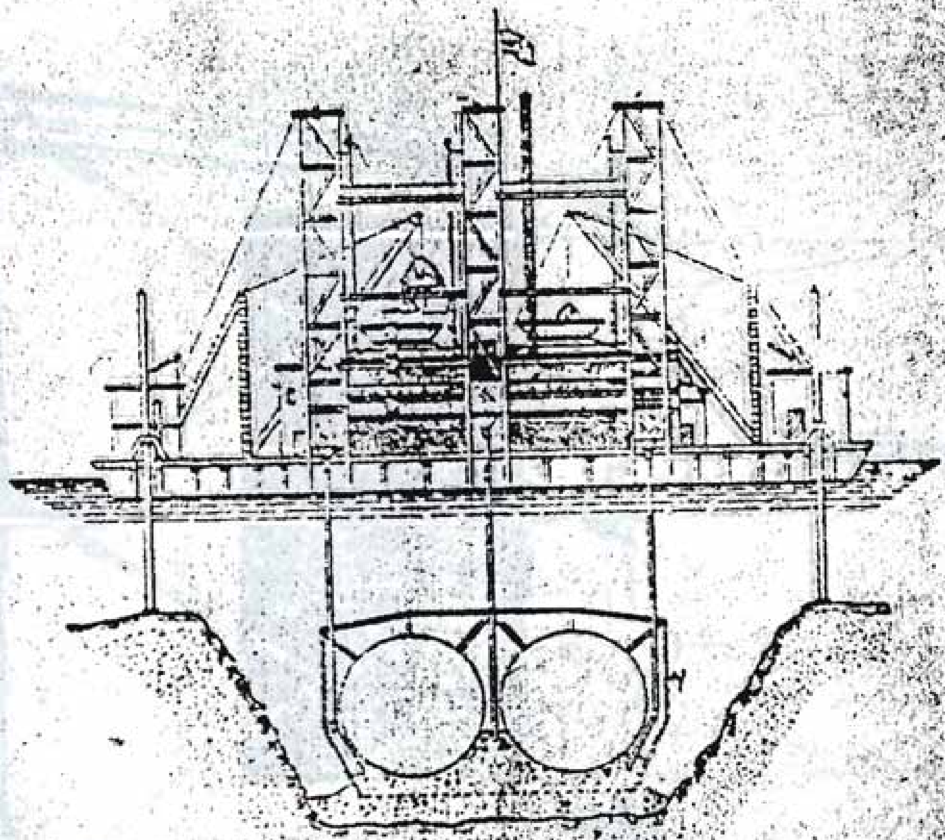




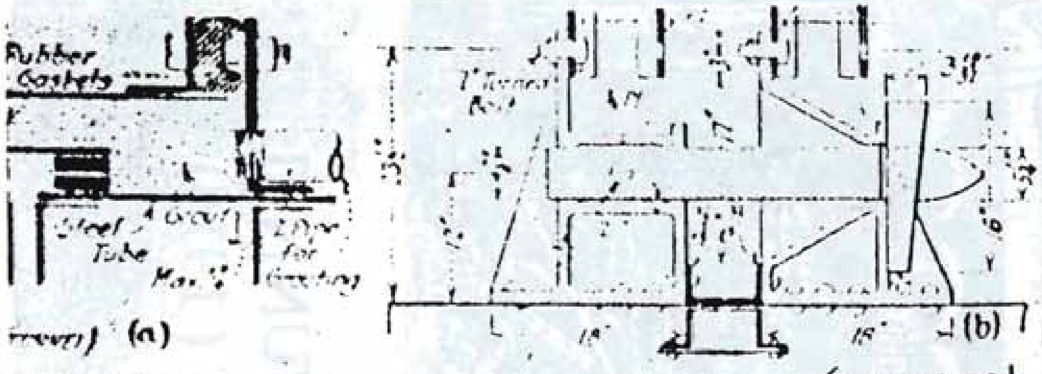
WYATT/HAWKINS
CROSS SECTIONS OF
TEST ELEMENT



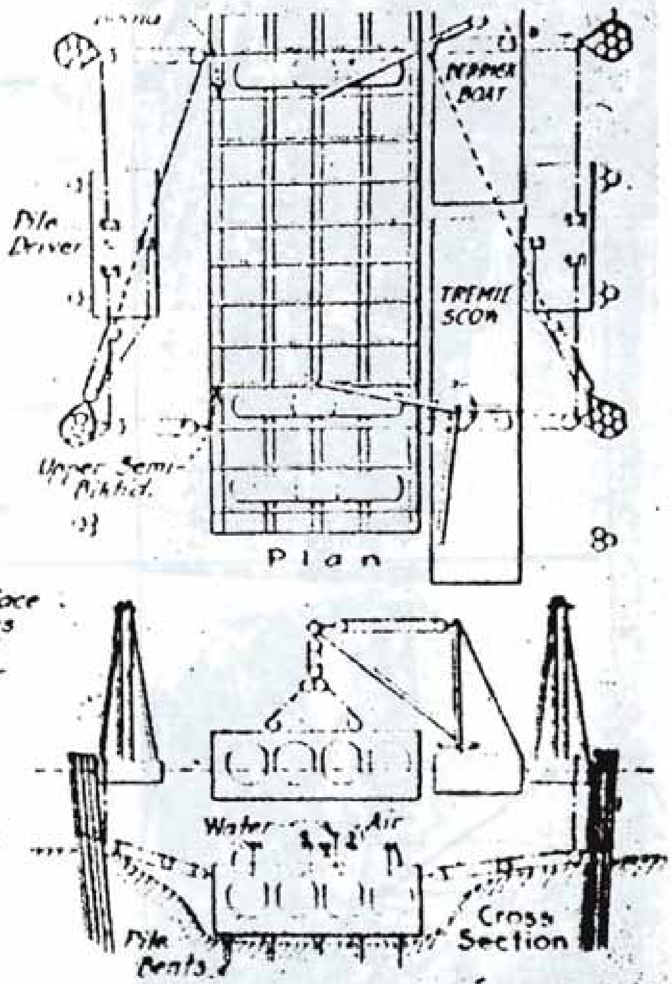
1810 WYATT/HAWKINS TEST



DETROIT RIVER TUNNEL
(1910)
Method of Construction



77-DETAILS OF PILOT PIN AND JOINT



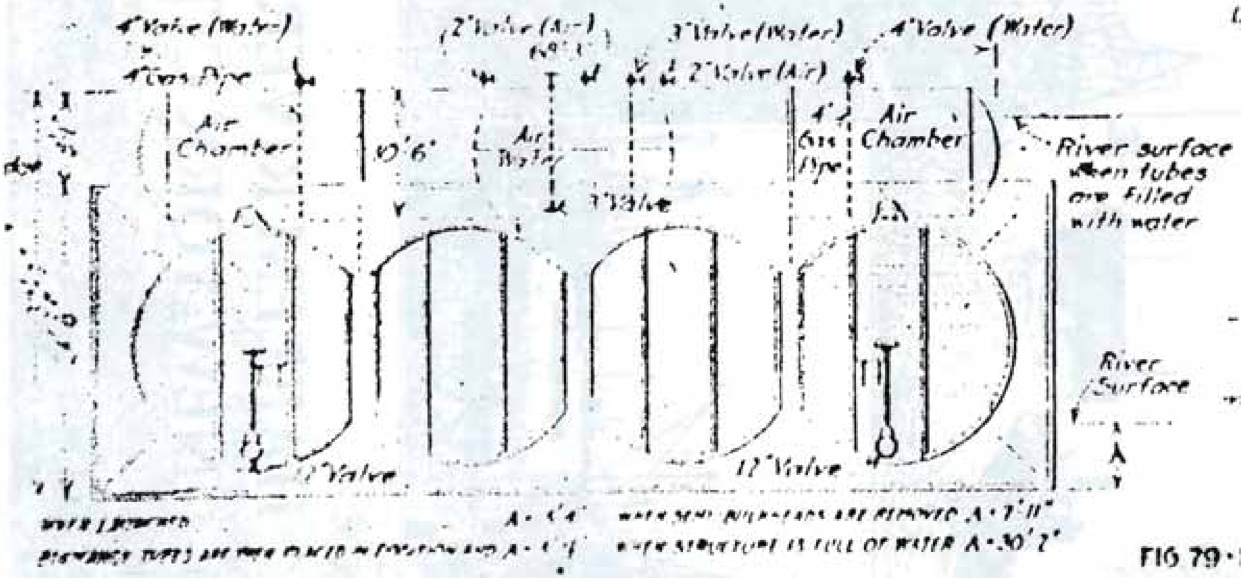
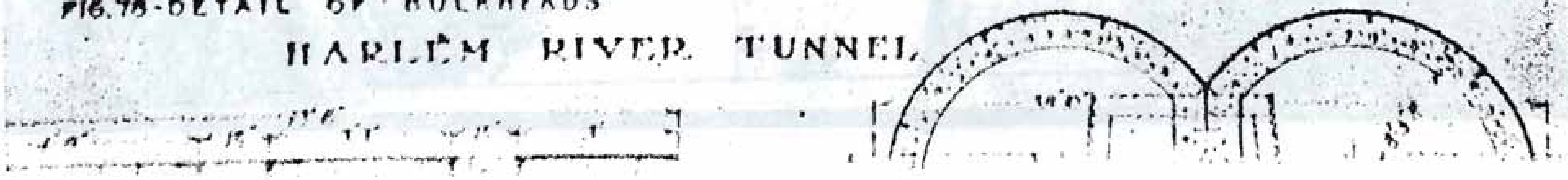
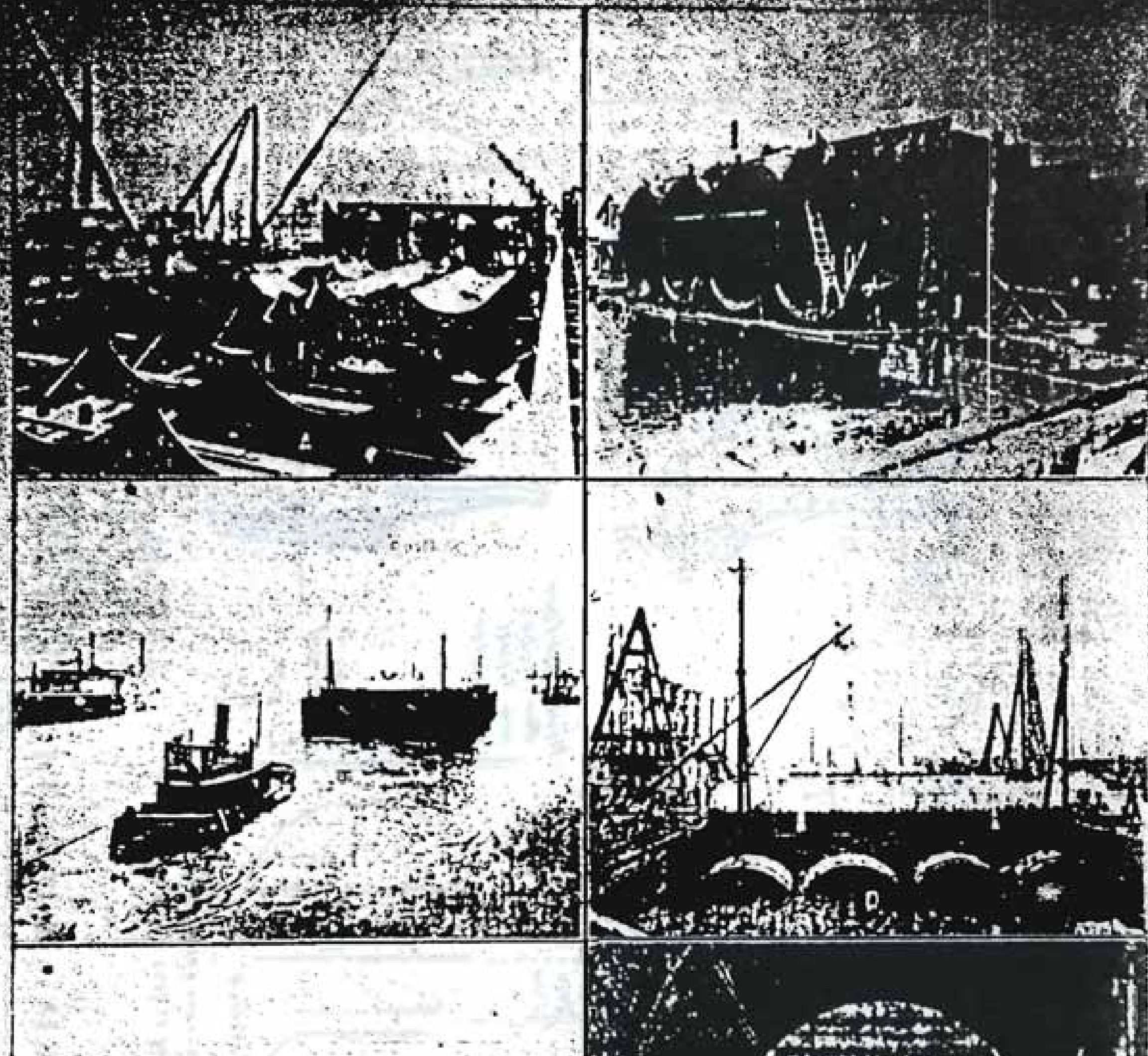


FIG.78-DETAIL OF BULKHEADS

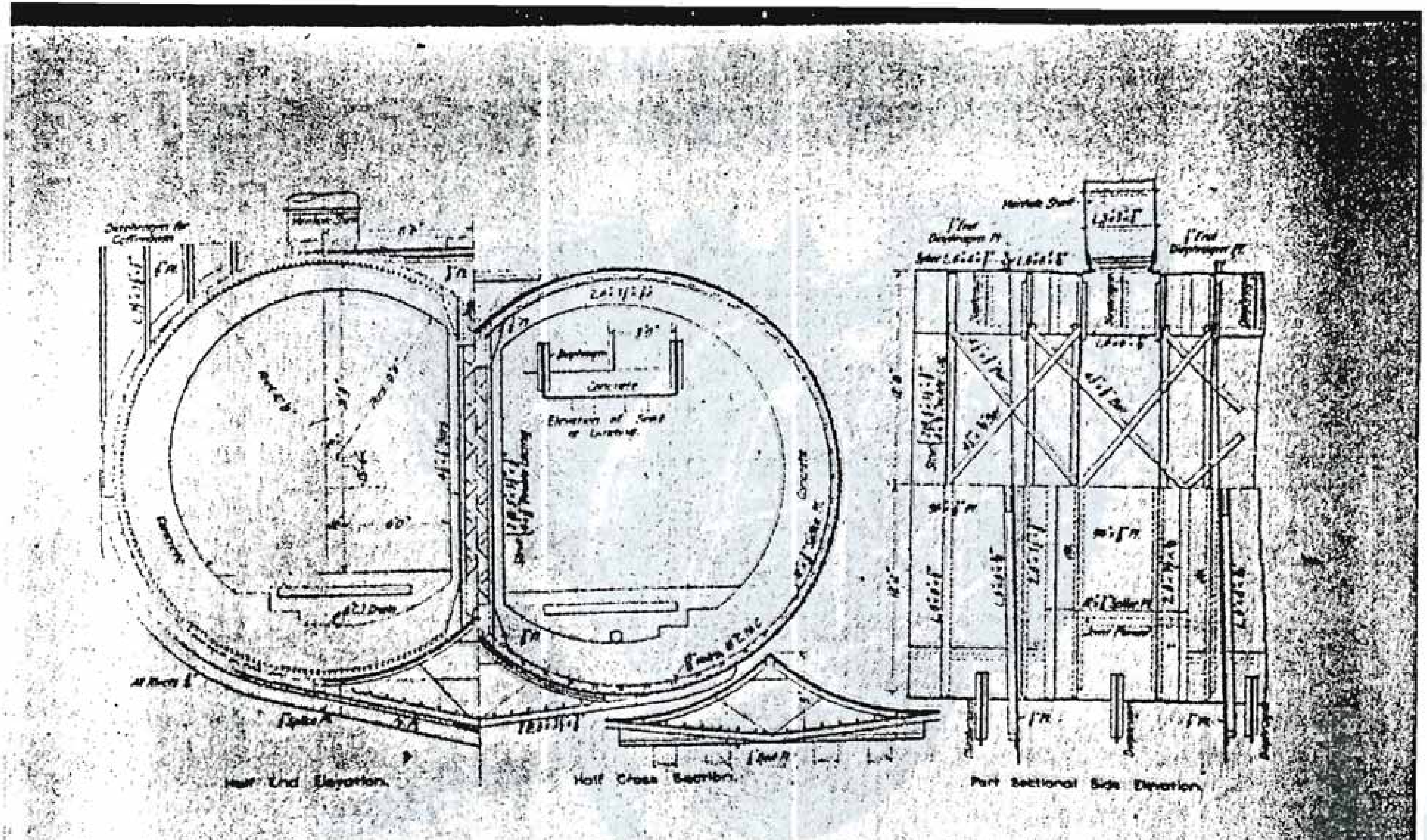
FIG 79·FLOATING EQUIPMENT FOR SINKING TUBES

HARLEM RIVER TUNNEL

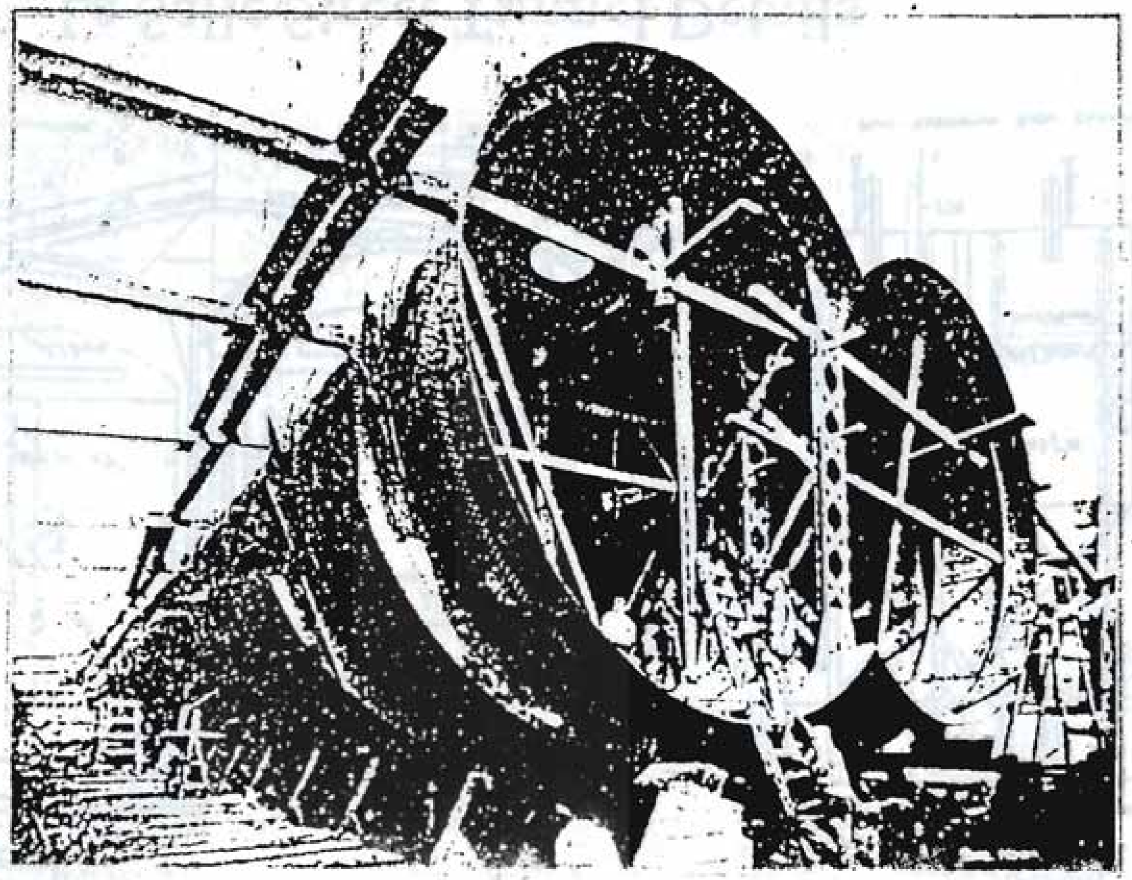




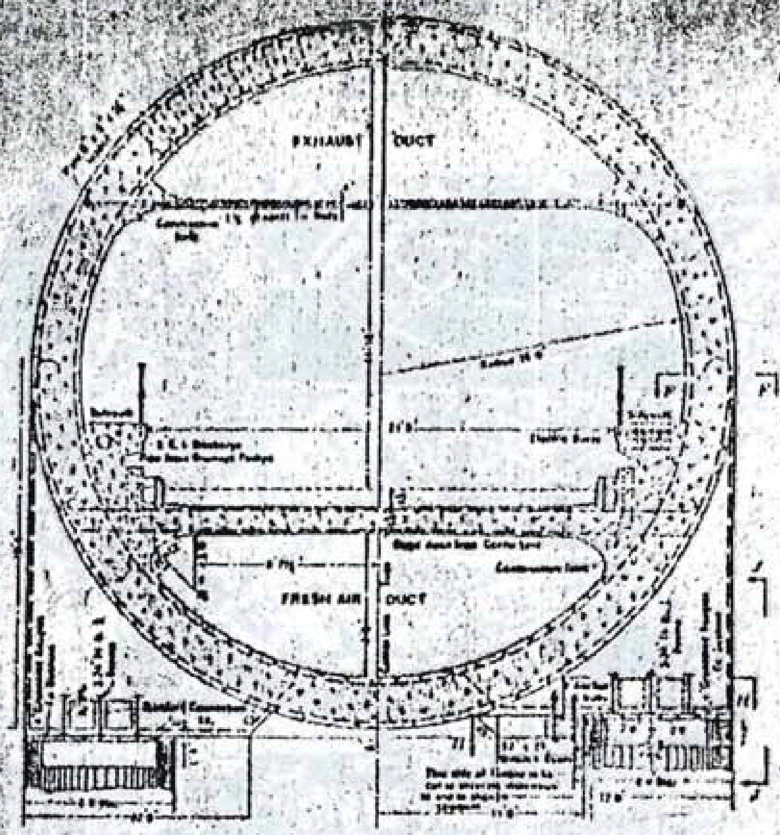
HARLEM RIVER TUNNEL
NEW YORK CITY (1914)



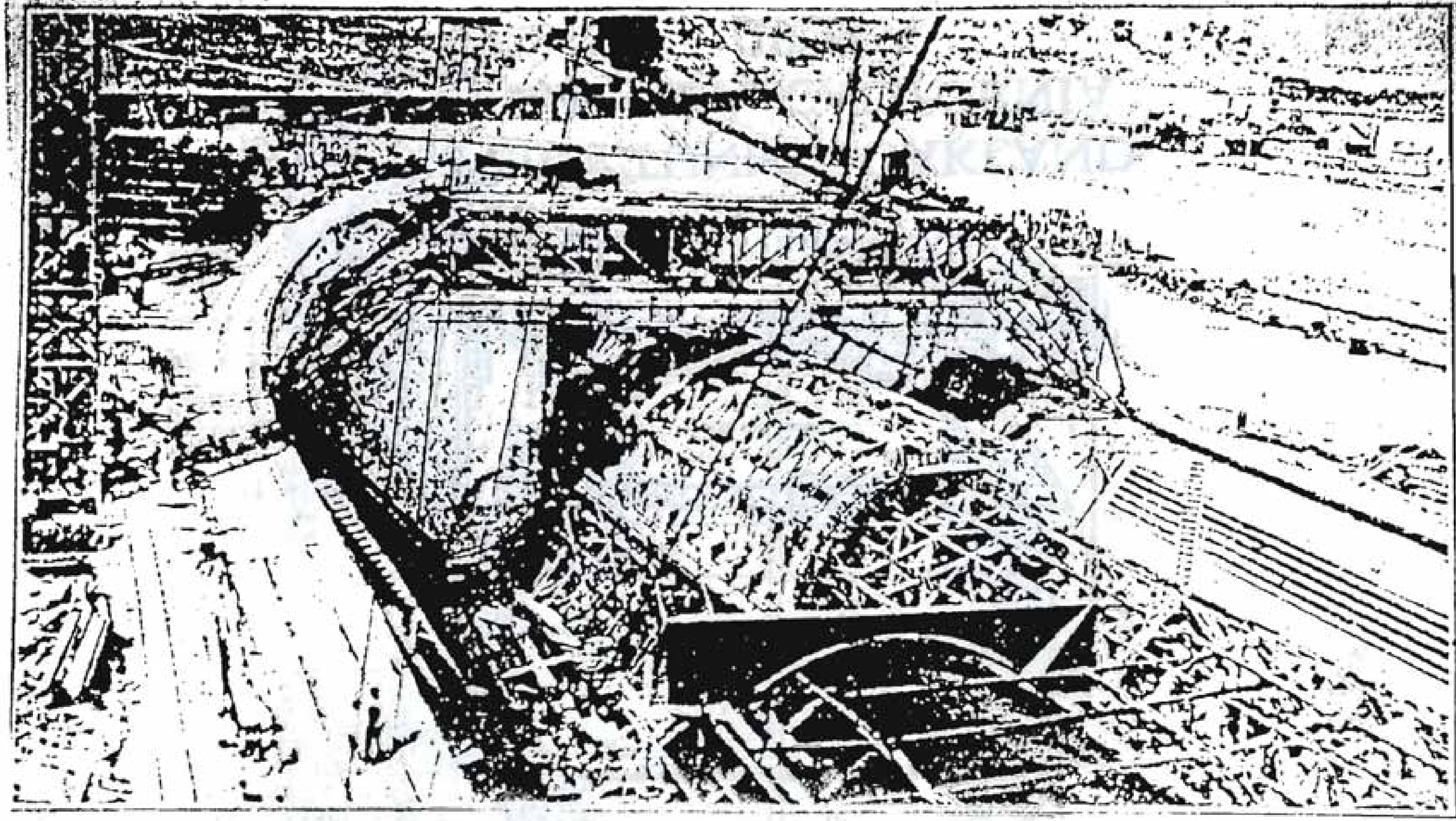
La Salle Street Tunnel Details



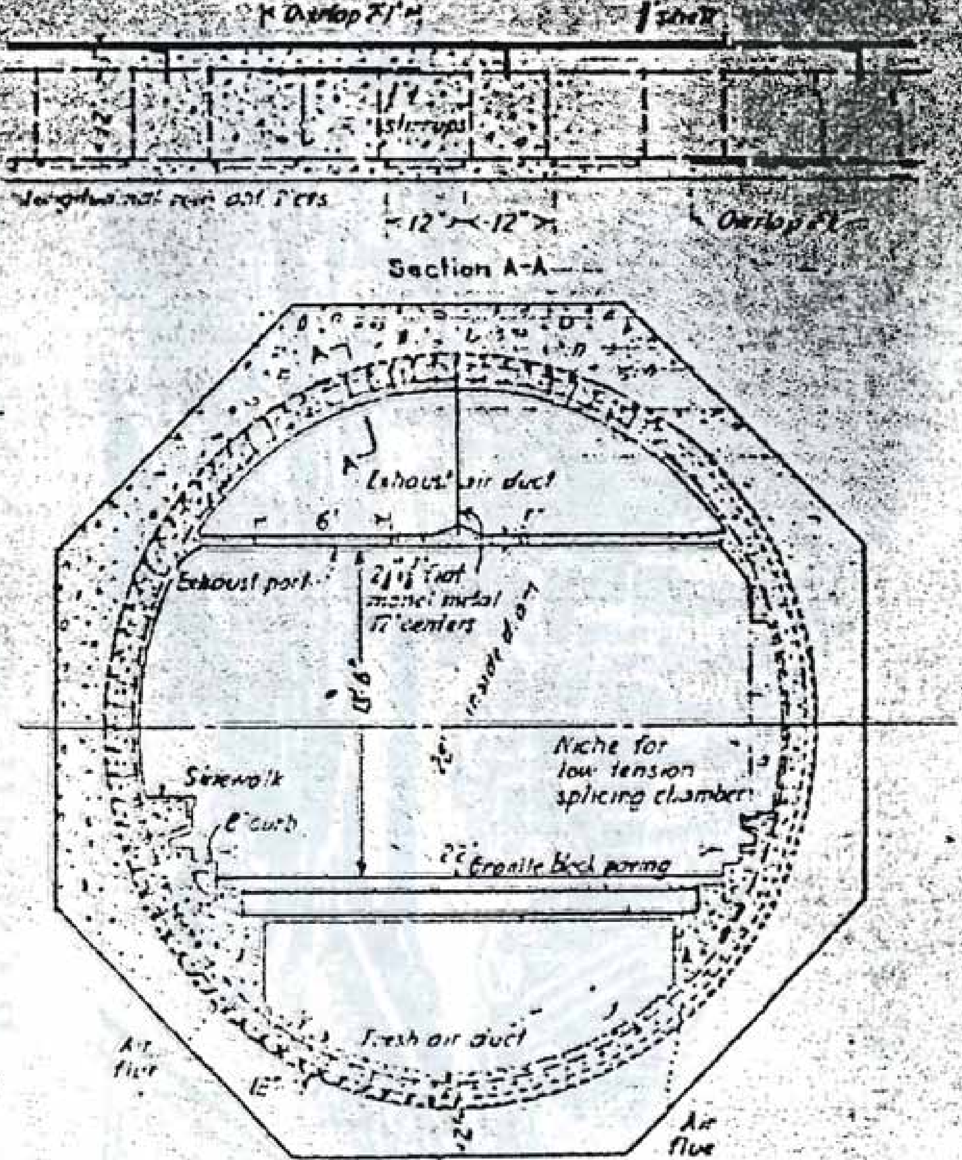
LA SALLE ST. TUNNEL
CHICAGO (1912)



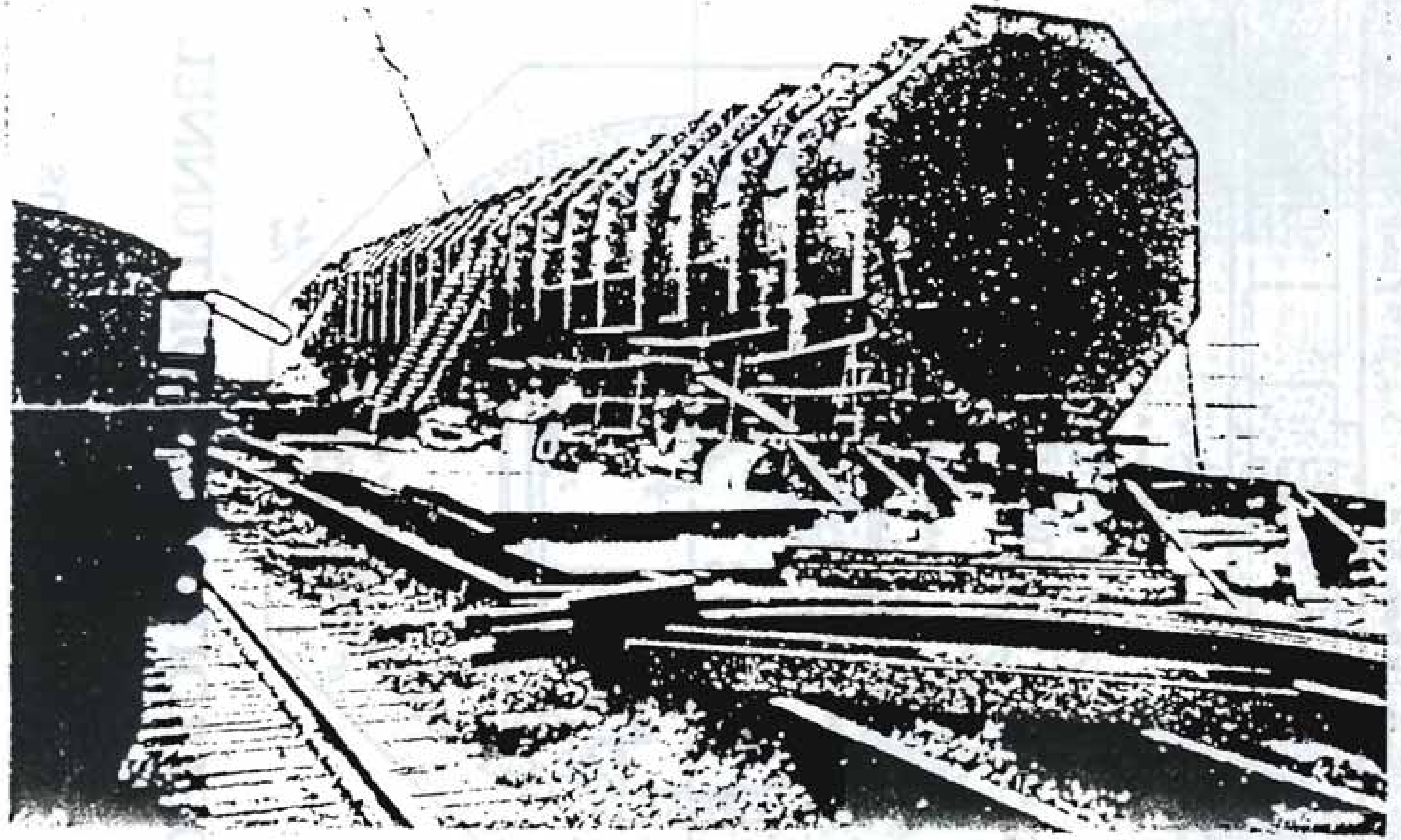
POSEY TUNNEL OAKLAND-ALAMEDA, CALIFORNIA (1928)



POSEY TUNNEL ELEMENTS IN
GRAVING DOCK



DETROIT-WINDSOR TUNNEL
(1930)
Typical Cross Sections



BANKHEAD TUNNEL,
MOBILE, ALABAMA (1940)

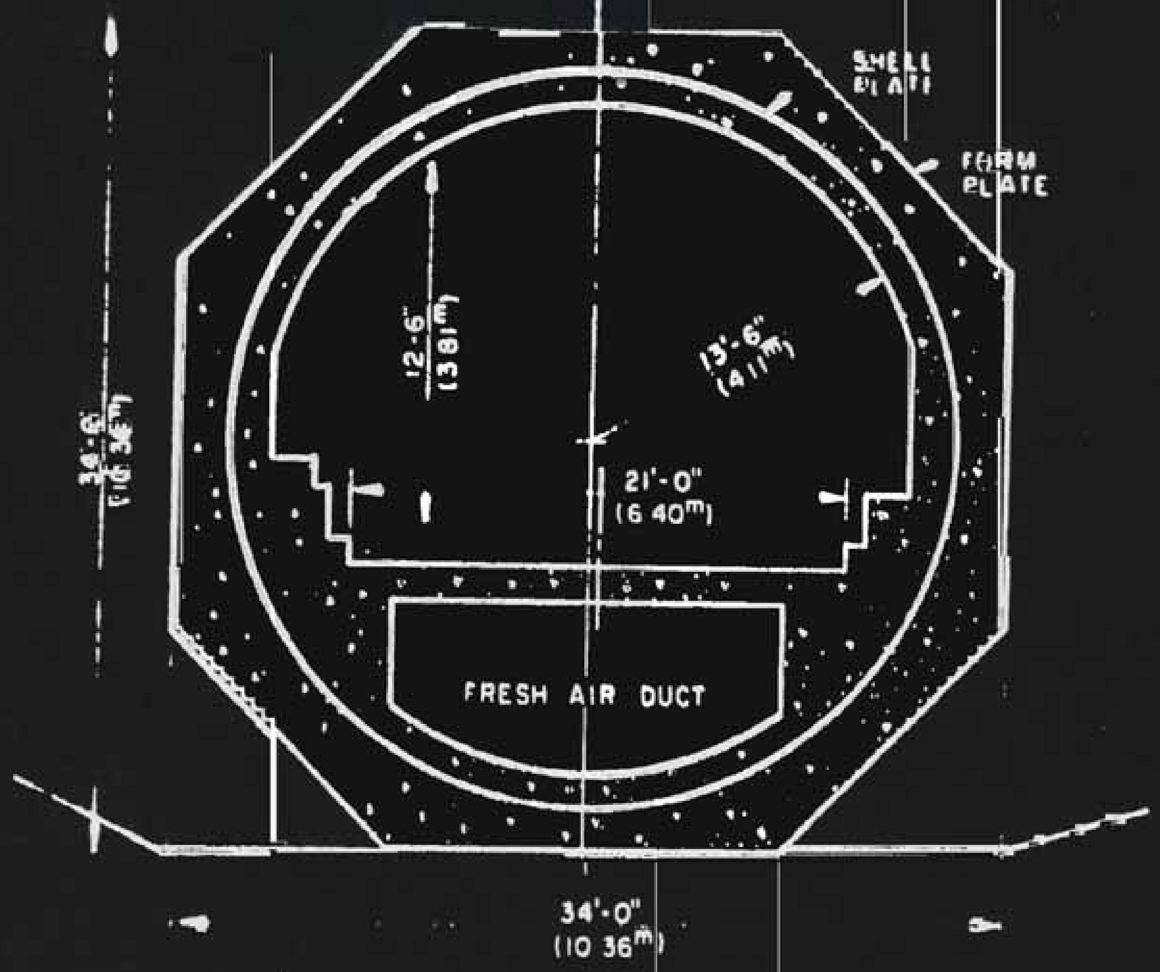


FIG. 4.—Bankhead Tunnel, Mobile, Ala.

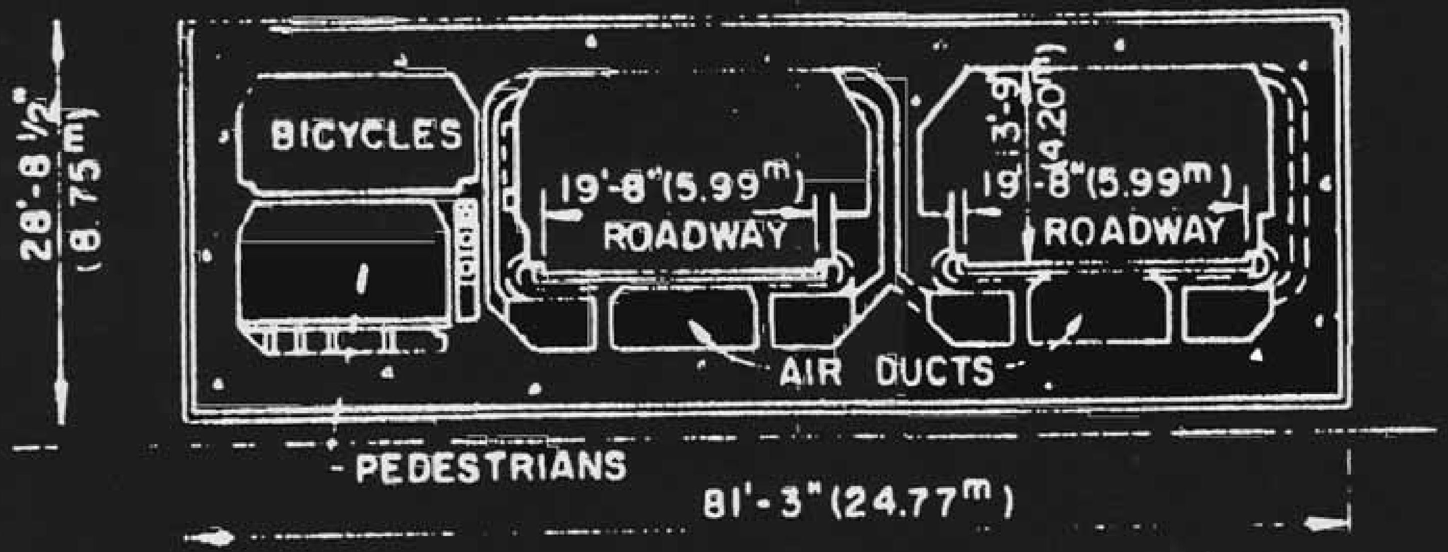
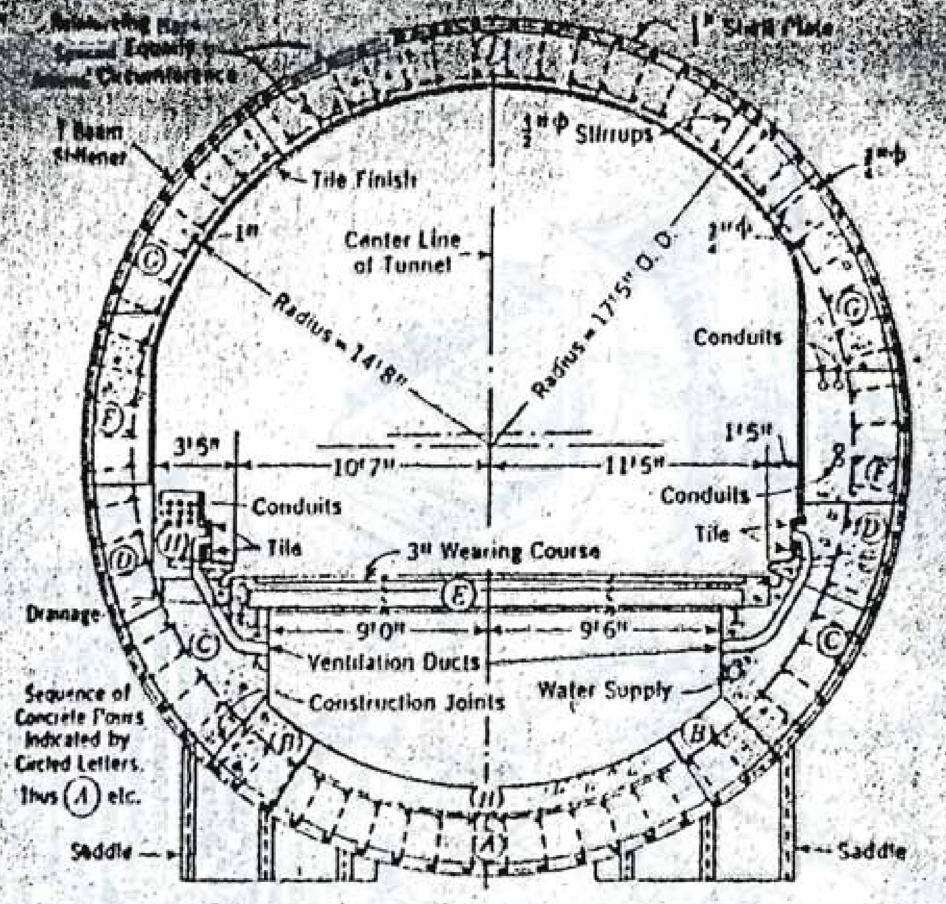
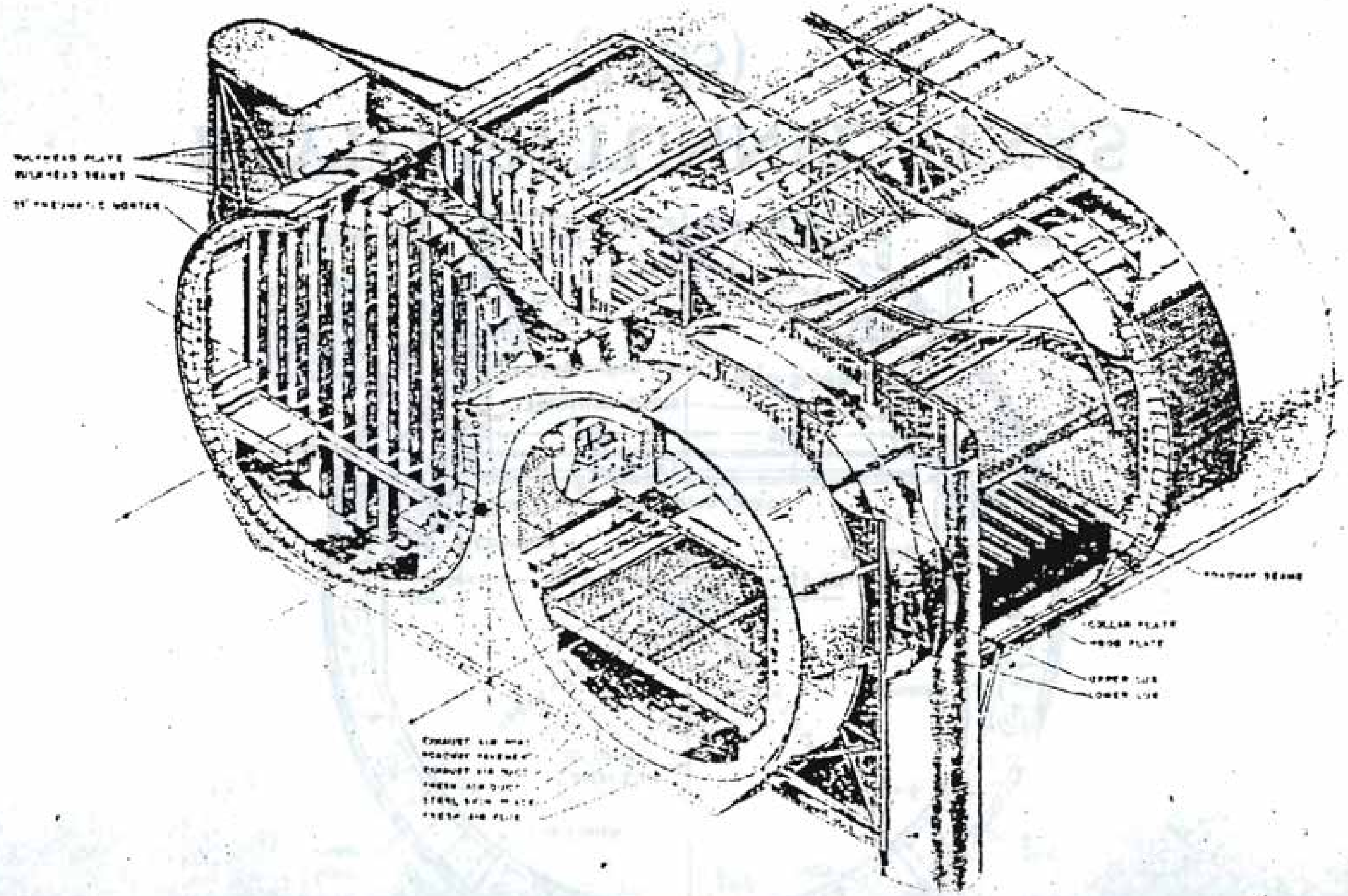


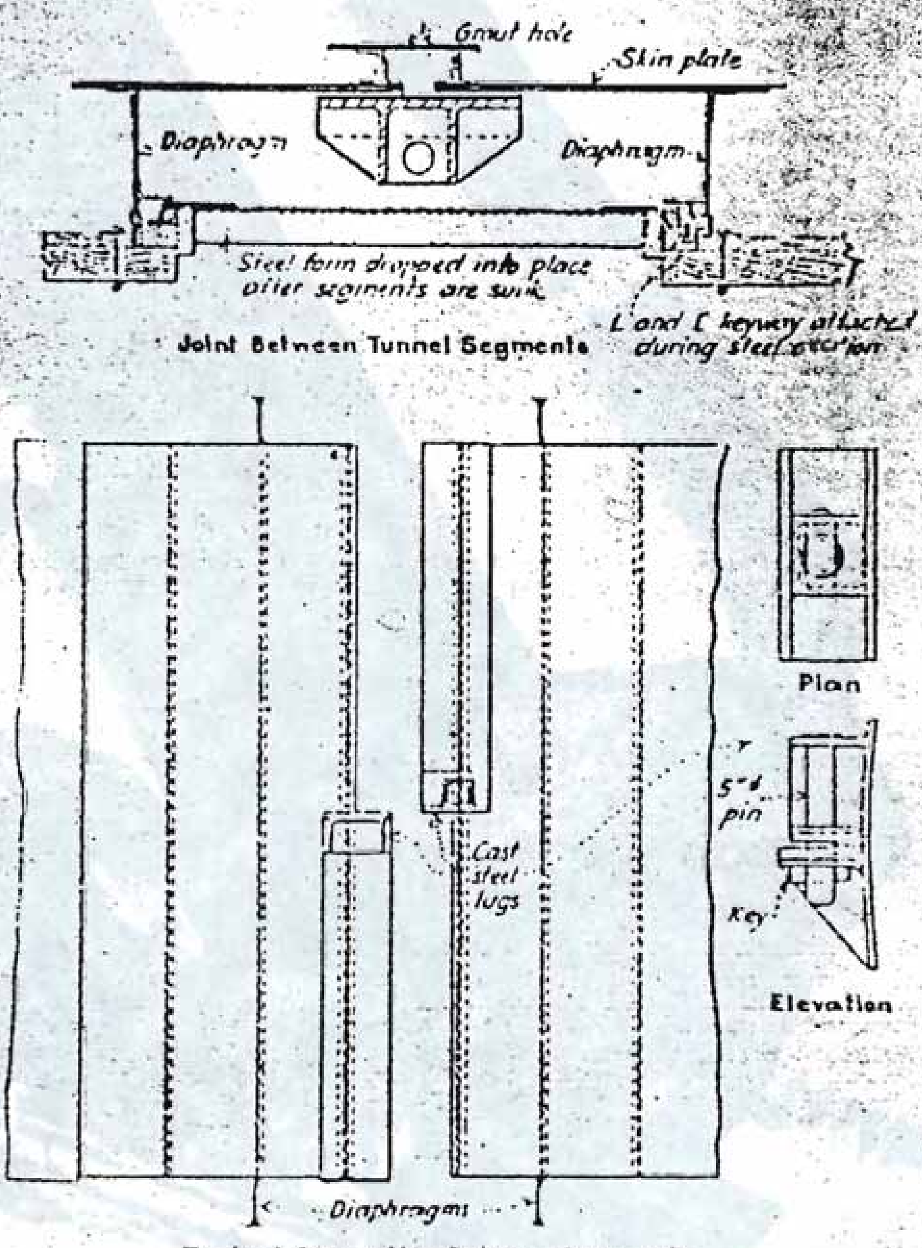
FIG. 5.—Maas River, Rotterdam, The Netherlands



BAYTOWN TUNNEL, TEXAS
(1953)

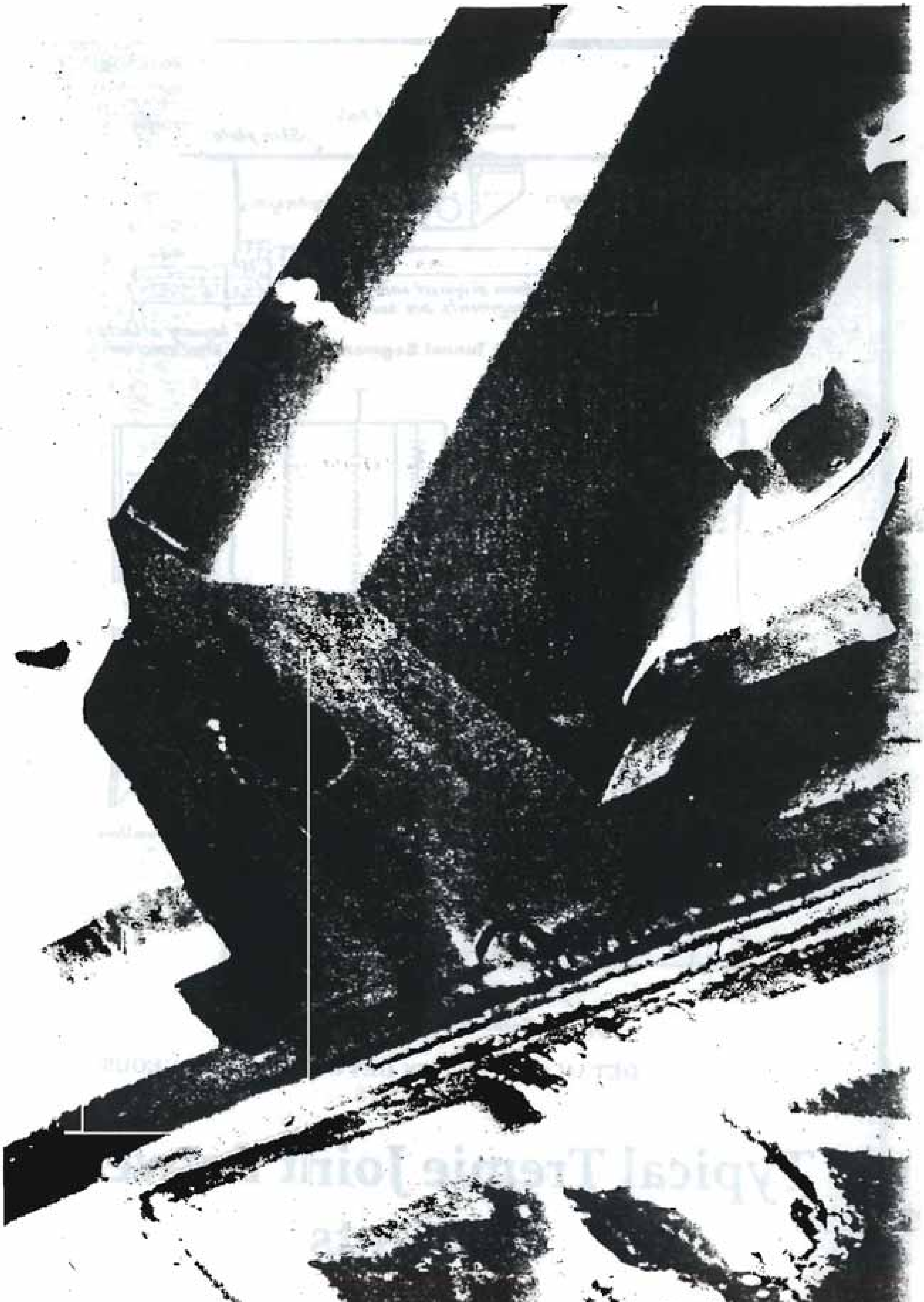


BALTIMORE HARBOR TUNNEL (1957)



DETAILS AT JOINTS BETWEEN SUBAQUEOUS SEGMENTS

Typical Tremie Joint Between Elements



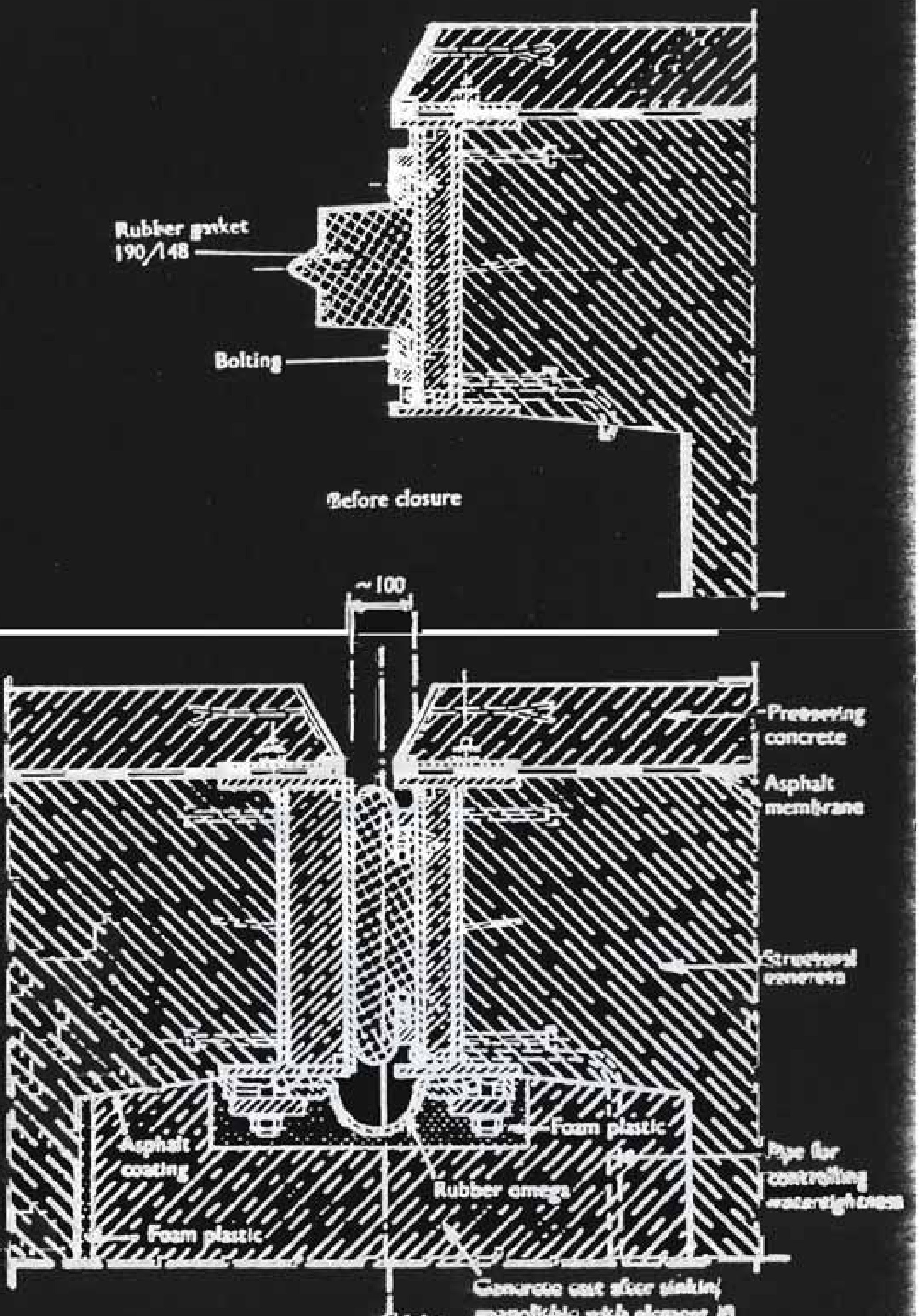
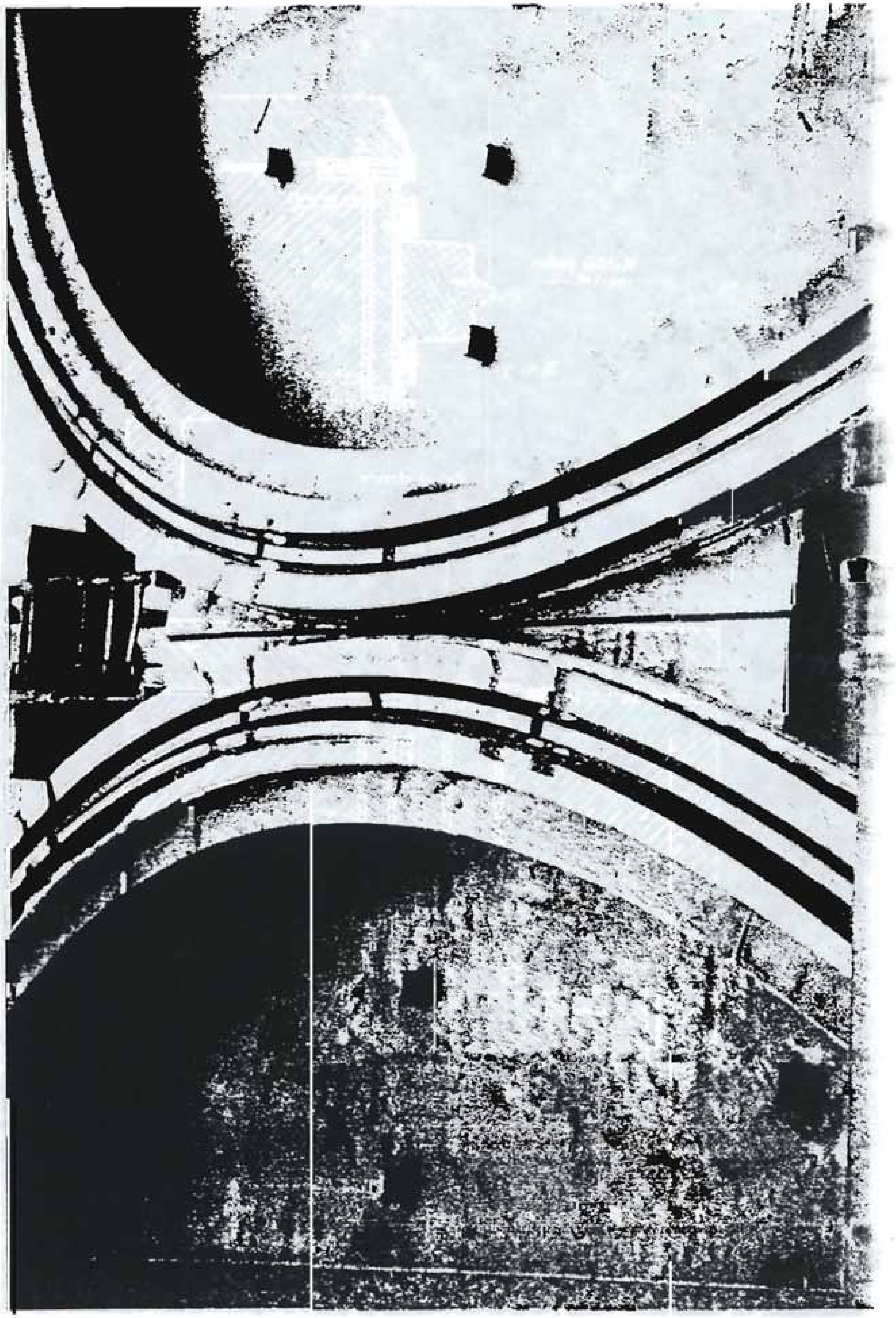
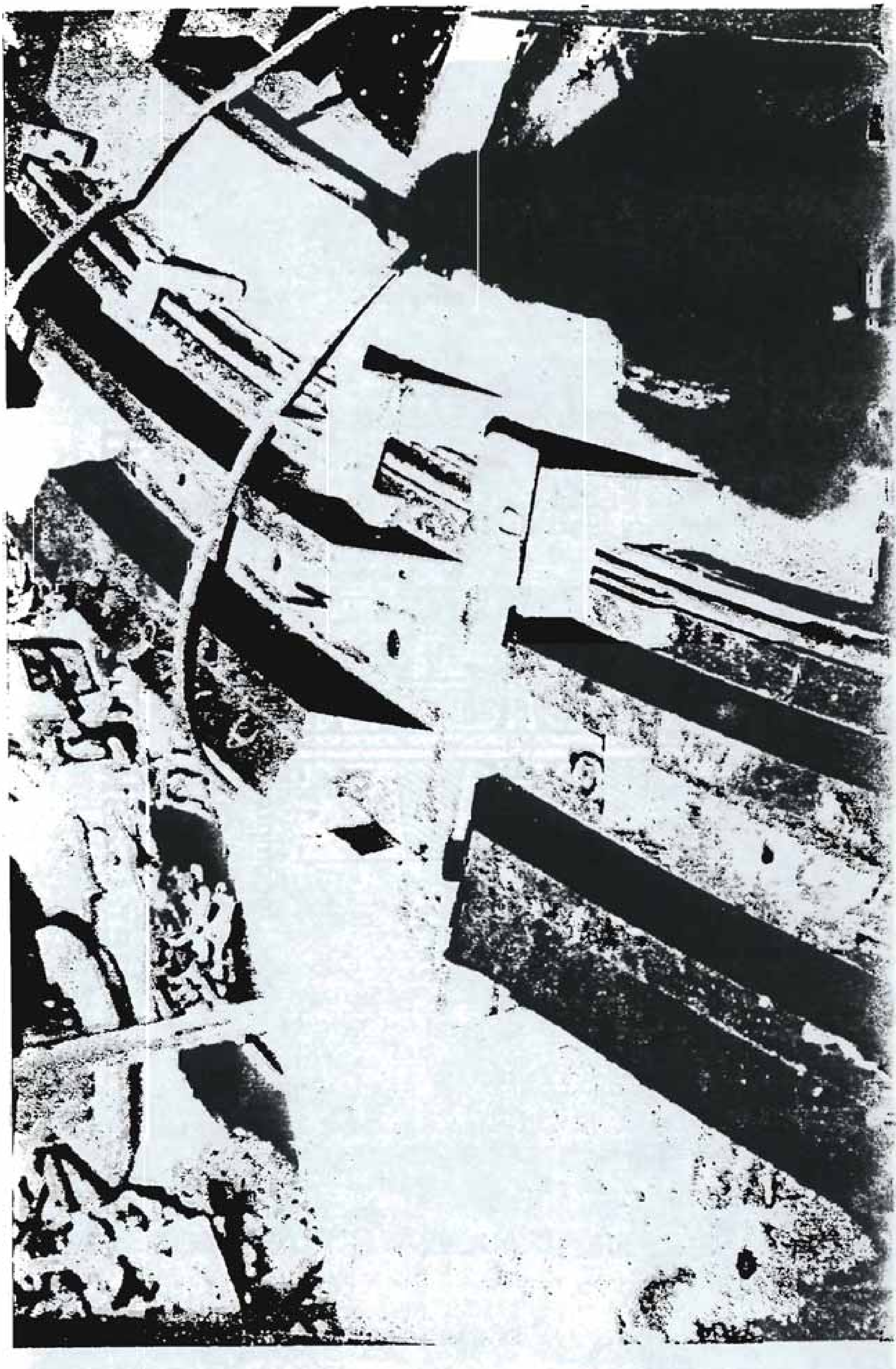


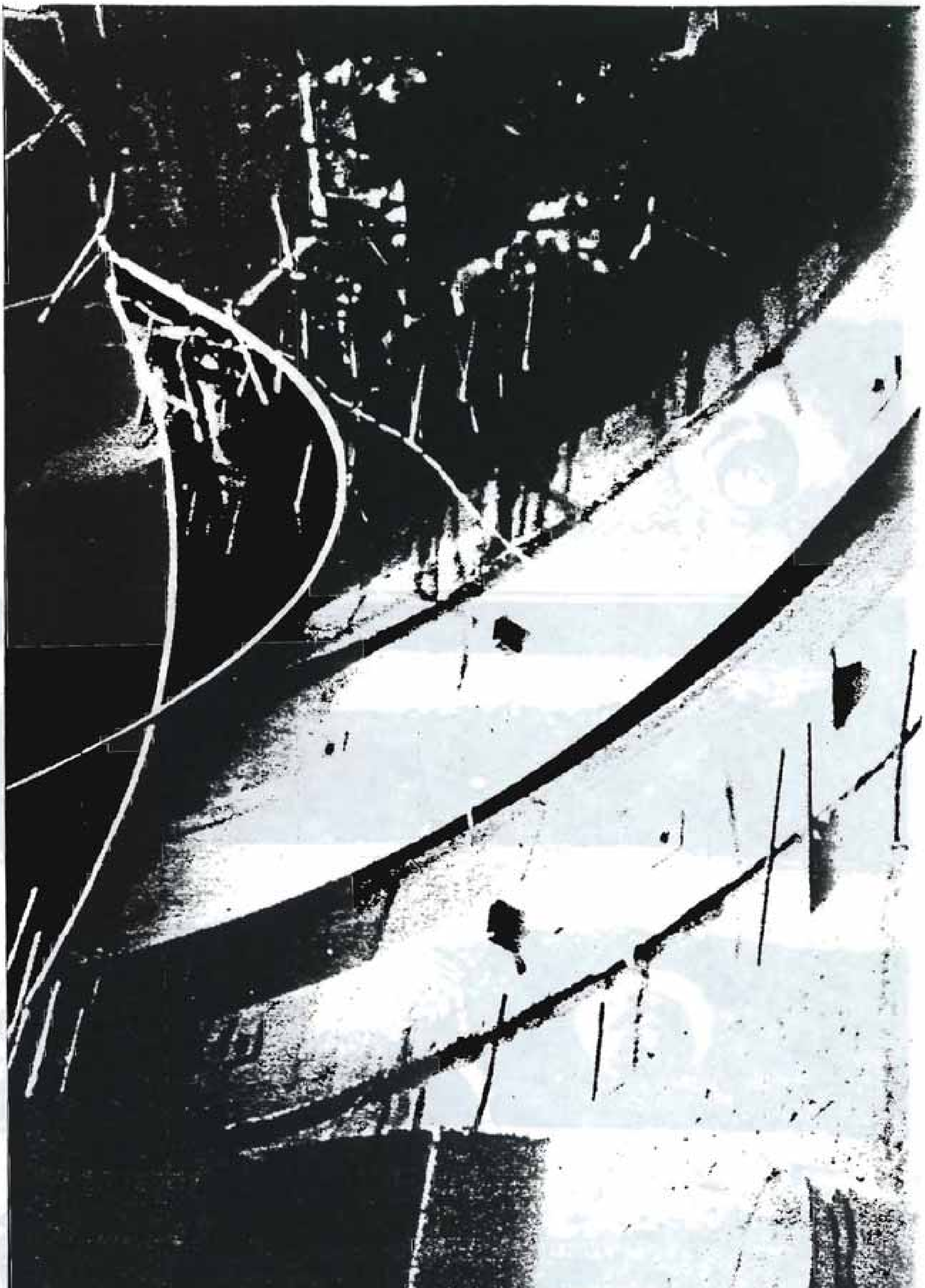
Fig. 16. Joint between elements : top slab

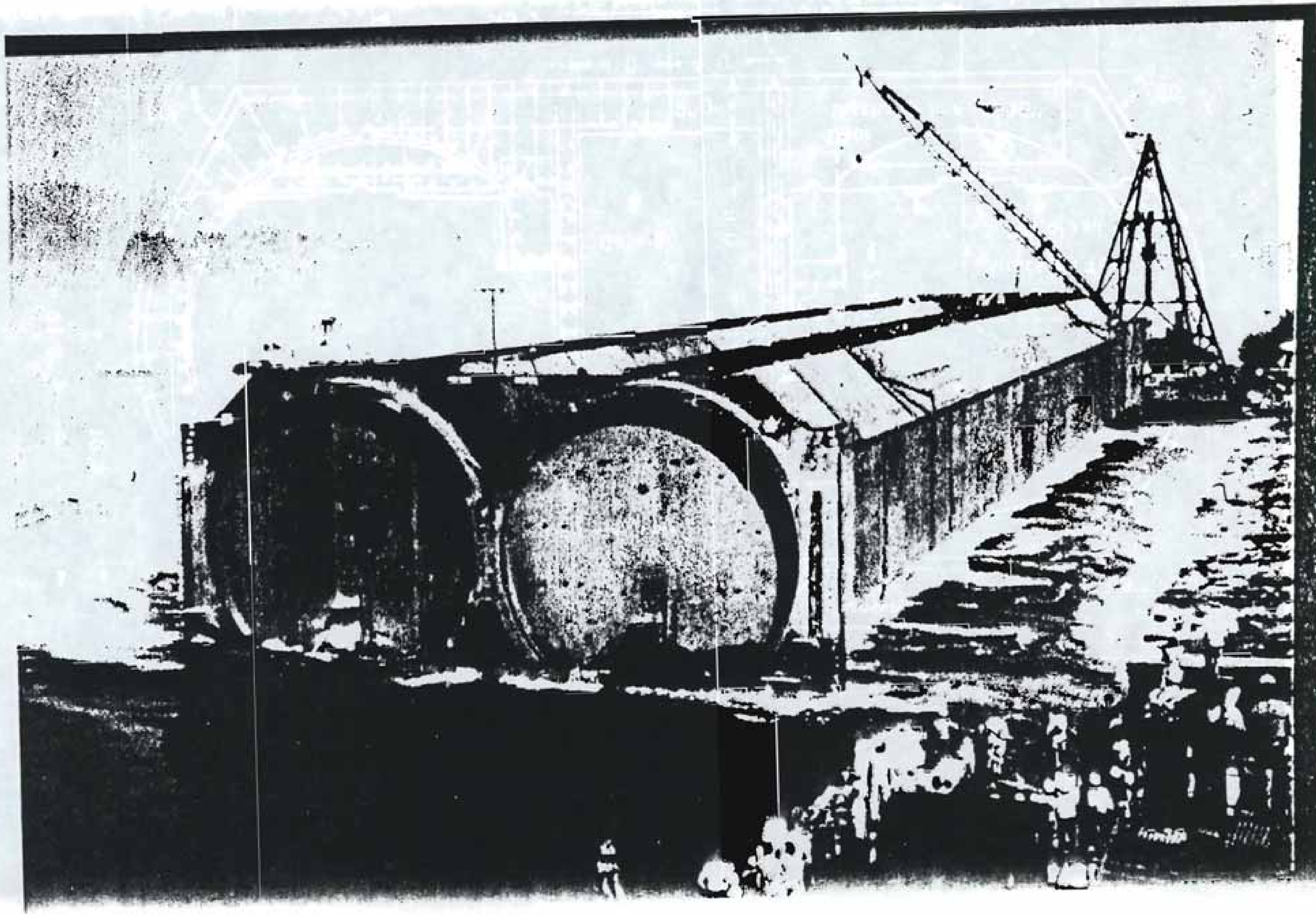


Uncompressed Gasket Detail

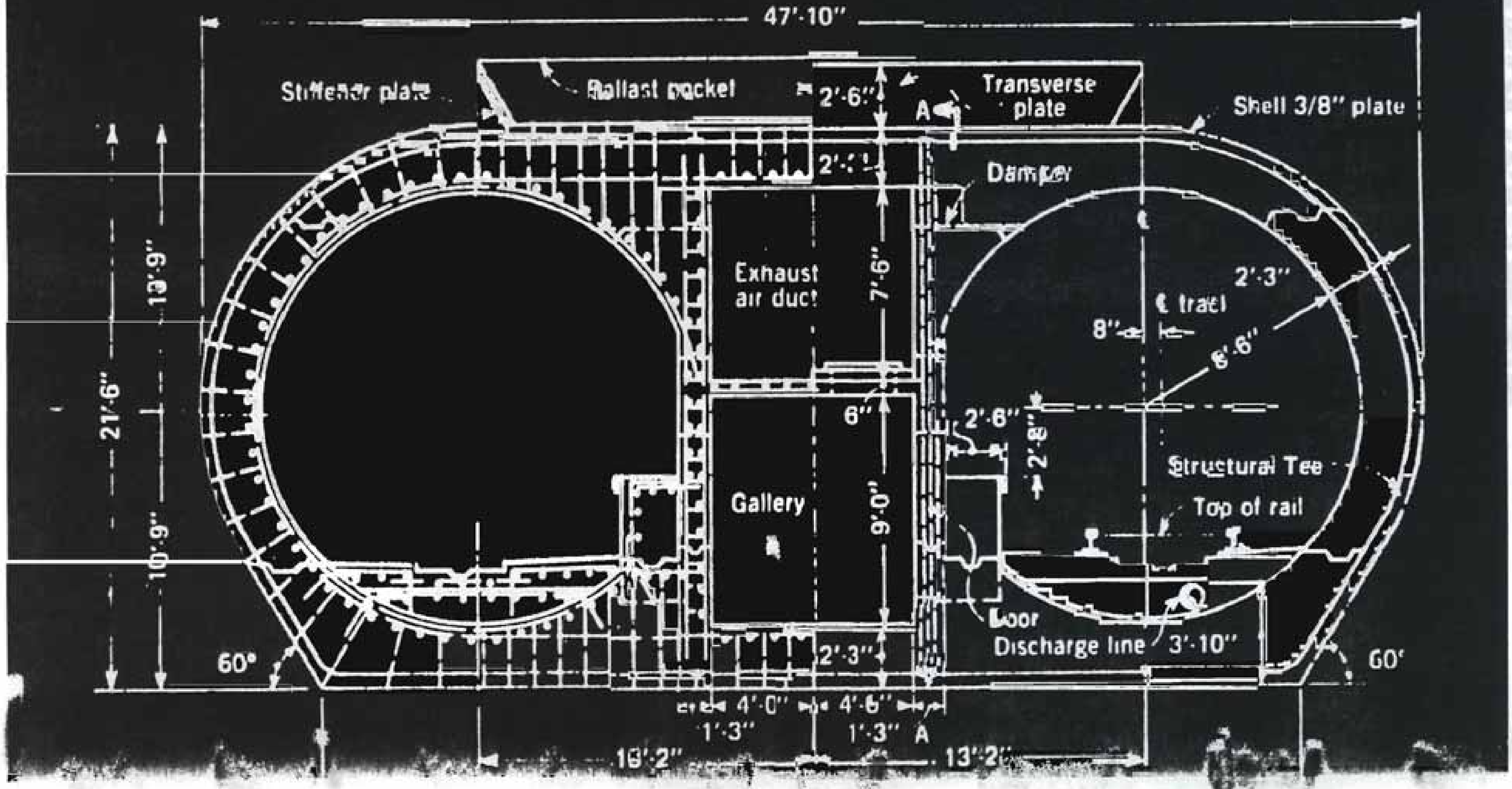
Rubber Gasket Joint Detail

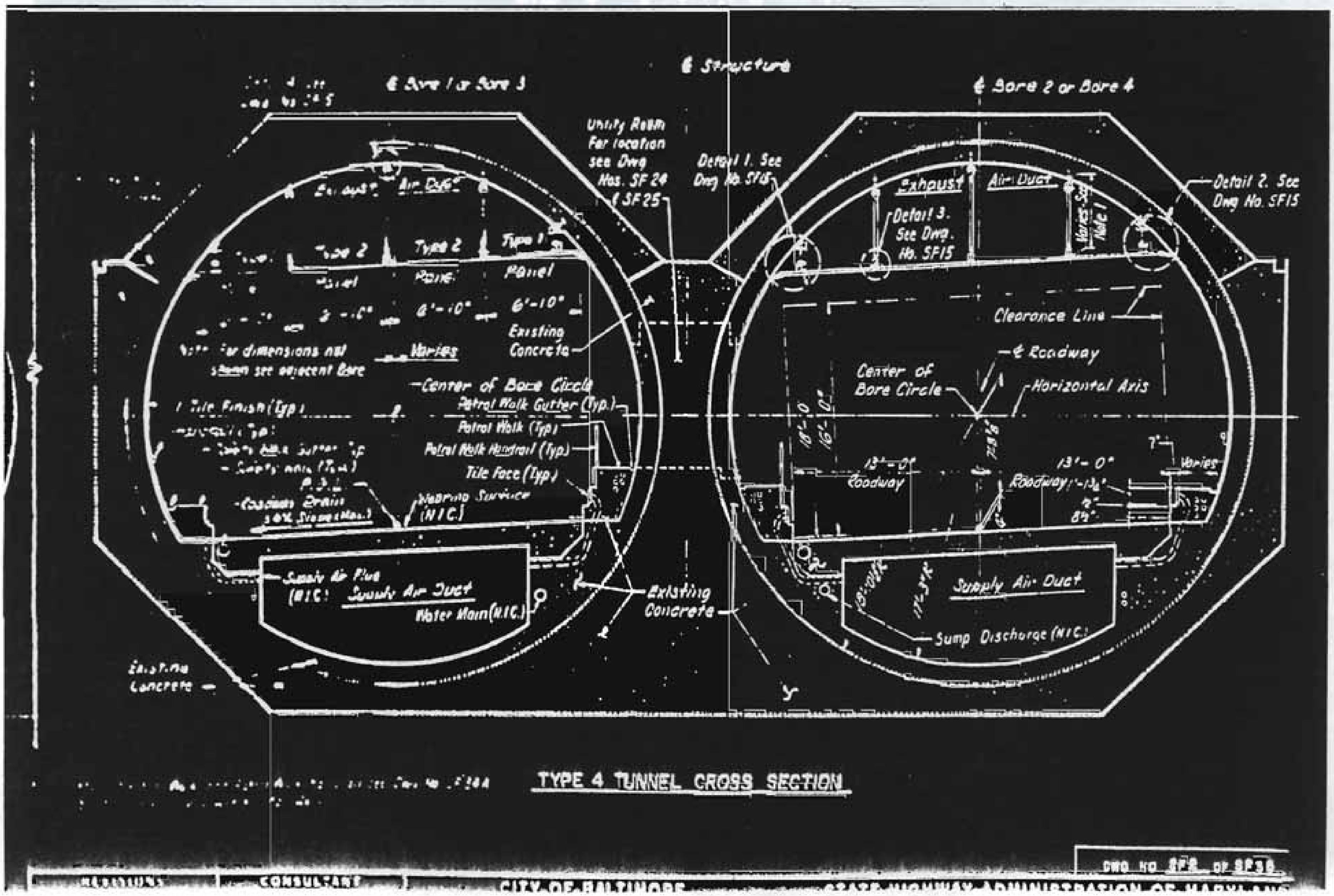






Trans Bay Tube—Cross Section





TYPE 4 TUNNEL CROSS SECTION

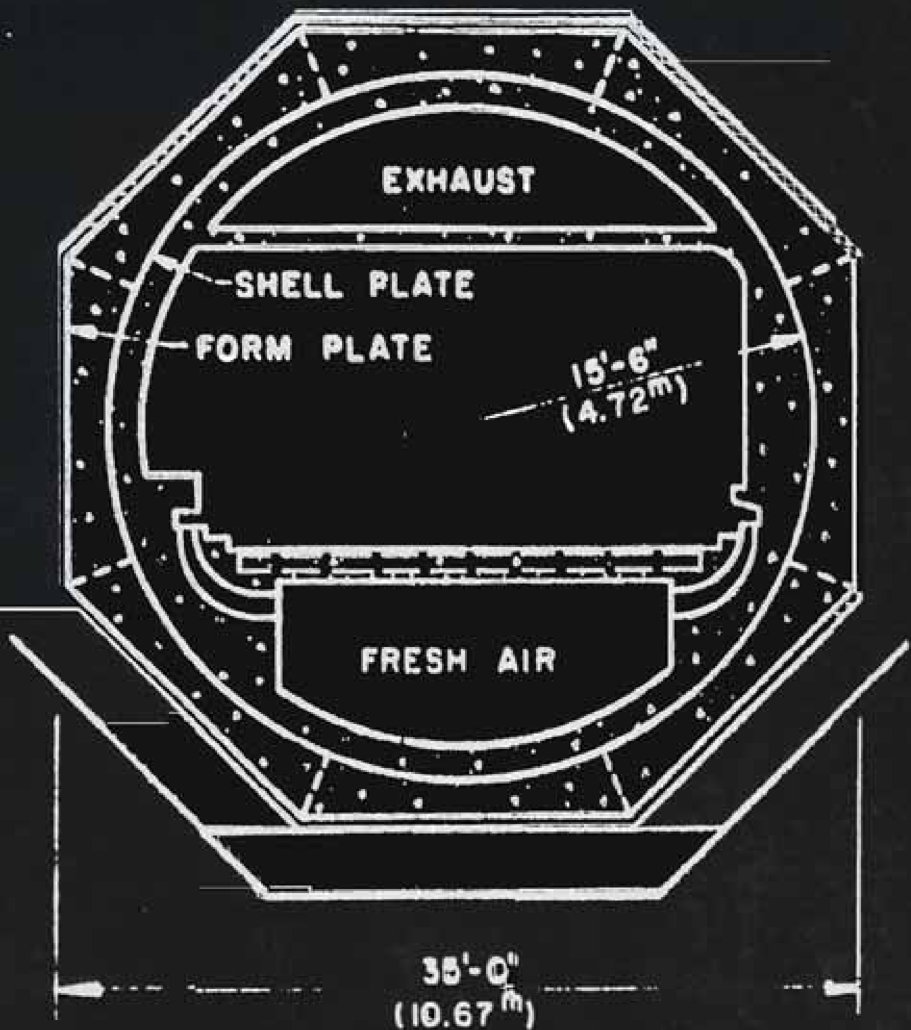
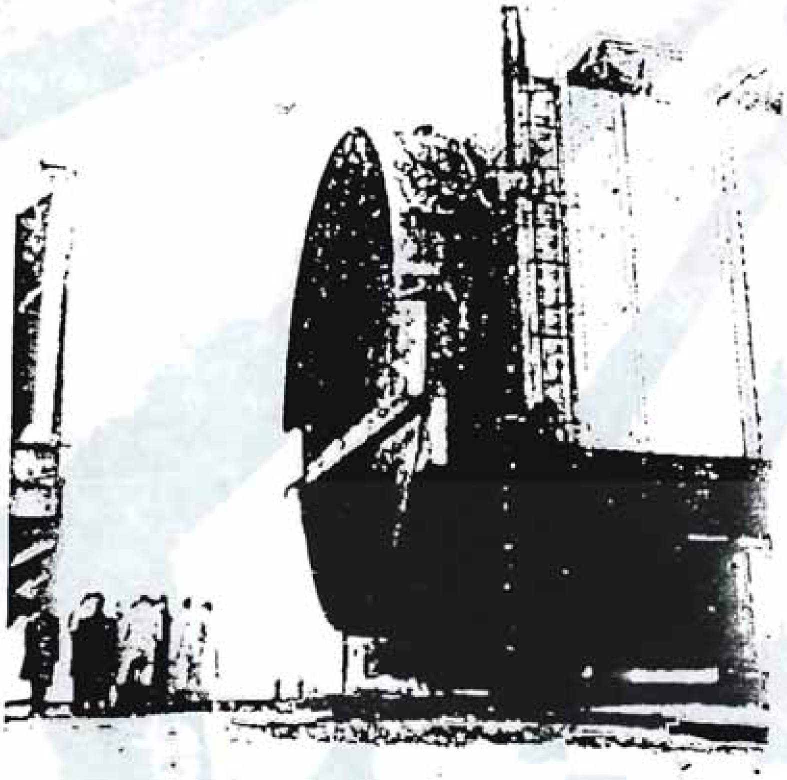
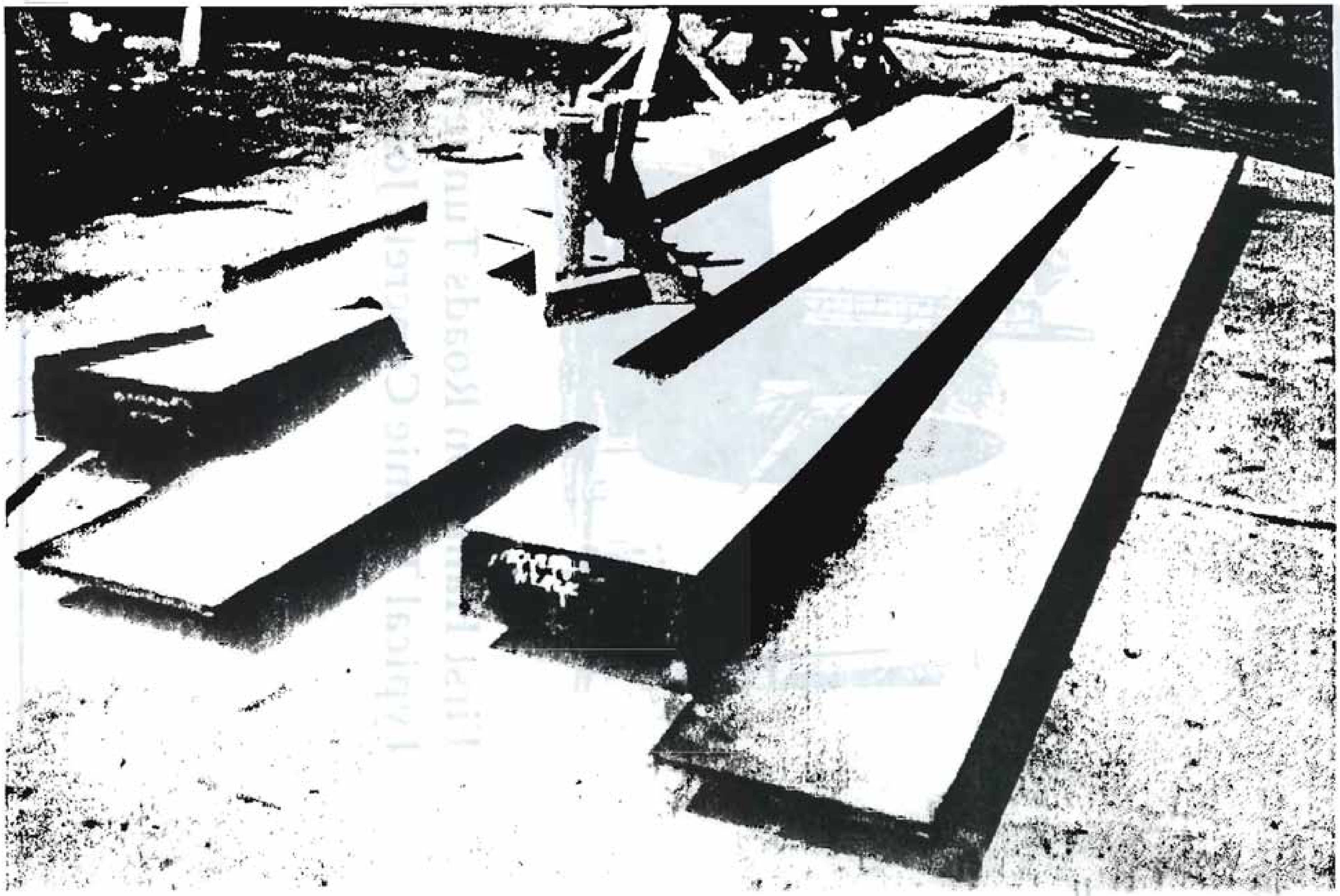
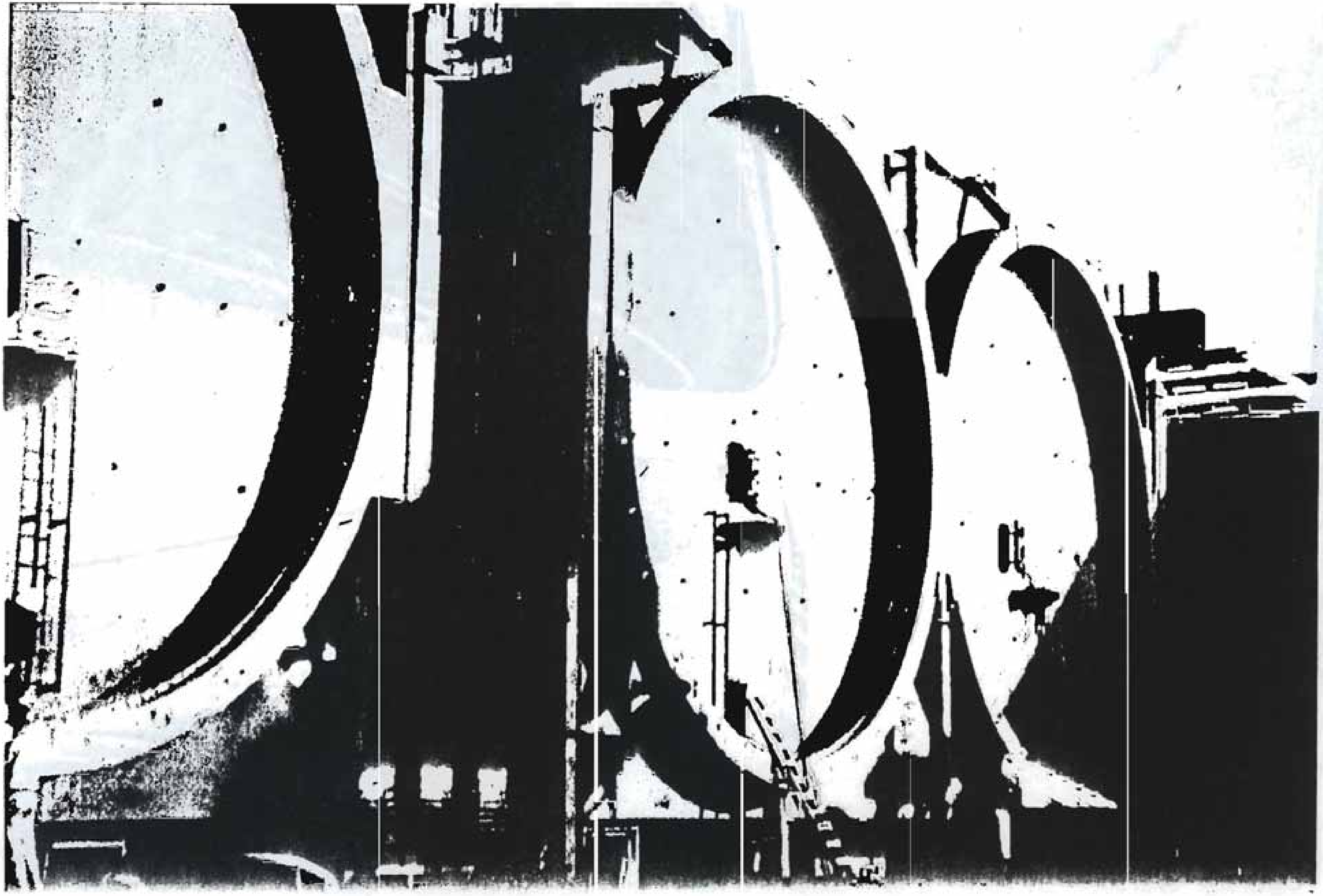


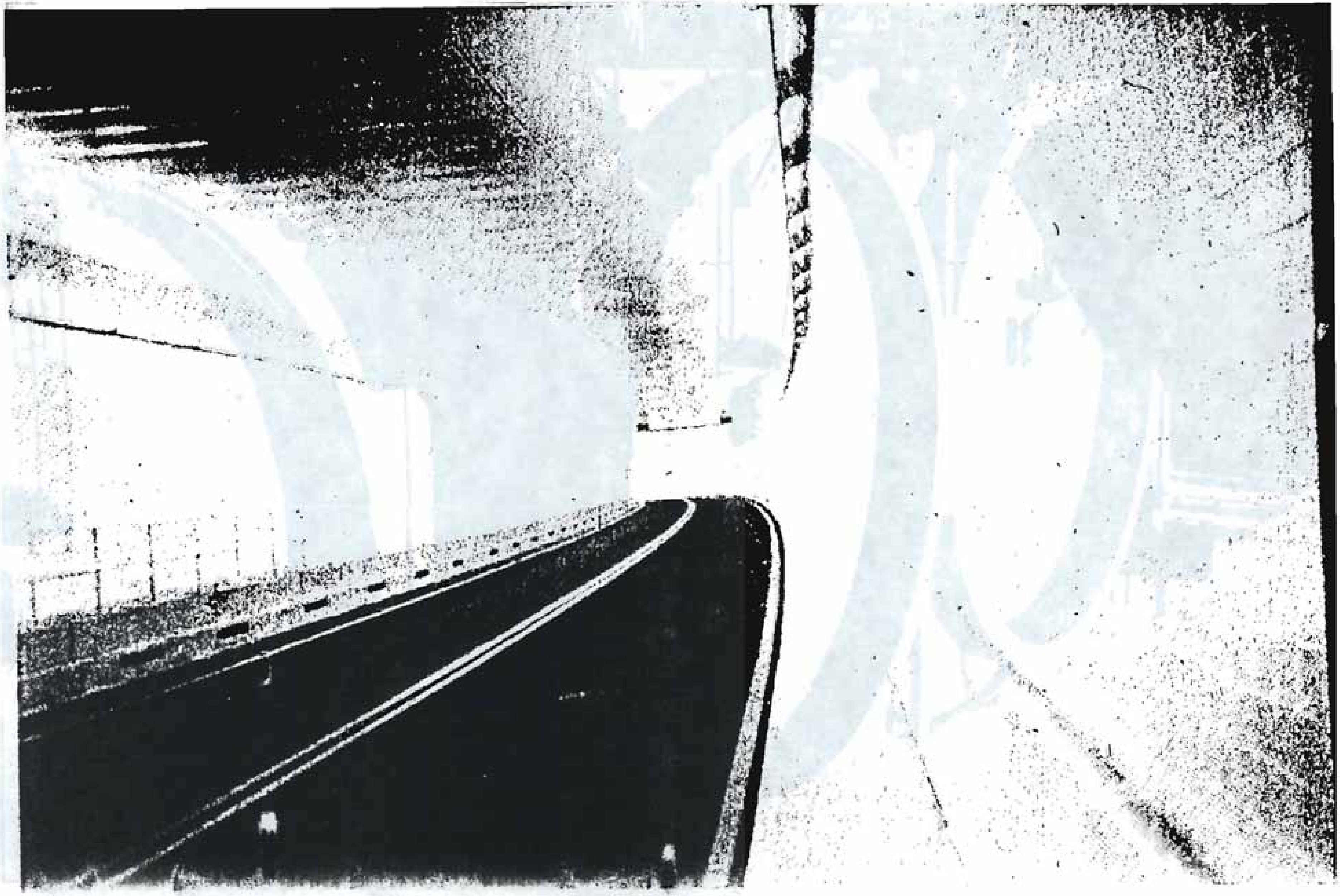
FIG. 3.—Detroit-Canada

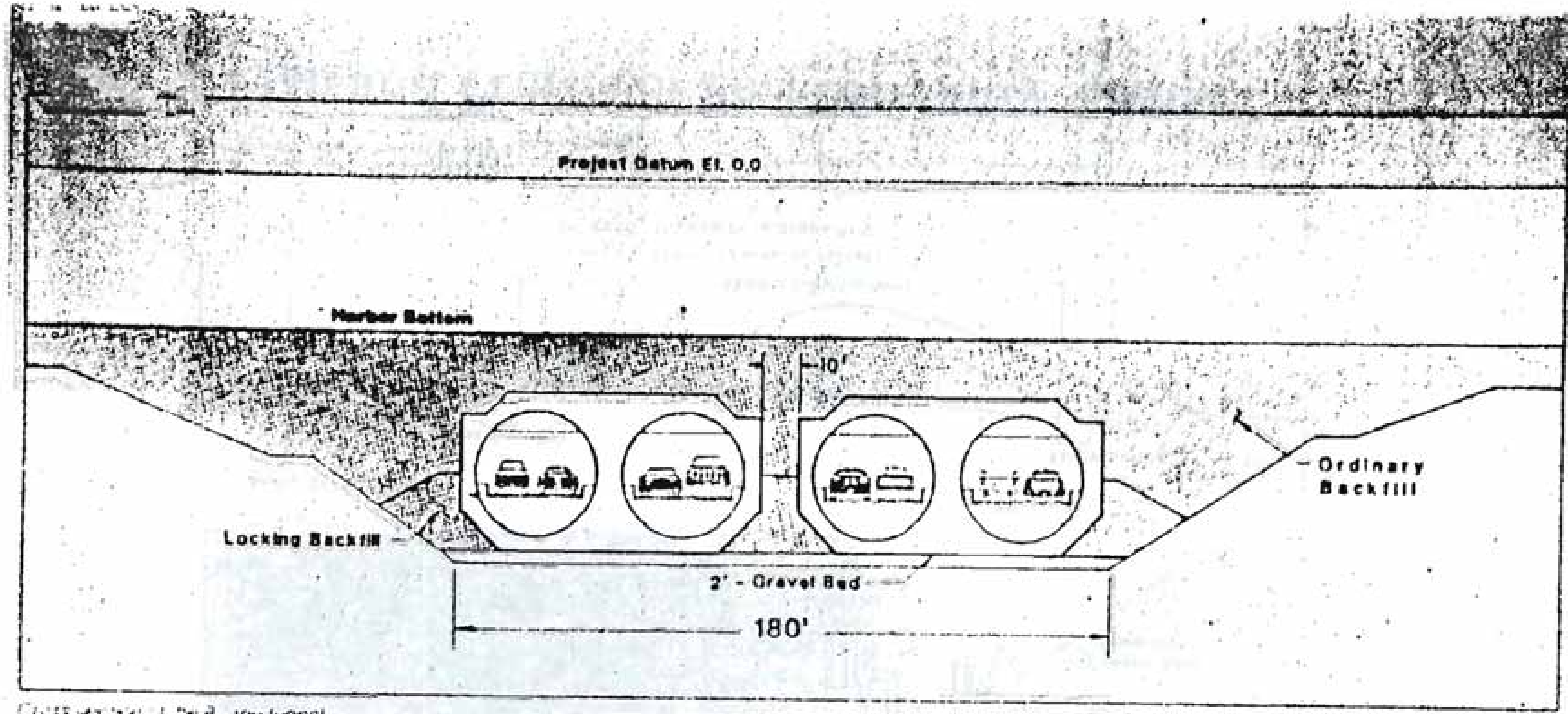


First Hampton Roads Tunnel
Typical Tremie Concrete Joint



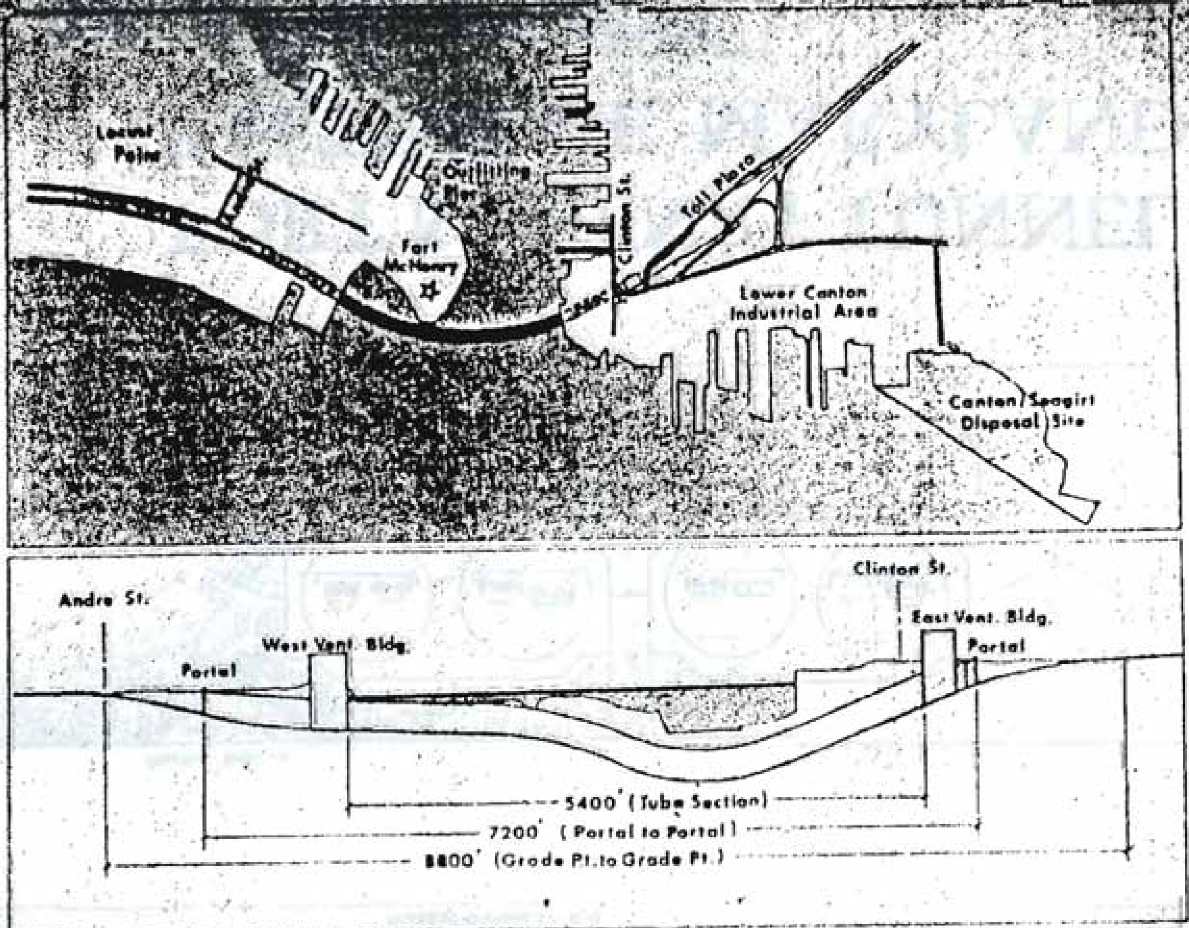




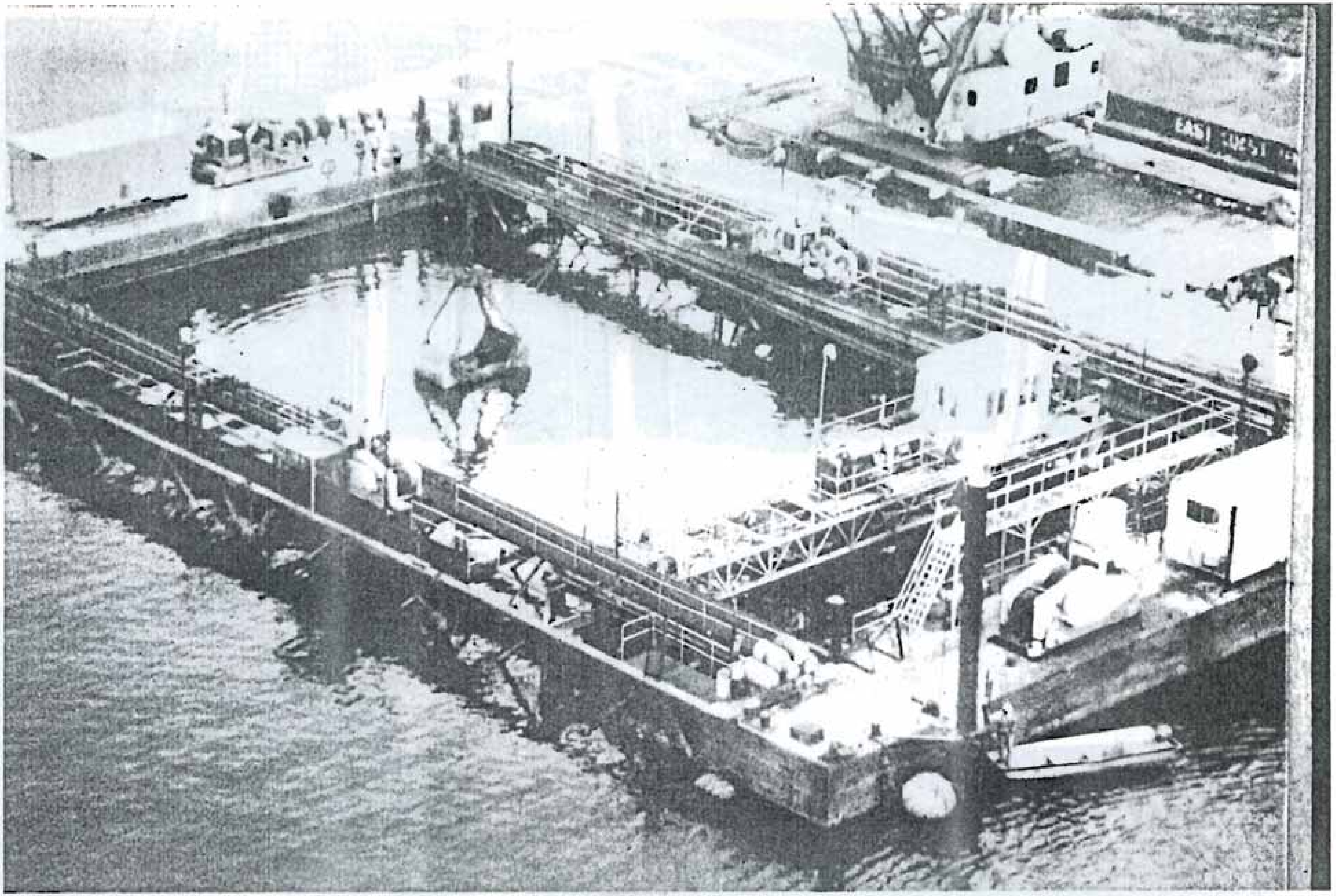


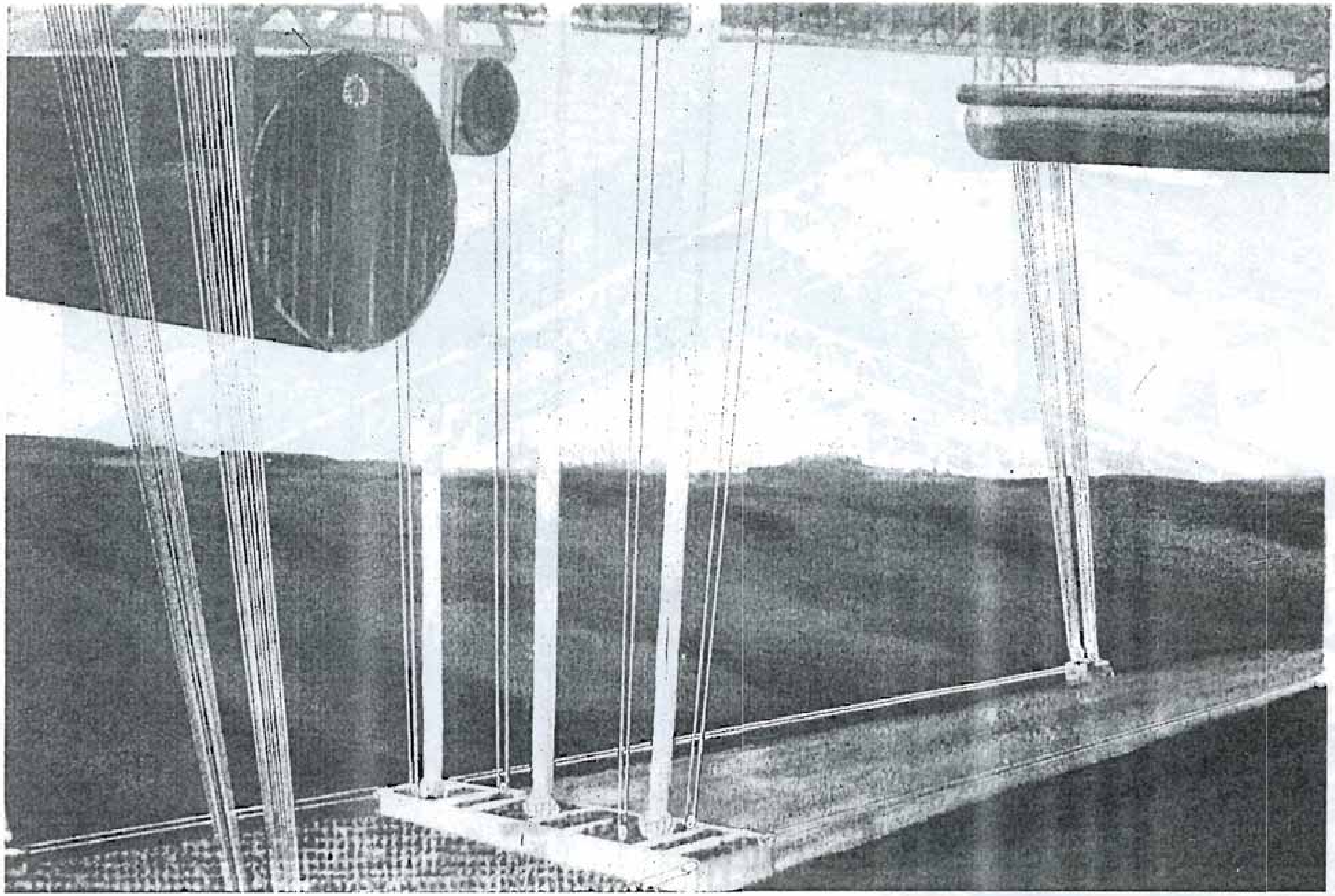
Cross section of the 8-lane tunnel

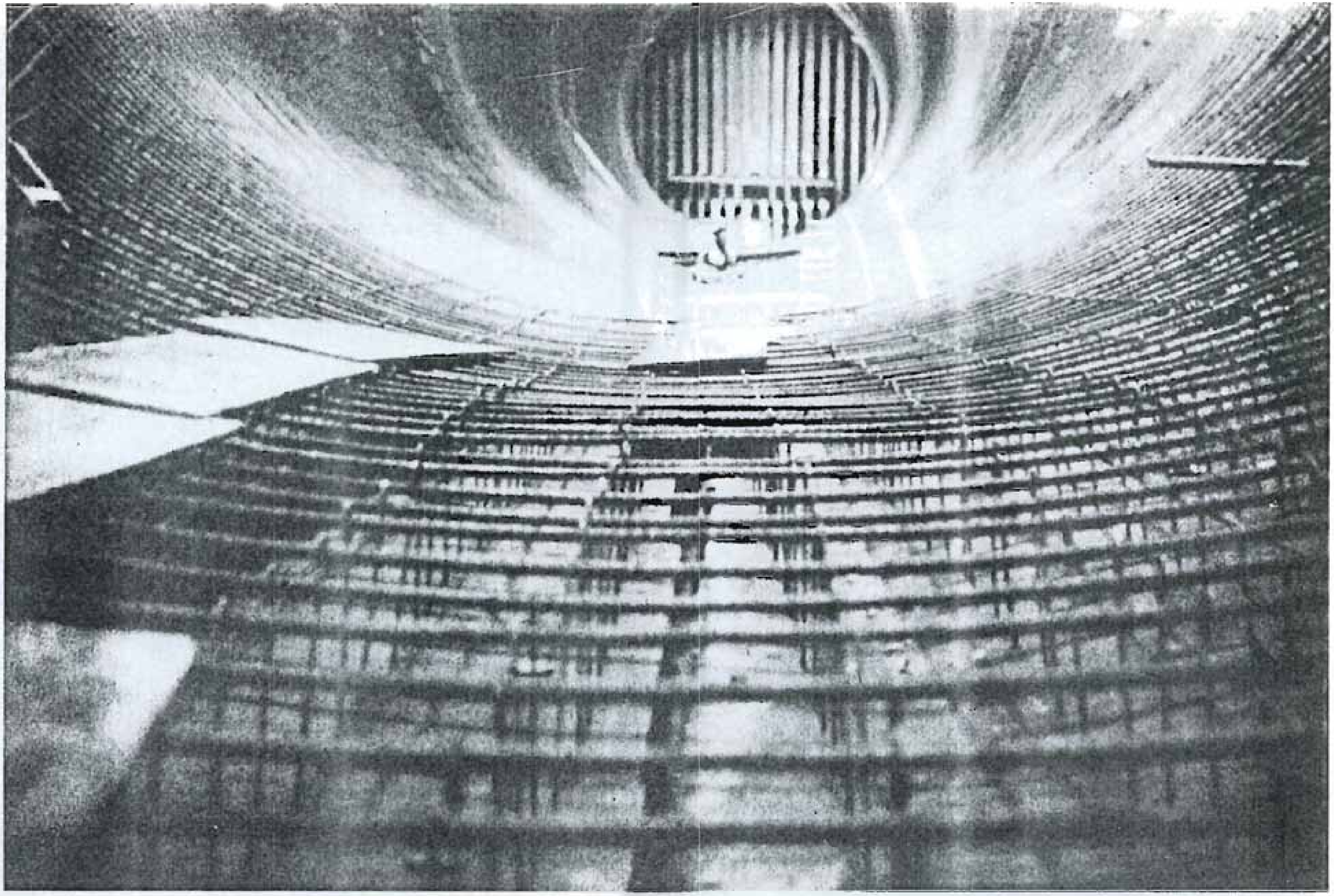
FORT McHENRY TUNNEL
BALTIMORE, MARYLAND
(1987)

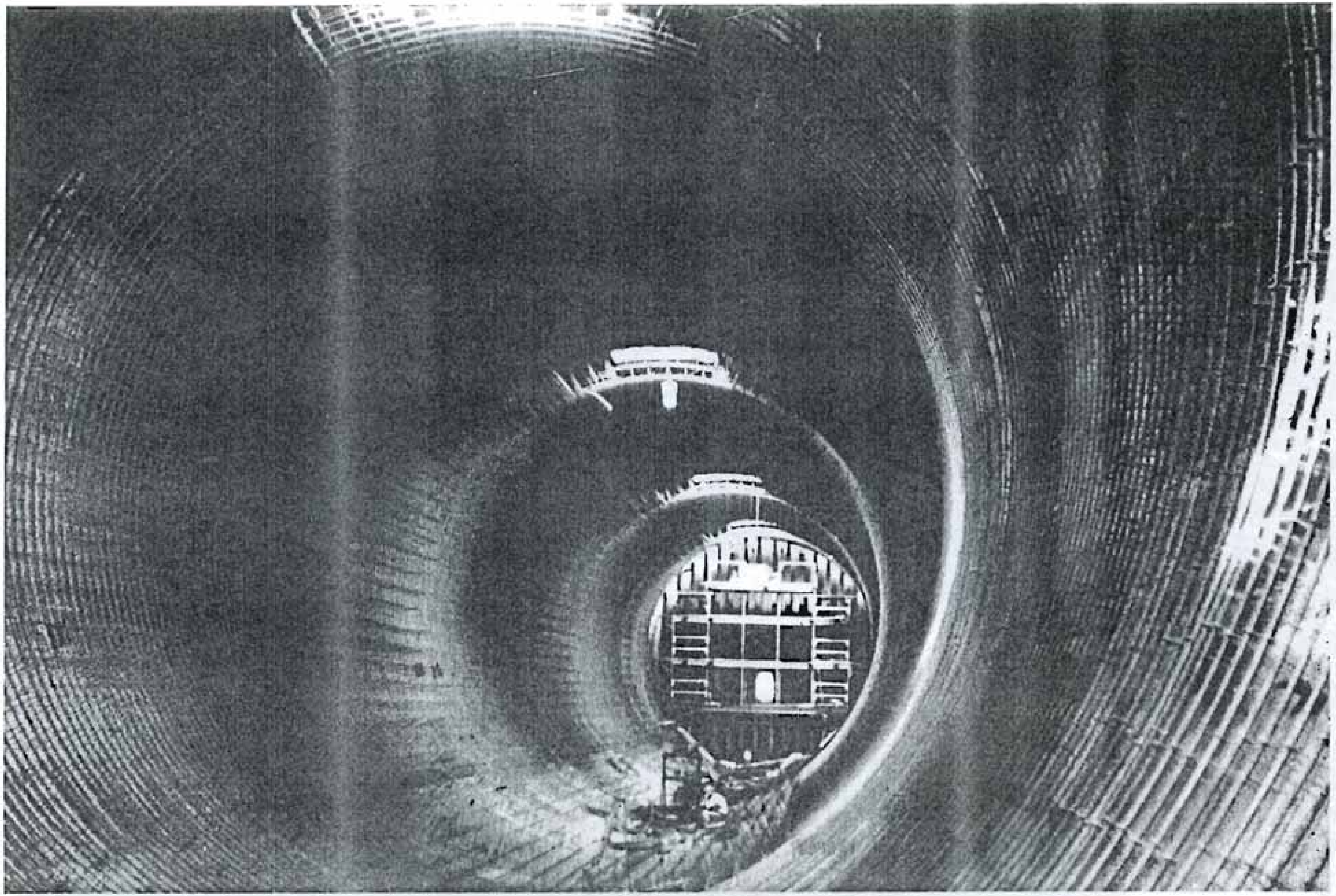


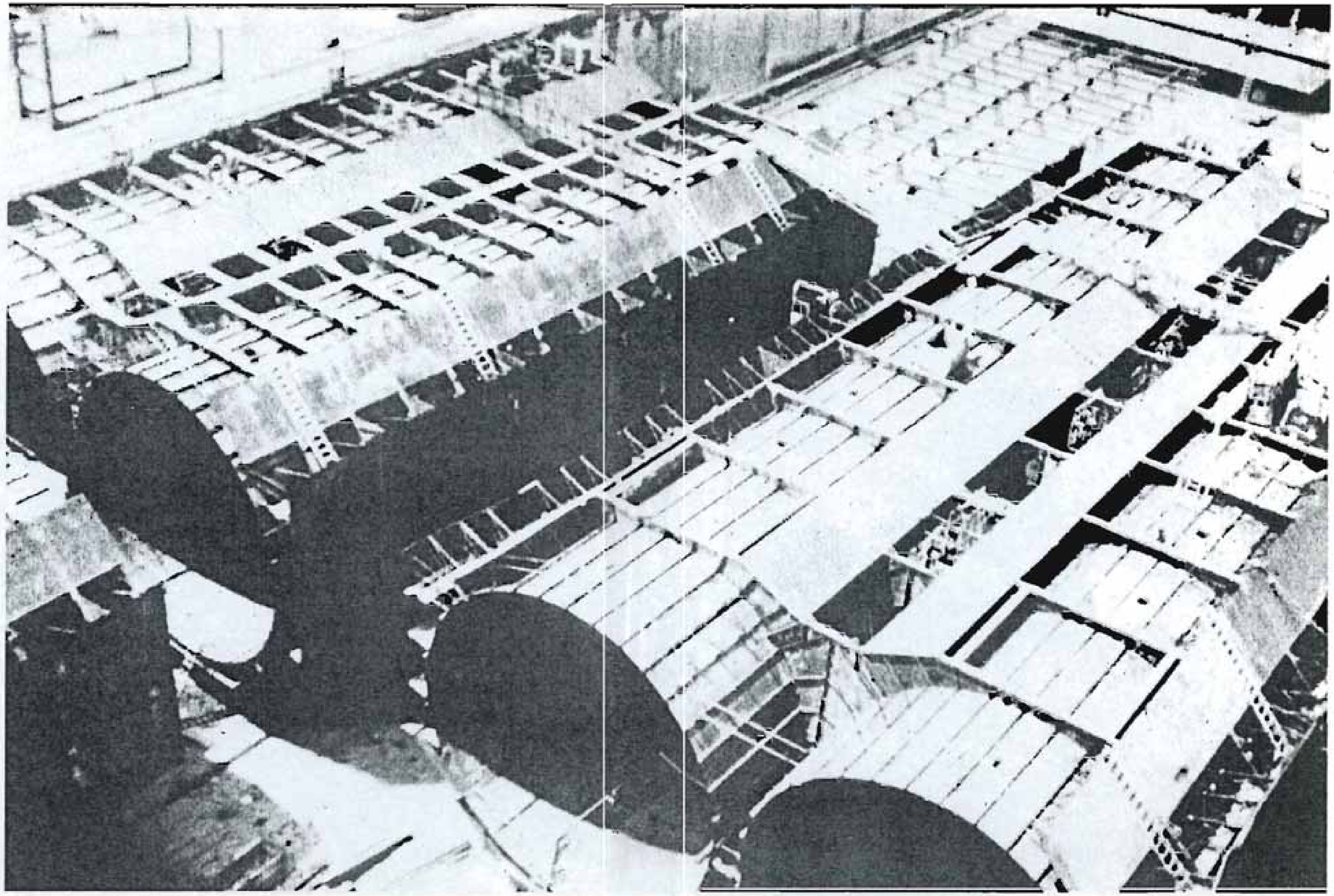
Plan and Profile of Fort McHenry Tunnel

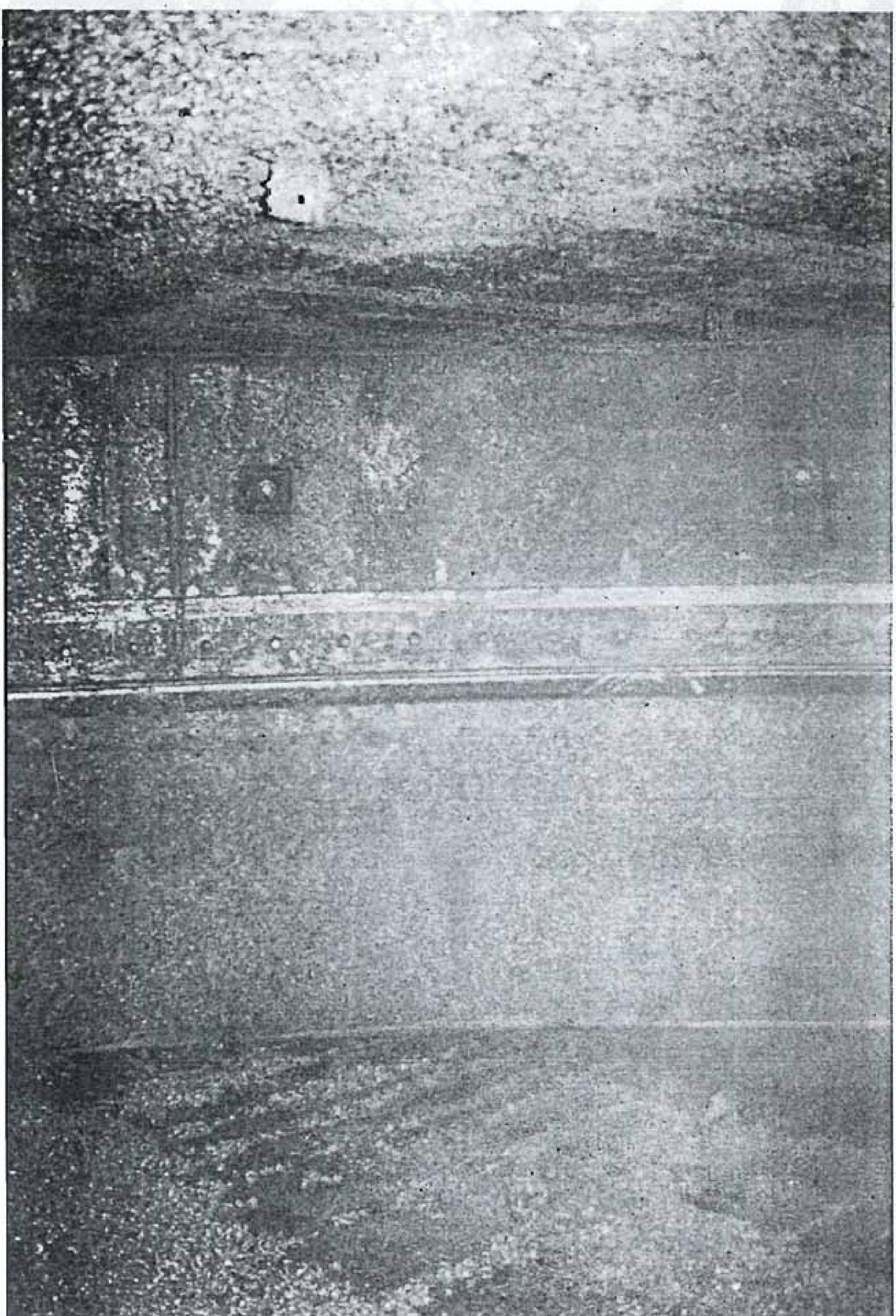

EAST

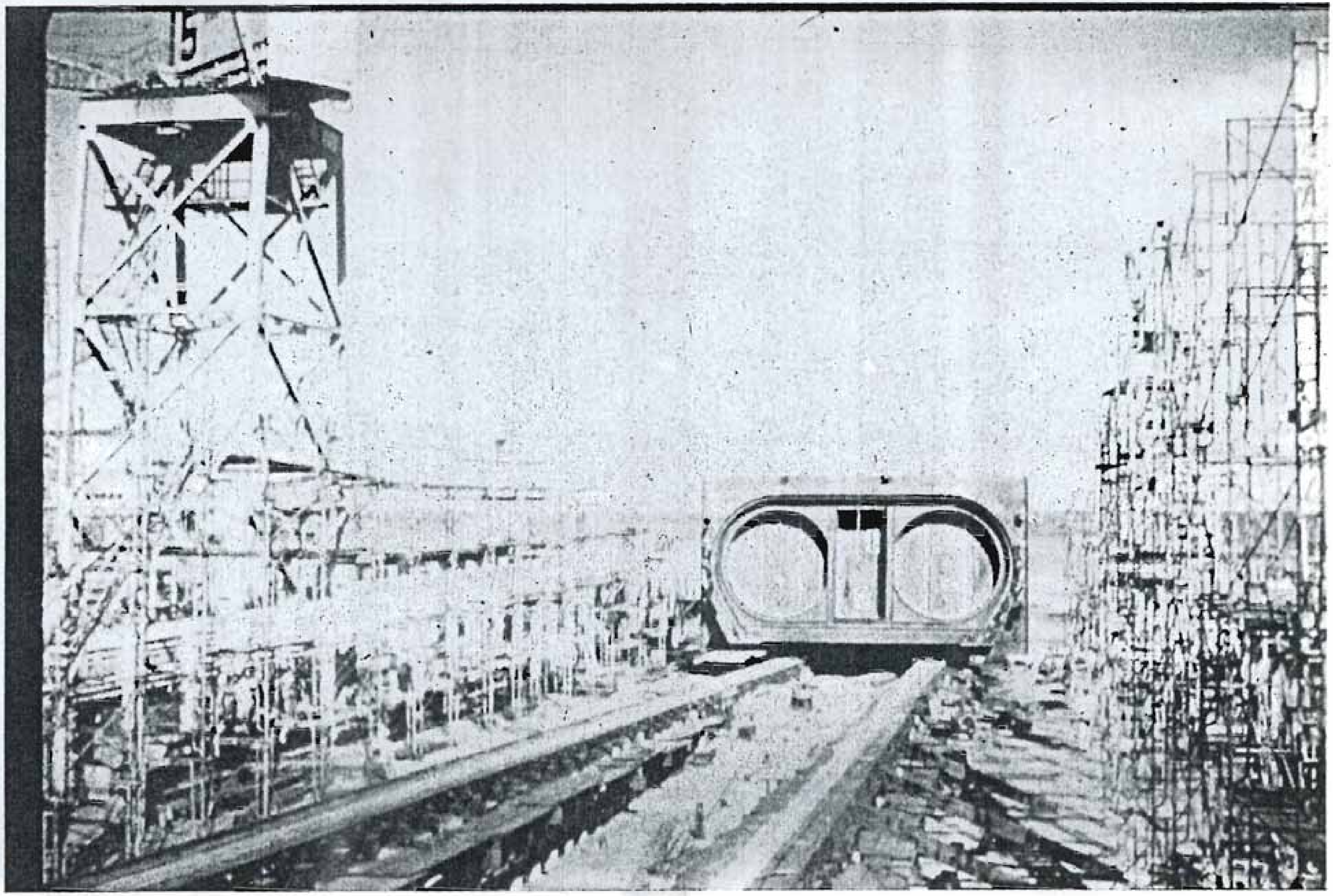








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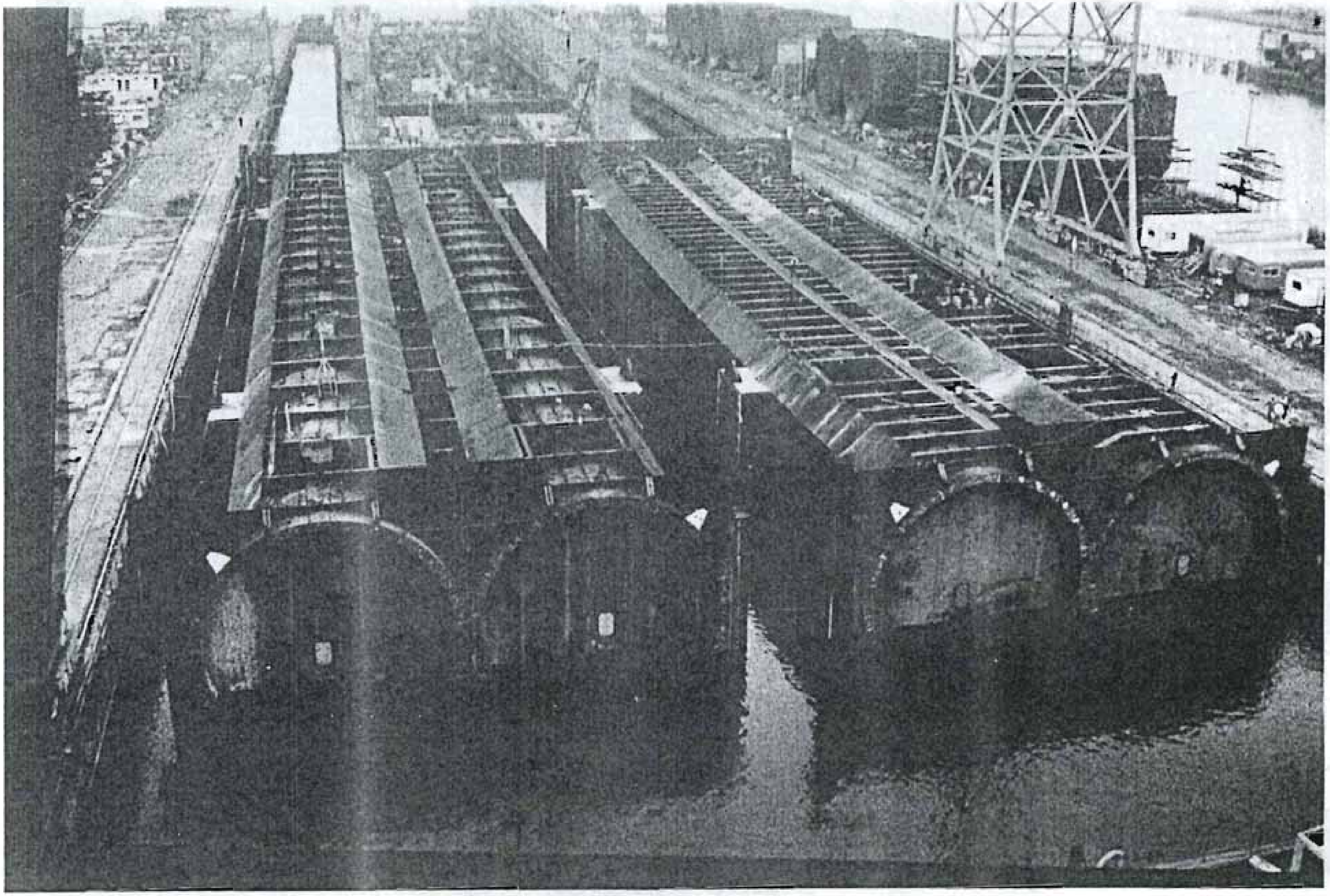
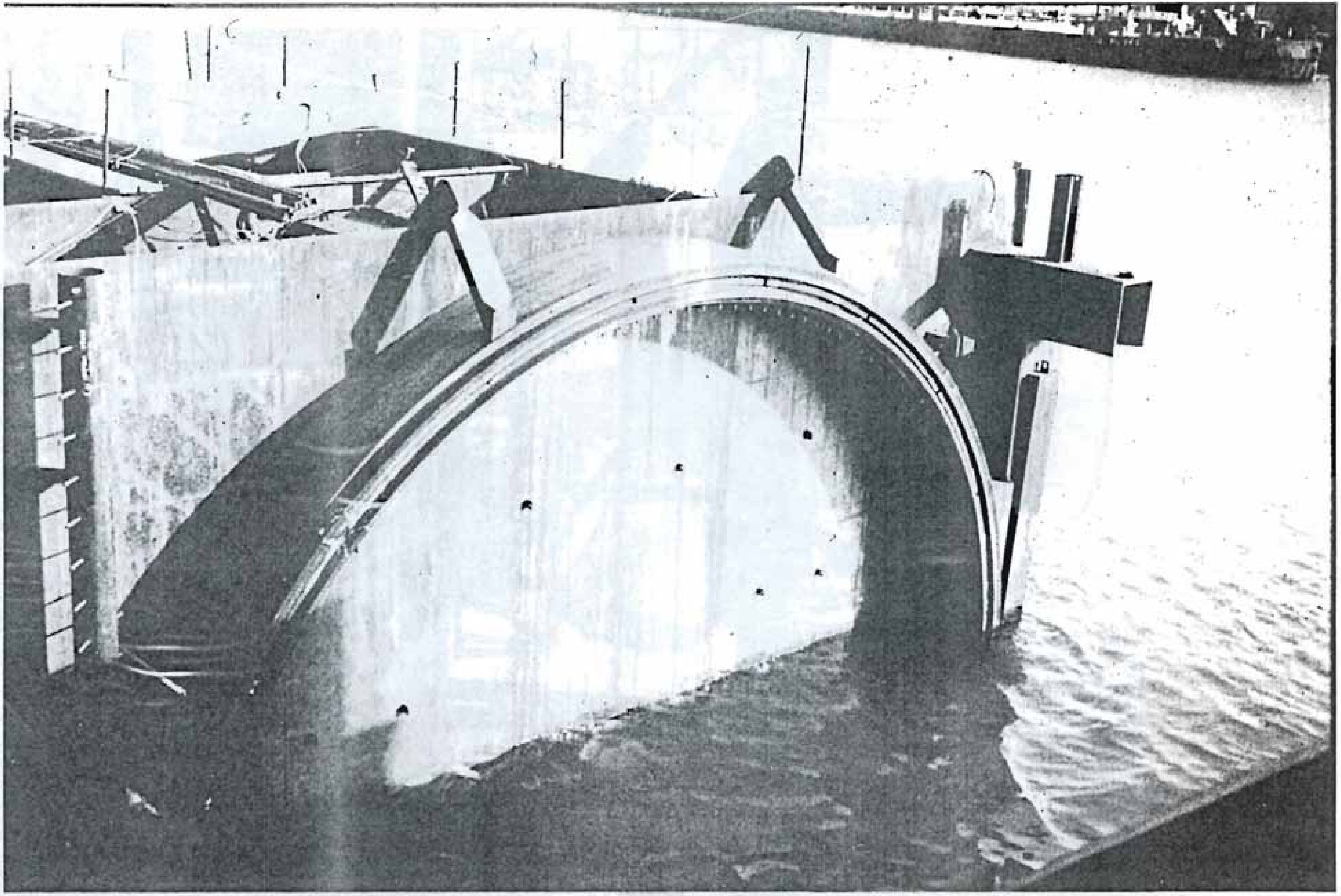
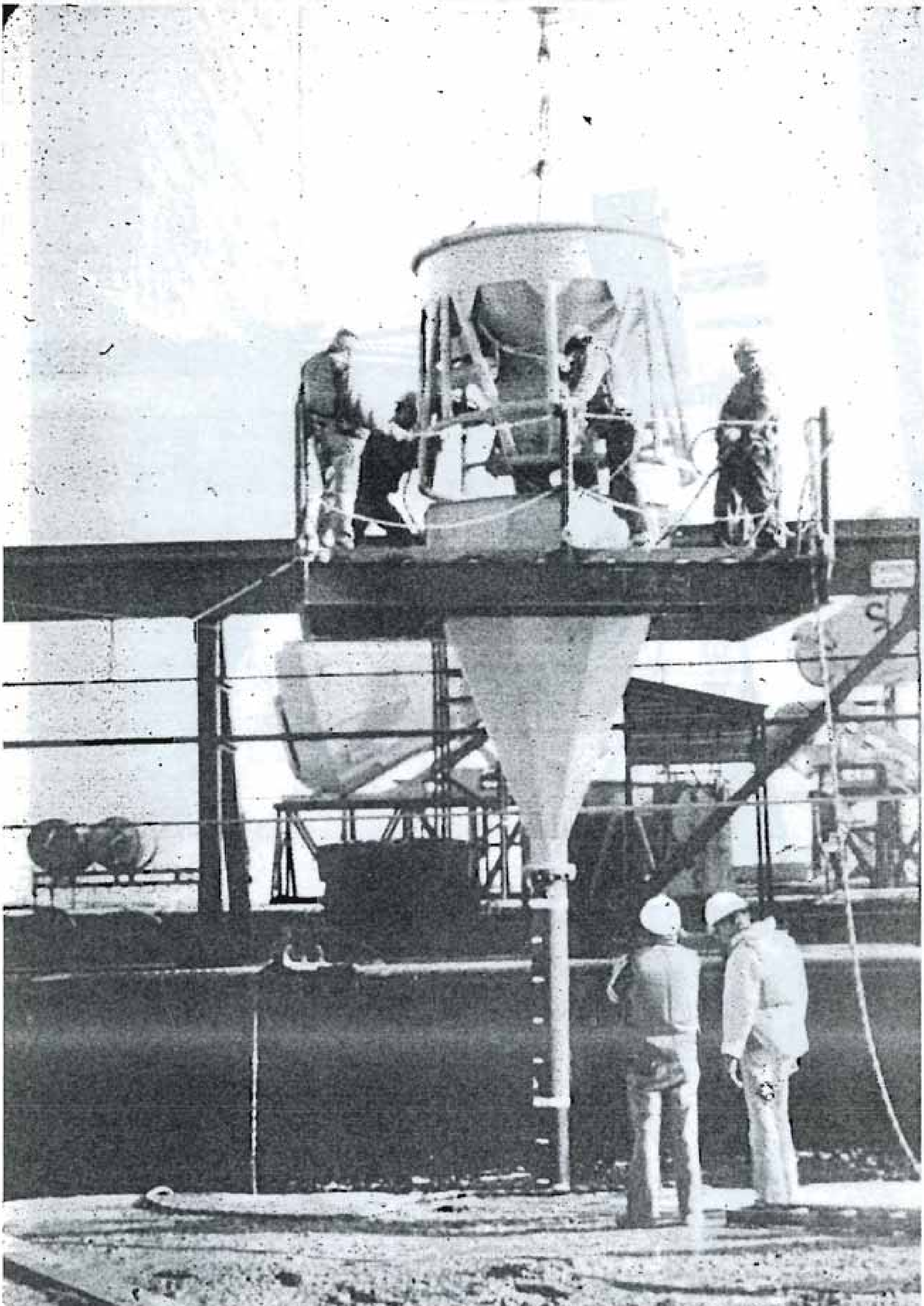
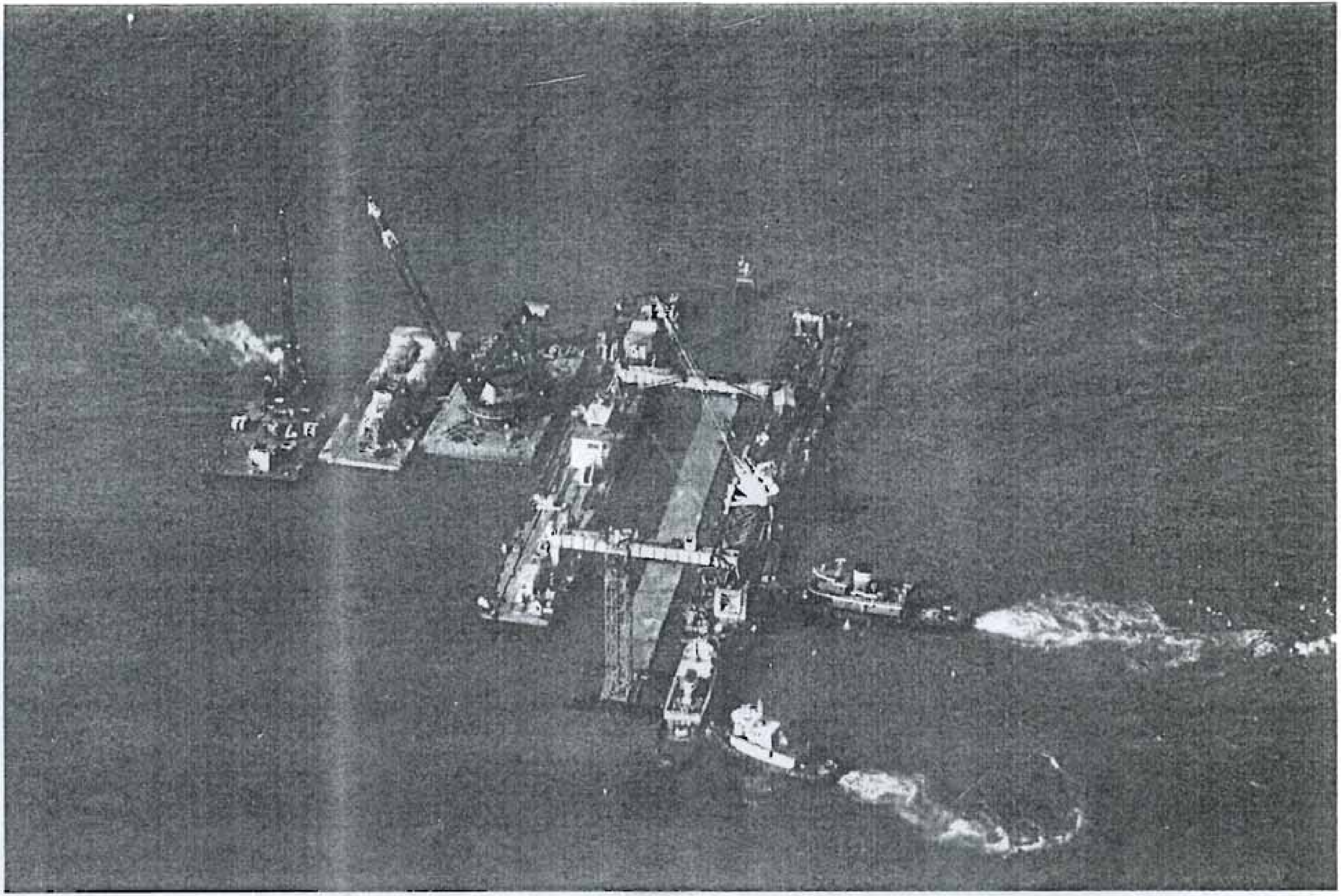
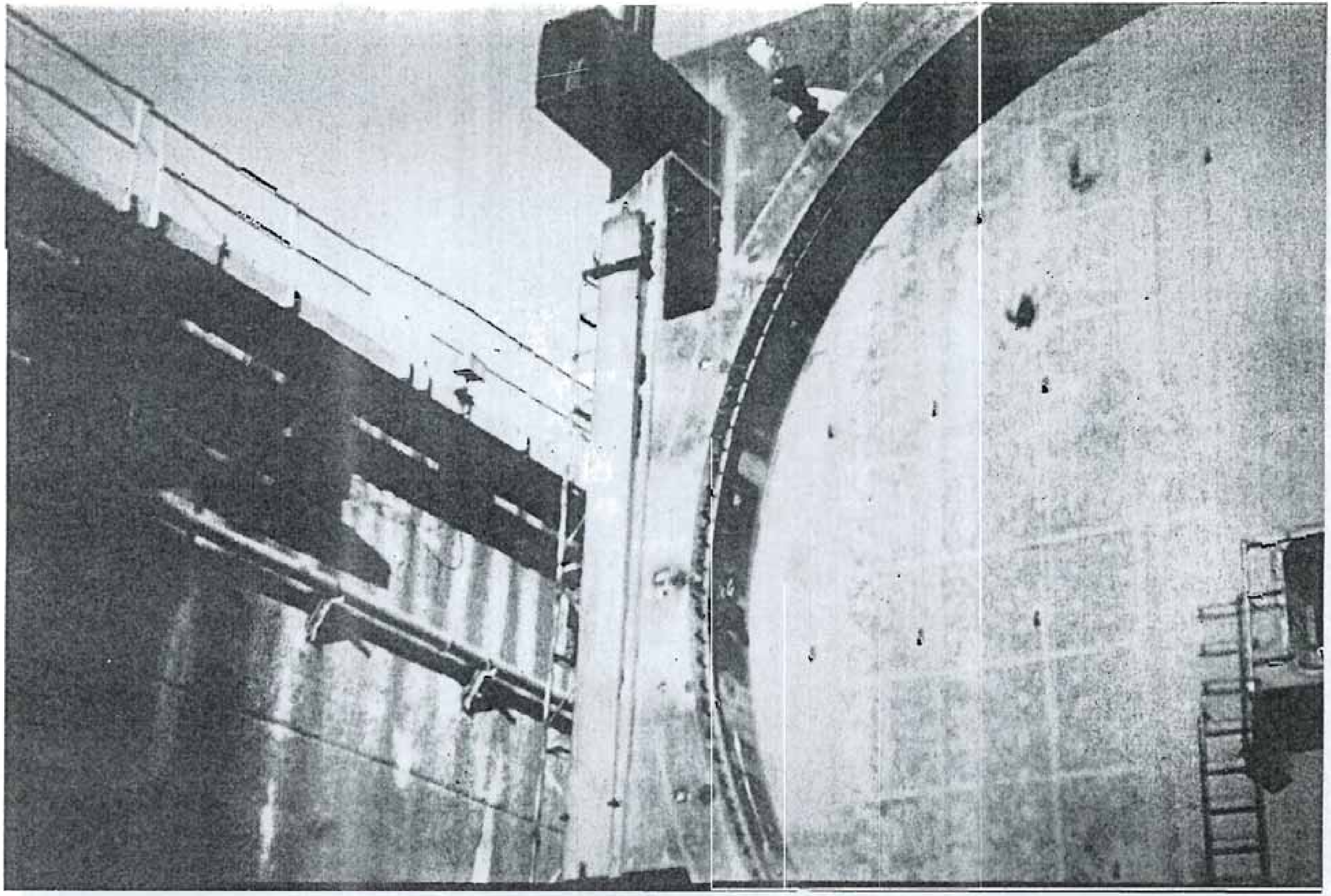
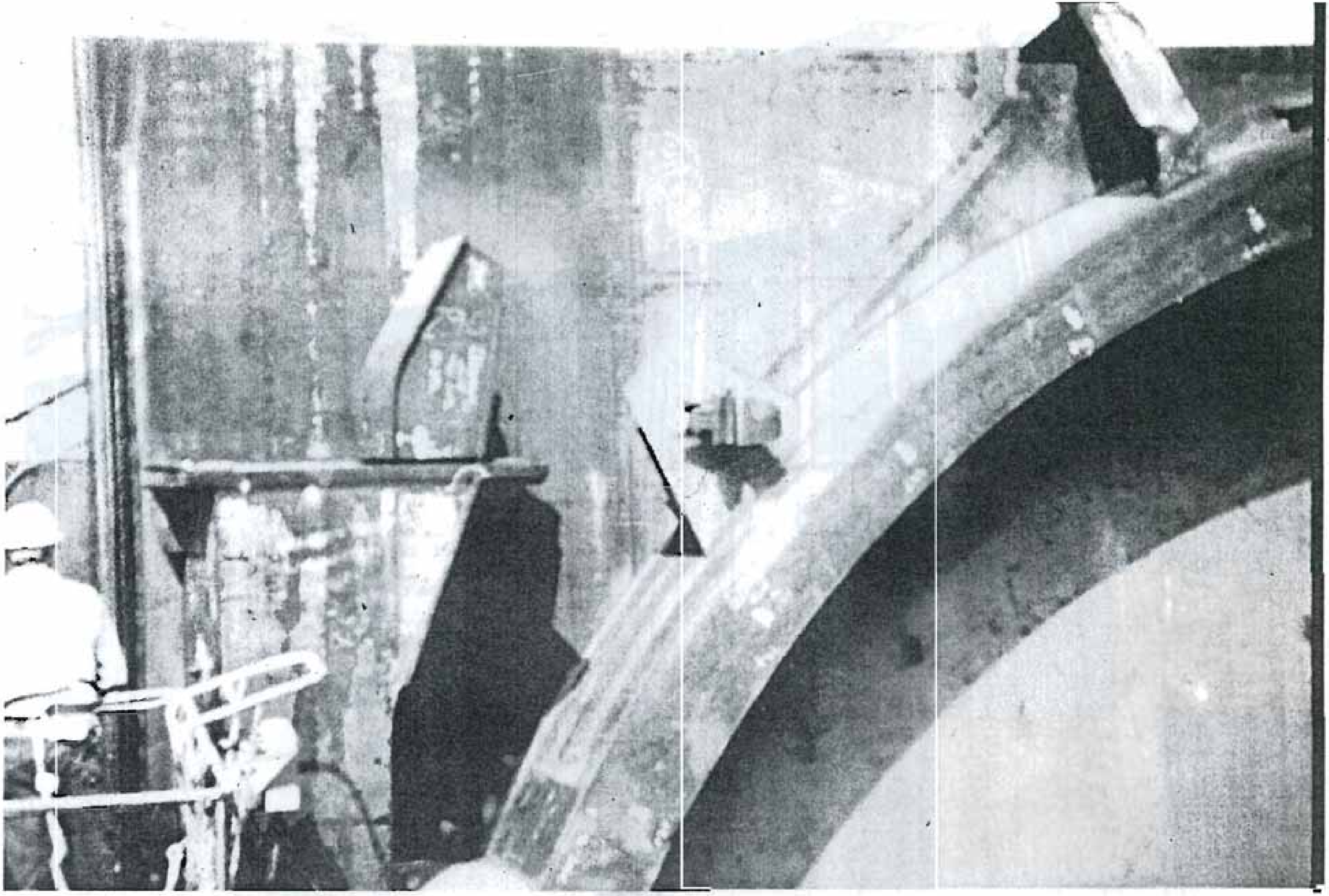


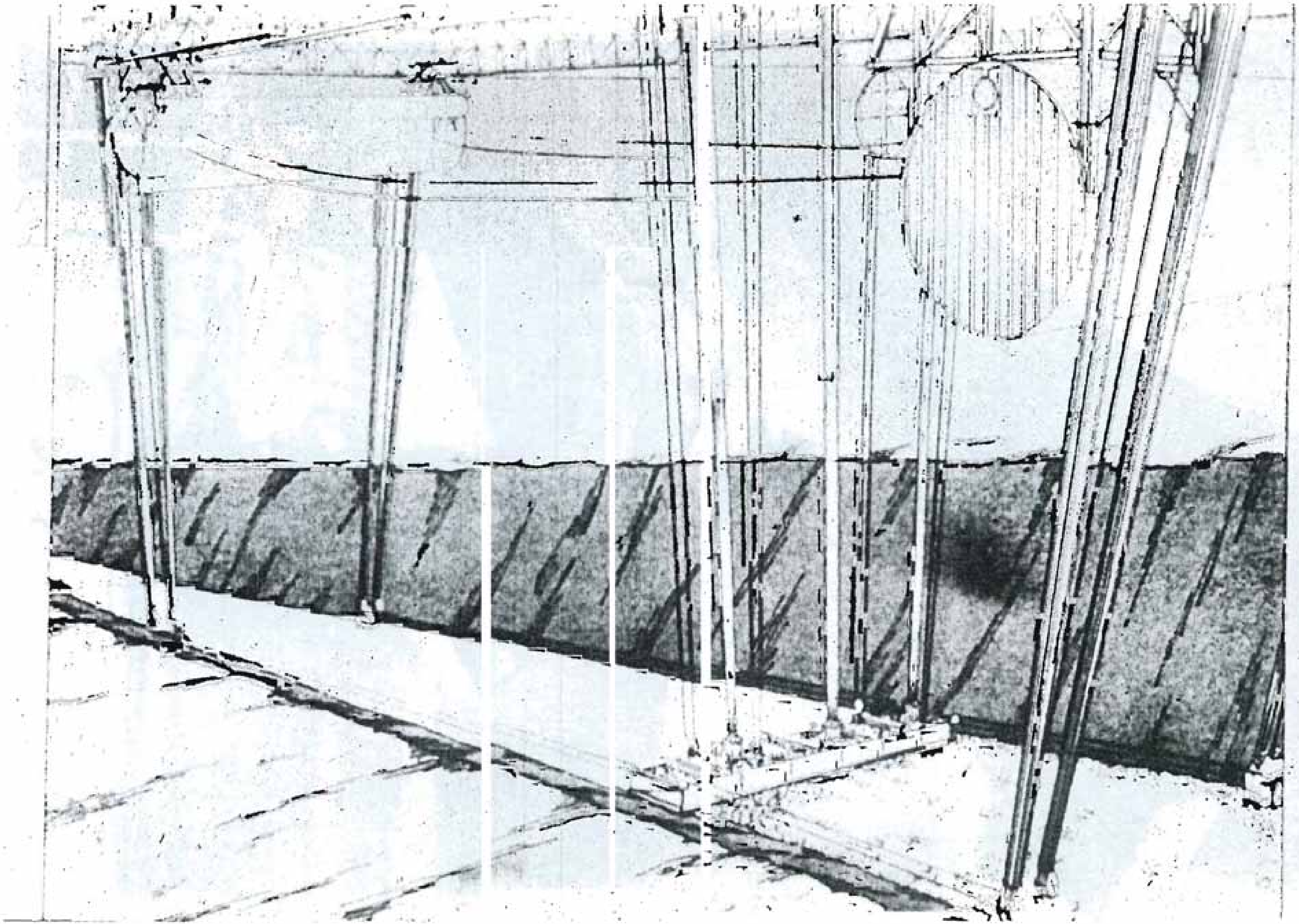


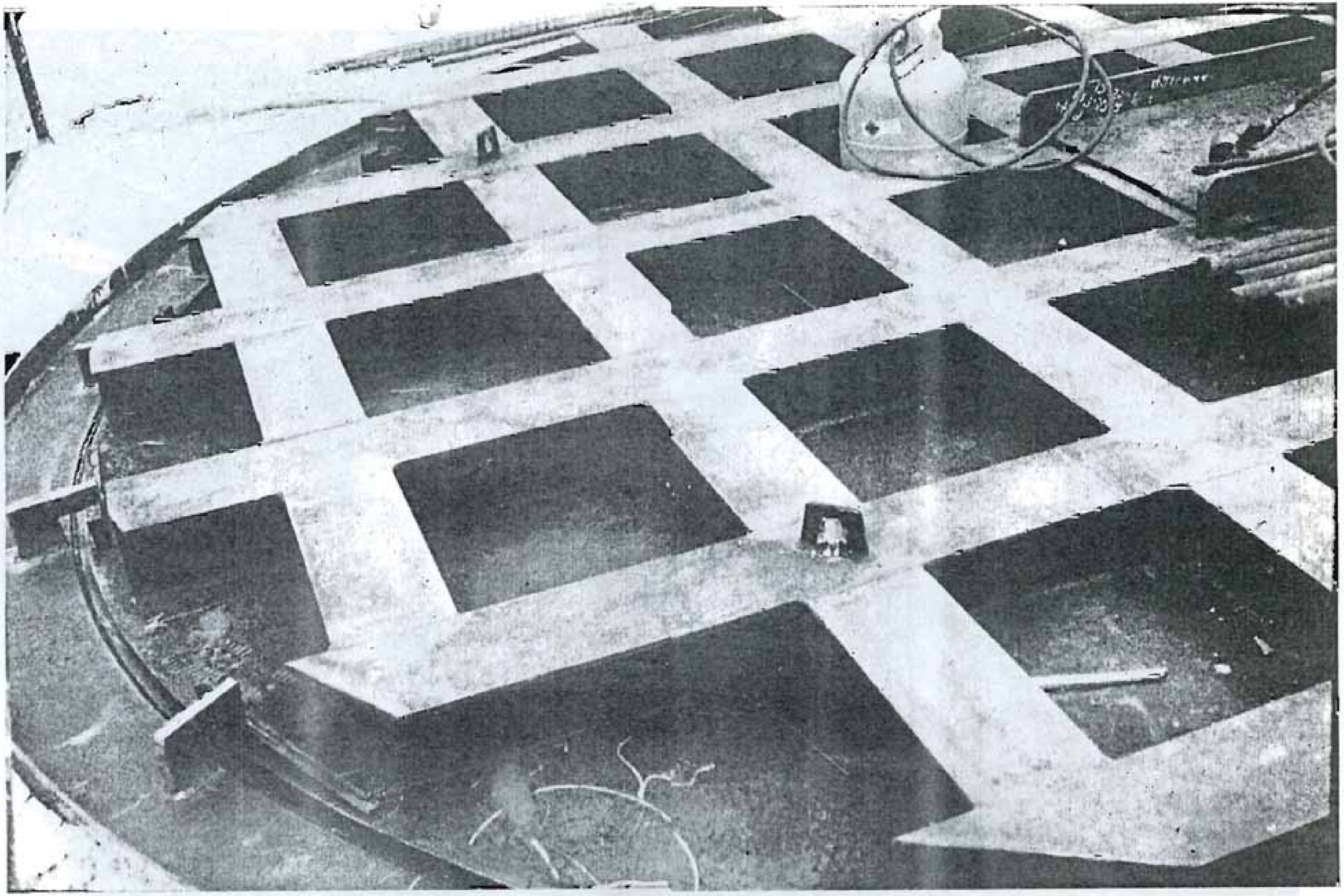


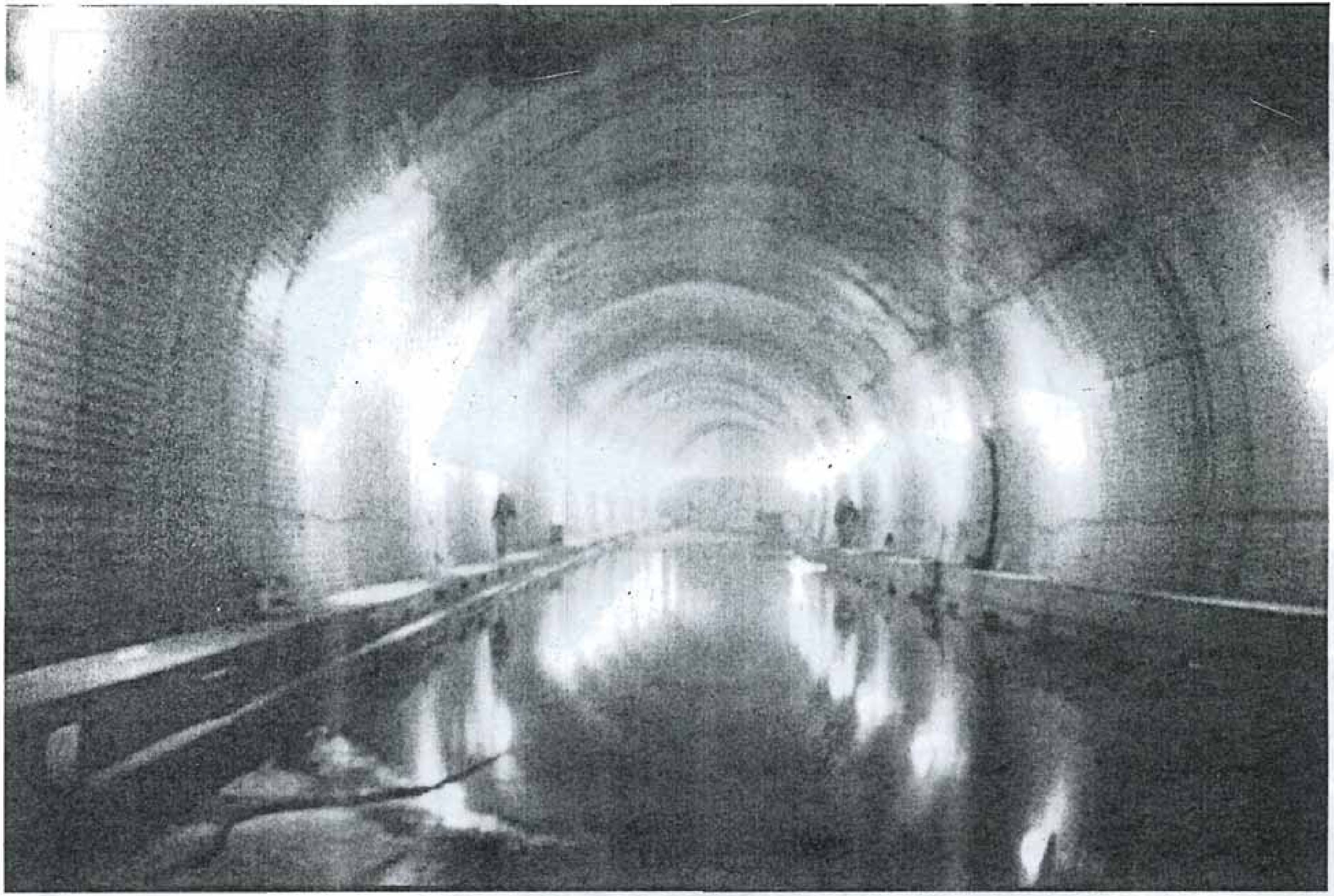


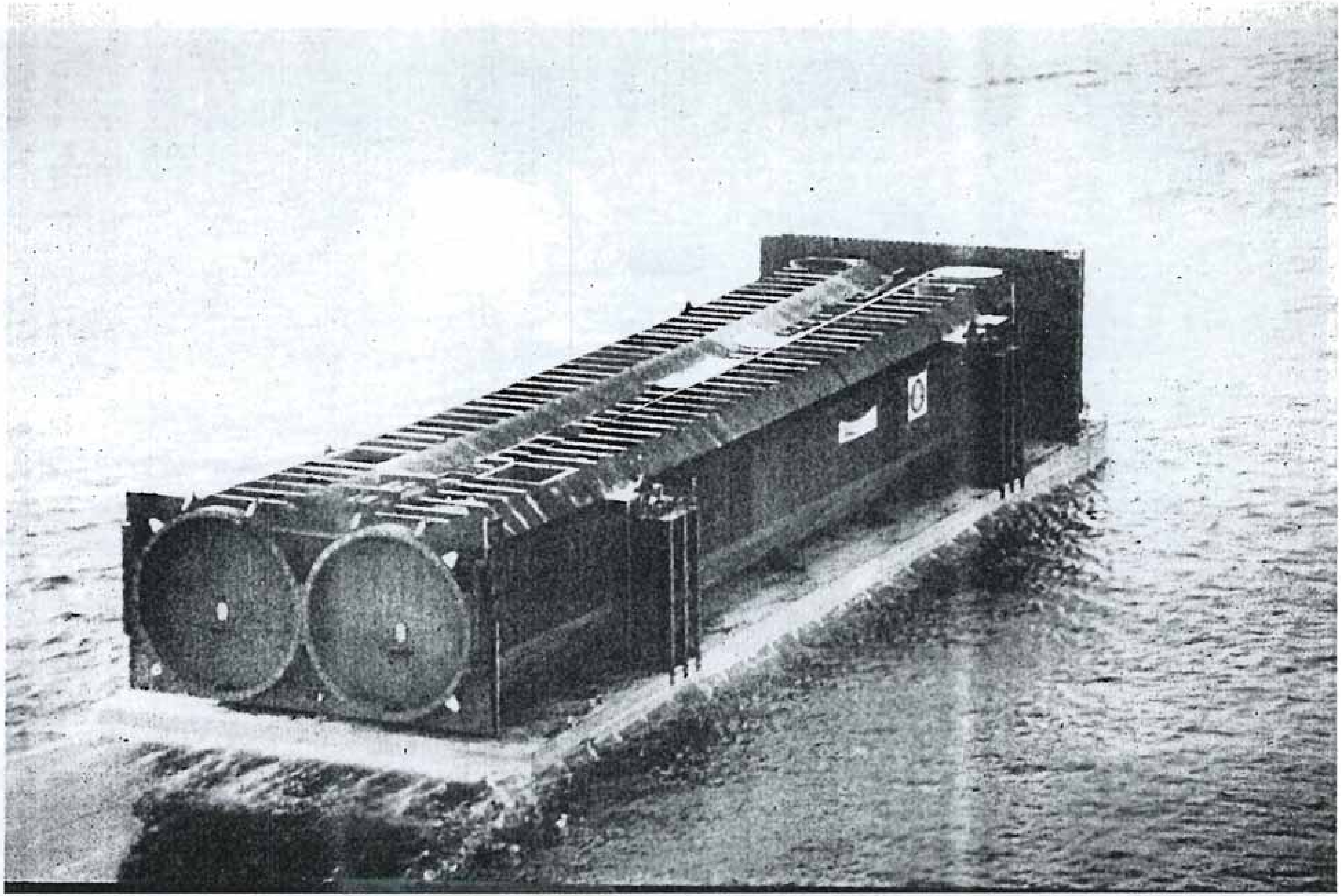


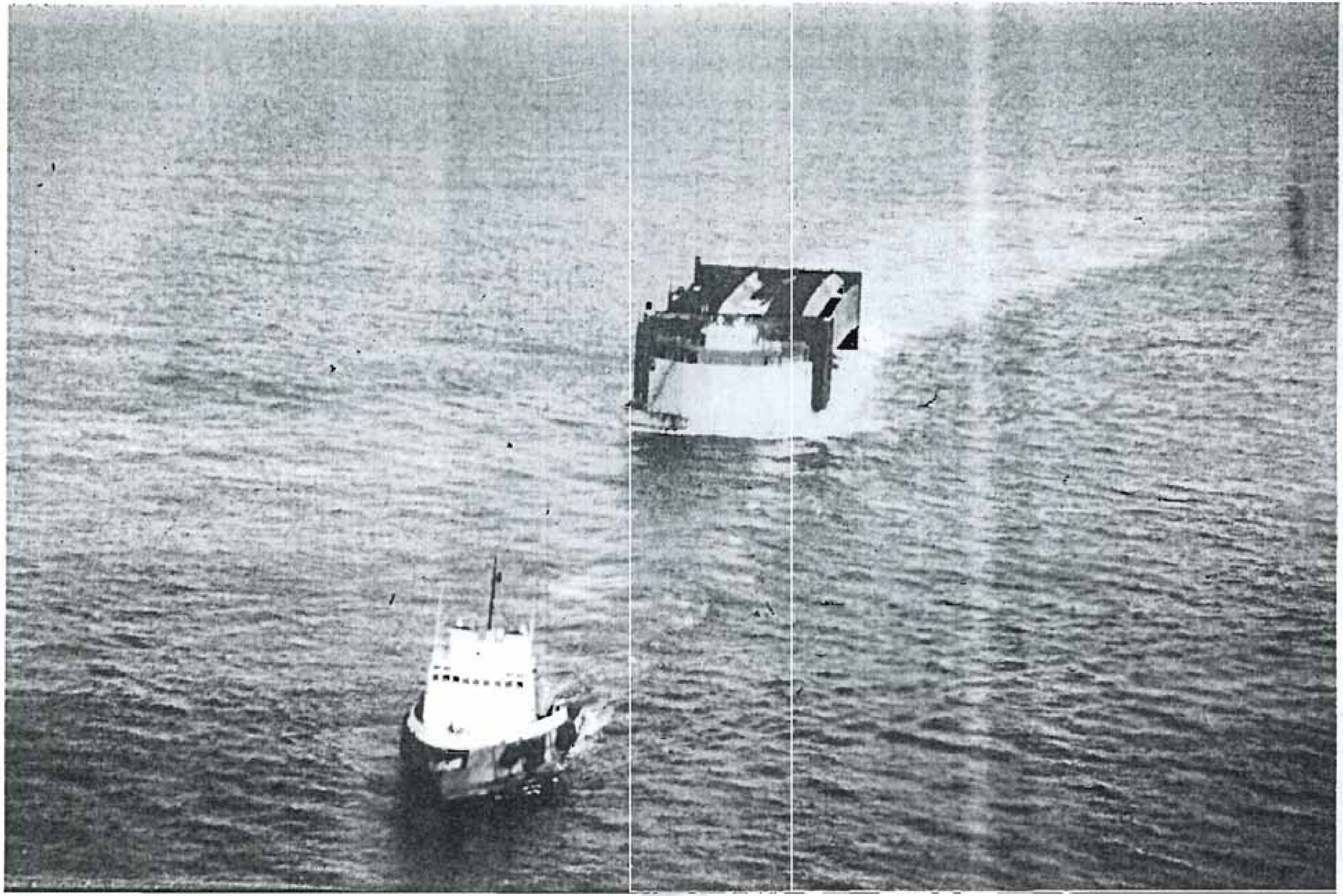












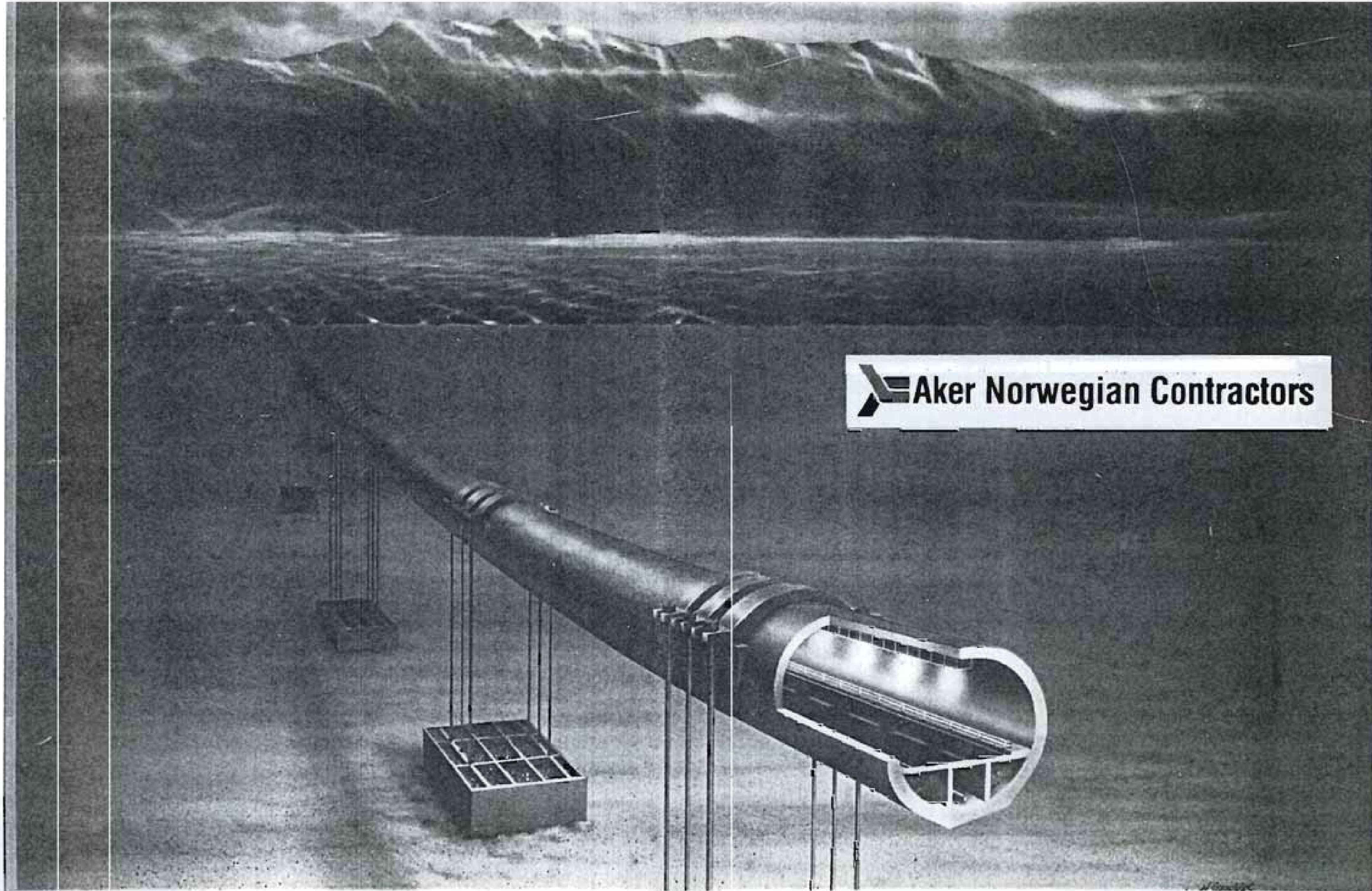
International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

12. Constructional details

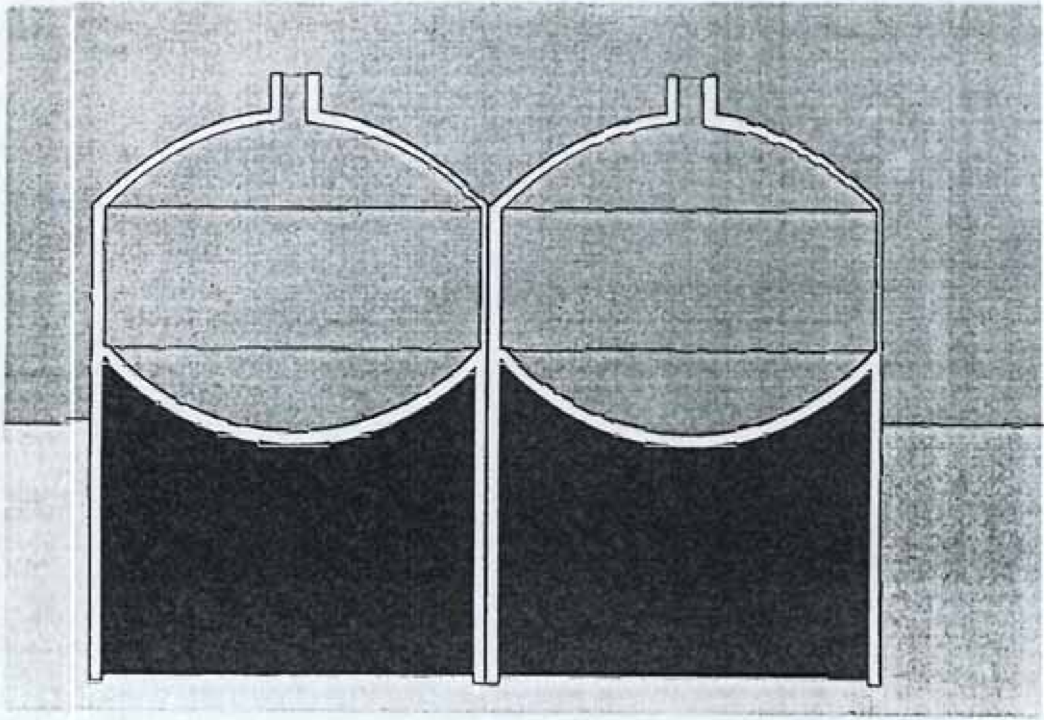
12.2 Tension anchoring

A. Nyhus
Aker Norwegian Contractors

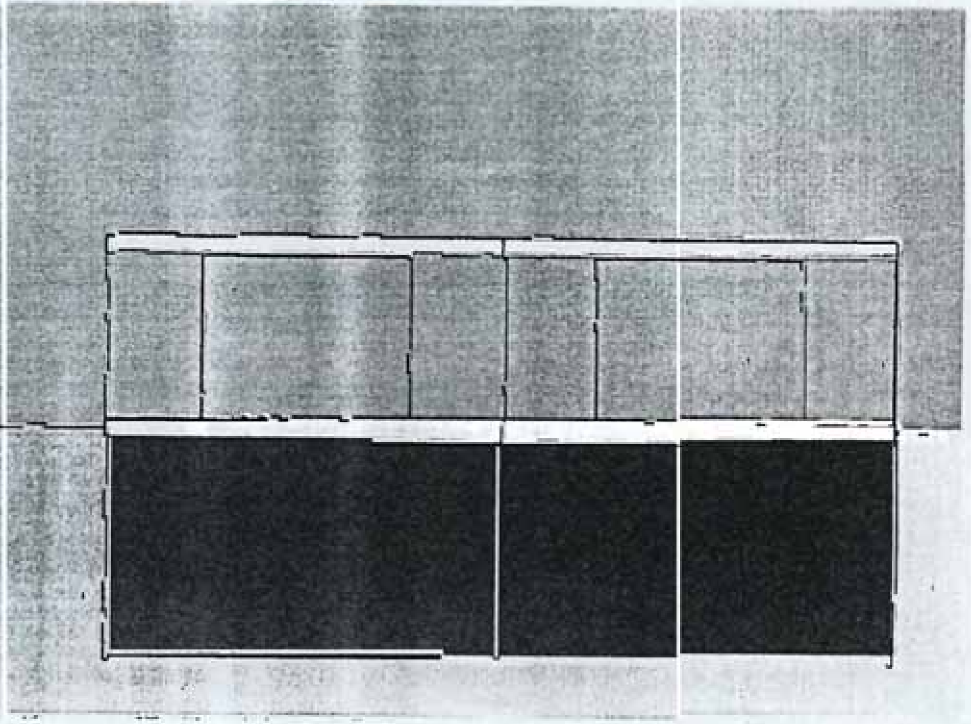

Aker Norwegian Contractors

"The Atlantic Alliance"

TLP anchor foundations
Suction piles



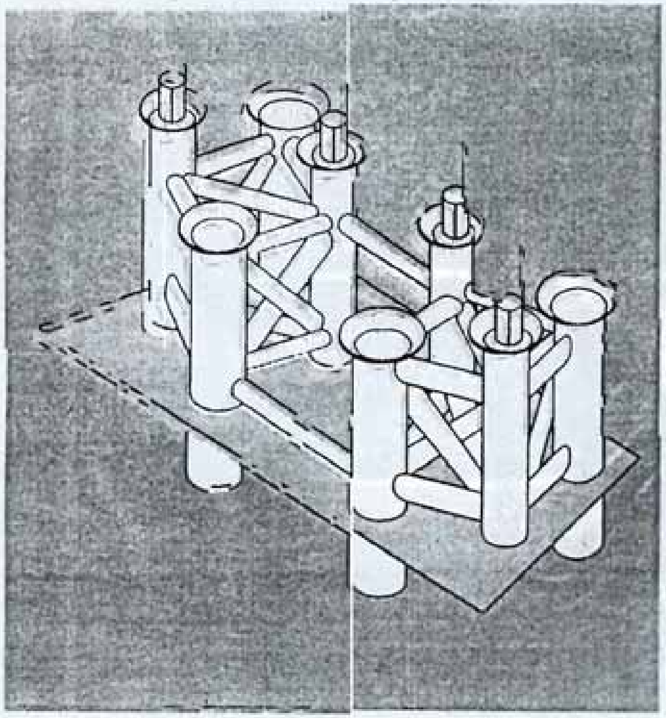
3-cell concrete self-float



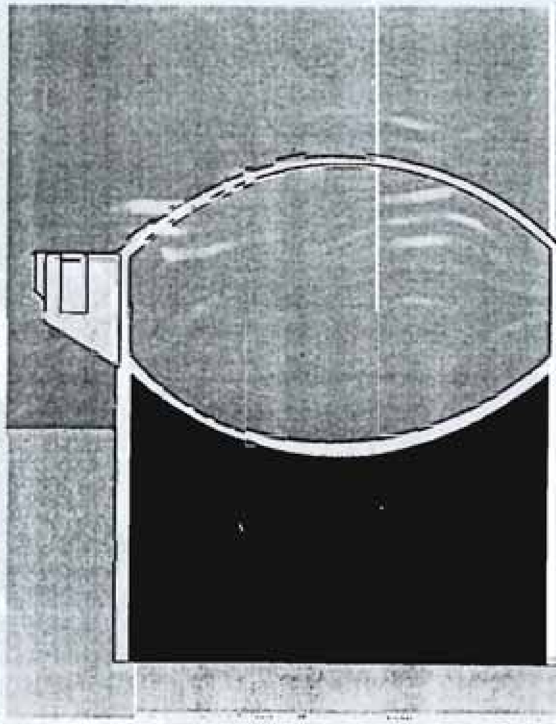
3-cell steel, self-float

"The Atlantic Alliance"

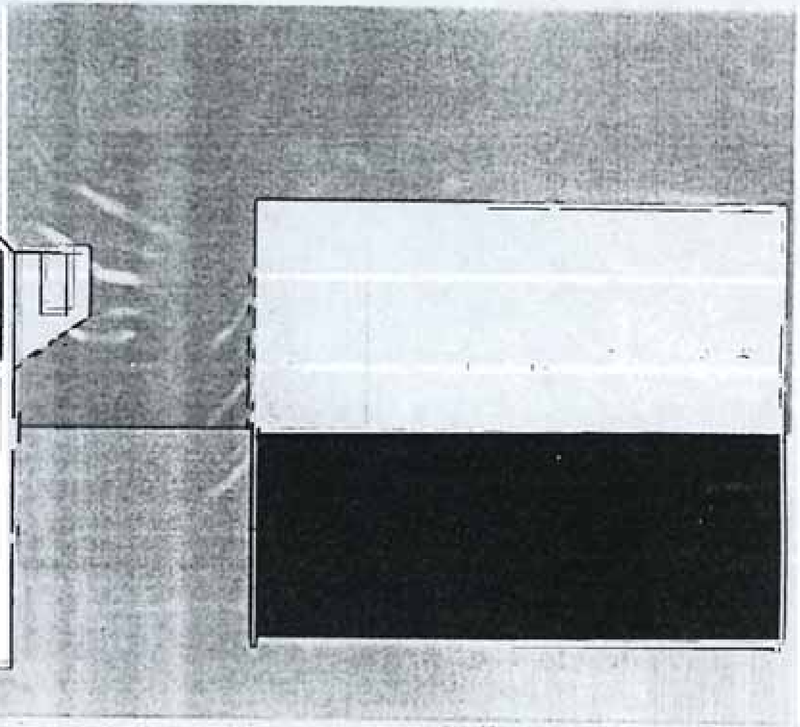
TLP anchor foundations
Piles



Alternative driven piles with frame

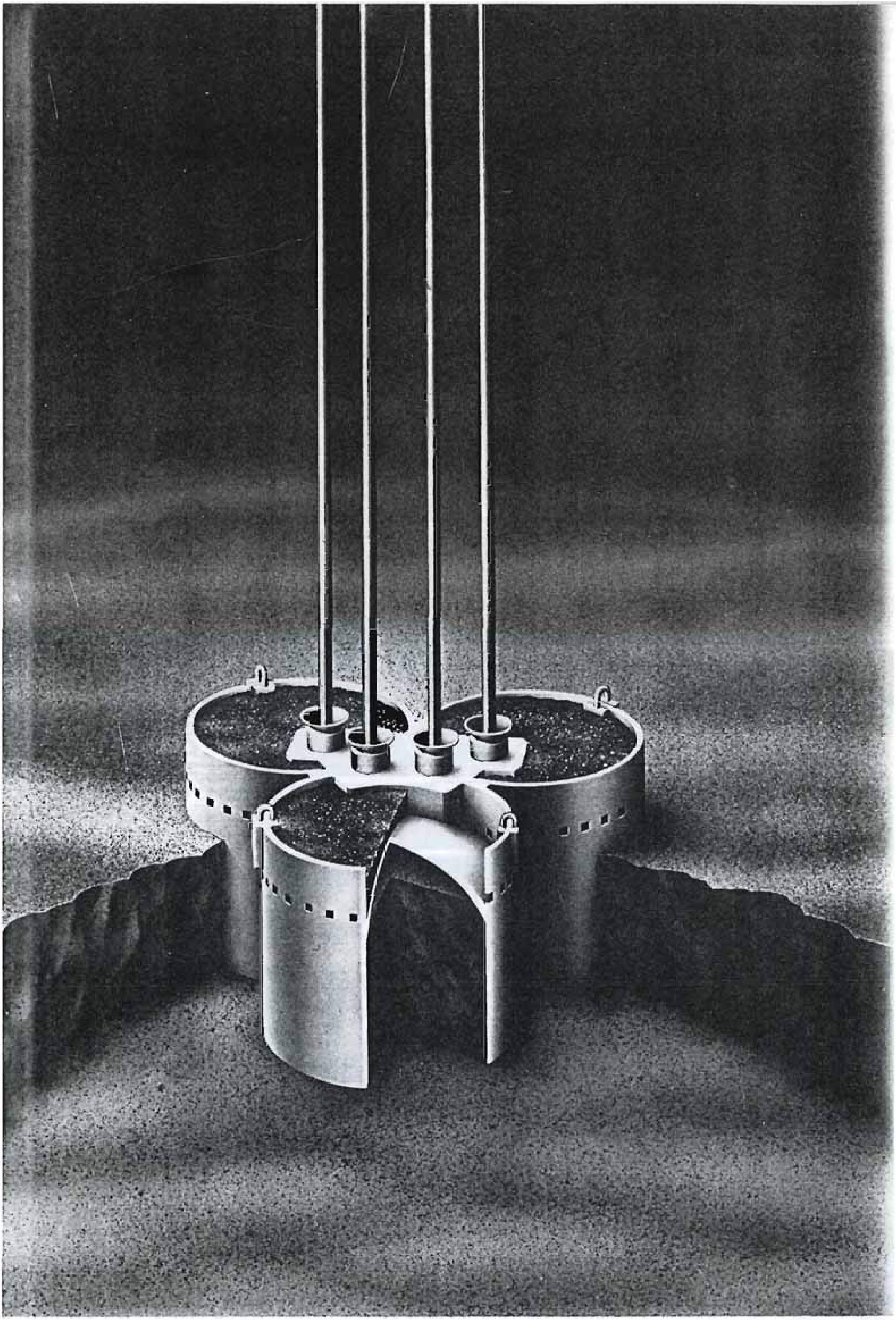


Super suction pile in concrete



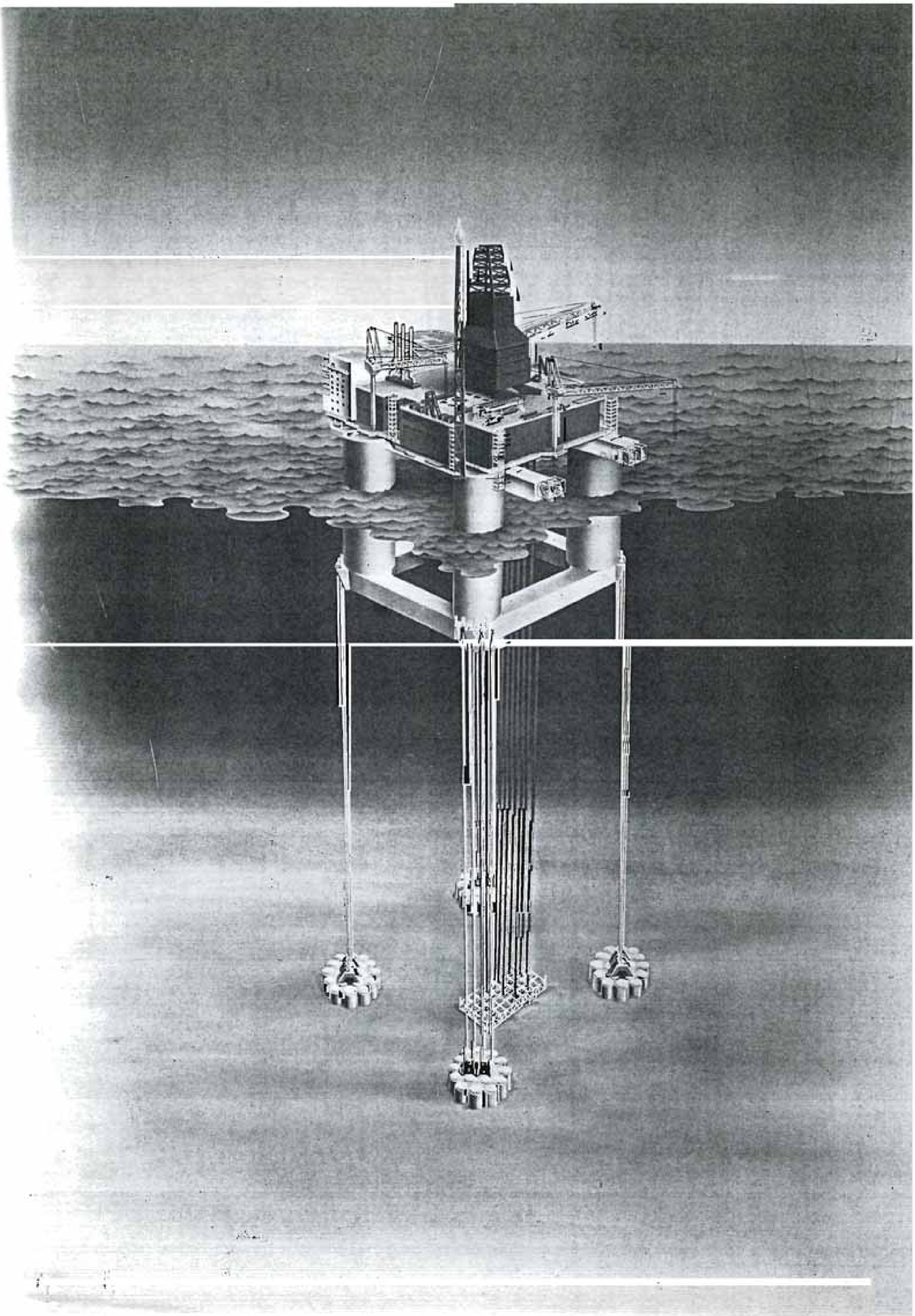
Super suction pile in steel

L-6562

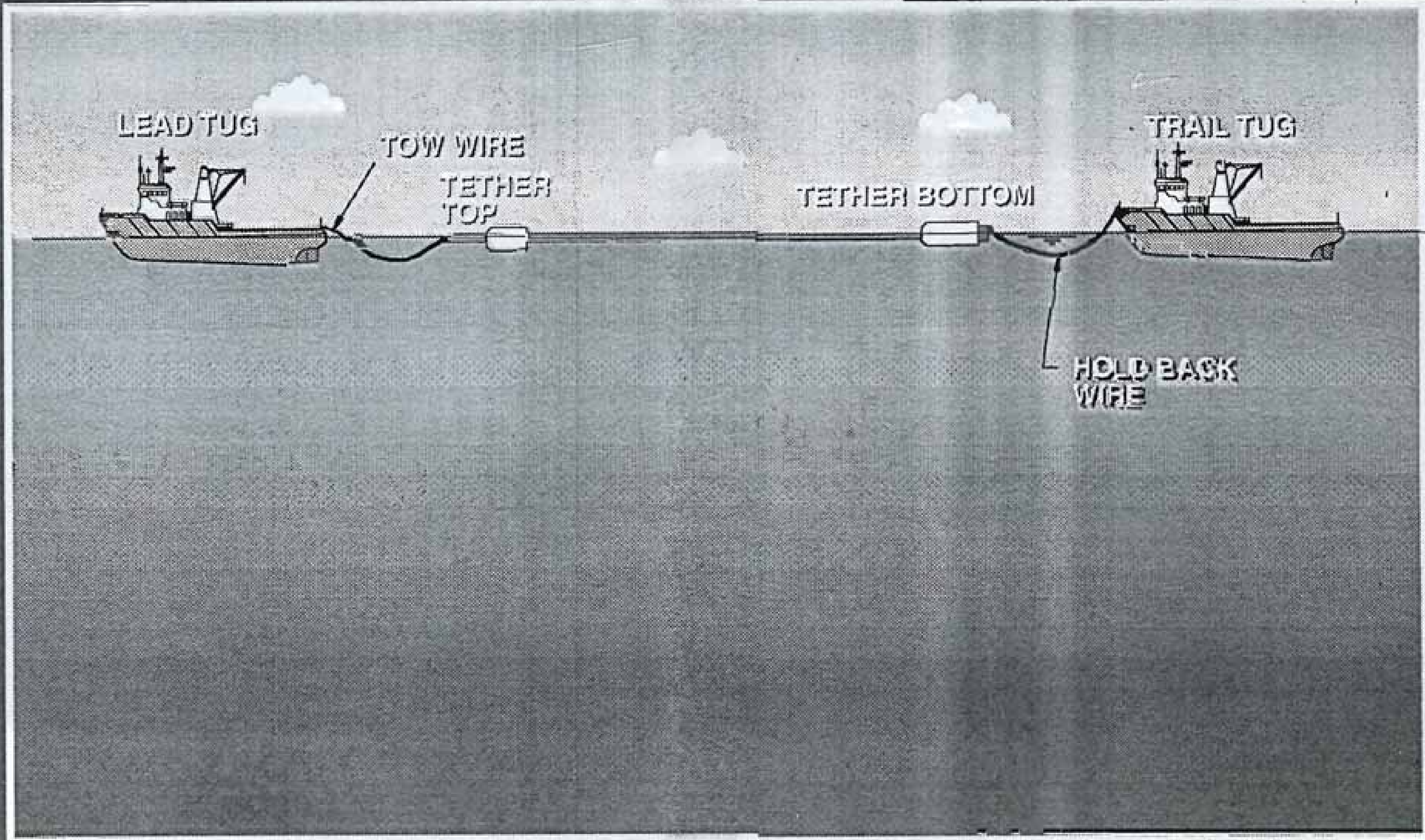


Tension Leg Platforms in the Offshore Industri

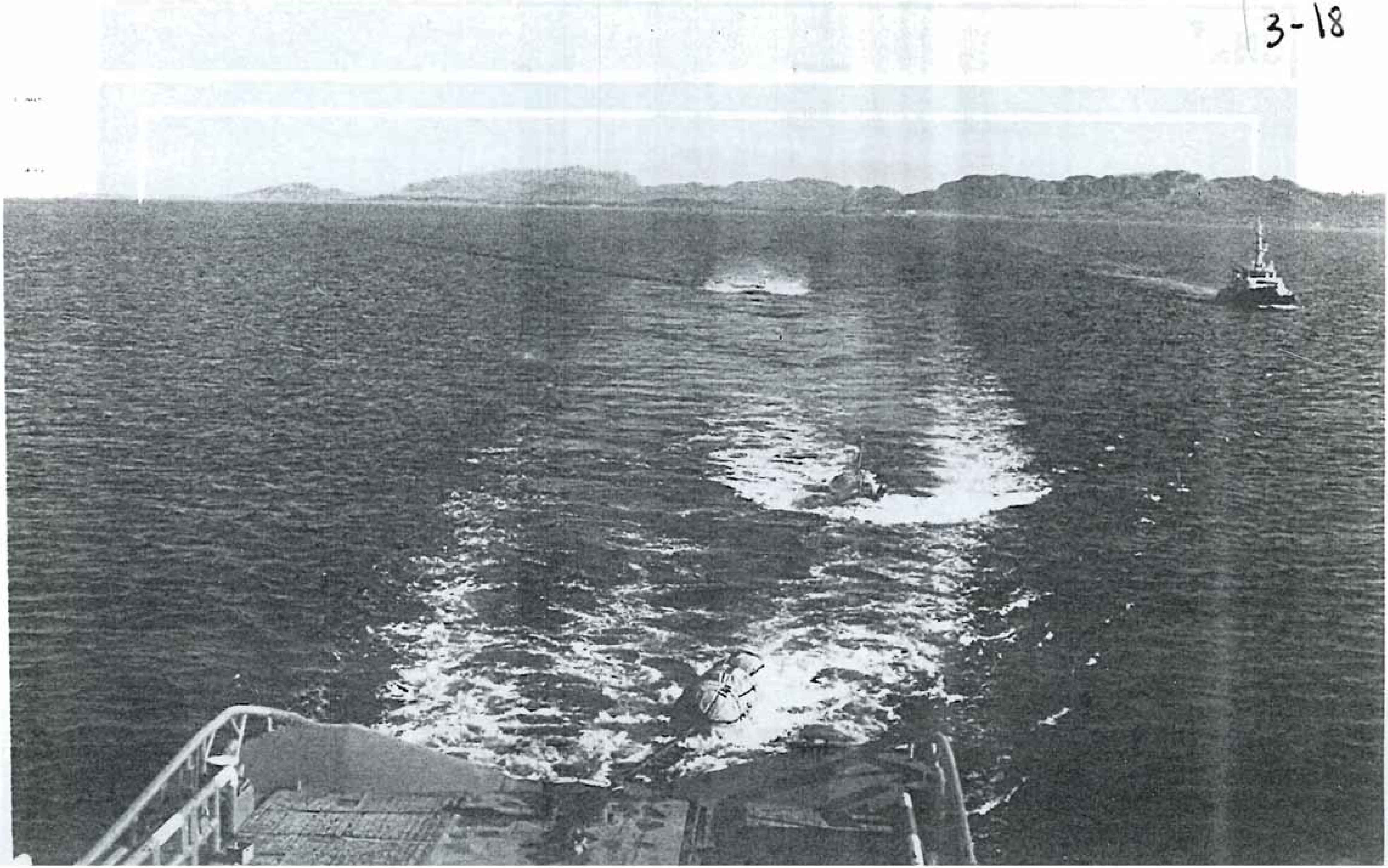
Platform	Inst.	Depth	Found.	Tethers
Hutton	1984	140m	Piled	Skrewed
Juliet	1989	540m	Piled	Full length
Snorre	1990	310m	Gravity	Screwed
Auger	1994	800?m	Piled	Screwed
Heidrun	1995	345m	Gravity	Full length
Mars	1996	910m	Piled	Screwed



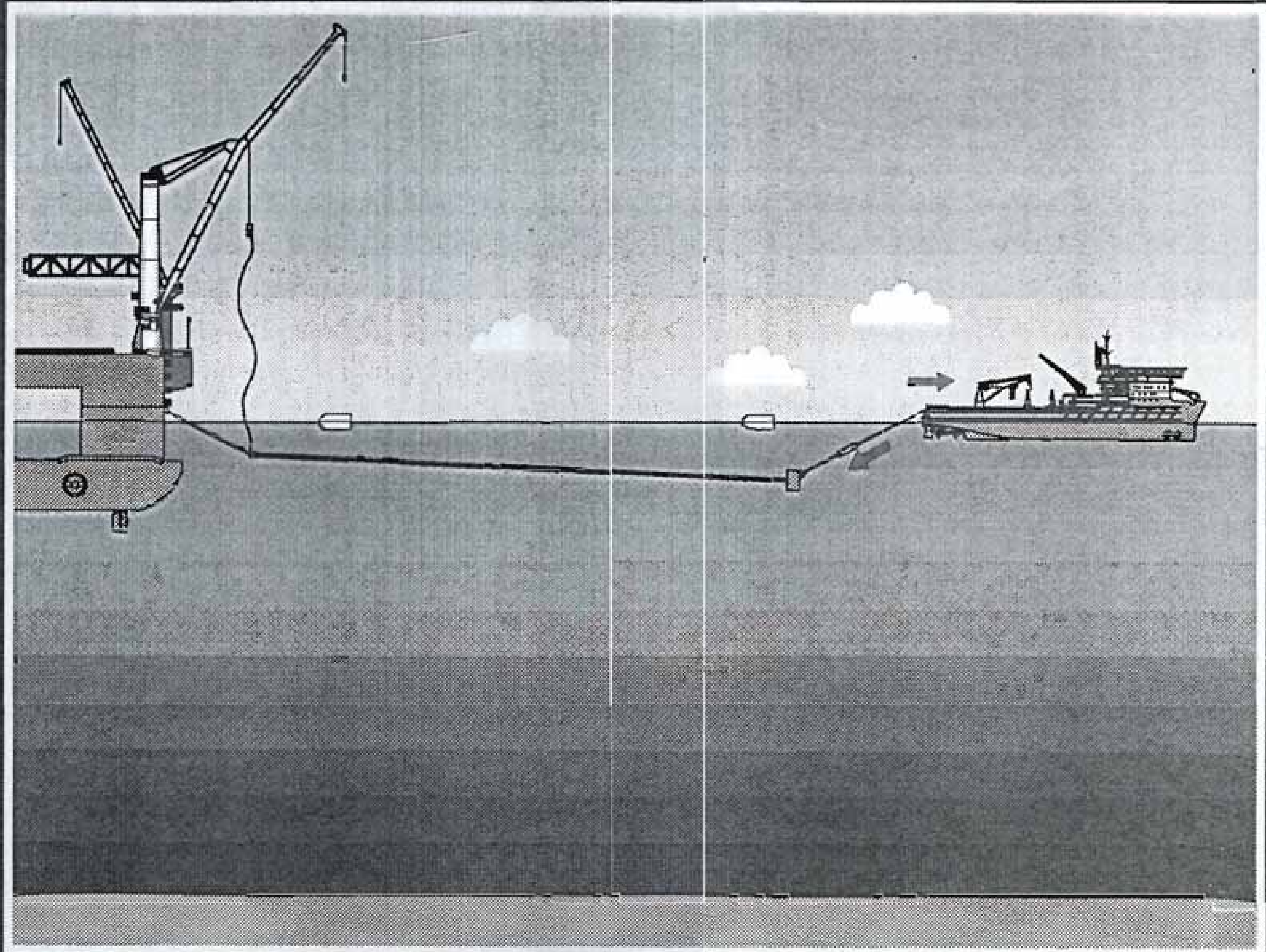
OFFSHORE TOW ARRANGEMENT



20.03.93



UPENDING COMMENCES



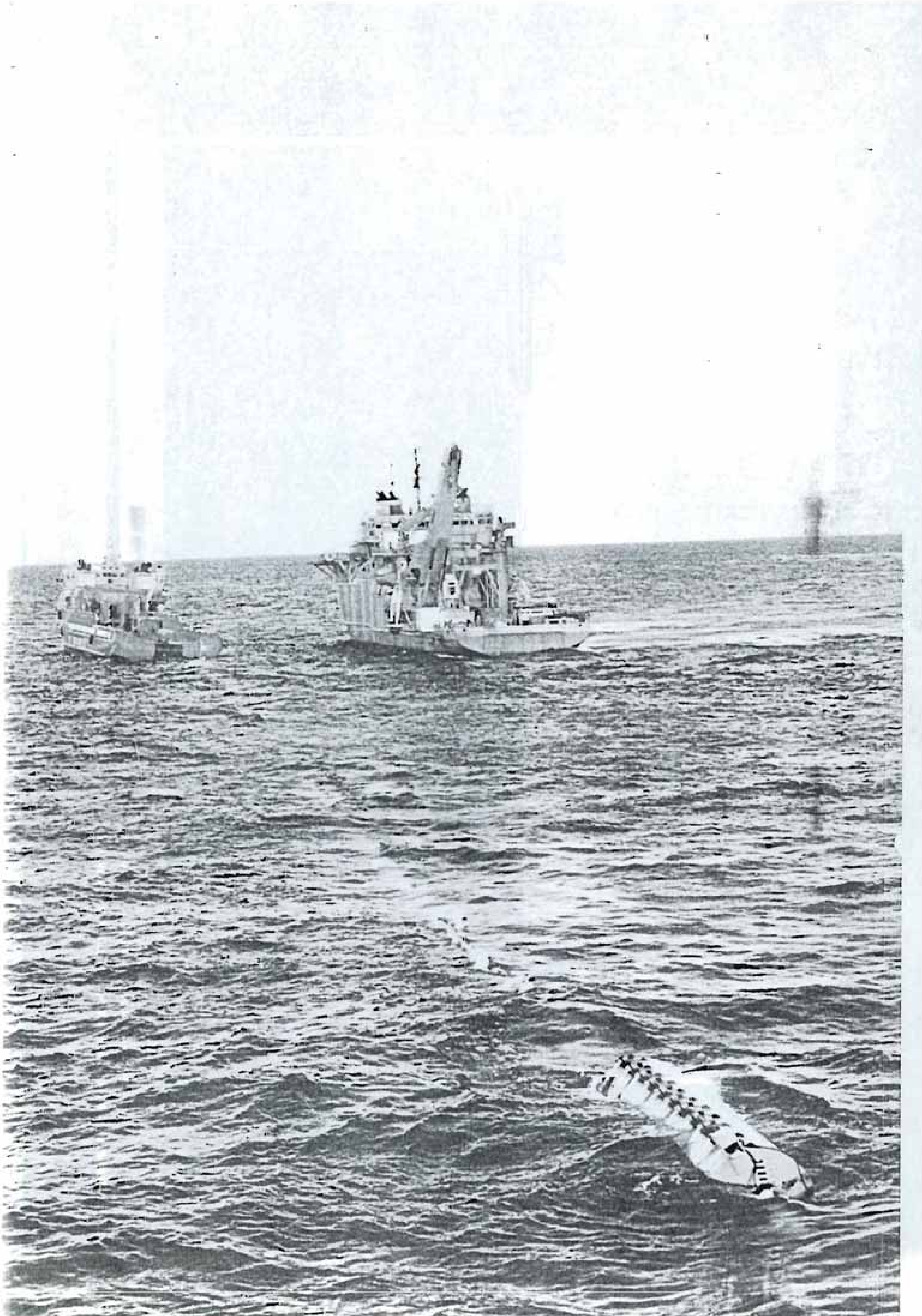


FIG. 1 TETHER LIFTING

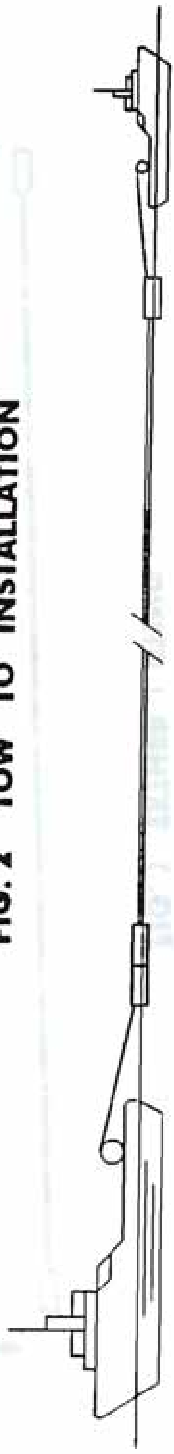


FIG. 2 TOW TO INSTALLATION

FIG. 3 START UPENDING

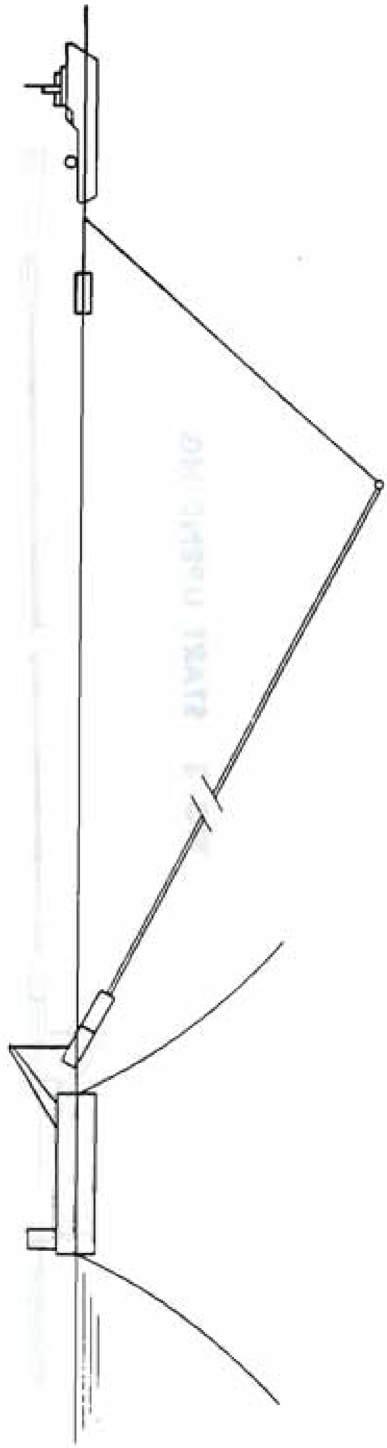


FIG. 4 TETHER UPENDING

FIG. 5 REMOVAL OF TOP BUOY

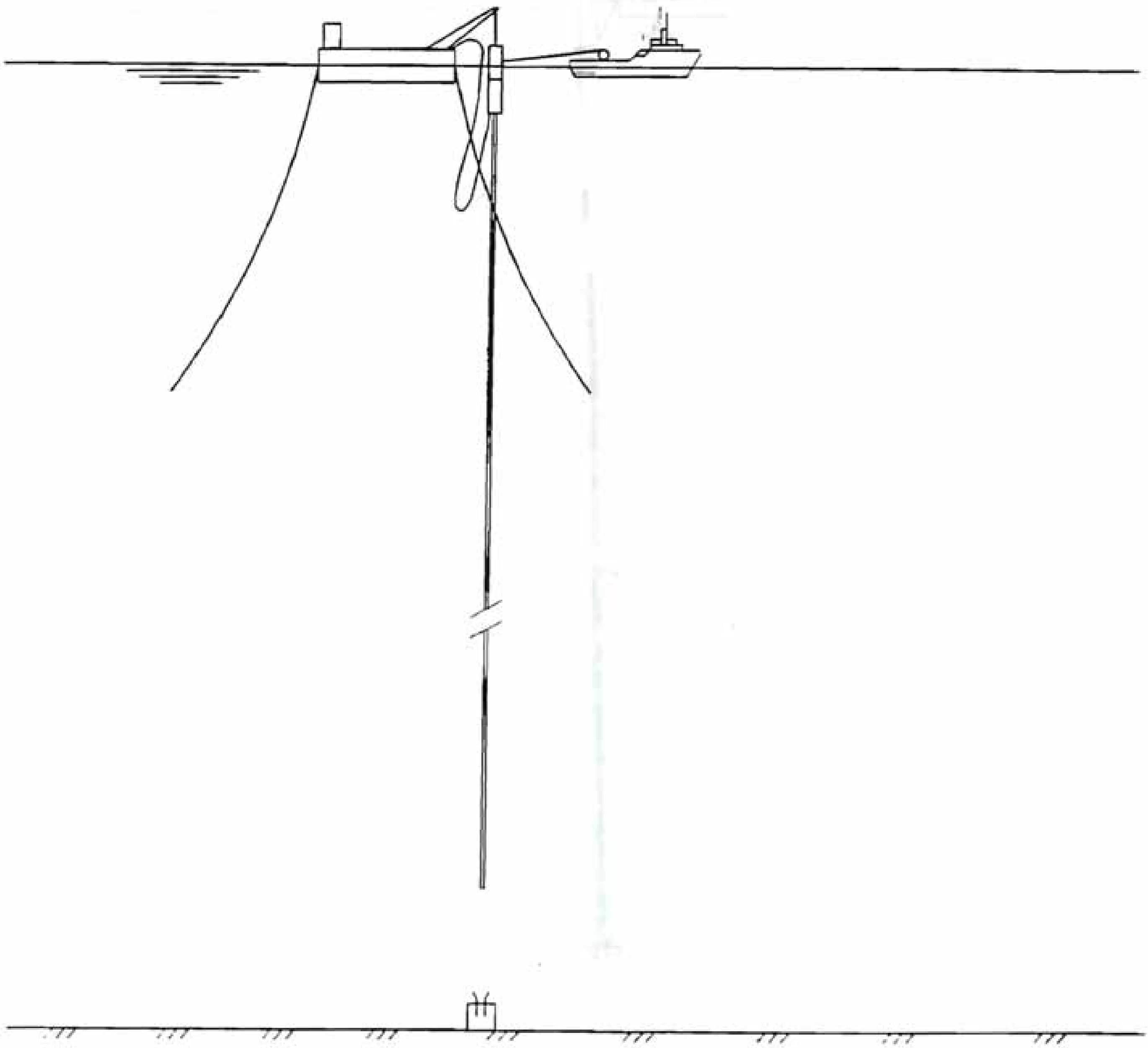


FIG. 6 STABBING OF TETHER

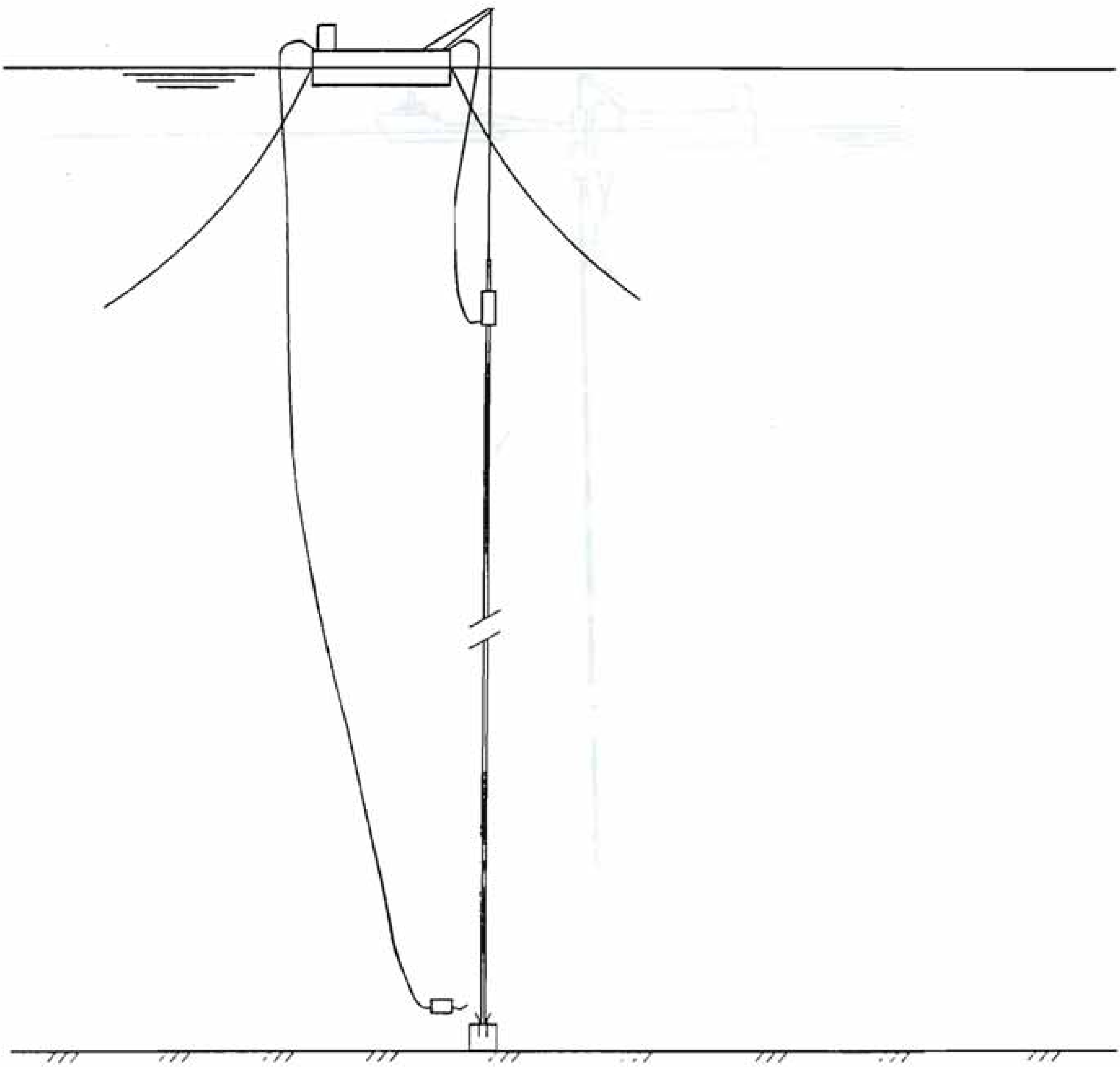
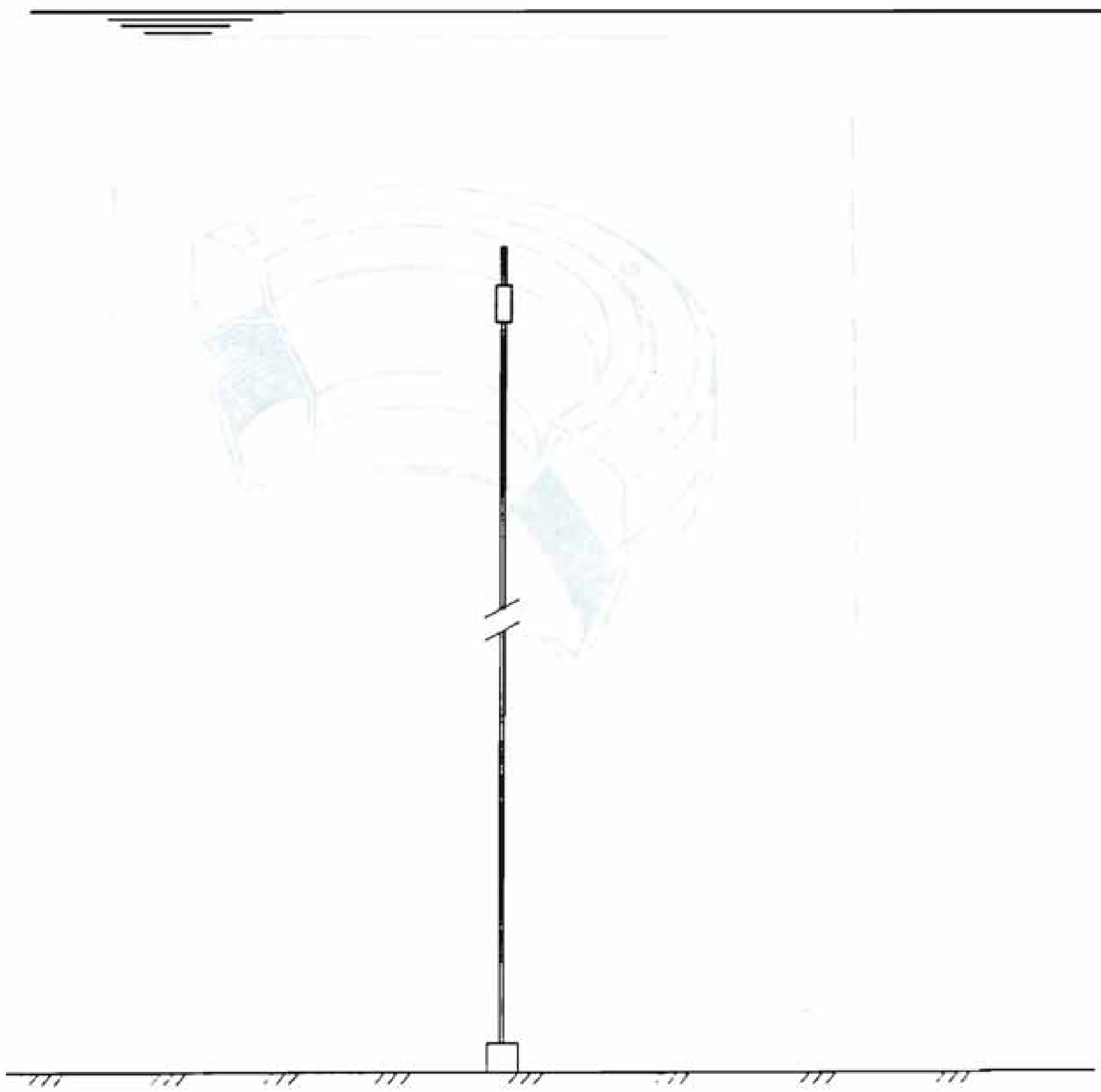


FIG. 7 FREE STANDING TETHER



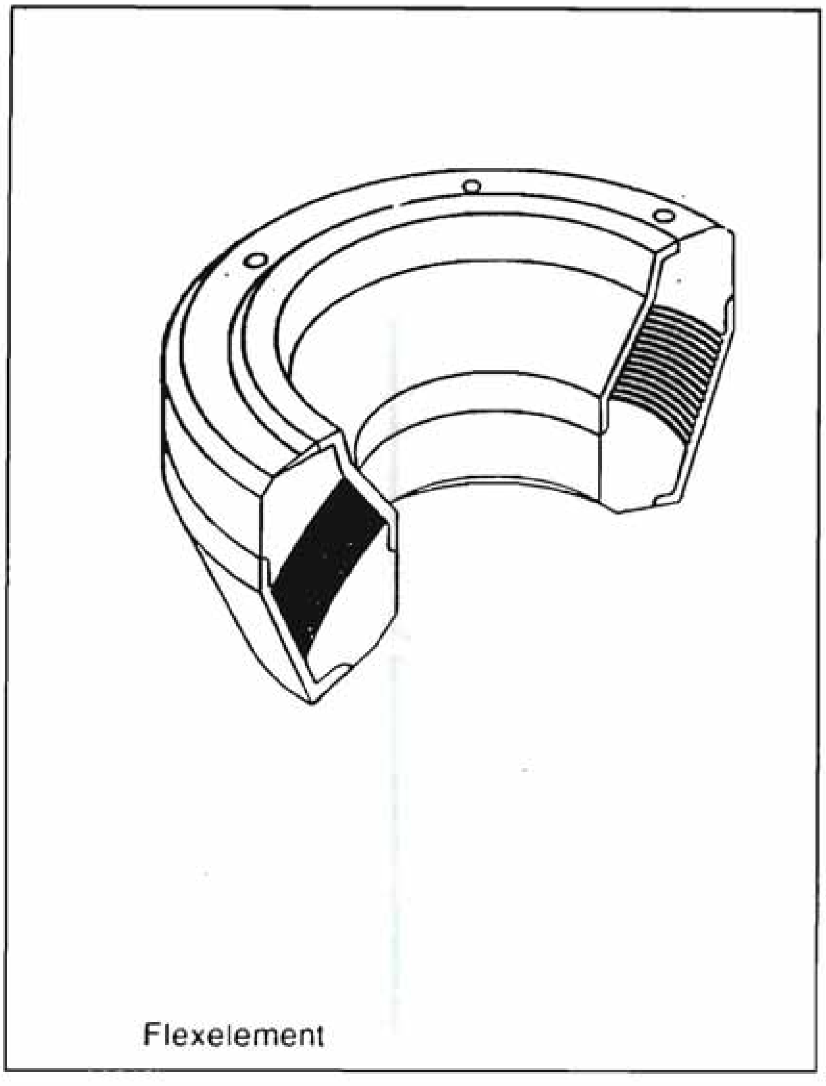


Figure 4 **Flexelement (cut through)**

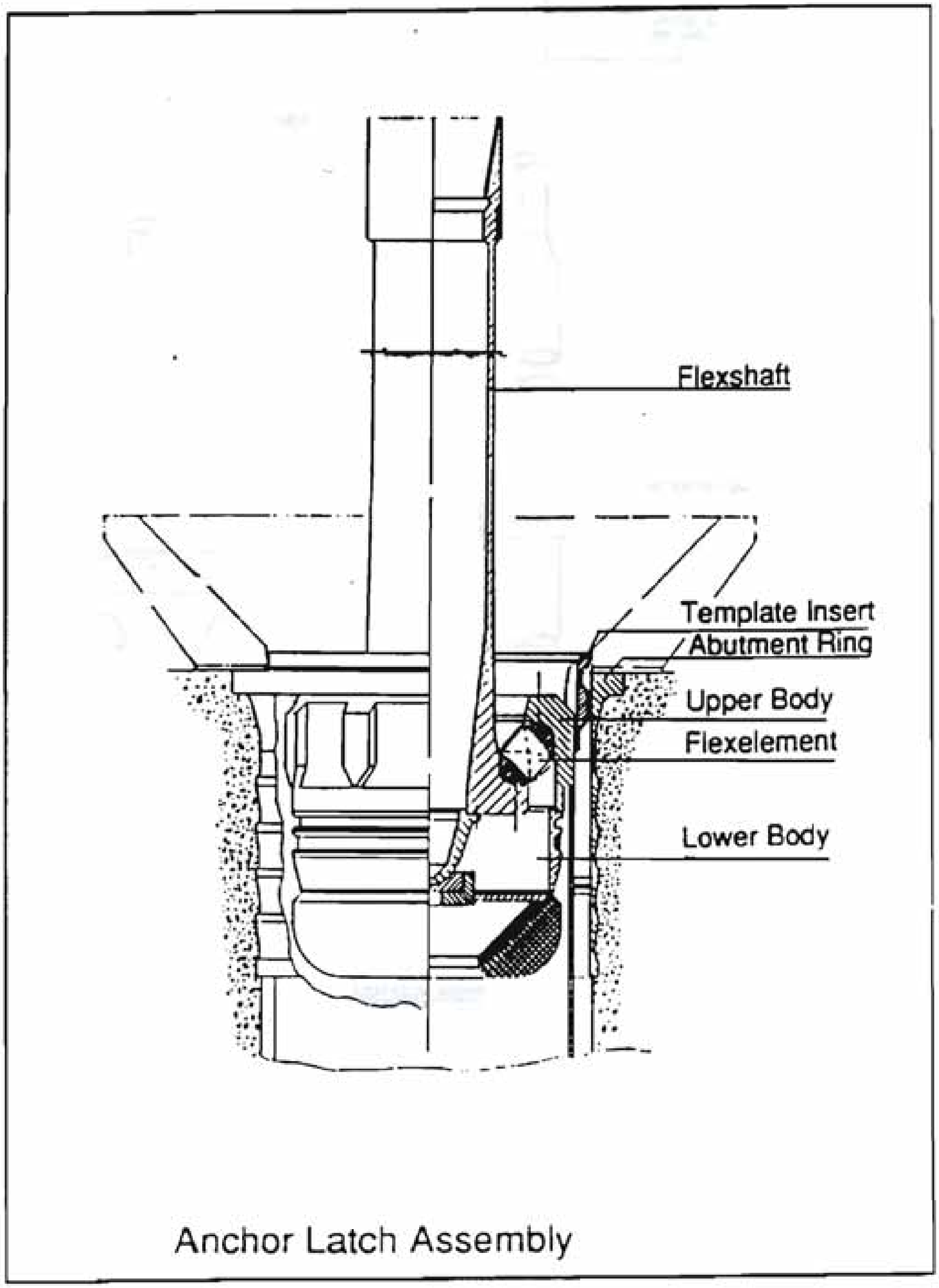
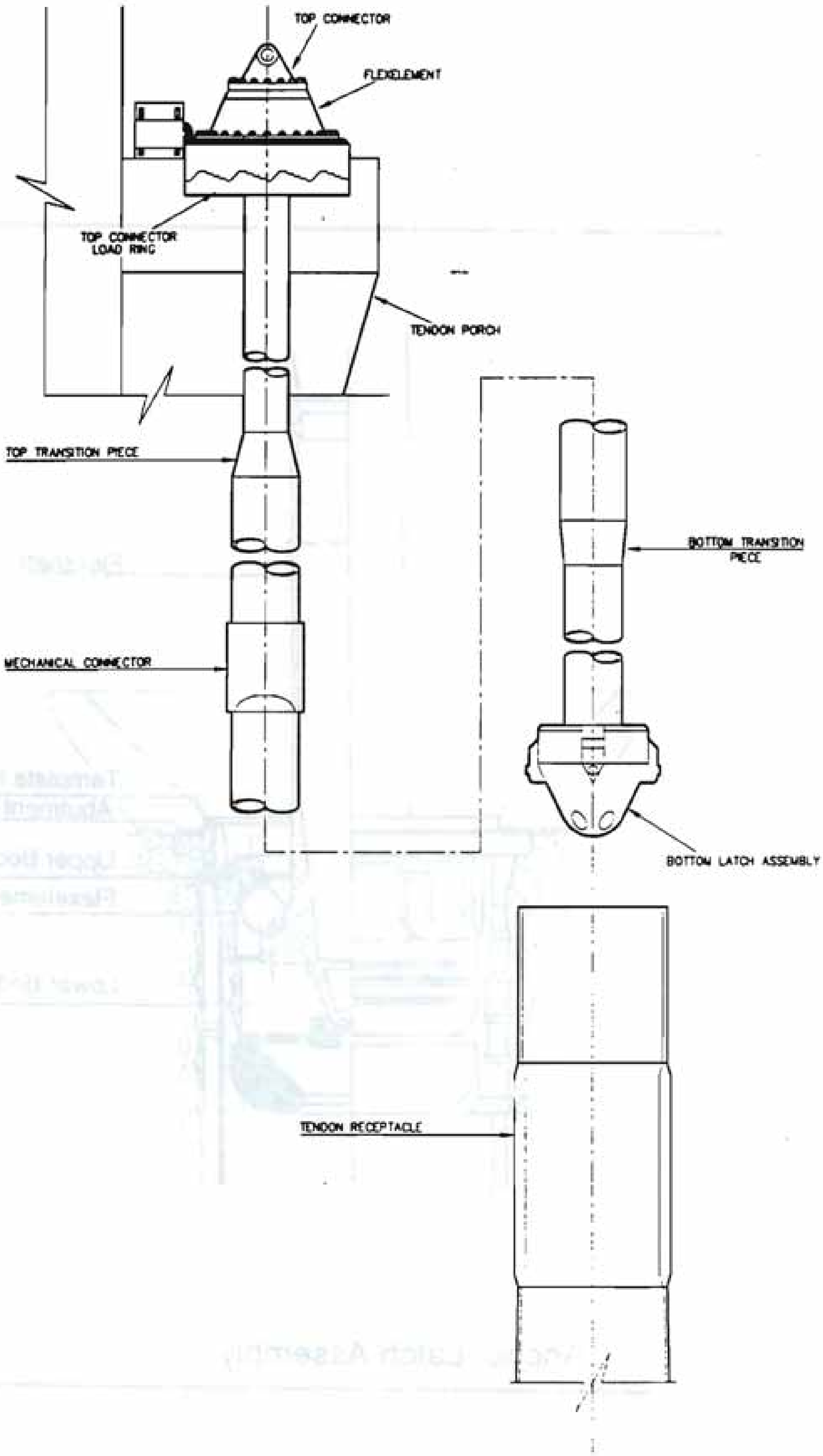
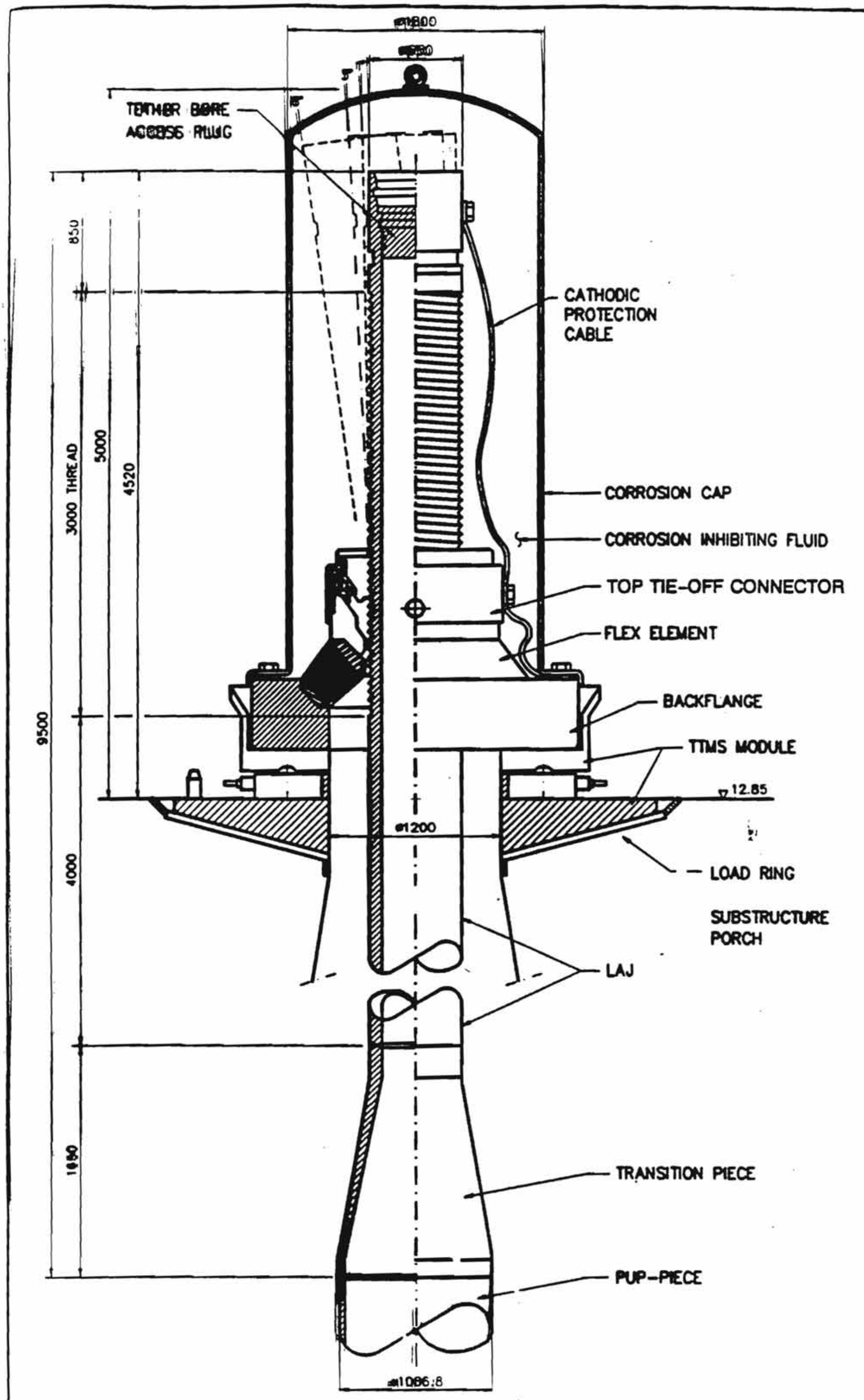


Figure 5 **Bottom Connector**



TLP TETHER ASSEMBLY

FIGURE 2



TETHER TOP CONNECTOR ASSEMBLY

FIGURE 3.2

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

12. Construction details

12. 3 Joints for Immersed Tunnels and Submerged Floating Tunnels

J. Saveur
Volker Stevin Construction Europe BV

INTERNATIONAL CONFERENCE ON SUBMERGED FLOATING TUNNELS, MAY 29-30 1996, SANDNESS, NORWAY

TITLE: JOINTS FOR IMMERSED TUNNELS AND SUBMERGED FLOATING TUNNELS.

by Ir. J. Saveur, Chief Design Engineer of Volker Stevin Construction Europe BV, The Netherlands

Abstract
This contribution is based on the experience with design and construction of immersed concrete tunnels, where joints are of paramount importance for water tightness of the whole of the sructure
The philosophy of joint design of immersed tunnels is discussed. Details are given of the types of joints. Finally the use of joints for SFT`s is considered.

Introduction

A joint is a transition between homogeneous parts of a structure. Joints can be rigid or flexible, but should be watertight.
The use of joints dictated by the mechanical behaviour of the tunnelstructure, which is distinctly different for SFT`s and immersed tunnels.
Details are given of the typical joints used in the presentday practice of immersed tunnel engineering. A practical consideration is made for the use of joints for SFT`s, based on the experience with immersed tunnels.

PHILOSOPHY OF JOINTS FOR IMMERSED TUNNELS

A major requirement for tunnels exposed to ambient water or groundwater is watertightness. The structural material of the tunnelshell itself is deemed to be watertight. Therefore it is prefered to compose the tunnelshell of as large as possible homogeneous watertight sections. At the transition between such sections a joint is required which must act as a seal.
This can be a weld between steelplates or a rigid construction joint in concrete or a flexible joint. Bored tunnels with segmented concrete lining may have as much as 1 meter of jointseal per 0.5 square meter of shell.
Immersed concrete tunnels according to Dutch design practice have 1 meter of flexible joint per 25 square meter of shell. In steel shell tunnels permanent flexible joints can be almost ommited.

By the nature of the structural material used for the shells a steel shell tunnel is more flexible than a concrete tunnel. The rigidity is about the same, but a steel tunnel can be subjected to much larger deformations than concrete tunnels that would be homogeneous over the same length. The deformations of concrete tunnels have to be limited to avoid tension cracking of the concrete. The solution is apply flexible joints.

The typical logistics of immersed tunnelconstruction allow the transportation and installation of fairly large prefabricated tunnelelements that are mated together under water inside the dredged trench. In case of a double shell steeltunnel the

interior structural shells of the two mating elements can be spliced by weldring behind a temporay enveloping jointseal.
In case of a concrete tunnel the temporay joint seal is kept in place as a permanent seal. It is in fact a rubber compression gasket of sufficient flexiblity to allow longitudinal and rotational deformation between the tunnelelements. This reduces the building up of longitudinal strain in the structure. For the same reason permanent rubber compression joints can be used for steel tunnels as well for instance at foundation discontinuities.
For concrete tunnels the presence of only a flexible joint every 100 m is not enough to provide watertightness. The concrete material itself is watertight as long as there are no through wall cracks. The critical longitudinal precompression level is not enough to avoid transverse throughwall cracking if the tunnelelement is subjected to differential settlements and temperature gradients. The method to avoid these cracks in The Netherlands is to use expansion joints at 20 to 25 m distance which coincide with the vertical construction joints. In this manner a tunnelelement is broken up in segments. Longitudinal tension forces are thereby eliminated.
The segments themselves however need to be fabricated without transverse cracks. This is achived by proper control of the mix and curing procedures of the concrete. It is noted that immersed tunnels have fairly thick wallls without complicated details. Because the stress levels are low sophisticated high grade concrete is not necessary. All these factors facilitate the proper placing and compaction and therfore the homogenity of the concrete.

For construction reasons there is one rigid construction joints inside a segment. This is a horizontal construction joint between the base slab and the walls. These joints are made as simple as possible without keyways and waterstops. Additional sealing provisions are not required.

The completed immersed tunnel has one more joint. Namely the Closurejoint. The last tunnelelement to be installed can only be mated at one end with the regular immersion joint. At the other end there must be a gap to allow passage of the tunnelelement in between the others. In this gap which is about 1 meter wide a small tunnelsegment of the lenth of the gap has to be constructed in situ.

A special type of rigid vertical construction joint is sometimes used in immersed tunnels of small diameter like services tunnels. Individually precast segments of say 6 m length (poured straight up) are assembled to form a tunnel element with the use of cast joints in between the segments. Longitudinal post tensioning is applied to provide structural homogenity.

THE EXPANSION JOINT

The location of expansion joints are at the vertical construction joints. The end of a segment is provided with a flexible rubber metal waterstop that is continous around the perimeter. The surface finish of the concrete face is smooth as provided by a resinfinished plywood formpanels. The smooth face helps to evacuate air when the concrete of the next segment is cast against it.

Water stops act as an obstruction for the proper pouring of concrete. Therefore, without special care and special provisions, many waterstop joints leak.

The tips of the waterstop are provided with filtertubes or spongeducts for postconcrete grouting of voids that may be left behind. In the horizontal sections the wings are held up by temporay rods to avoid air entrapment .

The sealing capicity of these expansion joints is not dependent on longitudinal compression. Widening of the joint is practicably not limited by the properties of this type of waterstop. At the outside of the concrete a small sand seal is provided.

THE FLEXIBLE IMMERSION JOINT

The permanent flexible compression gasket, so called "Ginagasket" is a very practicle and highly reliable jointing and sealing device. It is only mounted on the primary end of a tunnelelement. The secundary end is just flush face. The Ginagasket is first used as an initial seal for the immersion installation. When the primary end of the tunnelelement to be installed is pulled gently against the secondary end of the previous one it seals the chamber between the end bulkheads of these two elements. The subsequent dewatering of this chamber makes that the hydro static pressure is transfered from the bulkhead to the gasket. This compression is locked in permanently.

The Gina gasket acts as a spring. The characteristics depend also on the cross-sectional dimensions.
The compression of the gasket will vary with the longifudinal movement of the concrete faces of the joint. The compression force somewhat reduced by relaxation of the rubbers should at minimum compression be sufficient for its sealing capacity at the maximum expected waterhead.

In the Dutch construction practice the contactfaces for the Ginagasket are especially lined with a backgrouted steel plate to match the two jointfaces with a tolerance of + or - 5 mm.

Not withstanding the sealing reliability of the Ginagasket, a second gasket is applied inside of it. This is the socalled "Omege-gasket", which is a curved rubber membrane with fibrous inlays, that is clamped on by bolted down steel strips.

The annular space between the two seals is pressure tested to spot leakages of the Omega-gasket connection. Under operational conditions this annular space can drain off into the tunneldrainage system. This enables early warning for irregular performance. So far on all Dutch tunnels these joints keep performing according to their design.

The Gina gasket transfers compression but practically not shear.
If there is a relative shear movement betwee the two faces, the Ginagasket will slide along the free face, without losing its sealing capicity.
The Omega gasket is fixed to both tunnelelemen tends it has to be designed to follow the expected longitudinal and shear deformations with sufficient marging to hold the water pressure.
Often shear deformation in the immersion jount is restrained by telescopic dowels or by shearkeys inward of the OMEGA membrane.

A special application of Gina gaskets is used in Japan for earthquake conditions. The design of the joint is very similar to the design described above, but longitudinal prestressing is applied across the joint.
This is to increase the decompression range of the joint (gapwidening). Beyond total decompression there is still longitudinal integrity provided by the strain capacity of the prestressing tendons.

THE CLOSURE JOINT

The closure joint has to be fabricated inside a cofferdam. Very often the cofferdam also has to enclose the roof of the closure joint.

The closure joint is like a short segment of about 1 m long between the adjacent faces. Each face is provided with a similar waterstop as is used for the expansion joint.
Only to one face steel reinforcement bars are connected for longitudinal continuity. So the other face can act as an expansion joint.

MORTAR JOINT WITH LONGITUDINAL PRESTRESS

Reference is made to immersed longitudinal prestressed concrete service tunnels. There are two examples in The Netherlands both with cylindrical internal diameter of 4.00 m and wall thickness of 0.325 m. These tunnels are underneath busy waterways with about 15 m waterhead.
Both tunnels consist of tunnelelements of 60 m long coupled by Ginagaskets.The tunnelelements are composed of 10 precast segments of 6 m long cast as vertical cylinders. The segment joints are about 20 mm wide and filled with epoxy mortar applied under pressure. The assembly of 60 m is longitudinally prestressed.
These tunnels were completed more than 20 years ago. Unequal settlements have caused some cracking, which could be repaired, however, there was no cracking in the joints. Apart from these flaws the relatively thin walls and the joints perform very well with regard to watertightness.

JOINTS FOR SUBMERGED FLOATING TUNNELS (SFT-JOINTS)

Mechanically an SFT differs from conventional immersed tunnels with regard to its discrete support points versus continous supports and, when the supports are more or ~~and~~ less flexible, the tunnel shell itself has to provide continuity. For immersed tunnels the continuity is provided by the continous ~~relatively rigid~~ embedment in the ground.

For immersed tunnels longitudinal bending they can be practically eliminated, but the tunnel shell of an SFT must transfer longitudinal moments , which can be relatively large. A concrete SFT will be longitudinal prestressed and will be without flexible joints, ~~except maybe at the Abutments~~. However, there will always be ~~rigid~~ construction joints for the fabrication of the tunnel elements and for the in situ coupling of the tunnel elements, which have to be rigid. The concern for these joints is watertightness.

If SFT-elements would be fabricated as an assembly of individually vertically poured segments, there would only be construction joints prependicular to the tunnel axis. The joints would be made similary as described for the fabrication of services tunnel elements, namely as pressure poured mortar innfill joints. For SFT`s for traffic the segments could be too heavy for this practice.
The alternative is to cast SFT-elements in the same manner as conventional immersed tunnel elements. Namely in segments in line, which a horizontal construction joint between the bottom part and the top in each segment. Following the experience of immersed tunnels this joint would not need special provisions for watertightness other than proper cleaning of any laitance prior to the pouring of the concrete on top of it.

The function of the vertical construction joints between the segments is not the same as for immersed tunnels. They will become permanently prestressed. Opening up prior to receiving prestress could lead to non uniform compression. The provision of a waterstop would be helpfull for watertightness, but not to maintain structural continuity. A solution could be to apply a part of the permanent prestress directly after the pour. The advantage of a waterstop is questionalble. The grouting could be facilitated by the embedding of a continous porous grout tube along the perimeter.

For the rigid joints temporary sealing is required. To avoid weakening of the cross-section at the joint the seal could be positioned in a collar. A Ginagasket can be used and stay in place permanently. The complexity of the joint is the coupling of the prestressing ducts. This will require a relatively wide joint, which is to be filled with concrete. With post pour grouting to fill any voids left behind.

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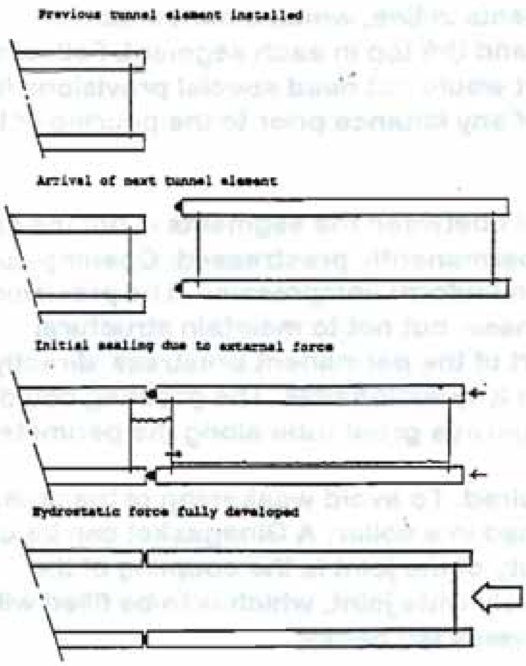

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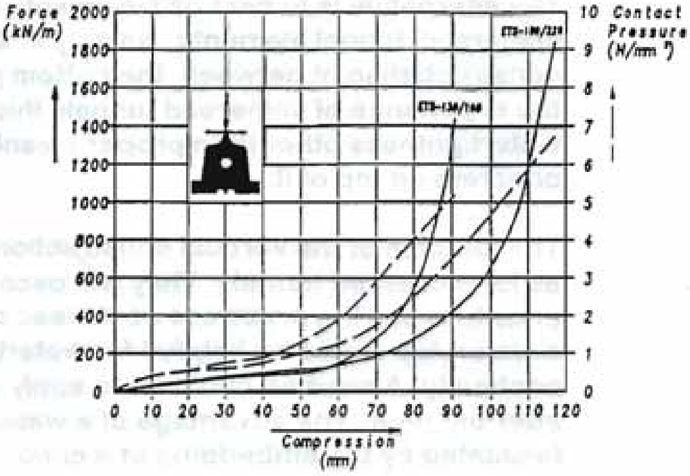

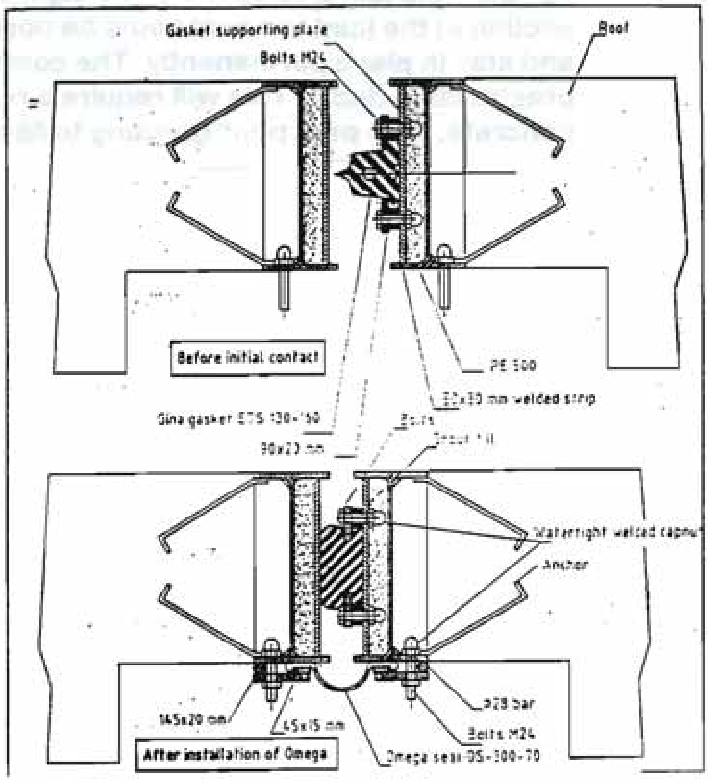

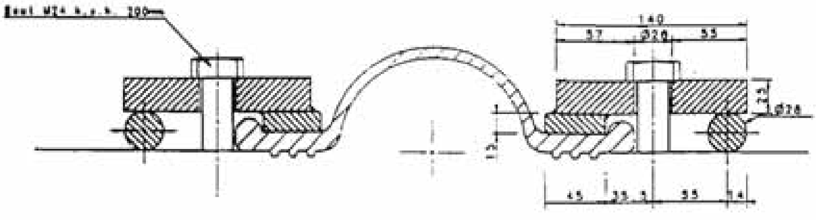

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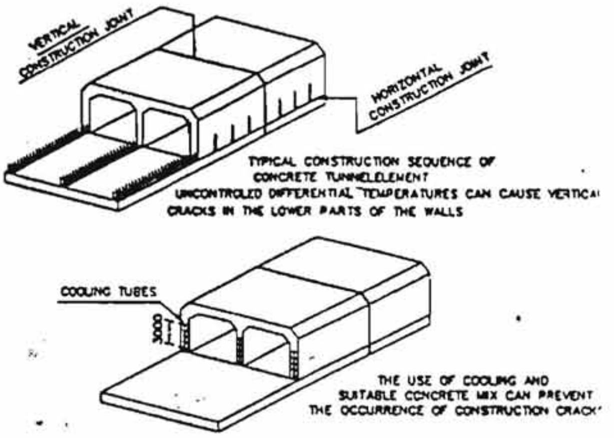
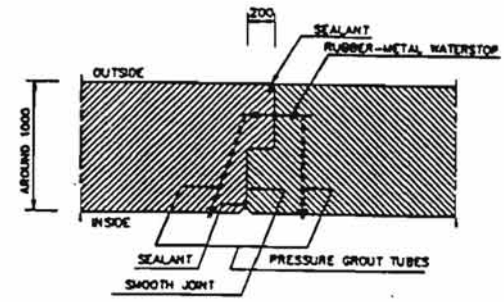
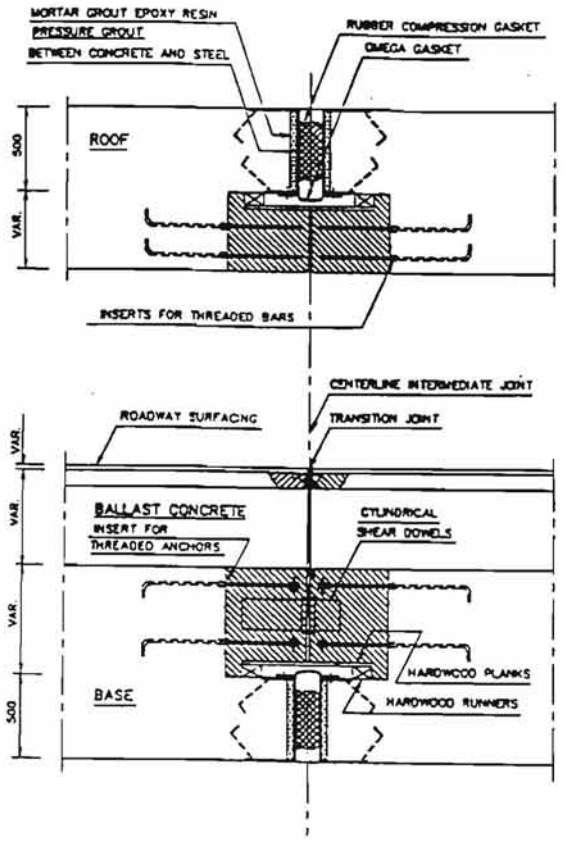


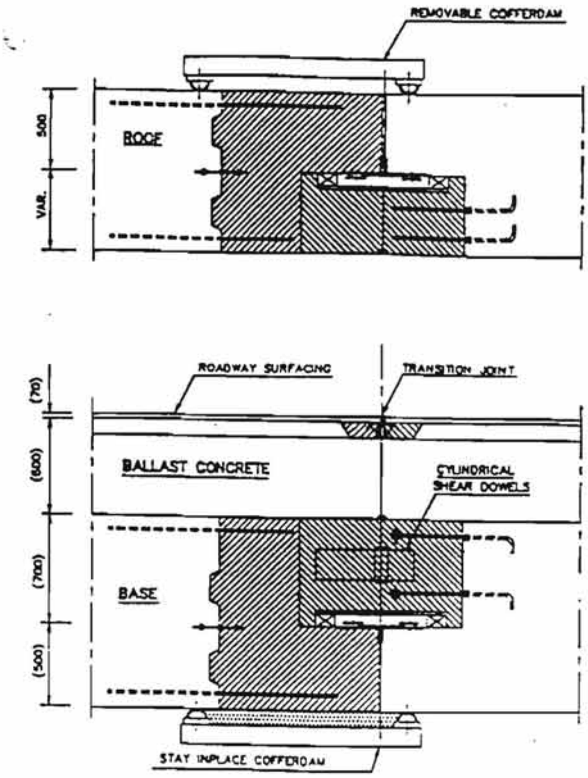
THERMAL SHRINKAGE PROBLEM AND SOLUTION

TYPICAL DUTCH EXPANSION JOINT



INTERMEDIATE FLEXIBLE JOINT FOR CONCRETE TUNNELS
(DUTCH SOLUTION)





FINAL JOINT
EXAMPLE FOR CONCRETE TUNNELS

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

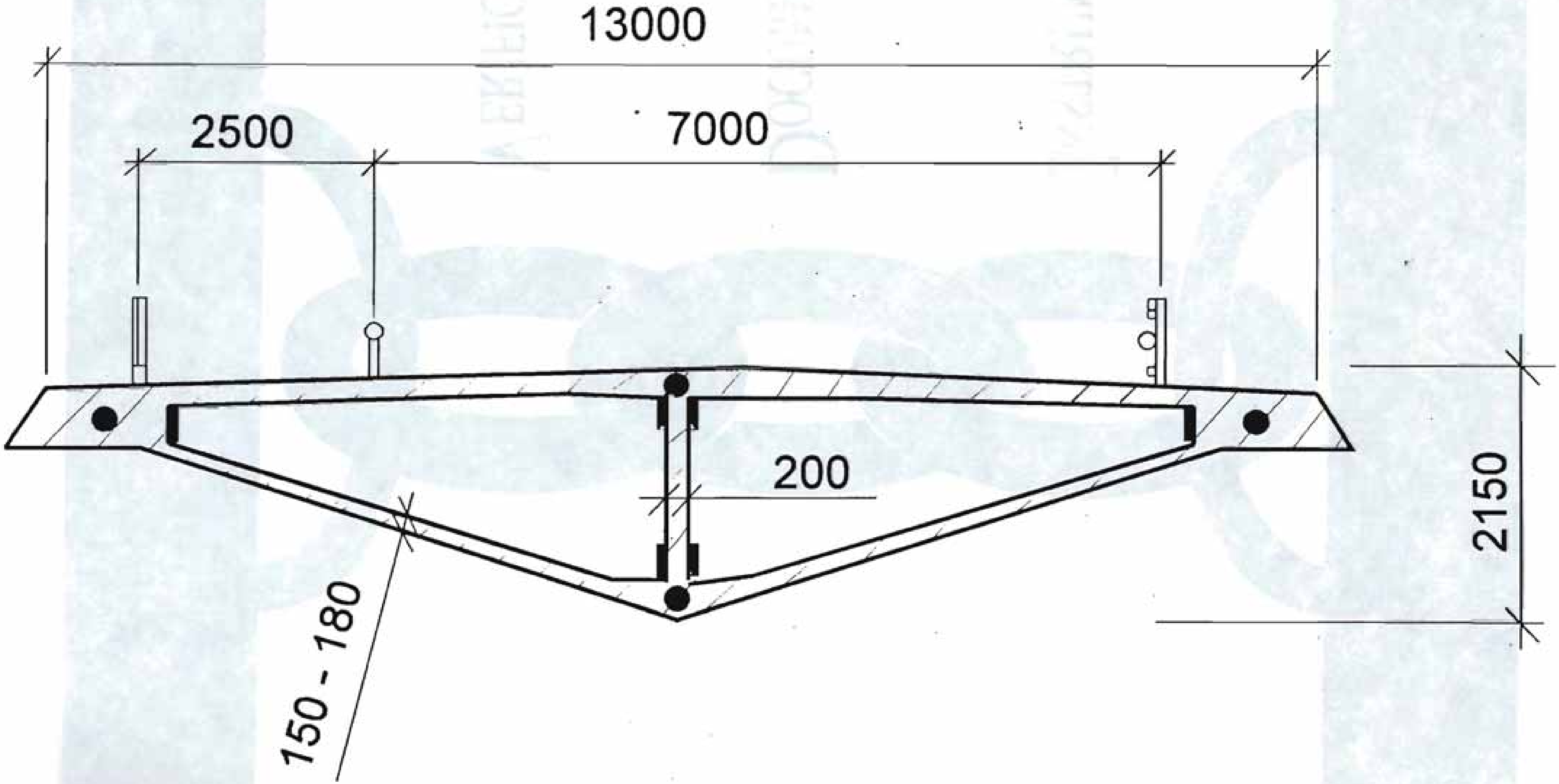
13. Instrumentation, Documentation and Verification

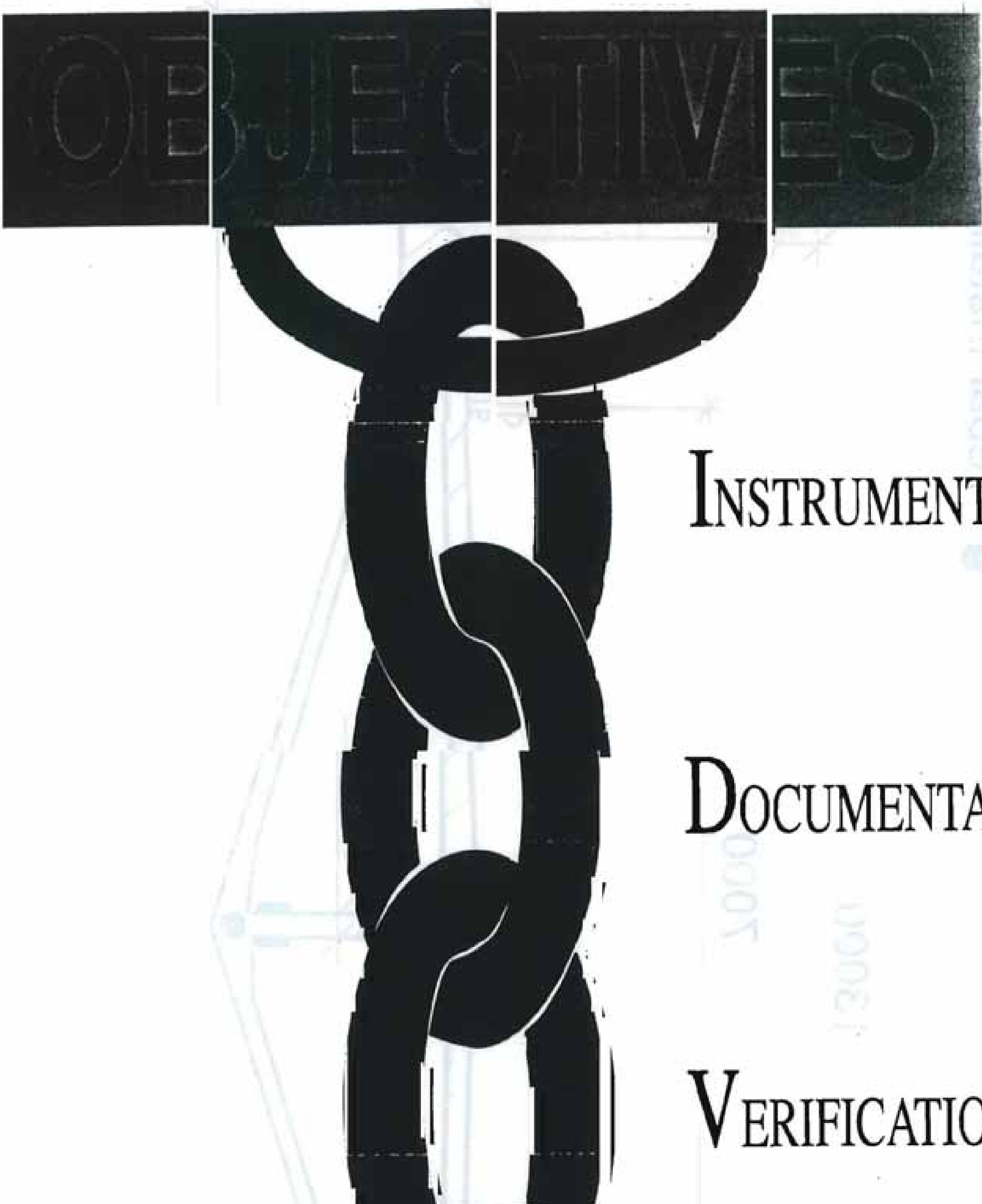
H. Østlid / I. Markey
Norwegian Public Roads Administration

Vibrating Wire Strain Gauge

▮ **surface mounted**

● rebar installation





INSTRUMENTATION

DOCUMENTATION

VERIFICATION

International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

14. Design of a submerged floating tube with a free span of 1750 m

P. Tveit
Agder Distriksthøgskole

(Handout)

DESIGN OF A SUBMERGED FLOATING TUBE WITH A FREE SPAN OF 1750m

Handout for the International Conference on Submerged Floating Tunnels

Sandnes, Norway, May 29-30, 1996

Including a few changes and additions made in June 1996

Per Tveit

Faculty of Engineering
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4890 Grimstad, Norway

THIS REPORT INDICATES THAT DOWNWARD ARCHED, ABUTMENT ANCHORED SFTS WITH MOVEABLE WATER BALLAST IS THE MOST ECONOMICAL SOLUTION FOR TUNNEL LENGTHS UP TO 2,5 KM

PREAMBLE TO THE SECOND EDITION WRITTEN FOR THE INTERNATIONAL CONFERENCE ON SUBMERGED FLOATING TUNNELS, SANDNES MAY 29-30 1996.

Most readers are recommended first to read my contribution to Strait Crossings 1994. It is to be found on page 12. Then finish this preamble and go on to choosing chapters from the contents on page II and page 3.

This edition has been done in a hurry. It is not completely satisfactory and it is not easy to read. Too much knowledge is taken for granted. The purpose of this edition is to present results and ideas that I consider important. Maybe it would be fair to say that it is written for engineers who are brighter and more well informed than myself.

Many good reasons for my choices are omitted for lack of time. I apologise and hope to do better some day. Still, the publication contains sufficient data for my statements to be checked by any engineer with a computer of reasonable size. Many calculations which could be done to prove my points, have had to be omitted due to lack of time.

For instance calculations in the deflected state have not been done, because the decisive bending moments occur when there is small normal force in the concrete tube. See column (10) and (11) on page XX. Then the usual linear calculation gives about the same results as the calculation in the deflected state. It is much simpler and more illustrative to use the usual linear calculation, because then influence lines can be used due to the principle of superposition.

Buckling analysis has not been done. Instead an educated guess has been used, based on the Euler load for a straight column. If anybody can put forward a good reason why that guess is wrong, the buckling analysis can be performed very quickly.

A dynamic computer analysis has not been performed. Instead the assumed periods of oscillations have been based on the periods of oscillation of a straight member. Previous experience indicate that the periods of oscillation found by this method, deviate less than 5% from the times found by more advanced methods (Page 42, Tveit 73).

In this edition I have increased the cross-section of the concrete tube. See page VIII. Many of the results based on the smaller cross-section are still of interest and value. Thus they have not been discarded.

This new edition is written in Century Schoolbook and the pagination is done with roman numerals. The old edition was written with Times New Roman and ordinary numbers were used for pagination This will make it easy to distinguish between the new and the old parts of this publication.

At the back of this publication are to be found my contributions to international symposiums on strait crossings in 1986, 1990 and 1994. The three publications show how my ideas have developed over the years, but basically they have stayed the same.

When I read my old publications it sometimes surprises me to find ideas that I thought were relatively new. New in this publication is:
1 Calculation of effects of continuos adjusting of the shape of the SFT by altering the water ballast. These calculations will be refined at a later date. Recalculation is not likely to lead to increase in the need for prestress.
2 Calculations to show that the axial force in the SFT can be controlled by measuring the vibrations of the concrete tube. By altering the water ballast a suitable distance can be kept between the Euler load for sideways buckling and the axial force in the SFT.
3 The reasons for the choice of vertical curvature for the SFT have never been presented before. See page XI. The author is not quite convinced by his own reasoning.
4 A calculation to show that if the design current assumed for the Høgsfjorden project lasted for one hour, then the sea level in the fjord would increase approximately 1,5 meters. Thus the maximum current can not occur at all depths of the fjord at the same time.

Finally I would like to thank Anker-Zeemer Engineering A/S for permission to publish the results that I obtained using ANSYS. I also thank Lars Line for helping me getting good use of the latest version of ANSYS. Thanks also goes to John Conway for correcting my English and to Håvard Østlid for the interest he has shown in my work over the years.

Future work on this publication will give me a chance to present my ideas in greater detail. That can usually not be done in a contribution to a congress. This publication will take its final form some time after my retirement. Please let me know if you have any suggestions or there is anything that you wonder about. I am also very interested in any mistakes that you might have found.

Page 41 and 18 gives the reasons for thinking that the downward arched, abutment anchored SFT is the most economical solution for tunnel lengths up to 1,5 and 2,0 kilometres. If water ballasting is used, this SFT is probably competitive up to 2,5 kilometres. This is why this structure continues to be of such interest to the author.

Grimstad May 20th - June 12th 1996

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CONTENTS

BALLASTING TO ACHIEVE OPTIMAL SHAPE OF THE CONCRETE TUBE AND TO AVOID BENDING DUE TO UNEVEN WEIGHT ALONG THE SFT

For the Høgsfjorden SFT project a variable load equal to ±2% of dead load has been used. This load should correspond to max. variation of dead load along the SFT. For this STF, spanning 1750 metres, this load would lead to deflections over 2,6 metres.

It seems expedient and economical to counteract these deflections by varying the thickness of asphalt and redistributing ballast water in dams underneath the roadway. See Strait Crossing 1994 p.13. To find the optimal distribution of ballast, we would have to measure the exact shape of the tube just after it had been cast, and keep measuring the shape of the concrete tube during its lifetime.

There are two sources of undesirable stresses that we, in our ignorance, can not quite counteract by the water ballast in the concrete tube. One source is the unavoidable and partly unknown variation of dead load along the concrete tube. The other is our lack of exact knowledge of the shape of the tube that would give us the stresses aimed at.

First we want to find the maximum stresses due to variation in the dead load. The following reasoning and calculations indicate what maximum stresses can possibly arise due to a variation in dead load when deflection deviates from an ideal shape by less than a predetermined minimum.

Let us assume that the concrete tube has a ballast tank every 50 meters and that we continuously measure the shape of the SFT at each ballast tank. According to H. Østlid there is a great likelihood of continuous remote measurement of the deflections of the shape of the first SFT in Norway. Examples of remote measurements are found in: "The Norwegian Road to Improved Bridge Quality" Norwegian Public Roads Administration. Contact H. Østlid. P.O. Box 8142, Dep. 0033 Oslo, Norway

Given a maximum deviation δ_{max} from the shape that causes ≈no bending and a max. variation ΔG of the dead load, it is the load case in fig. I that gives the biggest stress due to bending.

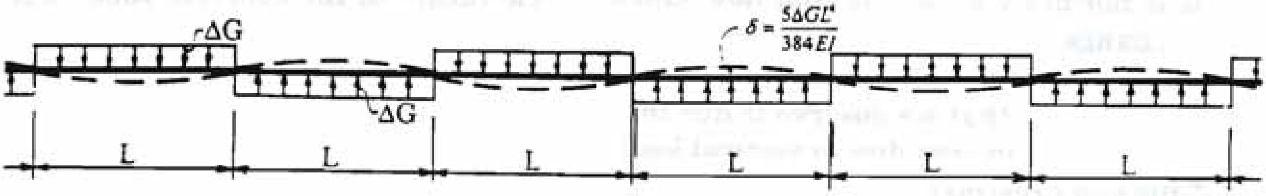


Fig. I. Load case which gives the biggest bending stresses for a given max. deflection and a given max. load.

Below is a table that gives maximum stresses for various combinations of $\pm\delta_{max}$ (max. deviation from originally measured shape) and $\pm\Delta G$ (max. deviation from average weight).

		$\delta_{max}=\pm10$ mm	$\delta_{max}=\pm20$ mm	$\delta_{max}=\pm30$ mm
Max. variation in weight of tube in % and kN/m	±1% ±11,2 kN/m	σ=0,5 MPa L=191 m	σ=0,7 MPa L=227 m	σ=0,9 MPa L=251 m
	±2% ±22,3 kN/m	σ=0,7 MPa L=160 m	σ=1,0 MPa L=191 m	σ=1,2 MPa L=211 m

It can be seen from the table that the stresses due to variation in weight depend on how well the deflection is controlled. For the calculation of this 1,75 km long SFT, the author assumes that the shape is not allowed to stray more than ±20 mm from the shape assumed to be right. Secondly the author assumes that the change in weight is ±1% or ±11,2 kN/m. This would lead to a maximum stress of 0,7 MPa.

The calculation leading to the 0,7 MPa is based on an assumption of abrupt change in weight every 227 metres. This could be due to abrupt change in the thickness of laid asphalt and abrupt change in the thickness of the 1,25 m thick concrete wall.

Changes in weight of the concrete, wear of asphalt on the roadway, dust and debris inside the tunnel and marine growth on the tunnel, contribute much less to abrupt change in weight along the tunnel.

It can be seen from the table that ±10 mm deviation from desired shape and ±2% or ±22,3 kN/m change in weight every 160 metres would lead to the same maximum stress.

The observed deviations from the "optimal" shape could be put into a computer program, the output of which could be max. stress due to the observed deviation and suggestions for making the dams higher or lower to reduce this stress. Design of dams is indicated in fig. 2 page 13.

The concrete is prestressed. This prestress leads to changes in the shape of the concrete tube. For our purposes we shall consider shortening and bending of the tube. The shortening is easy to observe and a good shortened shape can be found. It is normally harder to find how much the curvature of the concrete tube ought to change.

The bending that we observe is due to:
* bending moments due to vertical loads
* uneven prestress
* other loads
* modulus of elasticity
* creep

Careful calculation will be necessary to determine what deflection and stresses this involuntary bending will introduce into the concrete tube. These calculations become much simpler if the initial prestressing is constant at each cross-section. By varying the buoyancy along the concrete tube the prestressing can be kept constant or almost constant in each cross-section of the tube. See page 35. If there has to be a small part of prestress that causes bending, this could be applied after the tube has been put in between the abutments. See page XII.

With a prestress that is uniform over each cross-section, deflection of the concrete tube should be kept to a minimum until it is put in place between the abutments. After the tube has been put in place, loads will be introduced that give considerable bending. Careful calculation would be necessary to find what shape of the tube should be aimed at after bending has been introduced.

This publication has no calculation of the stresses that might be introduced because of insufficient knowledge of how the concrete tube deflects due to the bending it is subjected to. Of course these calculations would be easier, if the bending was introduced only after the start of the process of putting the tube to its final position.

After the concrete tube is in place, the modulus of elasticity could be found by loading and unloading the tube and measuring the deflection. Furthermore the creep is very much reduced for bending that occurs when the concrete is one to two years old.

The main source of deformations, imperfectly known, is the buoyancy introduced after the concrete tube has been fastened to the abutments. Some minor sources of deformations that we do not quite know are:
* creep due to bending of the concrete tube to place it in between the abutments.
* deformations due to prestress, if any, that is applied after the concrete tube is fixed to the abutments.

Since we do not exactly know the modulus of elasticity and the coefficient of creep, we don't quite know what shape to aim for when ballasting the concrete tube. Thus we do not quite know what stresses we introduce. Careful calculation would be appropriate to decide what involuntary stresses could be introduced in the concrete tube.

We shall not be very wrong if we measure the maximum deflection due to the buoyancy introduced. By ballasting we aim for a calculated shape that corresponds to this deflection. Special attention must be paid to deflections near the abutments because the biggest stresses occur there.

In the table on page XXI 0,8 MPa has been added due to uncertainty in the calculated deflections. Stresses due to 10 kN/m buoyancy, found in the same table, have influenced this choice.

BUCKLING AND VIBRATIONS OF THE CONCRETE TUBE

The modulus of elasticity is assumed to be 31000 N/mm² for buckling and 35000 N/mm² for vibrations. The most likely period of oscillations is T≅100 sec.

Weight of concrete tube plus an equal weight of water that vibrates with the tube:

$$\mu=\pi R^2 \cdot 1{,}027 \cdot 2=\pi \cdot 6^2 \cdot 1{,}027 \cdot 2=232 \cdot 10^3 \text{ kg/m}$$

If the tube is assumed straight, the frequency of oscillations would be:

$$f_n=\frac{4{,}73^2}{2\cdot\pi\cdot L^2}\sqrt{\frac{EI}{\mu}}=\frac{4{,}73^2}{2\cdot\pi\cdot 1750^2}\sqrt{\frac{35\cdot 10^9\cdot 618}{232\cdot 10^3}}=0{,}01123 \text{ sec}^{-1} \Rightarrow T=89 \text{ sec}$$

Because the concrete tube is not straight, the period of vibration is slightly longer. The compressive force in the concrete tube will also increase the period of oscillation. For a straight bar the period of oscillation T is proportional to:

$\sqrt{(P_E+N)/P_E}$ N is positive when compression.

When using the dynamic modulus of elasticity we get the Euler load:

$$P_E=\frac{\pi^2 \, EI}{\frac{L^2}{4}}=\frac{\pi^2 35000\cdot 618}{(\frac{1750}{2})^2}=279\,MN$$

Previous worked examples (page 47, Tveit 82) show that P_E is lower when the concrete tube is not straight. For this reason we assume that P_E=270 MN. A 10 kN/m buoyancy of the tube makes N=55 MN. This gives:

$$\sqrt{(P_E+N)/P_E}=\sqrt{(270+55)/\ 270}=109{,}7$$

This gives a 10% increase in the frequency of oscillation. Thus T≈100 sec.

Professor Geir Moe (1989), reference on page 19, says that in-line vibrations due to symmetric vortex wakes occur when the speed the of current is $U=U_R f_n D$. U_R lies between 1,2 and 2,5.

This corresponds to a speed of current $U=1{,}2 \cdot 0{,}01123 \cdot 0{,}892 \cdot 1.2=0{,}144$ m/sec and 0,3 m/sec.

According to (Eidnes 89) and to (Eide 74), references on page 19, it seems likely that such a speed of current may occur at least once a year. Whether the current will occur long enough and over sufficient length of the SFT, to cause hydroelastic vibrations, should be subject to careful consideration and, if necessary, research.

In-line vibrations due to an asymmetric vortex wake could occur for currents between 0,31 and 0,5 m/sec. These speeds seem unlikely to occur long enough and over sufficient length to cause hydroelastic vibrations, but if they did, it would not impair safety or comfort.

Comment added on June 4th 1996: The reduction of modulus of elasticity found on page XX is probably compensated for by a reduction in the added mass. In The International Conference on Submerged Floating Tunnels in Sandnes May 29-30, 1996 somebody mentioned that the added mass of water was likely to be only 70 to 80% of the weight of the tube for the very high Reynolds numbers found in SFTs.

Transverse (vertical) vibration of the SFT does not seem likely. The least impossible maximum deflection is sketched to the right.

$$f_n = \frac{3{,}927^2}{2\pi(\frac{L}{2})^2}\sqrt{\frac{EI}{\mu}} = \frac{3{,}927^2}{2\cdot\pi\cdot 875^2}\sqrt{\frac{36\cdot 10^9\cdot 618}{232\cdot 10^3}} = 0{,}0314\ \text{sec}^{-1} \quad \Rightarrow \quad T=31.4\ \text{sec}$$

A worked example (Tveit 83), reference on page 42, indicates that the natural frequency is about 10 % lower. (Moe 89), reference on page 19, says that atransversal vortex wake occurs when the speed of current is $U=U_r f_n D$. U_r lies between 4,5 and 10. This corresponds to a speed of current between

$$U=U_r f_n D = 4{,}5\cdot 0{,}0314.\ 0.9\ 12=1{,}5\ \text{m/sec and } 3{,}3\ \text{m/sec}$$

Such speeds of current will hardly occur long enough and over sufficient lengths of a SFT in a dead end fjord.

ACCELERATION DUE TO IN-LINE VIBRATIONS OF THE SFT.

Maximum amplitudes due to in-line vibrations will be around 0,1·D=1,2 metres. (Moe 89). Reference on page 19. This corresponds to a maximum acceleration:

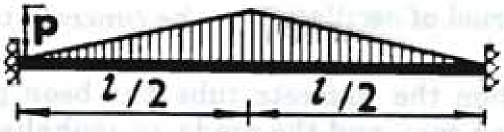
$$\ddot{x} = A\omega^2 = 1{,}2(2\pi f_n)^2 = 1{,}2(2\cdot\pi\cdot 0{,}01123\cdot 0{,}891)^2 = 0{,}0047\ \text{m/sec}^2$$

This is unlikely to be noticed by the public. Jensen (86), reference on page 19, says that 0,5 m/sec² is acceptable for off-shore platforms when T=16 sec.

The period of oscillation is more than 100 sec. Most cars will take less than this to go through the SFT. Thus the motorists are not in the SFT long enough to notice the vibration. SFTs would usually replace ferries. As far as unpleasant accelerations are concerned, the SFT will always represent a great step forward.

STRESSES DUE TO IN-LINE VIBRATIONS OF THE SFT.

The deflection δ will be roughly 0,1 D=1,2 metres. Shedding of the vortex wake in step with the natural frequency of the SFT will happen in the middle portion of the concrete tube. At the ends of the tube, the speed of current is likely to be so different from the average speed of current, that the vortex wake will not be in step with the natural frequency. Thus we assume that the deflection is roughly due to a triangular load on a straight beam with fixed supports at both ends. See drawing above.



The bending moment at each end is $M_A = M_B = M_{max} = \frac{5pL^2}{96}$.

Max. deflection is $\delta = \frac{pL^4}{549EI}$. By inserting one formula into the other we get:

$$M_{max} = 28{,}6\frac{\delta EI}{L^2} \quad \Rightarrow \quad \sigma = \frac{M_{max}}{W} = \frac{28{,}6\cdot\delta\cdot ER}{L^2} = \frac{28{,}6\cdot 1200\cdot 35\cdot 10^3\cdot 6000}{1750^2\cdot 10^6} = 2{,}35\ \text{N/mm}^2$$

This is insignificant because the moment is about the vertical axis. The important moments are about the horizontal axis. See page 24 and XX.

In page 15 the max. amplitude due to in-line vibration is said to be 20%. After the conference in Sandnes I contacted the professors Alf Tørum and Geir Moe. They sent me their publication: "On Current Induced In-line Ocillations of Pipelines in free Spans." They told me that it is not clear how big the amplitudes due to in-line vibrations will be. Amplitudes of 12 to 15% seemed likely. It is lucky that stresses due to in-line vibrations will not be decisive in any case.

CONTROLLING THE AXIAL FORCE

HOW THE FREQUENCY OF HORIZONTAL VIBRATIONS SHOWS IF THE NORMAL FORCE IN THE CONCRETE TUBE IS ACCEPTABLE.

Redistribution of water is used for counteracting bending moments due to slowly changing loads. See page IV. Slowly changing loads are: Change of concrete weight, marine growth, dust and debris inside the tunnel, wear of asphalt, and to a lesser degree: temperature loads.

We also need to make sure that the normal force stays within acceptable limits. This can be achieved by measuring changes in the frequency of the horizontal vibrations of the concrete tube. This can be done either by observing horizontal, current induced, hydroelastic vibrations of the concrete tube, or by inducing horizontal vibrations of the tube by applying sideways loads.

It is not difficult to calculate the frequency of the horizontal vibrations of the concrete tube. This has not been done. Instead the formula for a straight beam has been used. On page VII the formula for the frequency of a straight rod fixed at both ends with and without axial force has been used.

Below is a table that shows the connection between the normal force N and the period of oscillation according to the formulas on page VII:

Buoyancy kN/m	-20	-10	0	10	20	30
Weight $\mu \cdot 10$· kg/m	234	233	232	231	230	229
N (tension is pos.) MN	110	55	0	-55	-110	-165
Period of oscillation T for E=35000 MPa in sec	69	80	89	97	105	112
Period of oscillation T for E=37500 MPa	67	77	86	94	101	108

The table shows that a change in the axial force causes significant change in the period of oscillation of the concrete tube. This can be utilised as follows:

When the concrete tube has been placed between the abutments, the lane has been cast, and the roadway asphalted, we can know the buoyancy of the tube. For a start the weight of the tube, when it was towed to its final position, is known. After that, the removal of ballast water and other changes in weight can be controlled meticulously.

The first time the finished concrete tube has in-line vibrations, determines the period of oscillations that should be aimed at in the life of the SFT.

5% or ≈5 sec of change in the period of oscillation gives 33 MN change in the axial force in the concrete tube. This corresponds to a 6 MN change in the buoyancy and a 0.8 MPa change in the longitudinal stress. The change in the longitudinal stress is not frightening compared to the stresses in the table on page XXI. The change in buoyancy has to be distributed in such a way that it gives little bending. See page V.

The change in the axial force in the concrete tube is ≈12% of the buckling load. This is not critical. The usual reason for keeping far from the buckling load is the load enhancement factor. This is not so important here because the bending moments due to currents are small compared to the bending moments due to vertical loads. See table on page XX.

The buckling load for buckling in the vertical plane is ≈ twice as big as the buckling load for sideways buckling. Thus the moment enhancement factor for vertical loads is much smaller than the enhancement factor for the horizontal loads.

The modulus of elasticity can be found by measuring deflections due to known traffic loads. When the concrete tube is put in between the abutments, most of it is over one year old. Still there will be a slight change in the modulus of elasticity.

With high strength concrete the modulus of elasticity depends more on the modulus of elasticity of the aggregate than in the case of ordinary concrete. The change in E depends on the change in strength of the cement glue. Thus the change in E is smaller for high strength concretes than for ordinary concretes.

The last two lines in the table on the previous page give T for E=35000 MPa and E=37500 MPa. Even this exaggerated chance in E gives only ≈3% change in T. This is not important. The 7% increase in P_E enhance the safety of the structure.

Substantial increase in marine growth will increase μ, which is the weight of the tube plus the weight of water that vibrates with the tube. This would increase the period of oscillation because T is proportional to the square root of μ. See page VII

If the increase in marine growth was not known, the period of oscillation would be kept constant by decreasing N. That is to say by reducing the buoyancy. There are at least two reasons why the marine growth would not represent a problem:

1. A reduced buoyancy would not give much extra stress as long as great changes in bending was avoided. A reduced buoyancy does not increase the danger of sideways buckling.

2. The water in dead end fjords is usually not rich in oxygen. Thus the marine growth is much smaller than for instance in the North Sea. The water in greater depths along the tunnel has the least oxygen and the least marine growth.

Thus the growth will be unevenly distributed. The uneven growth will be detected when adjusting the shape of the concrete tube to avoid bending stresses. Marine growth can be easily found by underwater inspection, and the weight of the growth can be deduced from the change in ballast water to keep the shape of the tube.

In short: remedy will be routine, because the first SFT of this type will be subject to continuous inspection.

REASONS FOR THE CHOSEN VERTICAL CURVATURE OF THE SFT

The chosen vertical curvature can be seen from the drawings on pages 12 and 20. The following reasons speak for this curvature:

1. The axial force in an arch is roughly inversely proportional to the rise of the arch. A big rise of the arch makes it easier to avoid sideways buckling due to the change in weight attached to the concrete tube. Given a maximal gradient of the road, the rise increase when long stretches of the arch has the maximal gradient. Thus the chosen curvature gives near minimum axial force for the maximum prescribed gradient of the tube.

2. Bending moments due to changes in the length of the tube are roughly proportional to the area between the centre line of the tube and a straight line between the abutments. The changes in length of the tube can be due to temperature variation, shrinkage and creep. With the same maximum gradient a parabolic or circular curvature gives rise to bigger bending moments due to longitudinal contraction or expansion, especially at the abutments.

3. If electricity is cut off and water slowly trickles into the tunnel, any SFT will eventually collapse. With the chosen curvature bending due to water in the middle of the tunnel will be small. See influence lines for bending moments on page XIV. The axial force is also nearly as small as possible for the maximum gradient. Since an SFT is likely to have an independent power supply, this "fail slow" might be considered not important.

4. Parabolic arches are efficient at carrying evenly distributed loads. In the suggested SFT the decisive loadcases are accompanied by small axial forces. See column (10) and (11) on page XIX. Since this is the case, bending is less influenced by the shape of the arch. If big axial forces had occurred in conjunction with decisive loadcases, this would have spoken for a more constant vertical curvature of the concrete tube.

5. Wind is the main reason for the biggest currents in dead end fjords. When the biggest currents occur, the water tends to move in layers. The surface layer moves fastest and in the opposite direction to the movement of the water further down. The bending moments in the concrete tube are reduced when it goes though layers of water that moves in opposite directions. Furthermore, bending moments are reduced when the tube goes deep into the fjord where there is less current. This might not be important, since bending moments due to current is usually not a decisive loadcase. See also next page.

6. With the chosen curvature one can sight directly from the abutment to the lowest point of the tunnel. This makes it easier to monitor the deflections and vibrations of the tunnel.

7. Forces due to waves are much smaller on SFTs that are placed deep in the water. Fig 3 page 14 gives an idea how movement due to waves decreases with increasing distance from the surface. See also page 9.

If the tunnel were straight for the first 700 metres nearest to the abutments, it would be easier for the public to see how the concrete tube deflects under the vehicles. As is, the assumed upwards force of 10 kN/m gives an upwards camber of 0,84 metres on the first 700 metres at each end of the tunnel. Loads that make these 700 metres straight will be very rare and build up slowly. Finally it should be mentioned that a concrete tube with constant curvature is easier to produce.

A NOTE ON THE DESIGN CURRENT OF THE HØGSFJORDEN PROJECT

A simple calculation can be performed to show that the design outward current can not occur simultaneously at all depths of the fjord. The cross-section of the fjord is $A_{f1} \approx 125000\ m^2$. The surface area of fjord inside of the SFT is $A_{f2} \approx 60\ km^2$. We assume that the average speed of current is $u \approx 0{,}2$ m/sec. ~~This is a~~ low estimate based on fig 5 page 15.

In one hour the increase in the water level of the fjord inside of the SFT will be:

$$\Delta h = \frac{uA_{f1} \cdot 60^2}{A_{f2}} = \frac{0{,}2 \cdot 125000 \cdot 60^2}{60 \cdot 1000^2} = 1{,}5m$$

Max. ebb is 0.4 m. Max. flood is 0.5 m. (100 years) Eidnes (89), reference on page. 19.

From this very simple calculation it can be seen that when very strong currents occur in dead end fjords, the water moves both inwards and outwards at the same time. The volume of the water moving out is roughly the same as the volume of the water moving in. Thus the SFT suggested in this publication is subject to currents in opposite directions. This is not of any great importance, because bending moments and stresses due to the design-current are not important. See also page 15.

I have been told by Eidnes that the strongest currents occur after long periods of westerly winds. Lightish water is then piled up in the Lysefjord on the inside of the site for the SFT. If the wind changes direction, the piled up lightish water flows out of the fjord. Then the depth where the water changes direction must be lower down than usual. This reduces bending in any SFT that has substantial portions over and under the depth where the current changes direction. As mentioned above this does not really matter, but it is interesting all the same.

PRESTRESSING

The author has up to now assumed that each section of the concrete tube is posttensioned with anchors in the concrete wall at the end of each section. When high quality concrete and a great amount of prestress are used near both ends of the concrete tube, it might be difficult to find room for all the anchors. Additional prestressing tendons might be pulled into ducts in the wall and prestressed.

It might be advantageous for these cables to be put in and stressed after the concrete is more than 28 days old. This will give less loss of prestress due to creep and shrinkage, and less chance of microcracks due to high levels of prestress.

It would be easier to control the shape of the concrete tube after it has been cast if the initial prestressing were uniform over each cross-section. See page 35. It seems likely that the initial prestressing could be 80 to 100 % of the total prestressing. When the concrete tube has been put in place, the prestress that causes bending could be put in and tensioned. - Extra prestressing cables can be put in and tensioned if the SFT shows tendencies towards sideways buckling due to creep.

NEW CALCULATION OF CROSS-SECTION OF CONCRETE TUBE SPANNING 1750 METRES

Inner radius 4.75 meters. This is the radius used for the Høgsfjorden project for an SFT. It accommodates a 7 metres wide road, with 4,6 metres headroom, and one metre of pavement on each side. A SFT spanning 1,75 km would probably not be built unless a wider road is necessary.

The pedestrian traffic will be very small, so narrower pavements can be used. In the deepest part of the tunnel the level of the pavement should be the same as the level of the roadway. The pavement should be indicated by colour.

Cars and lorries will sometimes have to stop in the SFT. For instance if they have run out petrol. It would then be an advantage if they could roll on to the pavement and thereby be less of an obstruction to traffic. They would then be an obstruction to the pedestrians, but pedestrians will be few and far between and could easily cross over to the other pavement. The pavement is probably there not so much for the pedestrians as for storage space when work is done in the SFT.

Outer radius 6 metres.
Unit weight of concrete including reinforcement: 24,5 kN/m^3.
Area of concrete inside the concrete tube: $0{,}24 \cdot 9+0{,}54=2{,}7$ m^3.
Area of asphalt on the roadway: $7 \cdot 0{,}10= 0{,}7$ m^3.
Unit weight of asphalt: 23 kN/m^3.
Weight of concrete structure inside the tube: $2{,}7 \cdot 24{,}5+0{,}7 \cdot 23=82{,}3$ kN/m.
Area of concrete in the tube: $A=\pi(6{,}0^2-4{,}75^2)= 42{,}2$m^3.

Weight of concrete tube: $42{,}2 \cdot 24{,}5=$	1033,9 kN/m
Total weight of tube:	1116,2 kN/m

Weight of suppressed water: $1027(\text{kg/m}^3) \cdot 9{,}81 \cdot 10^{-3}\, \pi \cdot 6^2=1139{,}4$ kN/m.

Upward pressure on the tube without water inside the dams:	23,2 kN/m.
Weight of water inside the dams:	13, 2 kN/m.
Normal upward pressure on the tube:	10,0 kN/m.

Extra upward pressure on the tube if the weight of water increases from 1027 to 1028 kg/m: $1 \cdot 9{,}81 \cdot 10^{-3} \cdot \pi \cdot 6^2=1{,}1$ kN/m.

This cross-section will be used for the calculations made for Sandnes (May 1996):
$I= \pi (R^4-r^4)/4=\pi\,(6{,}0^4-4{,}75^4)/4= 618$ m^4 $\quad$ $W=I/R=618/6=103$ m^3

The concrete inside the tube will be cast after the tube has been put in place. It is convenient and on the safe side to disregard all stresses in the concrete inside the tube. Creep and shrinkage will diminish longitudinal stresses inside this concrete.

INFLUENCE LINES for SFT spanning 1750 metres.
Outer diametre 12 metres. Inner diametre 9,75 metres

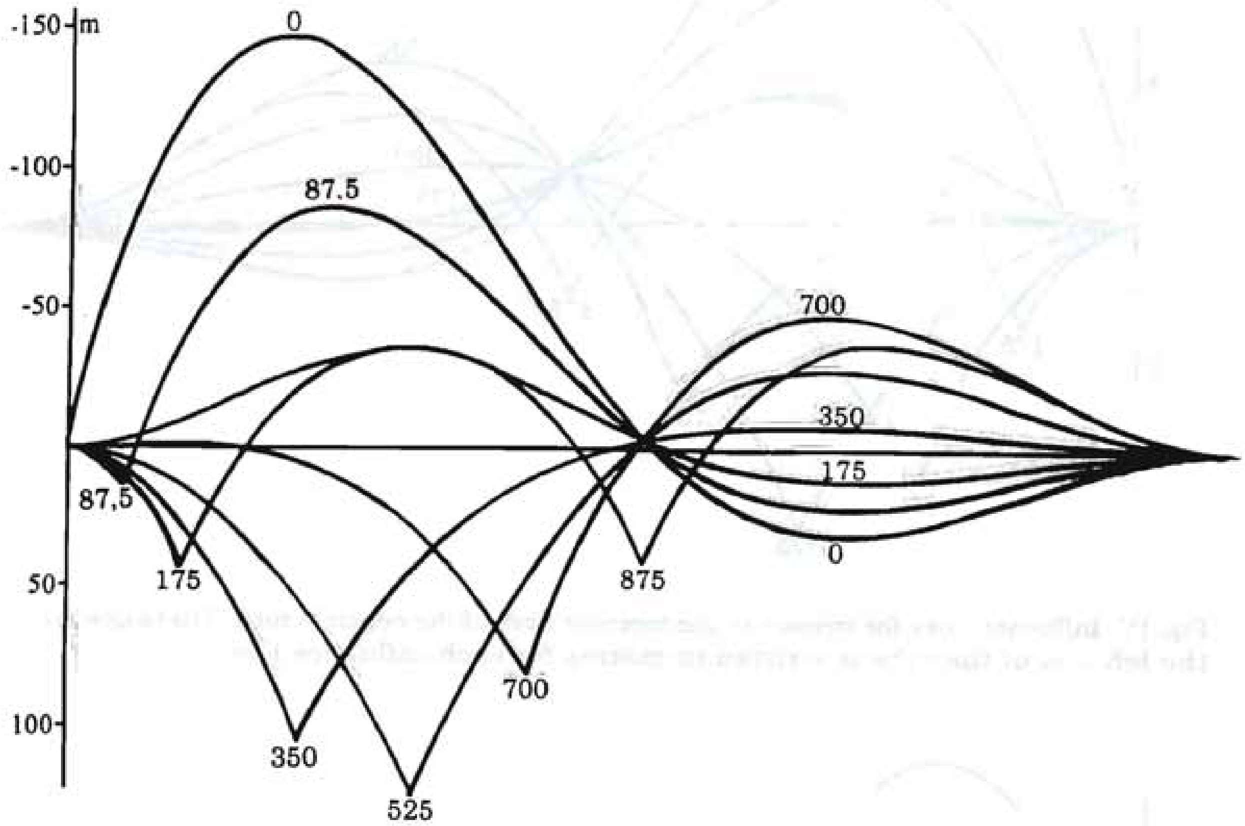


Fig. II. Influence lines for bending moments. Distance from the left end of the tube is written in metres for each influence line.

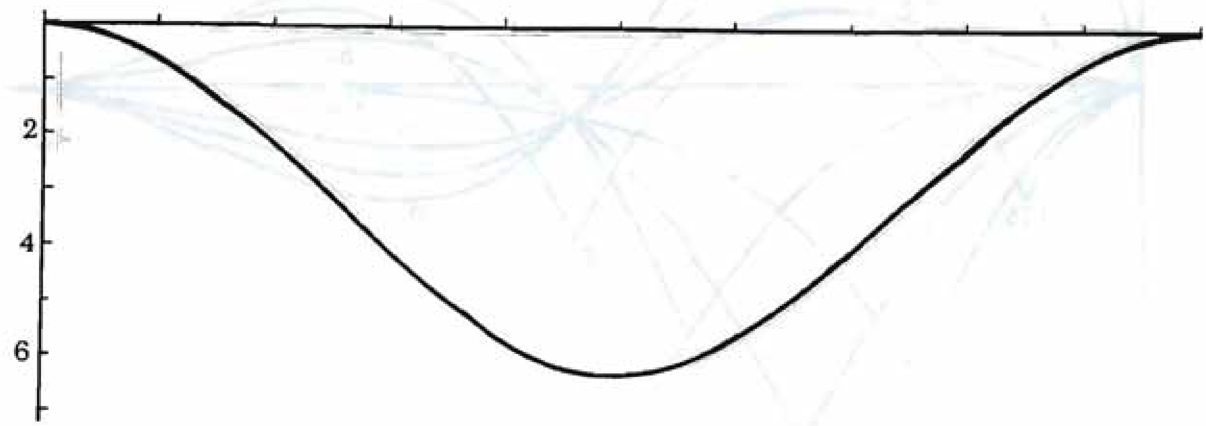


Fig. III. Influence line for axial force in the middle of the concrete tube.

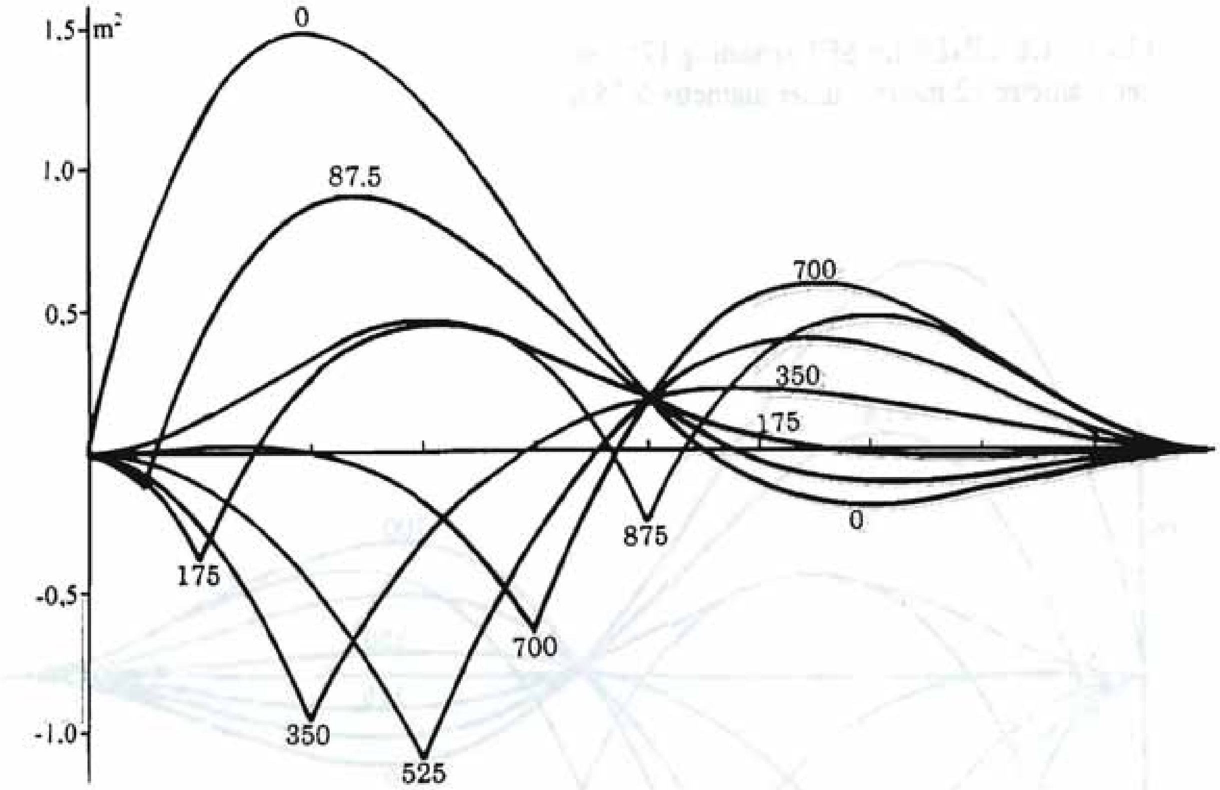


Fig. IV. Influence lines for stresses in the topmost fibre of the concrete tube. Distance from the left end of the tube is written in metres for each influence line.

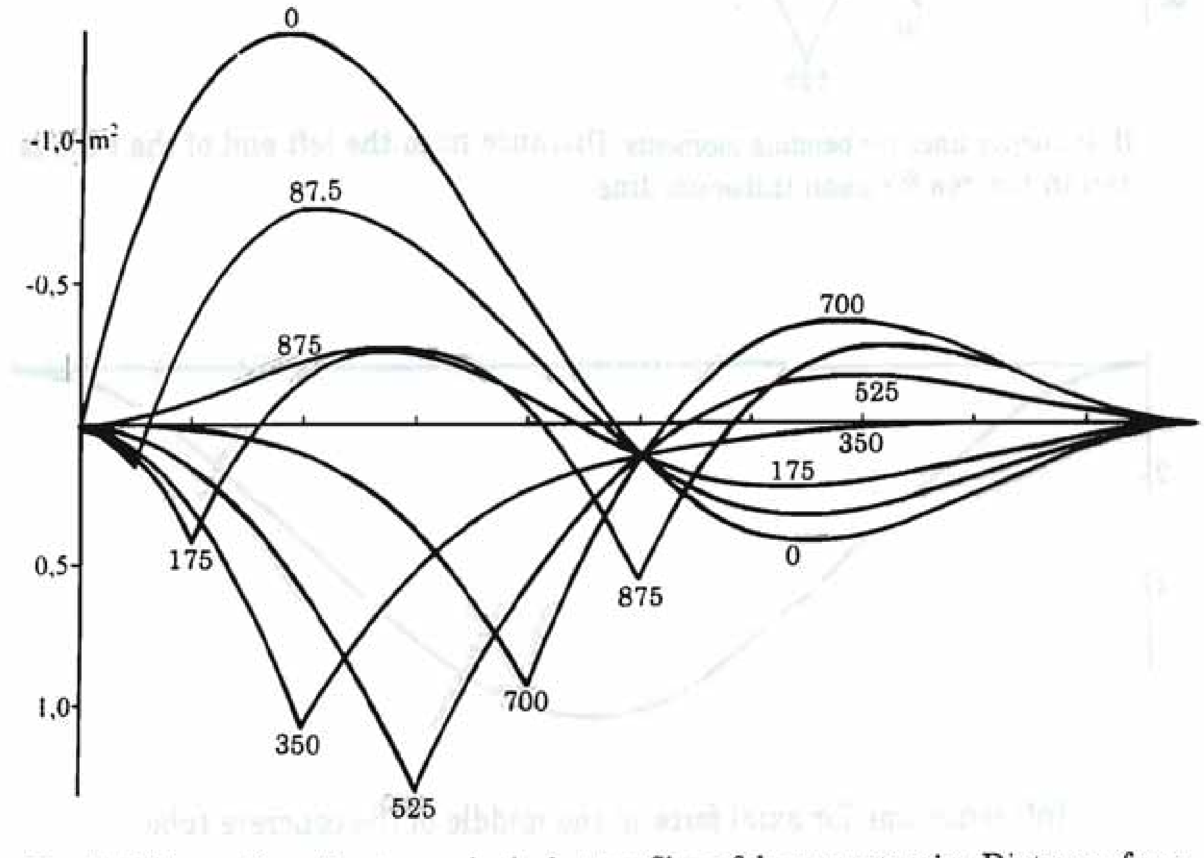


Fig. V. Influence lines for stresses in the bottom fibre of the concrete tube. Distance from the left end of the tube is written in metres for each influence line.

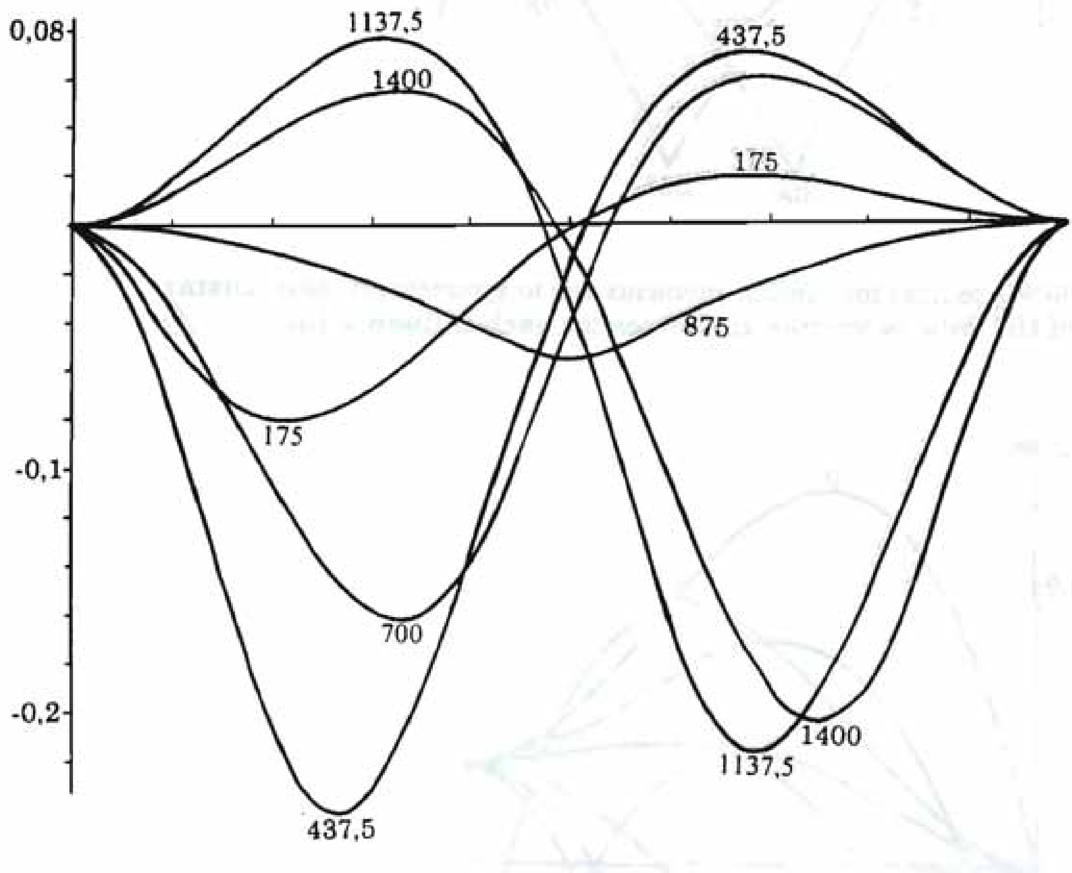


Fig. VI. Influence lines for deflection. m/MN. Distance from the left end of the tube is written in metres for each influence line.

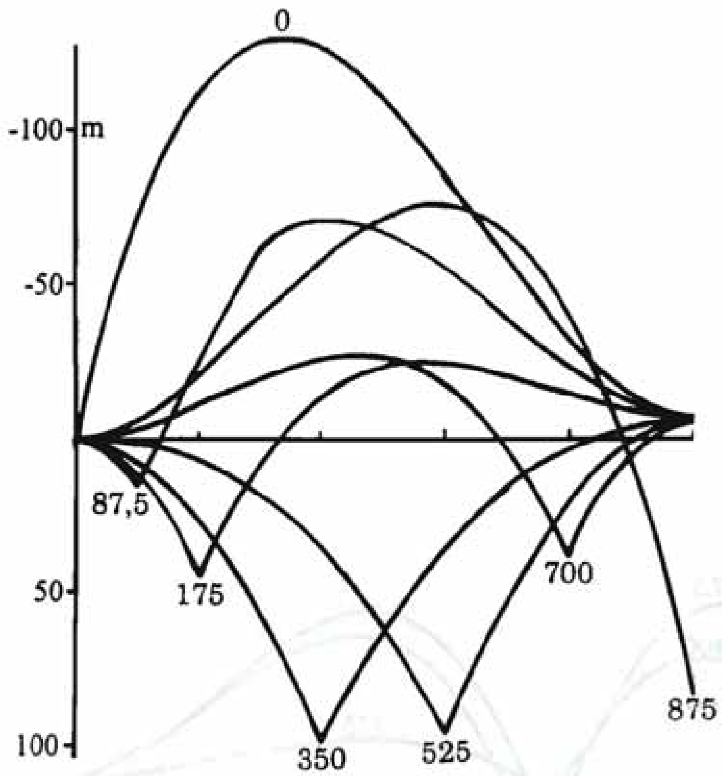


Fig. VII. Influence lines for bending moments due to symmetrical loads. Distance from the left end of the tube is written in metres for each influence line.

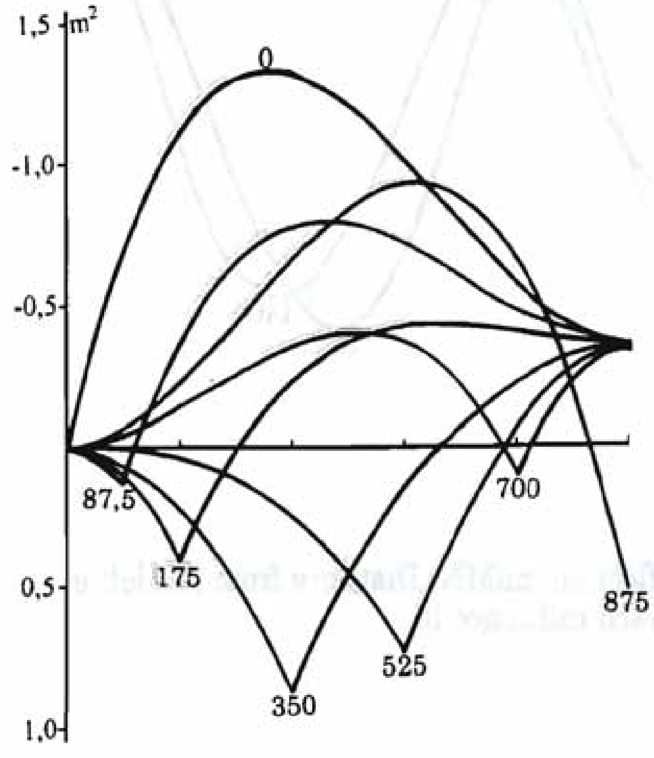


Fig. VIII. Influence lines for stresses in the topmost fibre of the concrete tube due to symmetrical loads. Distance from the left end of the tube is written in metres for each influence line.

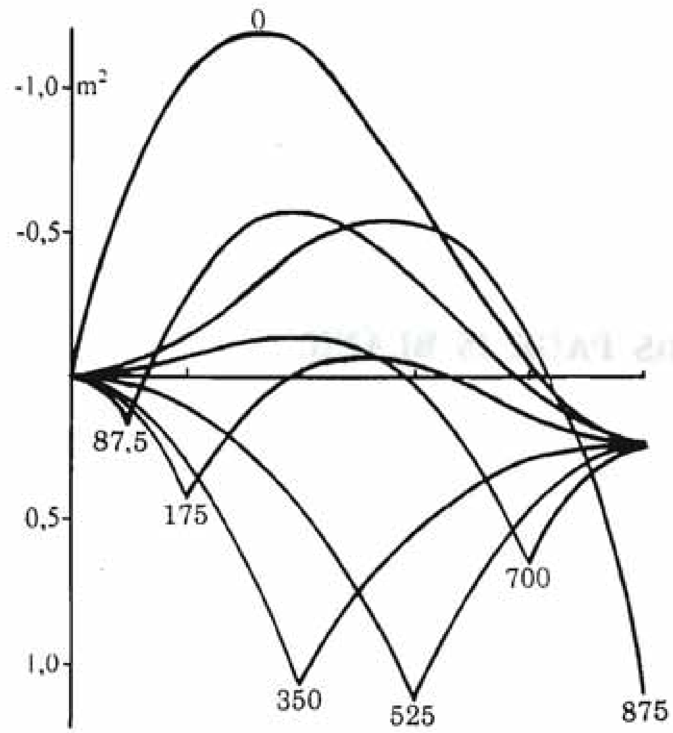


Fig. IX. Influence lines for stresses in the bottom fibre of the concrete tube due to symmetrical loads. Distance from the left end of the tube is written in metres for each influence line.

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Distance from end of tube in metres.	N=Axial load. M=Bending moment	Traffic load: Evenly distributed 18 kN/m.	Concentrated 1,26 MN	Due to 10 kN/m buoyancy	Water	density	Due to 10° drop in temperature	Current.(Moment vector in the plane of the arch.)	Forces due to 0,01 rad. rotation at each abutment.	Forces due to incomplete adjustment for variation in dead load and deformation.	Collapse limit state (1) or (2)·1,2+(3)·1,0	(4) or (5)·0,8
		M_{max}	M_{min}		High	Low					M_{max}	M_{min}
	M	314	-1637	654	150	-349	213	-188	-933	155	1151	-1589
0												
	N	55	51	-55	-8	20	6	0	-20	0	5	22
	M	220	-860	311	58	-139	170	-105	-791	155	155	-860
87,5												
	N	56	53	-55	-8	20	6	0	-20	0	6	25
	M	230	-302	44	-4	5	127	-49	-648	155	324	-321
175												
	N	53	52	-55	-8	20	6	0	-20	0	25	23
	M	676	-91	-258	-56	129	40	12	-365	155	682	-423
350												
	N	47	61	-55	-8	20	6	0	-20	0	17	34
	M	868	-297	-253	-40	98	-46	37	-81	155	867	-649
525												
	N	54	59	-55	-8	20	6	0	-20	0	25	10
	M	431	-495	59	16	-35	-133	50	203	155	589	-563
700												
	N	54	58	-55	-8	20	6	0	-20	0	16	31
	M	122	-641	291	53	-103	-176	54	345	155	479	-434
875												
	N	27	85	-55	-8	20	6	0	-20	0	-29	63
		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Deflections in m.												
(+ up, - down, s sidew.)				0,84	0,15	-0,4	1,31	s 0,6	1,26			
In point metres				438	438	438			350			
from left end.				1312	1312	1312	875	875	1400			

The bending moments in the SFT are found in the table above. The load factors used for calculating the collapse limit state are the same as for the 1989 competition for an SFT in the Høgsfjord. The load-carrying capacity in the collapse limit state has not been calculated. Previous experience is that the dimensions are decided by the serviceability limit state.

The necessary concrete strength will be C55 near the abutments and C35 in the rest of the concrete tube. The modulus of elasticity for C45 is 27836 N/mm². E=31000 N/mm² has been used in the calculations. This mistake has come about because the author had been taken by surprise by the low stresses.

E will be ≈10% lower in the structure than the value used in the calculations. Therefore the deflections stated in the table above will be ≈10% too low. Other effects of the low E will be less important.

Distance from end of tube in metres.	Maximum tensile stress	Due to traffic load	Due to 10 kN/m buoyancy	Due to variation in water density		Max. tensile stresses due to variation in temperature	Max. longitudinal stress due to water pressure. Minus local water pressure in	Stresses due to 0,01 rad. rotation at each abutment	Necessary prestress to counteract uncertainty in calculating deflections	Necessary prestress to counteract uncertainty in calculating deflections	Prestress needed to suppress all tensile stresses. (1)·0,8+(2) or (3)·1,0±(4)·1,0+(6)+(8) and (9)·0,5
				High	Low						
	σ_t	17,3	-7,6	-1,3	3,9	0,1	-0,1	8,5	0,7	0,8	10,9
0											
	σ_b	4,4	5	1,3	-2,9	3,8	-0,3	-9,5	0,7	0,8	14,0
	σ_t	9,8	-4,3	-0,8	1,8	1	-0,2	7,2	0,7	0,8	6,9
87,5											
	σ_b	3,5	1,7	0,4	-0,8	2,9	-0,4	-8,2	0,7	0,8	8,4
	σ_t	4,5	-1,7	-0,2	0,5	1,2	-0,3	5,8	0,7	0,8	4,1
175											
	σ_b	3,5	-0,9	-0,2	0,5	2,7	-0,5	-6,8	0,7	0,8	5,4
	σ_t	2,6	1,2	0,3	-0,8	1,8	-0,5	3	0,7	0,8	5,6
350											
	σ_b	8,2	-3,8	-0,7	1,8	2,2	-0,7	-4	0,7	0,8	7,8
	σ_t	4,3	1,2	0,2	-0,5	2,2	-0,8	0,3	0,7	0,8	7,0
525											
	σ_b	9,8	-3,8	-0,6	1,5	1,8	-1	1,3	0,7	0,8	7,1
	σ_t	6,2	-1,9	-0,4	0,8	2,7	-1	-2,5	0,7	0,8	6,3
700											
	σ_b	5,6	-0,7	0	0,2	1,2	-1,2	1,5	0,7	0,8	4,7
	σ_t	8,3	-4,1	-0,7	1,8	2,9	-1,1	-3,8	0,7	0,8	6,9
875											
	σ_b	2	1,5	0,3	-0,8	1	-1,3	2,8	0,7	0,8	3,9
		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)

The bending stresses in the SFT are found in the table above. The load factors here used for calculating the serviceability limit state, are the same as for the 1989 competition for a SFT in the Høgsfjord. Maybe a load factor higher than 0,5 should have been used for columns (8) and (9) when calculating "Prestress needed to suppress all tensile stresses" in the right column.

From the table it can be seen that we would get a more even distribution of prestress if the buoyancy in column (2) was 8 kN/m instead of 10 kN/m. There is no reason why this should not be done.

If concrete quality C95 is used for SFTs for road traffic in dead end fjords, two-lane abutment-anchored SFTs would probably be the most competitive structure for tunnel lengths up to 2,5 kilometres. See reasoning on economic lengths on page 18 and 42.

Due to less current and no change in salinity, inland lakes are even more favourable for this type of SFTs. If four lane tunnels are needed, the longest free spanning bridge in the world would not present any great technical problem. Compared to some of the things done in the North Sea in the last 20 years, this would be a technological anti-climax.

A lot more could have been said about these SFTs, but there was insufficient time available. Those who have questions are most welcome to write or ring me.

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AID-SERIEN REPORT FOU-RAPPORT

DESIGN OF A SUBMERGED FLOATING TUBE WITH A FREE SPAN OF 1750m

Preliminary edition - Handout for the Third Symposium on Strait Crossings

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DESIGN OF A SUBMERGED FLOATING TUBE WITH A FREE SPAN OF 1750m

Preamble:

It is now nearly 30 years since I was first involved in the design of a submerged tube. Over 25 years ago I became interested in the abutment anchored submerged floating tube. Years of investigations have made me more and more convinced that the abutment anchored STF is the most economical solution for deep fjords between 1 and 2 km wide.

Very few engineers share this opinion. Of course most engineers do not care one way or the other. Engineers that have been involved in the design of SFT's have the good sense to say very little about abutment anchored STF's, but the level of interest is low.

The situation seems well illustrated by the following statement: "When we have meetings on SFT's and I start talking about abutment anchored SFT's with a free span over one kilometer, then the audience seem to loose interest and start wondering when the next meal will take place."

The main reason why SFT's can be made so slender is that they float in near equilibrium with the ambient water. Thus constant dead weight is carried by the water. - It is a bit like designing a structure on the moon. The forces of gravity are much reduced.

-------- " --------

The purpose of this handout is to present detailed reasoning and calculations for an abutment anchored STF spanning 1750 meters. It is my intention to make the calculations in future editions of this publication relatively simple to control. Anybody is welcome to ask for more detailed treatment of whatever section of this book that they doubt or find interesting.

Administrative work and illness have prevented me from doing the calculations that were planned for the first edition of this publication. Thus this is only a very preliminary edition. The publication is mainly intended to show the scope of planned work. Some influence lines and a bit of reasoning is also included. At the back of the publication my three contributions to the three Symposiums on Strait Crossings can be found.

When I had finished my contribution for The Third Symposium on Strait Crossings I sent it to individuals that I thought might be interested in abutment anchored SFT's and asked for questions and comments. I heard from professor Milcho Brainow in Bulgaria, from professor Geir Moe and Erik Ødegård in Norway. I will take this opportunity to thank them for their letters. The letters did not contain any questions that demanded answers.

I would also like to thank docent John Conway for correcting the language of my third contribution to the Symposium of Strait Crossings. At present he is away in Canada and that has an influence on my English. Finally I would like to thank dr. Håvard Østlid for many interesting conversations on SFT's.

Grimstad, June 1994

Per Tveit

Plan for handout for the Third Symposium on Strait Crossings, Ålesund, 1994.
The handout will be completed later.

I. Influence lines: Influence lines for deflection. See page 4.
Influence lines for tension in the topmost fibre. See page 5.
Influence lines for tension in the bottom fibre. See page 6.
A note on data used in the calculations of influence lines. See page 7.

II. Calculation of SFT in the deflected state.
Calculation of stresses considering prestress, creep and shrinkage. See note page 7.
Sideways buckling and effect of current on SFT. See note page 7.

III. Vibrations.
Modes of vibration.
Movements of water particles due to deep water waves and stresses due to movements of the SFT with the same frequency. See note page 8

IV. Calculation of the SFT when the vertical curvature of the tube is part of a circle.

V. Possible benefits of deviation from a circle in the cross-section of the tube.

VI. Comments on the contribution to the Third Symposium on Strait Crossings, June 1994.

A Loads on SFTs.

B Vibrations of the SFT and how it is influenced by the axial force.
Tolerable levels of vibrations.
Fins to suppress vibrations of SFTs.

C Deflections of the SFT.
Measurement of deflections of the SFT. See page 11.
Accelerations when the SFT does deflect.

D Various ways of casting the concrete tube for the SFT.
Variation of concrete quality along the SFT.
Extra reinforcement, increased concrete quality or increased concrete cross-section near the abutments.
Prestressing with visible or grouted cables.

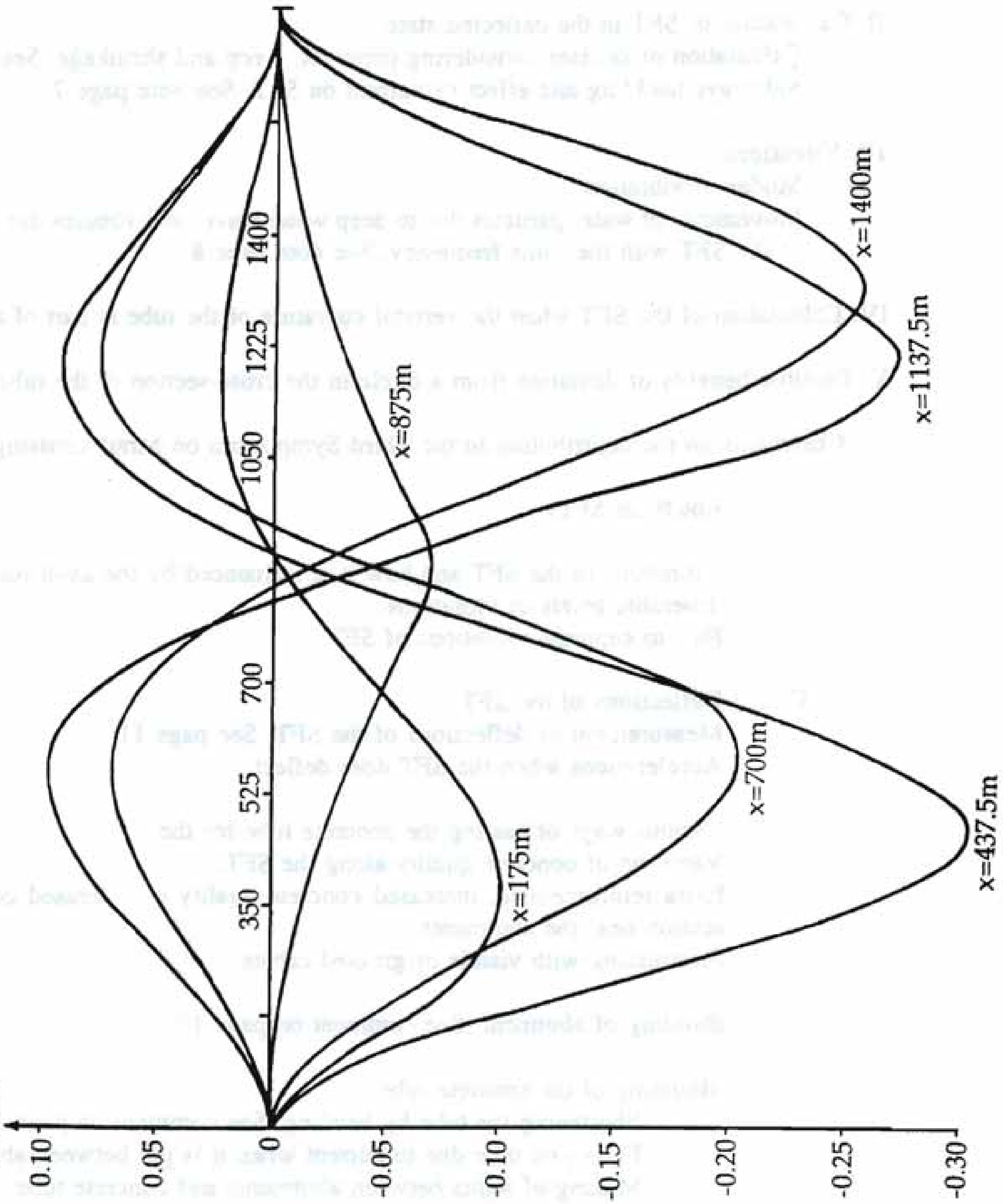
E Building of abutment. See comment on page 10.

F Mounting of the concrete tube.
Shortening the tube by bending. See comment on page 11.
Forces on tube due to current when it is put between abutments.
Making of joints between abutments and concrete tube.

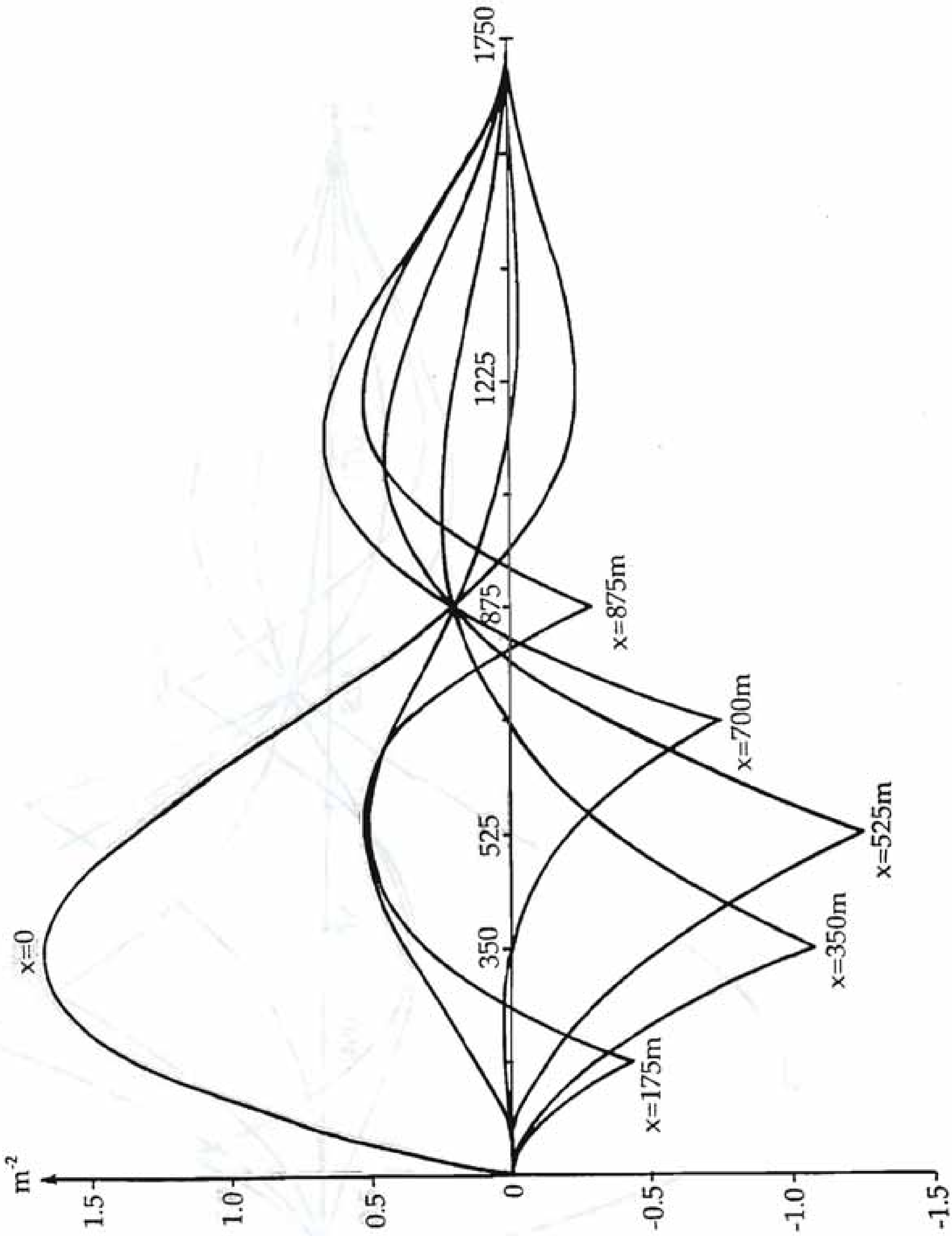
G Economy and costs.

H Abutment anchored SFT's for other spans.

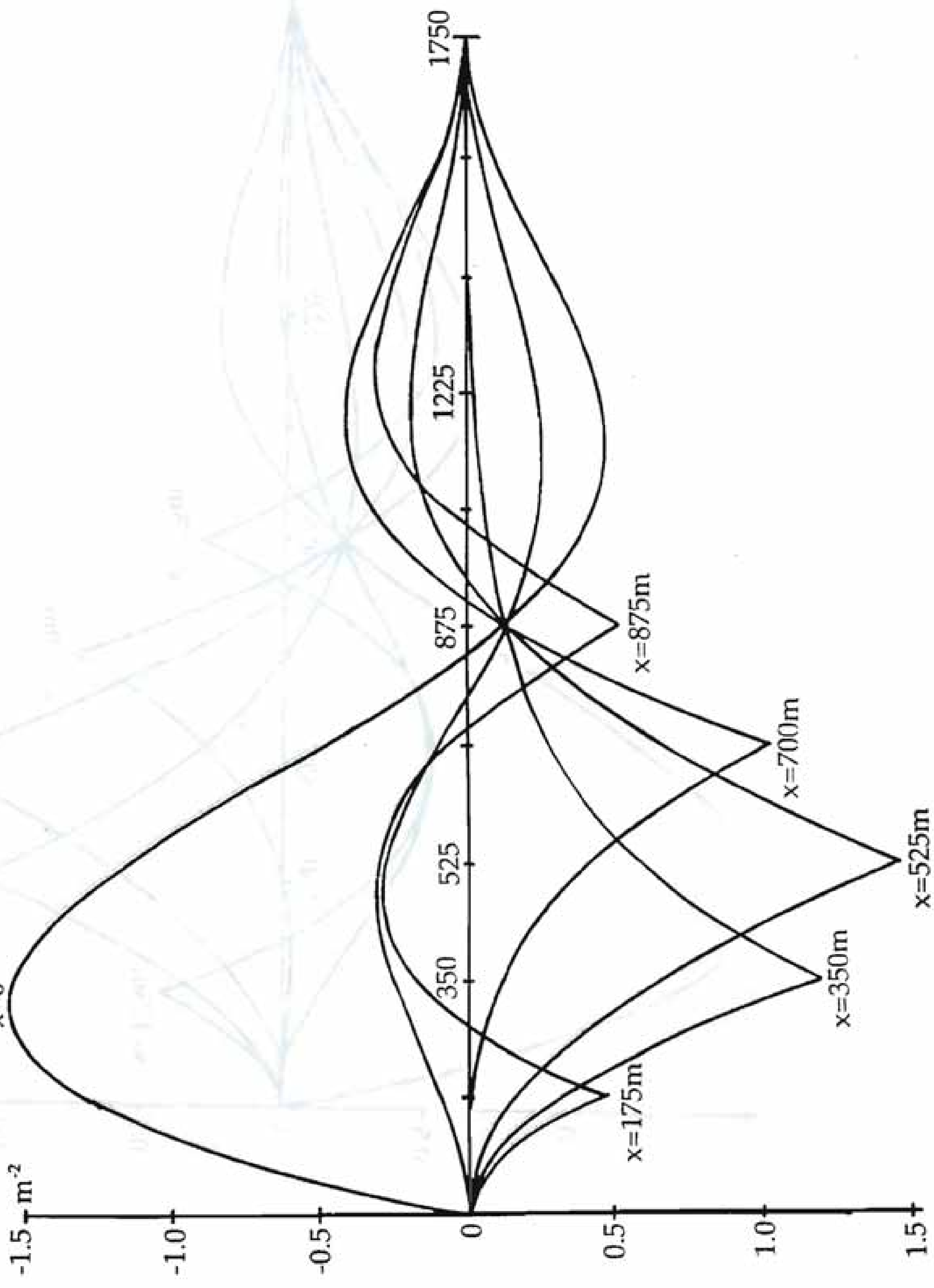
At the back of the handout are the author's contributions to Symposium on Strait Crossings in 1994 page 12, in 1990 page 20, and in 1986 page 28.



Influence lines for deflection



Influence lines for tension in the topmost fibre.



Influence lines for tension in the bottom fiber.

A NOTE ON DATA USED IN THE CALCULATION OF INFLUENCE LINES

The influence lines were calculated by means of the computer program ANSYS. In the middle 20% of the tube curved pipe elements are used. In the rest of the SFT straight pipe elements are used. So far all calculations have been made disregarding the concrete inside the concrete tube. That is the concrete in and supporting the roadway. The inner and outer radii of the concrete tube are 4.75 and 5.58 meters respectively. This gives: $A=40.7\ m^2$, $I=539\ m^4$, $W= 91.3\ m^3$.

A NOTE ON MAXIMUM STRESSES IN THE DEFLECTED STATE INCLUDING EFFECTS OF PRESTRESS, CREEP AND SHRINKAGE

For the abutment anchored SFTs that the author has examined, the dimensions are decided by the serviceable limit state. For the decisive load cases of the 1750m SFT, there will be a moderate tensile axial force in the serviceability limit state. When there is a tensile force in the tube, stresses in deflected state will be slightly smaller than the stresses we find by means of the influence lines.

The biggest stresses in the SFT occur just in front of the abutment. Preliminary calculations show that concrete C85 will hardly be strong enough to take the stresses that occur here. Maybe we will have to use C95. We could also use extra compressive reinforcement.

Yet another solution can be suggested. Extra concrete could be cast under the lane after the tube is put in between the abutments. This should be done before the prestressing cables that joins the concrete tube to the abutment, are stressed. Less than fifty meters from the abutment, a tube made of concrete C85 would have enough strength. For the middle 85 % of the tube C55 would probably be a sufficient concrete strength.

A NOTE ON SIDEWAYS BUCKLING AND STRESSES DUE TO CURRENT

If we assume a modulus of elasticity 30 GPa, the Euler load of a straight tube with fixed ends and the same cross-section and length as the SFT would be 208 MN. It is assumed that the buckling load for sideways buckling of the STF is about 180 MN.

It seems reasonable to limit the maximum compressive force in the SFT to 45 MN. This is about a quarter of the sideways buckling load. The shape of the tube would be under constant observation. Thus a slow sideways buckling due to creep would not be allowed to take place. Bending moments due to current would be enhanced due to the relatively high compressive force in the SFT, but stresses due to currents are small, so this does not matter.

The buckling load for buckling in the vertical plane is about 416 MN. Vertical loads that cause big bending stresses, greatly reduce the axial force. Thus the bending moments due to vertical loads are not much enhanced due to the SFT's tendency for buckling in the vertical plane.

A NOTE ON THE QUESTION OF THE STRESSES IN THE SFT DUE TO WAVES.

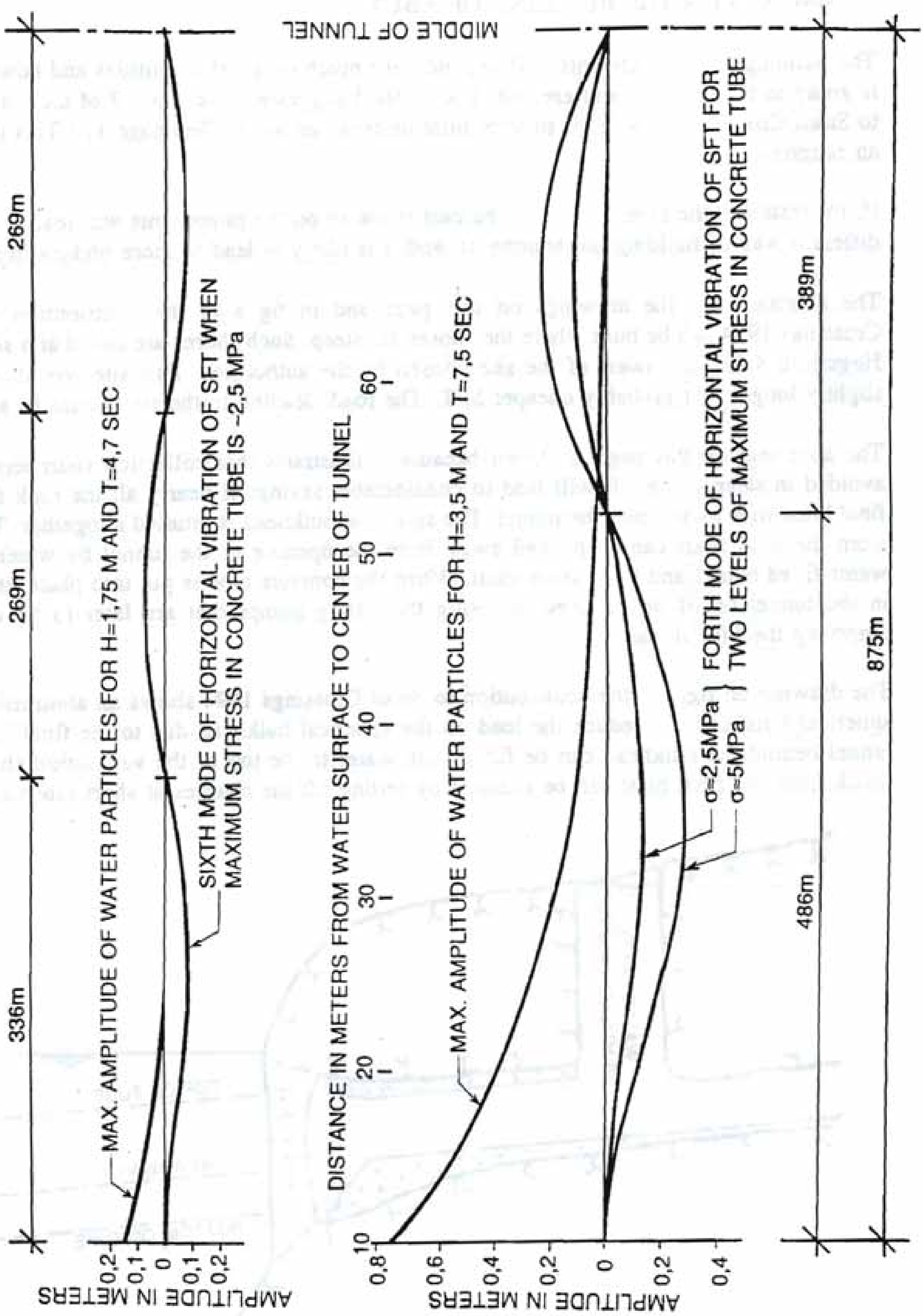
When waves and SFT have the same frequency and there is a suitably small angle between the SFT and the wavefront, resonance can cause movements and stresses in the SFT. A diagram on the next page shows max. amplitudes of water particles for waves with H=1.75m and T=4.7 sec. Another line shows max. amplitudes of the SFT when T=4.7 sec. and max. stress is ~2.5 Mpa.

It seems unlikely that the movement of the water will cause much movement of the SFT. Thus stresses due to these waves must be insignificant. This is important because most sites in Norwegian fjords have maximum significant wave heights under 1.75 m. It is also worth mentioning that the movement due to the drawn vibration of the SFT, gives a maximum acceleration of ~0.18 $^{m}/_{sec}{}^{2}$. This lies nicely between allowable accelerations for permanent offshore structures and the allowable acceleration for skyscrapers.

The other diagram on the next page shows max. amplitudes of water particles for waves with H=3.5m and T=7,5 sec. Other lines show max. amplitudes of the SFT when T=7.5 sec. and the max. concrete stresses are 2.5 Mpa and 5.0 Mpa. Here it is hard to say what is an upper limit to the stresses due to the waves.

Good mathematicians will work out what stresses will occur if the waves have the shape of a sinusoidally corrugated sheet of iron and hit the SFT at the most unfavorable angle over a long period of time. This is a most impressive exercise. One might doubt that the waves will ever look like a corrugated sheet of iron along the whole SFT for a considerable length of time. With time the more mathematically gifted engineers will be modelling the waves in a realistic way.

At the moment engineers with lesser learning can take comfort from the fact that waves of 3.5m occur very seldom in Norwegian fjords where SFT might ever be asked for. Secondly there will be very little traffic when the biggest waves occur, and then a stress of 5 Mpa is not dangerous.



Diagrams that compare amplitudes of the SFT, when it has a certain max. stress, with how water particles move due to waves. The SFT and the waves have the same period of oscillation.

COMMENTS ON THE BUILDING OF ABUTMENTS

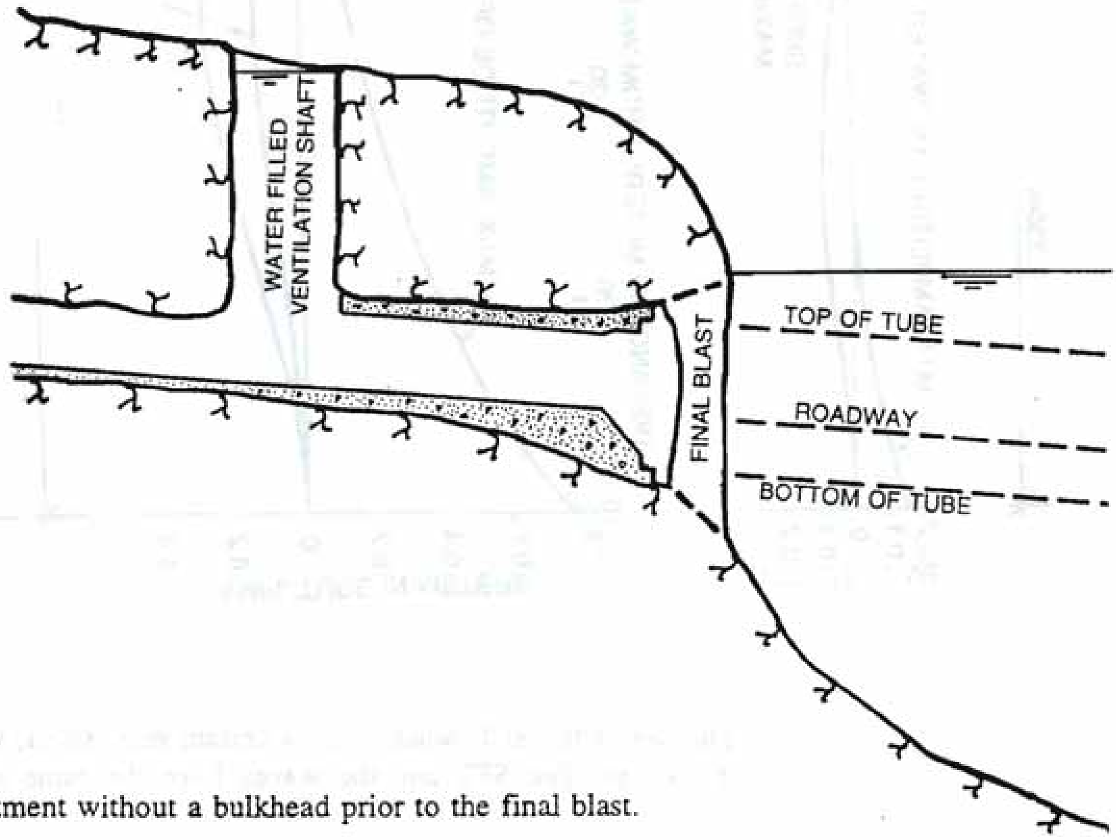
The building of the abutments will depend very much on local conditions and how the SFT is going to be produced and erected. The method suggested in chapter 7 of the contribution to Strait Crossings 1994 leads to very little underwater work. (See page 17.) This is usually an economic advantage.

If, for instance, the tube is going to be cast stepwise on the ramps, this will lead to a totally different way of building the abutments, and it is likely to lead to more underwater work.

The abutments in the drawings on this page and in fig 8 of the contribution to Strait Crossings 1994 can be built where the shores are steep. Such shores are found at a site in the Høgsfjord 4 km northwest of the site chosen by the authorities. This site would lead to a slightly longer, but probably cheaper SFT. The roads leading to the site would be shorter.

The abutment on this page is shown because it illustrates that collection chambers can be avoided in steep shores. It will lead to considerable savings if nearly all the rock from the final blast slide away from the tunnel. The spherical bulkhead is omitted altogether. The rock from the final blast can be pushed away from the opening of the tunnel by water from a water-filled tunnel and ventilation shaft. When the concrete tube is put into place, the water in the tunnel could be removed by using the strong pumps that are later to be used for emptying the SFT in cases of emergency.

The drawing on fig 8 in the contribution to Strait Crossings 1994 shows an abutment with a spherical bulkhead. To reduce the load on the spherical bulkhead due to the final blast, the tunnel behind the bulkhead can be filled with water to the top of the ventilation shaft. The shock from the final blast can be reduced by setting off the charges at short intervals.



Abutment without a bulkhead prior to the final blast.

A NOTE ON MEASURING DEFLECTIONS OF THE TUBE

When the STF is finished we can see from the abutments to the lowest point of the roadway in the middle of the concrete tube. This makes it straightforward to monitor the deflection of the SFT. While the concrete tube is being cast, the shape of the part that is finished must be measured carefully.

Significant bending in the concrete tube due to change in deadweight of the tube can be detected by measuring the deflection. The bending can be counteracted by redistribution of ballast water. Creep and shrinkage will also cause change in the deflection of the tube, but it will probably not be too difficult to tell what is a near optimal shape of the SFT and distribute the ballast water accordingly.

A NOTE ON SHORTENING THE TUBE BY BENDING

Before the concrete tube is placed between the abutments, it must be shortened by bending. Tensile stresses can be obtained in the lower fibre of the concrete tube by concentrating the ballast water to introduce a bending moment near each end of the tube. A tensile stress of 10 Mpa in the lowest fibre all along the concrete would shorten it by 3.7 meters. Since the average compressive stresses in the lowest fibre in the middle 80% of the tube is about 10 Mpa, it should be risk free and easy to shorten the tube by about ~2 meters.

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Third Symposium on Strait Crossings / Ålesund / 12-15 June 1994

Design of a submerged floating tunnel with a free span of 1750 m

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ABSTRACT: Abutment-anchored submerged floating tunnels (SFTs) are probably the most economic permanent link across deep Norwegian fjords of over 800m width. This paper describes a two-lane abutment-anchored SFT with a span of 1.75 km. The SFT will probably have harmless in-line vibrations with a period of oscillation of ~100 sec. The frequency is influenced by the axial force in the tube. The change in frequency can be used to monitor the change in the deadweight of the SFT. This change can be counteracted by altering the ballast in the SFT. If there are no in-line vibrations, the SFT can be set in motion. Uneven deflection of the SFT, without traffic load, can be counteracted by redistribution of ballast. The SFT in this paper will be analyzed in the deflected state, and the results of the calculations will be available at the symposium in Ålesund.

1 INTRODUCTION

This paper describes a two-lane submerged floating tunnel (SFT) with a free span of 1750 m. It is anchored at the abutments only. See fig. 1 and 2.

The suggested structure is very slender. Thus many good engineers may feel skeptical about it. The author finds this easy to understand, because he is generally conservative regarding questions to which he has not given much thought or examined carefully.

This type of SFT was suggested by the author 25 years ago (Tveit 1969), but until 1985 the author did not understand that these tunnels were feasible for many sites. These SFTs are suggested for deep, dead-end fjords where waves and currents are moderate. There are many such fjords in Norway. The author leaves it to others to examine the use of these structures where waves are over 5 m and the current is over 0.5 m/sec 30 m below sea level.

SFTs spanning 1250 and 2500 m have been designed by Tveit (1986 and 1990). Such long free spans are possible because the tunnels float in near-equilibrium with the water which they displace. Depending on local conditions, these tunnels are competitive for spans from 800 m upwards.

Unlike these SFTs the tunnel in this paper will be analyzed in the deflected state. The calculations were not finished before the deadline for submission of this paper, so instead they will be presented to the participants in Ålesund.

Abutment-anchored SFTs are slender beams. Thus their analysis is simple. Determining the loads and load combinations to be considered is more difficult. The loads to be considered should depend on how the SFTs are to be built and on the inspection routines selected. Hakkaart (1993). Extensive monitoring of SFTs is recommended.

In the present design continuous monitoring of the frequency of in-line vibrations has been assumed. Changes in this frequency reflect changes in the axial force in the tunnel. These changes are due to changes in the permanent loads. Furthermore, uneven changes in permanent loads show up in the deflection of the tunnel. See chapter 5.

Slow changes in the permanent loads on the tunnel can be compensated for by altering the amount of ballast water in the tunnel. Ballast water is stored in small dams under the roadway. The amount of water is altered by varying the height of the dams. See fig. 2. Movable concrete ballast can also be used.

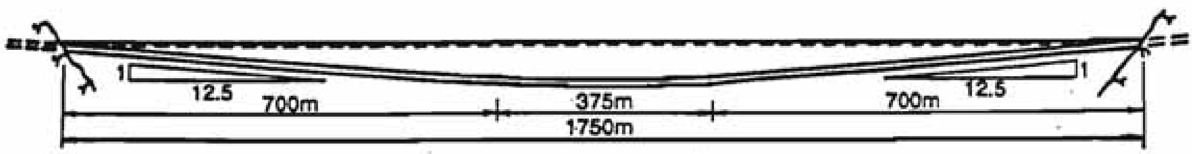

Fig. 1 Section through fjord with abutment-anchored SFT spanning 1750m

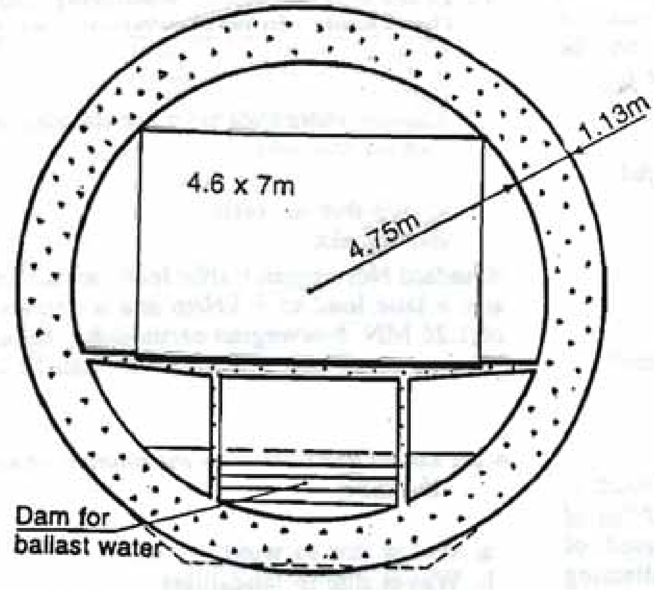

Fig.2 Cross-section of the SFT in fig. 1

Concrete quality will probably be C55 in most of the SFT and up to C85 near the ends. The cross-section in fig. 2 has been chosen to simplify the analysis. It would be economic to concentrate more of the concrete at the top and bottom of the cross-section. Due to the high outside pressure, the center-line of the cross-section should always be nearly circular.

The downward-arched submerged tubes can of course be built without the use of ballast to counteract the change in deadweight of the tube. Tveit (1986 and 1990). Monitoring the weight of the tube, altering the weight of ballast, or grinding down the asphalt on the roadway can reduce the amount of dead load that has to be taken into consideration in the analysis of the SFT. This makes longer free spans possible.

2. VERTICAL CURVATURE OF THE SFT.

The road inside the tube has a maximum gradient of 8%. This slope has been generally accepted for SFTs in Norway. This acceptance is influenced by the fact that the lanes in the tube have no snow, less curves, and are less likely to be wet or frozen than roads on normal hills. The steep slope gives reduced traffic capacity, but two-lane roads would have ample capacity in locations where Norwegian SFTs could be built.

The vertical curvature is concentrated in the middle 20% of the concrete tube. This gives near maximum rise to the arch of the SFT, which leads to less variation in the normal force due to vertical loads. Furthermore it gives smaller stresses due to temperature variation, shrinkage and axial creep.

A constant curvature in the concrete tube would make production easier. Such a tube can better carry evenly distributed loads without resulting in big bending moments. The constant curvature would give about 55% of the rise in the arch of the suggested SFT. Consequently an arch with constant curvature has nearly twice the variation in the axial force as compared to the present design. This would make sideways buckling a more important mode of failure.

If we compensate for changes in the weight of the tunnel by altering the amount and position of ballast, the variable loads along the tunnel will be less important. Furthermore, total vertical load will be smaller, but the evenly distributed load increases. This favors an SFT with a constant vertical curvature.

Some users of the tunnel would worry if they could see changes in the vertical curvature of the tunnel. This is a good reason for avoiding long constant slopes at both ends of the tunnel. If the straight portions at each end of the tunnel were given a camber of 1 m, the rise of the tunnel would alter by about 4 m. This would make it nearly impossible for motorists to see changes in the deflection of the tunnel and the rise of the tunnel would become only ~6% smaller.

3 INFLUENCE ON BENDING MOMENTS OF THE AXIAL FORCE IN THE TUNNEL.

In the two previous abutment-anchored SFTs (Tveit 86 and 90) the author did not analyze the deflected state of the tunnels. The SFT in this paper will be analyzed in the deflected state and the results of the calculations will be available at the symposium.

The tendency for horizontal buckling will increase the bending moments due to currents. This is of little importance since the bending moments due to currents will contribute less than 1% to the maximum bending moment.

In slender columns made of prestressed concrete the elastic buckling load is very predictable. Thus the author suggests that in the ultimate limit state the maximum axial compressive force in the tunnel be about 60 MN. This is about $^1/_3$ of the elastic buckling load.

The elastic buckling load for buckling in the vertical plane is about twice as high as the buckling load for horizontal buckling. Thus the tendency for buckling will cause little increase in bending moments due to vertical loads. Furthermore, the vertical loads which give the biggest bending moments also cause tension in the tube. Thus there is no tendency for buckling accompanying the biggest bending moments.

4 LOADS ON SFTS WITH LONG SPANS

When the effects of loads, such as axial force and deflection, are monitored, we can counteract the loads by ballast water, concrete ballast and grinding down or adding asphalt to the roadway. Periodic monitoring of the shape of the tunnel is needed, together with continuous recording of the movements of the tunnel. For the design of the SFT we can use reduced values for all loads which can be counteracted by ballasting. (See also chapter 6.)

4.1 *Loads which can be reduced when careful monitoring and ballasting are used*:

A. Change in weight of concrete
B. Wear of asphalt on the roadway
C. Dust and debris inside the tunnel
D. Marine growth on the surface of the tunnel
E. Shrinkage of concrete
F. Creep and relaxation
G. Tensing of prestressing cables

The loads A through D can be treated as one load. If they increase more than an average of 14 kN/m of tunnel, then the change in the lowest period of oscillation will decrease ~10% and reballasting should already have been undertaken.

To control uneven distribution of the loads A through D, we need to measure the shape of the tunnel. The shape of the tunnel will also be influenced by shortening of the tunnel after it has been installed. This shortening will give an upward movement of all points of the tunnel. We should aim at a maximum upward movement in the middle of the tunnel. In the 100 m nearest to the abutments the desired movement should be determined by loads such as creep, shrinkage, marine growth etc. In the rest of the tunnel we should aim for a curve which would cause near minimum bending stresses in the tunnel. If the shape of the tube deviates by more than 40 mm from this curve, redistribution of ballast should be performed.

Now comes a difficult question. If monitoring prevents more than 14 kN/m increase in the average load on the tube and more than 40 mm deviation from nearly ideal deflection, what loads should be used for A through D in the analysis of the suggested SFT? If no better answer presents itself the author will let the loads A through D correspond to 14 kN/m over the whole tunnel plus a concentrated load of 200 kN. For the SFTs in the Høgsfjord a load of ±20 kN/m distributed like a traffic load was specified.

Load E, shrinkage of concrete, is small in "old" concrete. Wet concrete swells a little and this compensates for much of the shrinkage due to drying on the inside of the tube.

Load F. Creep. These preliminary calculations have been done for t=∞, when creep and relaxation have already reduced the prestressing forces. Tveit (1990) p. 406 tells why creep and relaxation do not enter into the calculations at this stage.

Load G, the prestressing, has been applied before the tunnel has been mounted between the abutments. Thus there are no prestressing forces due to the constraints. The ballasting to maintain the shape of the tunnel will also assure that the loads E, F and G will give rise to small stresses.

4.2 *Loads independent of monitoring and ballasting.* These loads can be divided into two groups:

4.2.1 *Loads independent of the amount of water above the tube.*

α. Loads due to traffic
β. Earthquake

Standard Norwegian traffic loads are assumed, which are: a lane load of 9 kN/m and a concentrated load of 1.26 MN. Norwegian earthquakes have up to now been so small that this load case can be omitted.

4.2.2 *Loads dependent on the amount of water above the tube.*

a. Waves due to wind
b. Waves due to landslides
c. Variation in the density of water
d. Temperature variation
e. Collision with surface ships
f. Collision with submarines
g. Explosion loads
h. Static loads caused by currents
i. Dynamic loads caused by currents

a. Norwegian fjords are usually sheltered from ocean waves. At the Høgsfjord site significant wave height is 1.5 m. Looking at fig. 3 we get the impression that a tunnel which lies between 4 and 79 m below sea level is subjected to very small forces due to waves for wave heights under 4 m. Energy absorbed

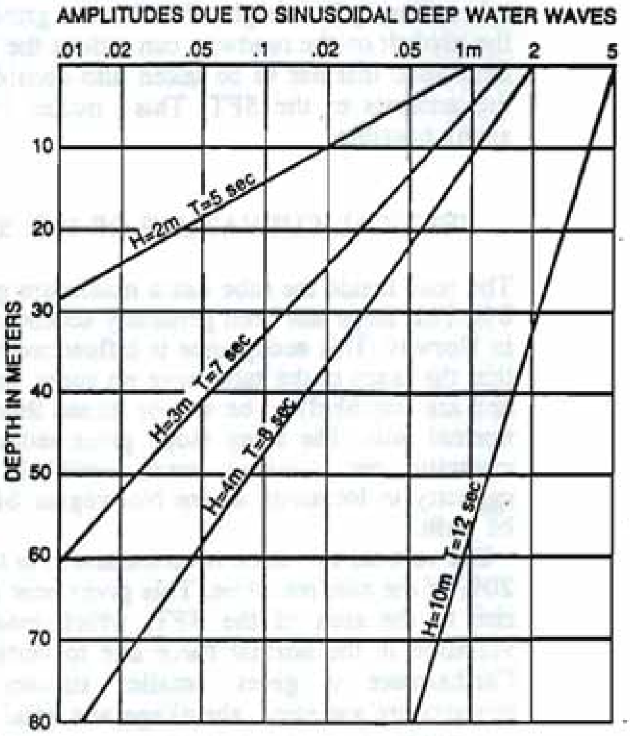

Fig. 3 Amplitude of oscillations due to waves

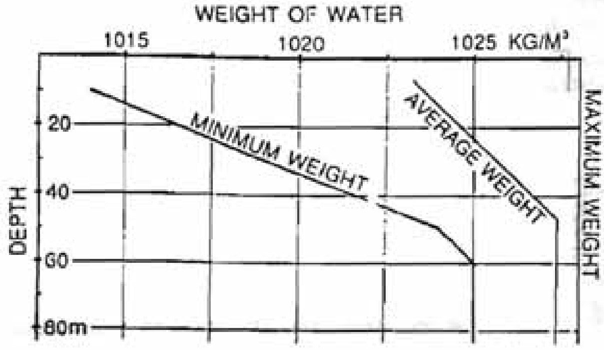

Fig. 4 Assumed variation in the density of water

in the upper part of the SFT will dissipate in deeper waters where there is hydroelastic damping.

b. In Norway most mountains were polished by glaciers during the last ice age. Thus waves due to landslides do not give a relevant loadcase in most fjords.

c. Fig. 4 shows the assumed variation in the density of water. Maximum density is the density of Atlantic water. The amount of less salty water depends on the volume of the rivers discharging into the fjord. The average density in fig. 4 is taken from Eidnes (1989). The minimum density chosen is influenced by the charts in Eide (1974).

d. In the fjord the temporal variation of temperature decreases with depth. The calculation of stresses due to temporal variation in temperature has been done assuming that the average variation in the concrete tube is ±10°C and the max temperature variation across the concrete wall is 10°C. This is believed to err on the safe side.

e. It has been assumed that collision with surface ships is so unlikely that this loadcase need not be considered. For fjords with considerable traffic the tunnel could be placed deeper.

f. Collision with submarines can be made even more unlikely by emitting ultrasound from the tunnel.

g. Explosion loads. The round cross-section is very good for taking the load from internal explosions. The tube is, like the Høgsfjord SFTs, designed for an explosion load due to gasoline of 0.7 MPa. Oud (1987) explains how explosion loads (2.3 MPa) and deflagration loads (0.3 MPa) can be avoided.

h. The speed of currents which has been used for the design of the SFT is shown with a full line in fig. 5. The 100-year maximum values found by Eidnes (1989) in the Høgsfjord are shown with a dotted line. Nothing is reported by Eidnes (1989) on simultaneous currents at various depths.

However, the following argument seems reasonable. When the strongest currents occur, the direction of the current near the surface is opposite to the direction of the current further down. The inward and the outward currents have about the same volume. The thin line in fig. 5 shows what the current might be like at one moment in time. The two shaded areas must be roughly the same. The boundary between the inward and outward currents is usually where the change in salinity is greatest.

If the current acting on the SFT is unidirectional, it is either uneven or very small because the volume above the SFT is very small. If the current on the SFT is not unidirectional, this will reduce the resulting bending moments due to currents.

In Tveit (1990) the contribution of current to the max bending moment in the tube was less than 1%. Thus more knowledge of the currents is not necessary in a feasibility study.

i. Dynamic loads due to currents. See next chapter.

5 VIBRATIONS AND DEFLECTIONS

Humans sometimes feel uncomfortable when subjected to accelerations. This is the main reason for trying to limit vibrations and deflections in structures. Tunnels like the suggested SFT will probably be closed to pedestrians. Motorists in motion are subject to acceleration at every curve. Thus they are not so sensitive to acceleration as pedestrians. Acceleration is not a problem in the proposed SFT.

The SFT lies between 5 and 79 m below sea level. At such depths the speed of the current is moderate and uneven. See fig. 5. For vertical vibrations the longest period of oscillation is ~34 sec. A current of 0.9 m/sec would give vertical hydroelastic vibrations, but such a speed of current will not occur. See fig. 5.

For horizontal vibrations the longest period of oscillation, ~100 sec, corresponds to currents between 0.16 and 0.33 m/sec . Such currents are likely to occur over long parts of the concrete tube. The maximal amplitude of the oscillation might be up to 20% of the diameter of the tube, which is 2.35

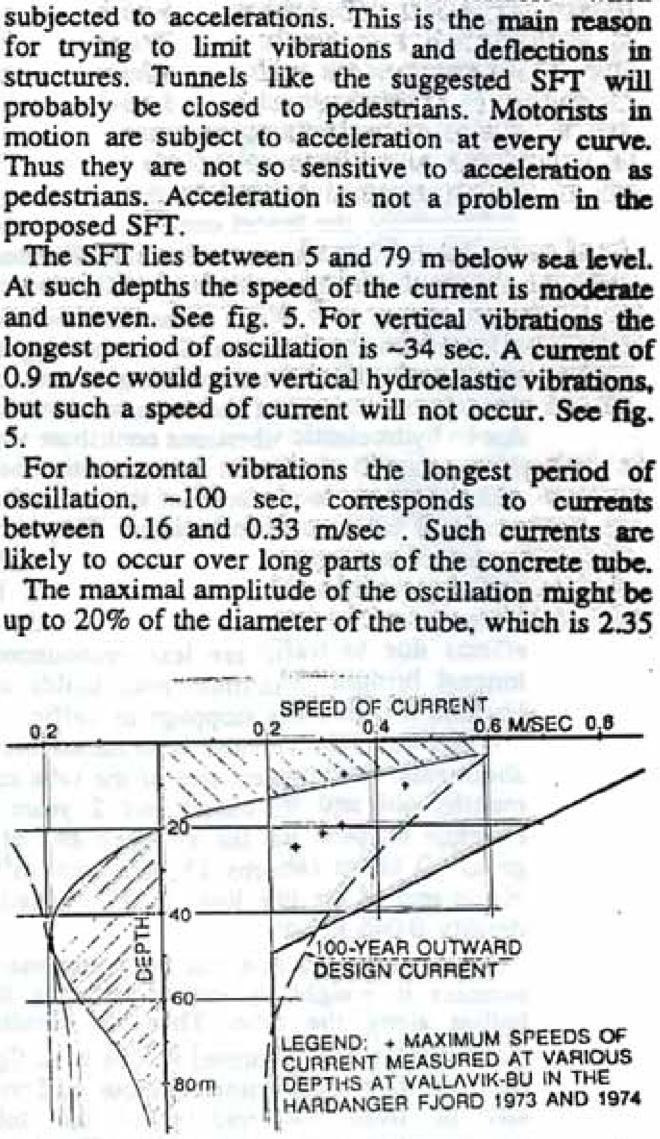

Fig. 5 Speeds of currents at various depths

m. (Moe 1989). The acceleration due to such a movement will not exceed 0.01 m/sec^2 . This acceleration is only ~2% of the max. allowable horizontal acceleration for oil platforms when the period of oscillation is 16 sec. Jenssen (89).

These oscillations will not be noticed by motorists because they will normally drive through the tunnel in about one period of oscillation. The bending of the tube is not easily noticed because of the concentrated curvature in the middle of the tube. This concentrated curvature makes sighting along the middle of the tunnel more difficult.

The horizontal hydroelastic vibration is due to vortex shedding symmetric about a horizontal plane through the axis of the tube. The oscillations could probably be suppressed by fins on the cross-section or maybe by giving the cross-section a horizontal sole as indicated by the dotted line in fig. 2.

There is no point in suppressing the horizontal vibrations. They are harmless and, most importantly, their frequency is influenced by the axial force in the tube. This axial force is determined by the total vertical force on the tube. Thus by monitoring the frequency of the horizontal vibrations, we can follow slow changes in the dead weight of the tube. Calculations on this phenomenon will be presented at the symposium in Ålesund.

If horizontal vibrations do not occur, or occur too infrequently, the tunnel can be set in motion to find its lowest frequency of horizontal vibration.

Stresses due to a sideways deflection of 2.35 m could be up to 5 MPa. These stresses occur at the sides of the cross-section. Since the biggest stresses are due to vertical loads, which give max. stresses at the top and bottom of the cross-section, the stresses due to hydroelastic vibrations contribute very little to the maximum stresses in the concrete tube.

The dynamic load effects of the traffic load on the proposed SFT can be calculated. They are believed to be somewhat similar to dynamic effects for ordinary bridges. The slenderness and flexibility enhance some dynamic effects, but the dynamic effects due to traffic are less pronounced for the longest bridges. Maximum load builds up slowly because it is due to a stoppage in traffic.

When the concrete tube is mounted between the abutments, the youngest end of the tube could be 3 months old and the oldest end 2 years old. For example suppose that the youngest end of the tube gains 10 kN/m (absorbs 23 liters per. m^3) and the oldest end of the tube loses 2 kN/m. (Reduction of density 0.046 kN/m.)

Let us further assume that we compensate for the increase in weight by evenly reducing the water ballast along the tube. Then the resulting load distribution along the tunnel will be as in fig. 6. This load will give a deflection of about ±0.5 m at about 400 m from the middle of the tube. The corresponding stress will be 1.5 MPa. It would be simple to counteract this deflection by redistributing ballast water in the tube.

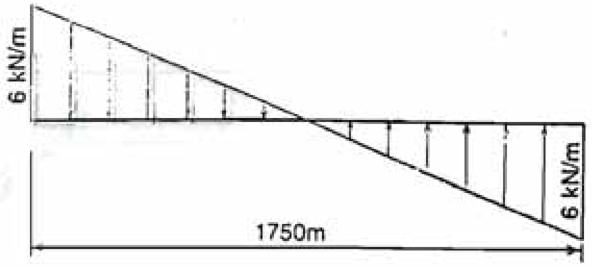

Fig. 6 Load on SFT due to change in weight of concrete

This example illustrates the advantages of using ballast to counteract uneven changes in loading along the tube. Ballast can be used to make the tube keep to the desired shape determined at casting, with additional changes of shape due to prestress, creep etc.

Ballast water can be stored in dams below the roadway. See fig. 3. The height of the dams can be altered by adding or removing horizontal prestressed concrete beams from the crowns of the dams.

Efficient use of ballast water to counteract slow changes in the dead weight of the tube necessitates careful measuring of the shape of the concrete tube during production and use. Whether this effort is worthwhile depends on the assumed uncertainty in the weight of the concrete tube. This method will give the greatest economic advantages for the longest free spans.

The method will lead to more knowledge of the change of dead load in SFTs. This will in turn lead to less need for ballast to counteract slow changes in dead weight of the concrete tubes of an SFT and undesirable changes of shape in SFTs.

6 CASTING OF CONCRETE TUBE

Many ways of casting a concrete tube are mentioned in Hakkaart (1993). The author prefers the stepwise method indicated by fig. 7. Here the formwork has been mounted on a pontoon. Tveit (1990)

The utmost care must be taken to give the concrete tube an evenly distributed weight. This means fabrication of concrete walls with constant thickness. By the suggested method of production the tube can be made to float freely when, for example, 100 m of the tube has been cast. How the tube floats show possible deviations from the prescribed weight of the last 100 m cast. These deviations can be adjusted for when deciding on the thickness of the lane or by the amount of ballast in the tube.

Casting could also be done from the shore in a sheltered fjord with high water density. The part of the tube already fabricated must be prevented from moving sideways, and the ballasting of the tube must not introduce too big bending moments in the part of the tube that has just been cast and prestressed.

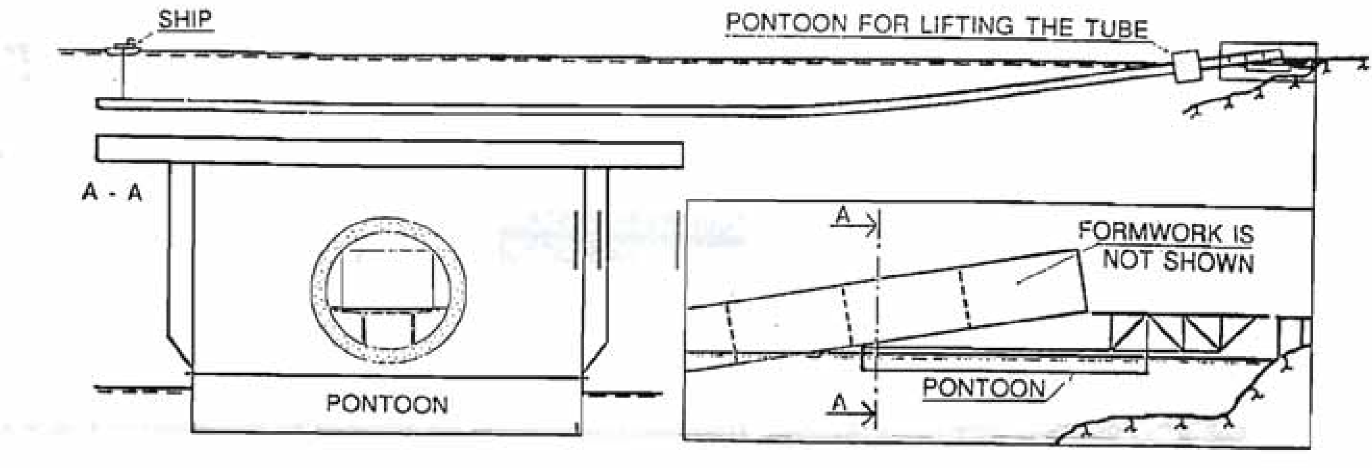

Fig. 7 Rig for stepwise casting of concrete tube

7 BUILDING OF ABUTMENTS

Where fjords are narrowest, there is usually good rock. It will often be practical to make the approach to the SFT through rock as indicated in fig. 8. Tveit (1969 and 1986) and Hakkaart (1993). A concrete tube to be joined to the SFT is made as a lining to the rock tunnel. This tube is sealed with a temporary spherical bulkhead.

At the shore where the concrete tube is put in place first, the concrete lining can end in a deep hole. Where the concrete tube is put in place last, the rock must not extend more than one or two metres over the concrete lining. Otherwise it will be impossible to move the floating concrete tube into position.

Before the final blast, the rock tunnel and the collection chamber are temporarily filled with water. An air pocket is left to help absorb the shock. The last bit of rock is then blasted away and falls into the collection chamber.

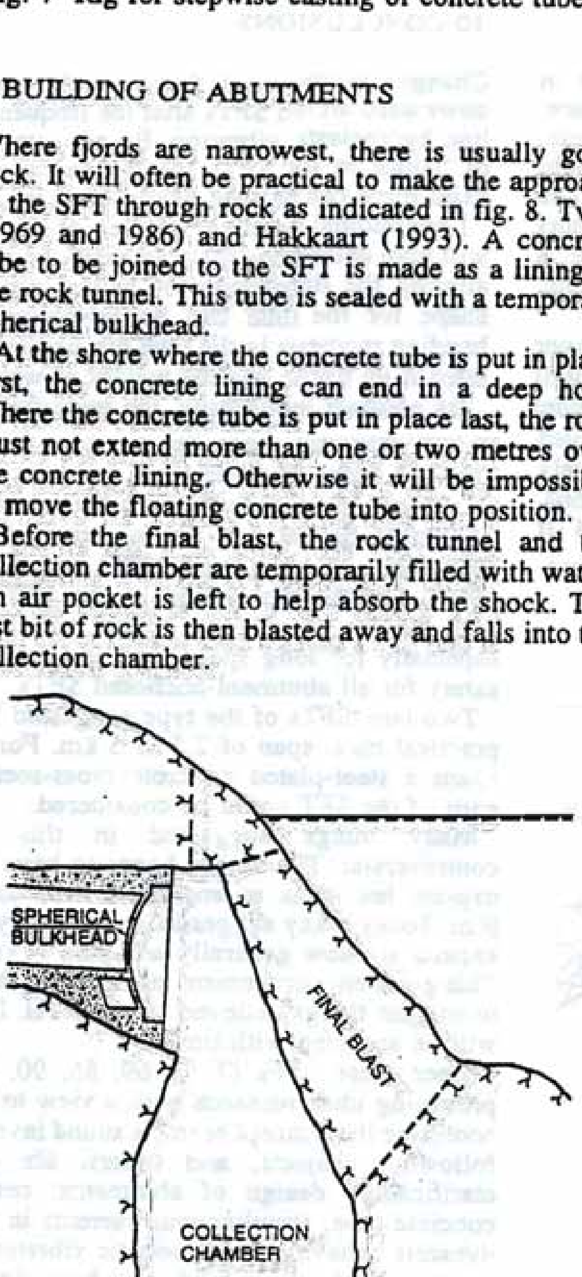

Fig. 8 Abutment for SFT prior to final detonation

8 MOUNTING OF CONCRETE TUBE

The roadway together with its supporting structure is cast before the tube is floated to the site. A Gina rubber gasket is attached to a flange around the circumference of the cross-section. The tube is then shortened ~2 m by concentrating the ballast water in the middle of the concrete tube. A ship keeps one end of the tube at the desired depth. The other end is kept out of the water, partly by a pontoon used when casting the tube. See figs. 7 and 9.

To control horizontal movements during installation, two wires from winches on shore are fastened to each end of the concrete tube. At the ship end of the tube, the Gina on the circumference of the cross-section is brought into contact with the abutment.

Sea water is now let out of the cavity on the inside of the Gina. The water pressure forces the end of the tube against the abutment. A uniform current of 0.1 m/sec will give the tube a tilt of ~0.2°. Since the current usually changes direction between flood and ebb, it should not be difficult to get the tube connected to the abutment in an upright position.

The other end of the tube is then lowered onto supports in front of the other abutment. The tube is then made longer by distributing the ballast water evenly along the tube, and the other abutment comes in contact with the Gina on the tube. When the water is removed from the cavity inside the Gina, the tube can then be held in place firmly.

Prior to casting the concrete of the cross-section, reinforcement is welded together and prestressing tendons are put in. When the prestressing tendons are tensed the tunnel is fully monolithic. At this point the first measurement of the lowest period of oscillation should be done. The standard practice of joining tunnel elements by means of Ginas is explained by Janssen (1978).

A better connection between the rock and the concrete tube can be obtained by means of grouting stones around the concrete tube with a concrete mix. See fig. 10. This connection can be further improved by anchoring the tube to the rock.

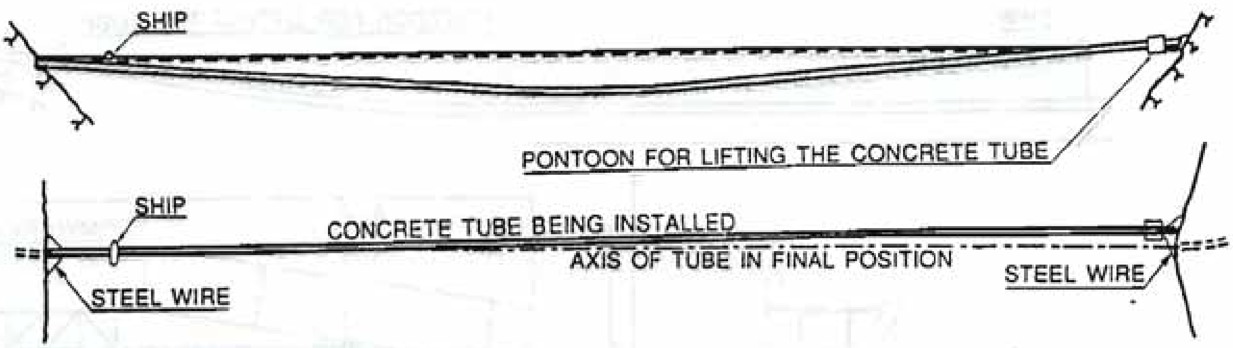

Fig. 9 1750m long SFT being installed. Horizontal movements are obtained by four wires to four winches.

9 COSTS

Most SFTs have round cross-sections and float in near equilibrium with the water which they displace. Thus they need about the same amount of concrete. Roughly the same high-bond minimum reinforcement will have to be used in all concrete SFTs. The mounting of the concrete tube between the abutments is not costly. The type of abutment shown in fig. 8 and 10 is about the cheapest solution possible, because it entails no underwater construction.

It is relevant to ask if other SFTs can be cheaper than the abutment-anchored SFT proposed. This will be the case only if the saving in prestressing steel and concrete quality can pay for the intermediate supports used in other tunnels. This is hardly likely for spans under 2 km. Thus the downward-arched, abutment-anchored SFT should be considered if SFTs are to be built in Norwegian fjords.

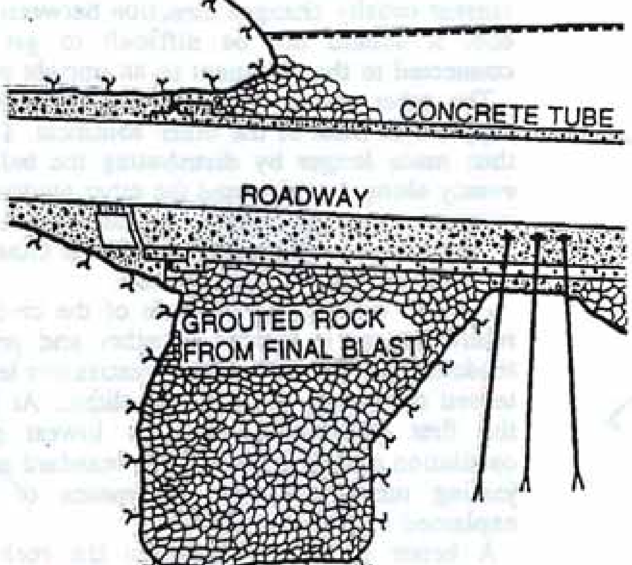

Fig. 10 Finished joint between abutment and SFT

10 CONCLUSIONS

Changes in the axial force in abutment-anchored downward-arched SFTs alter the frequency of the in-line hydroelastic vibration. By adjusting the amount of ballast in the SFTs the frequency and thus the axial force can be kept nearly constant.

Longitudinal bending stresses of any importance will give deflections that can easily be measured. By altering the distribution of ballast water a desired shape for the tube can be maintained. Thus the bending moments in the tube can be kept small when there is no traffic and the weight of water is normal.

By constant monitoring, the undesirable effects of slowly changing loads can be very much reduced. Thus the value of these loads used for analysis can be reduced in accordance with the selected routines.

Movable ballast was not necessary for the SFTs spanning 1.25 km (Tveit 1986) and 2.5 km (Tveit 1990). Careful monitoring of the SFTs, combined with the use of movable ballast will be economic, especially for long spans, and is likely to improve safety for all abutment-anchored SFTs.

Two-lane SFTs of the type suggested here, have a practical max. span of 2.5 to 3 km. For the longest spans a steel-plated concrete cross-section at both ends of the SFT could be considered.

Many things suggested in this paper are controversial. The author hopes to have a chance to explain his ideas to engineers who disagree with him. Today many suggestions previously rejected by experts are now generally accepted. (Tveit 1991). This gives encouragement, and adds to the obligation to suggest things believed to be useful. Perhaps they will be accepted with time.

Since these STFs (Tveit 69, 86, 90, 91) seem a promising idea, research with a view to rejecting or accepting the concept seems a sound investment. The following subjects, and others, are in need of clarification: design of abutments; casting of the concrete tube; simultaneous currents in deep fjords; dynamic behaviour; hydroelastic vibrations and their suppression by fins; loads and how they should be combined.

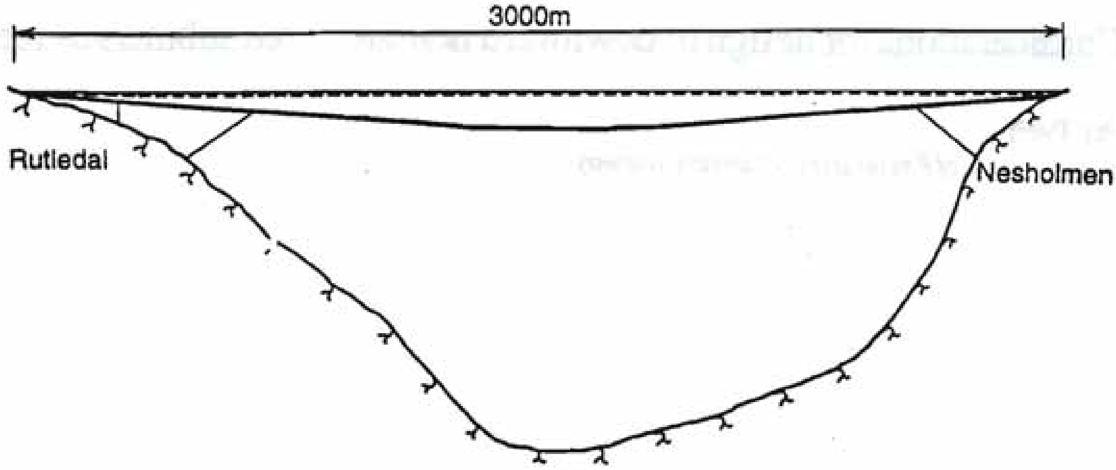

Fig. 11 STF Rutledal-Nesje. Free span 1900m. Stays closest to the shore are in pairs. Max. depth ~1050m.

11 A FURTHER DEVELOPMENT

Abutment-anchored downward-arched SFTs spanning over 2.5 km could be strengthened by inclined stays as in fig. 11. The site is located 30 km from the mouth of the Sognefjord. Rock anchors which are cheap and durable are assumed. The method of construction described in this paper could be used, and the stays added at the end.

The length of the STF and local conditions will decide whether stays should be inclined or vertical. The type of SFT shown in fig. 11 would have to compete with other SFTs using stays. It competes best when conditions for anchors are better close to the shore than in the middle half of the fjord.

If the middle third of the fjord is suitable for putting down anchors, the SFT in horizontal curve using vertical stays (Tveit 69 and 86, Hakkart 93) is likely to be a better solution. This will be the case between Vadheim and Høyanger 37 km further into the Sognefjord. Here the fjord is 1300 m deep.

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Considerations for design of downward non-anchored submerged tubes

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ABSTRACT: Long-span immersed tubular bridges weigh approximately the same as the volume of water they displace. Thus they carry only variable loads. This paper describes a design of a four lane bridge with a free span of 2.5 km. Cross-section depth is 16.5 m. Sufficient depth of water above the tube prevents collision from surface ships, and reduces wave action, variation in temperature and variation in weight of water. To ensure watertightness, prestress suppresses tensile stresses in the serviceability limit state. Redistribution of bending moments due to creep, shrinkage and relaxation do not increase the need for prestress. A new anchoraging and staying system for downward arched tubular bridges is described. Optimal curvature and methods of production are discussed. The downward arched, nonanchored tube for road traffic is competitive for spans over 1 km in deep fiords with steep cliff shores.

1 INTRODUCTION

This paper presents the results of a calculation of a downward arched tube for road traffic spanning 2.5 km between the shores of a deep fiord. The tube carries four lanes of traffic and has no supports between the abutments on the two rocky shores. See fig. 1. and 2. The tube has been designed to show structural possibilities and limitations at fiord sites.

2 VERTICAL CURVATURE AND CROSS-SECTION

The vertical curvature and cross-section chosen for the submerged tube in fig. 1 are probably not optimal, but the author hopes that they are not far off. The rise of the arched tube depends on the gradient of the road inside. Since the lanes in the suggested tube are straight and less likely to be wet or frozen than roads on normal hills, they can be made steeper than usual.

Given a certain gradient, concentrating the curvature at the middle of the tube, see fig. 1, gives more rise to the tube. This is favourable mainly because it leads to a smaller variation in axial force and this reduces the need for prestress. A steep tube also gives smaller bending moments due to temperature variation, shrinkage and axial creep. - If the tube were to carry a big uniform load, then the curvature should be parabolic. - A steep road gives a bigger rise in the tube. Steep roads have reduced traffic capacity. To obtain sufficient traffic capacity, more than two lanes may be necessary.

Since the tube must be floated into place, the area of the cross-section decides the amount of concrete in the tube. A four lane tube has a smaller cross-section than a three lane tube. Thus it is reasonable to assume that a four lane tube will - in most cases - be less costly than a three lane tube. The unidirectional traffic in the four lane tube helps ventilate and this gives smaller ventilation costs. This is because unidirectional traffic helps ventilate. Separate tunnels will also make the abutments of the four lane tunnel costlier. The optimal solution depends on local conditions.

The cross-section shown in fig. 2 is higher than its breadth and features extra thick tube walls at top and bottom. This reduces stresses due to

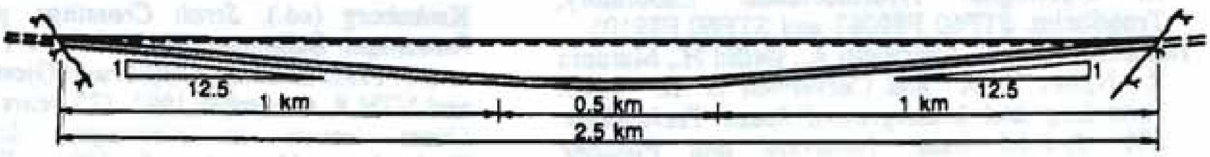

Fig. 1 Cross-section of fjord with tubular bridge spanning 2.5 km.

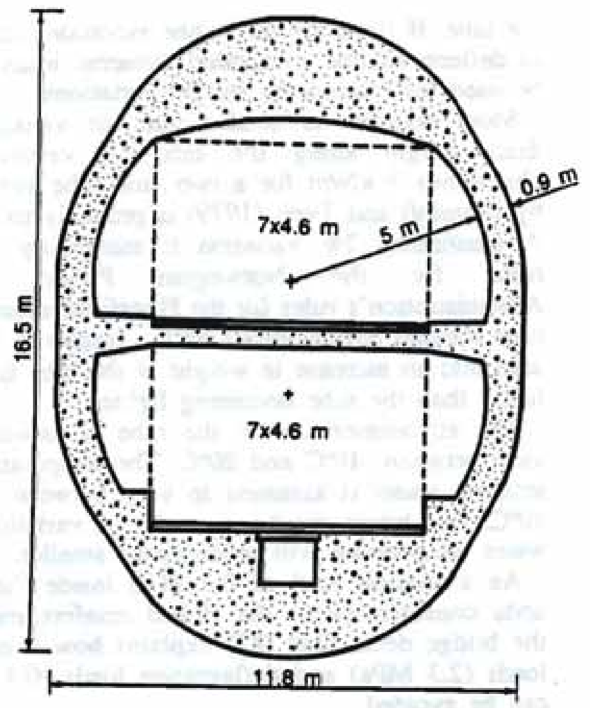

Fig. 2 Cross-section of the bridge in fig. 1

vertical loads in the serviceability limit state. Furthermore, it gives an increased capacity to resist bending moments due to vertical loads in the collapse limit state. Still, the capacity to withstand bending moments due to horizontal loads is ample.

3 LOADS ON SUBMERGED FLOATING TUBES

The loads have a great influence on the length of spans possible. Thus they will be discussed in some detail.

The horizontal loads are mainly due to the design speed of current, wave actions, and forces due to collision by ships. The design speed of current is shown in fig. 3. It is based on Eide (74), but is very much on the safe side, because the strongest currents move in the opposite direction to currents in deeper waters. This applies to fiords. By definition they are open at one end only.

The currents could be stronger in fiords with stronger winds and at sites where the fiord is much narrower than the fiord's average width. As can be seen from table 2, column (8) the static drag force due to currents gives bending moments that are small compared to other bending moments.

At the abutments the calculated tube has only 2 m of water above it. To reduce the risk of collision by ships the tube could be built deeper in the water. The extra depth would make the abutments and erection slightly costlier, but would not influence the cost of the tube.

To avoid collision by submarines, ultrasound should be emitted from the tube, even though this might frighten dogs, seals and whales.

Submerged tubes are best suited to sheltered waters. It is not normally feasible to build submerged tubes that have so many supports that they counteract wave actions. Wave action can be reduced by placing the tube deeper in the water. This gives an advantage to submerged tubes that are placed deep into the fiord.

The vertical loads are: traffic loads, buoyancy, variation in weight of ambient water, weight of marine growth and variation in the weight of the tube. The chosen traffic load is a concentrated load of 1.26 MN placed somewhere in the tube plus a lane load of 9 kN/m. Assumed variation in the weight of ambient water is shown in fig. 4. For the present design, the bending due to variation in ambient water is up to 5% of the total bending. Thus the obvious theoretical imperfections in the diagram in fig. 4. are unimportant. The depth of the less salty layer of surface water is influenced mainly by the volume of the rivers that discharge into the fiord and how quickly the surface layer moves out of the fiord.

With the method of erection suggested later in this paper, the total weight of the tube can be controlled with great precision when the tube is floated into place. In fact this applies to most methods of erection for submerged floating tubes.

The method of construction suggested later in this paper, facilitates good control of the weight distribution along the tube. Variation in the weight of concrete over time after the tube has been put in place is negligible.

Variation in dead load on the concrete tube, such as weight of road surface and marine growth is slow and can be monitored. Increase in dead weight, mainly marine growth, can be counteracted

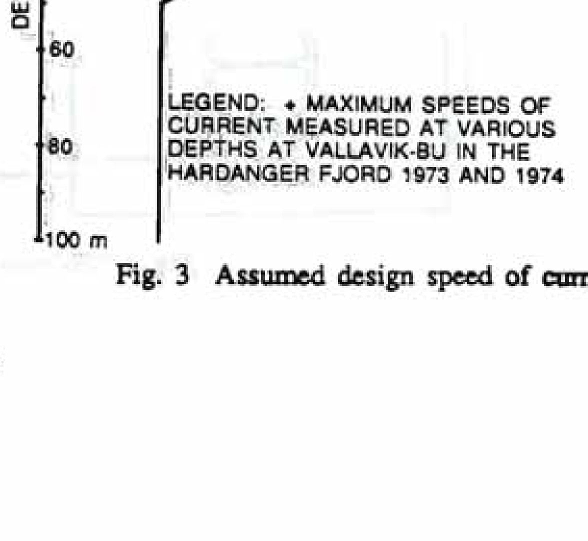

Fig. 3 Assumed design speed of current

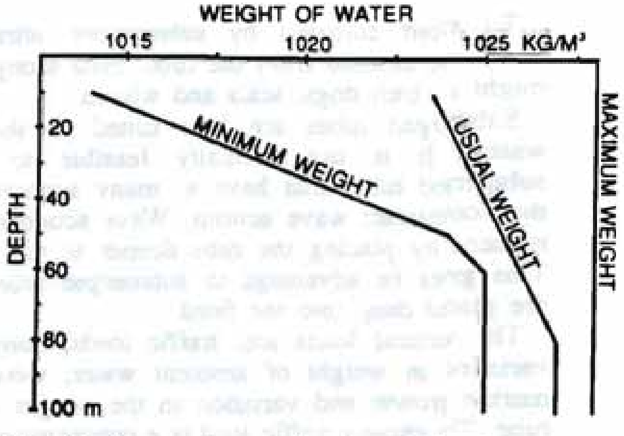

Fig. 4 Assumed variation in weight of water used for design

by grinding down the top of the road surface. Obviously, this change in the design weight should depend on the dead-load monitoring procedures.

For this design it is assumed that the increase in weight due to other dead loads is always larger than the reduction in weight due to road surface wear. The tube is designed for a 2% increace in total weight. This load is equal to 90% of the lane load. The two loads are placed on the tube in the same way.

The 2% increase may be too unfavourable in size and distribution, because it corresponds to a 0.24 m layer of marine growth with a specific density of 13 kN/m³ (3 kN/m³ in water). This will have to be mainly shells, because in water ordinary seaweed weighs practically nothing. Furthermore, the marine growth decreases with depth, but tends to be uniform along the length of the tube. The wear on the road surface also tends to be uniform along the whole length of the tube.

Weight variations can be monitored by inspection and/or determined by calculations based on changes in deflection and vibrations. Changes in the weight of the tube and the resulting changes in axial forces will influence the frequency of vibrations of the tube. If there are no in-line vibrations, changes in deflections due to applied dynamic loads could be used for monitoring weight variations.

More research is needed into the variation of dead weight along the tube. A variation of plus/minus 3 kN/m for a two lane tube suggested by Navntoft and Tveit (1979) is probably too little. A plus/minus 2% variation is mandatory in the rules for the Norwegian Public Roads Administration's rules for the Høgsfjord submerged tube concept competition (1987). Hansvold (1989) says that an increase in weight of the tube is more likely than the tube becoming lighter.

The air temperature in the tube is assumed to vary between -10°C and 20°C. The temperature of ambient water is assumed to vary between 0 and 10°C. For water depths over 50 m variations of water temperature will probably be smaller.

An explosion load of 0.7 MPa inside the tube adds considerably to the ribbed reinforcement in the bridge deck. Oud (87) explains how explosion loads (2.3 MPa) and deflagration loads (0.3 MPa) can be avoided.

4 CASTING OF SUBMERGED TUBES FOR ROAD TRAFFIC

Slipforming gives the best control of concrete quality. Slipforming (the Condeep way) a batch of parallel 100 to 200 m long concrete tubes could give be cost efficient. Afterwards the tubes would be joined together to form a submerged tube of required length.

A concrete tube can also be cast in the ramps that leads down to the finished tube. In this case the tube would be cast in sections that would be pushed into the water step by step.

The author has a preference for the construction method indicated in fig. 5. Here a two lane tube is cast section by section in a form on a pontoon. When one section is prestressed it glides towards

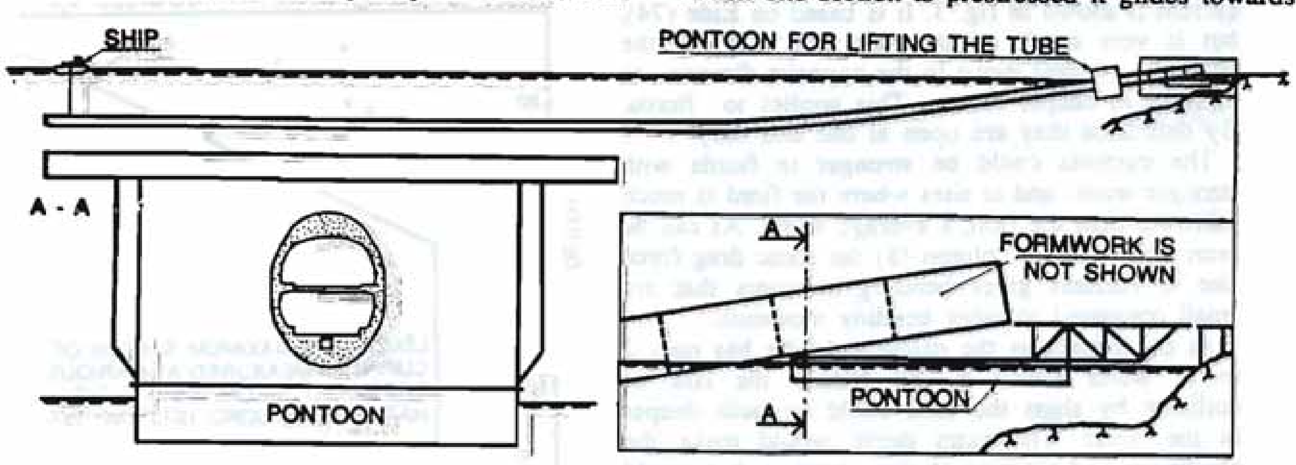

Fig. 5 Pontoon for casting of concrete tube

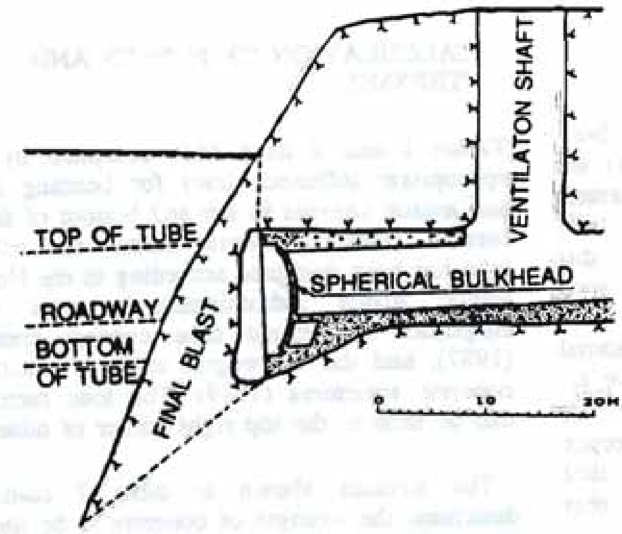

Fig. 6 Abutment for concrete tube prior to final detonation

the water and out of the form. Space does not permit details.

5 ON THE ERECTION OF THE 2.5 KM LONG SUBMERGED TUBE

5.1 Building of the abutments

Fiords gouged out by the ice are usually deep and have rock on both shores. The approaches to a submerged two lane tube could be built through rock as indicated in fig. 6.

When local conditions are favourable, rock from the final detonation would slide out into the fiord. In this way underwater drilling can be avoided. Another way of avoiding underwater drilling is suggested by Tveit (1986).

As can be seen from fig. 6, the rock tunnel is lined with concrete and ends in a spherical concrete shell. Before the final detonation the rock tunnel is temporarily filled with water, but an air pocket is left in front of the concrete shell to help absorb the shock. The shock from the final detonation will be reduced by setting off the charges at short intervals.

5.2 Placing the concrete tube

The finished concrete tube is floated to the site. It would be shortened by bending due to extra ballast water in the middle of the tube. A ship keeps one end of the tube at the desired depth. The other end is kept out of water, partly by a pontoon used when casting the tube. See fig. 7. Two wires from the shore are then fastened to each end of the concrete tube to control horizontal movements. High winds are the main cause of strong currents in fiords. Thus a reliable long-term weather forecast can secure a low speed of current while the submerged tube is put into place.

Both ends of the tube are fitted with rubber gaskets (Ginas) around the outer edge of the cross-section. The ship end of the concrete tube is moved towards the shore to bring the tip of the Gina in contact with the abutment. Sea water is then let out of the cavity on the inside of the Gina. The end of the tube is now squeezed to the abutment by the water pressure. The standard practice of joining tunnel elements by means of Ginas is explained by Janssen (78)

The other end of the tube is then lowered onto supports in front of the other abutment. Then the tube is made longer by distributing the ballast water evenly along the tube. The other end of the tube then come into contact with the Gina, so the water pressure can be removed from the cavity inside the Gina. The tube is then held firmly in place while ribbed reinforcement in the tube and abutment are joined by welding. Concrete can now be cast to make a monolithic connection between the tunnel lining and the concrete tube. Afterwards the prestressing cables between the tunnel lining and the concrete tube can be tightened.The ballast water is then removed as the asphalting of the road proceeds. Tveit et al. (86) tell more about the final stages of positioning the tube.

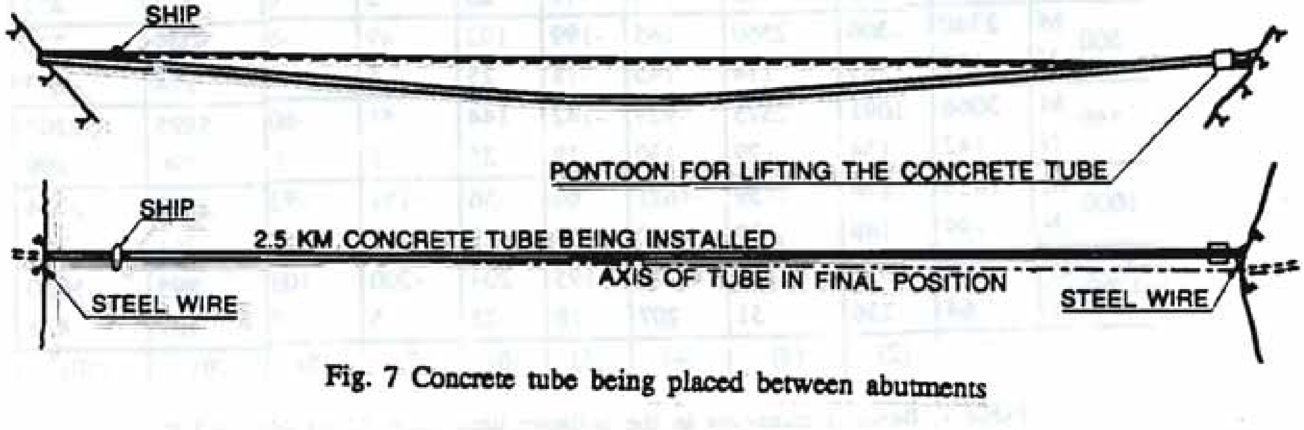

Fig. 7 Concrete tube being placed between abutments

6 VIBRATIONS

In calculations of hydroelastic vibrations a cylindrical tube with a diameter of 15 m has been assumed. Real values for weight and stiffness are used. For the submerged tube the lowest natural period of vertical oscilations is 46 sec. A 0.84 m/sec current would give resonance with this natural period. It can be seen from fig. 3 that such a current will never occur.

The lowest natural period of horizontal oscilations is 193 sec. A 0.1 m/sec current give in-line hydroelastic vibrations with this natural period. The vibrations would be due to symmetric vortex shedding. This gives a maximum acceleration that is less than 1% of the horizontal acceleration that can be tolerated according to Jensen (1986).

A 0.2 m/sec current would give in-line hydroelastic vibrations due to unsymmetric vortex shedding. Amplitudes are supposed to be moderate. More research into speed of current at 50 to 100 m depth in deep fiords and into in-line vibrations at high Reynolds' numbers would be most welcome.

7 CALCULATION OF FORCES AND STRESSES

Tables 1 and 2 have been compiled by loading appropriate influence lines for bending moments and tensile stresses in top and bottom of the cross-section. Unless otherwise stated, this submerged tube has been designed according to the Norwegian Public Roads Administration's rules for the Høgsfjord submerged tube concept competition (1987), and the Norwegian code of practice for concrete structures (1989). The load factors used can be seen in the top right corner of tables 1 and 2.

The stresses shown in table 2 column (7) determine the strength of concrete to be used. It is mainly C55 and C65, but qualities up to C105 is used at both ends of the tube. C55 means a concrete with a characteristic strength of 55 MPa tested on 200 mm concrete cubes.

The tube has no bouyancy when the ambient water has its usual weight. When the ambient water is heavy, compressive forces in the tube give a certain prestress in the tube. In smaller structures

Distance from end of tube in meters		Traffic load		2% increase in weight of tube		Water density		Due to 10° drop in temperature	Current (Moment vector in the plane of the arch)	Collapse limit state (1) or (2) · 1.2+ (3) or (4) · 0.8+ ±(5) · 0.8 ±(6) · 0.8 (8) · 1.0	
		M_{max}	M_{min}	M_{max}	M_{min}	High	Low			M_{max}	M_{min}
0	M	1273	-6278	1099	-5440	535	-660	248	345	3030	-12612
	N	148	148	129	131	-18	25	5	0	295	298
125	M	913	-3312	789	-2857	215	-221	198	182	2057	-6595
	N	149	148	130	130	-18	25	5	0	272	298
250	M	829	-1162	686	-991	-6	37	148	87	1692	-2310
	N	150	143	135	124	-18	25	5	0	312	252
500	M	2340	-306	2560	-265	-199	192	49	-6	5049	-778
	N	132	163	114	145	-18	25	5	0	274	293
750	M	3066	-1091	2575	-929	-142	144	-51	-60	5895	-2207
	N	142	154	129	130	-18	25	5	0	290	299
1000	M	1423	-1876	1159	-1621	66	-56	-151	-92	2808	-3714
	N	149	149	129	130	-18	25	5	0	264	282
1250	M	328	-2551	232	-2119	195	-204	-200	-103	895	-5080
	N	64	236	51	207	-18	25	5	0	107	473
		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)

Table 1. Bending moments in the collapse limit state. Units MN and m.

of this type, the compressive arch force can be used to suppress considerable tensile stresses. Sideways buckling limits the compressive arch force.

To ensure watertightness, prestress should suppress tensile stresses in the serviceability limit state. Relaxation, shrinkage and creep give loss of prestress and a certain redistribution of bending in the tube. The stresses due to redistribution are always smaller than the stresses due to loss of prestress.

This means that the redistribution of stresses influences the size of loss of prestress, but the change in concrete stresses due to relaxation, shrinkage and creep is unidirectional over the entire tube. Redistribution of stresses must be

Distance from end of tube in meters / Maximum tensile stress in upper and lower fibre		Due to traffic load	Due to 2% increase in weight of tube	Due to variation in water density		Maximum tensile stresses due to variation in temperature	Maximum longitudinal stress due to water pressure minus local water pressure	Prestress needed to suppress all tensile stresses $0.8\cdot(1)+0.5(2)+(3)or(4)$ if positive $+(5)+(6)$
				High	Low			
0	σ_o	31.8	27.7	-0.5	3.5	0.1	-0.1	42.8
	σ_u	8.6	7.4	-0.1	-2.7	3.7	-0.2	14.1
125	σ_o	18.2	15.7	-0.4	1.4	1.0	-0.3	24.5
	σ_u	6.9	6.0	-0.2	-0.6	2.8	-0.4	10.9
250	σ_o	8.5	7.3	-0.3	0.2	1.2	-0.5	11.4
	σ_u	6.7	5.7	-0.3	0.6	2.6	-0.6	10.8
500	σ_o	4.7	4.0	0.6	-0.5	1.7	-0.8	7.3
	σ_u	14.4	12.2	-1.2	1.3	2.1	-0.9	19.1
750	σ_o	7.9	6.9	0.4	-0.3	2.1	-1.1	11.2
	σ_u	16.9	13.5	-1.0	1.1	1.7	-1.2	21.9
1000	σ_o	11.6	10	-0.6	0.3	2.6	-1.5	15.7
	σ_u	9.2	7.6	0.0	0.1	1.2	-1.6	10.9
1250	σ_o	16.1	14	-1.2	1.4	2.8	-1.7	22.4
	σ_u	3.0	2.3	0.6	-0.6	1.0	-1.8	3.4
		(1)	(2)	(3)	(4)	(5)	(6)	(7)

Table 2. Stresses in the serviceability limit state. (Units MPa)

considered, but it does not influence the need for prestress. That is decided by the load cases at $t=\infty$. Thus column (7) in table 2 shows the need for prestress at $t=\infty$.

In the serviceability limit state, creep in the tube is largest on the inside of the tube. This leads to tensile stresses inside the tub: and compressive stresses on the outside of the tube. This effect has not been considered, but it would lead to a reduced need for prestress.

There is one overriding shortcoming in these calculations: The tube has not been calculated in the deflected state. - The tensile force in the tube is for many decisive load cases nearly half as large as the shorttime buckling load for buckling in the vertical plane. Exact calculations would give very considerable reductions in the bending moments, and thus in the need for prestress and concrete quality. These changes would, however, not have a great influence on the costs presented in the next section.

8 COSTS

The main purpose of this paper is to point out that a submerged concrete tube with a free span of 2.5 km can be built. Since this is the case, the cost of such a structure must be of interest. The author would like others to calculate the cost of this structure. Fearing that nobody will volunteer, he has used some worked examples that have come his way, to come up with the following estimate in thousands of Norwegian kroner (NOK). Values are for 1989. Seven NOK is equal to 1 US$.

Mobilization and rigging:			80 000
Tube:			
Formwork 232 500 m²x0.3	=	69 750	
Construct. joints 10 500 m²x.5	=	5 250	
Concrete 147 500 m³x1	=	145 000	
Reinforcement 38 500 tx7	=	269 500	
Prestressing 16 500 tx20	=	308 000	
Asphalt paving 7 000 m²x1.5	=	10 500	
Rigging	=	10 000	
+15%	=	123 075	
			943 575
Abutments:			150 000
			1173 575
Unforseen +5%:			58 679
Costs in thousands of NOK			1232 254

These costs do not include financing costs, technical installations, tunnels to abutments etc. The cost is equal to 17600 US$ per m lane.

The author is very keen to cooperate with anybody who would like to make a better estimate.

9 ECONOMY

The author is convinced that in deep fiords with good rock on both shores the downward arched submerged tube, Tveit (1969 and 1986), is the cheapest solution possible for spans between 1 and 1.4 km and cost competitive for larger spans.

The abutment shown in fig. 7 and the one described in Tveit (1986) may be the cheapest solution when local conditions are favourable. The amount of concrete and ballast for the tube are decided by the weight of the displaced volume of water. Thus the price of the tube will be about the same for all floating submerged tubes.

For two lane tubes, Tveit (1986), the high bound reinforcement is a minimum reinforcement. Thus it will be about the same for all floating submerged tubes. For the submerged, downward arched tube spanning 1 to 1.4 km the amount of prestressing steel required depends, among other things, on the grade of the road inside it and the assumed variation of tube weight. The amount of prestressing steel needed is probably between 500 and 5000 metric tons.

Some of this prestressing steel can be saved by introducing stays anchors at the bottom of the fiord. Anchors and stays are costly, and can hardly be compensated for by savings in prestressing steel costing 20000 NOK per ton.

If savings in prestressing steel can not compensate for anchors and stays, the downward arched tube must be less costly than other submerged tubes. This is especially obvious for the shorter spans. The author wonders why nobody else does research on this type of submerged tubes.

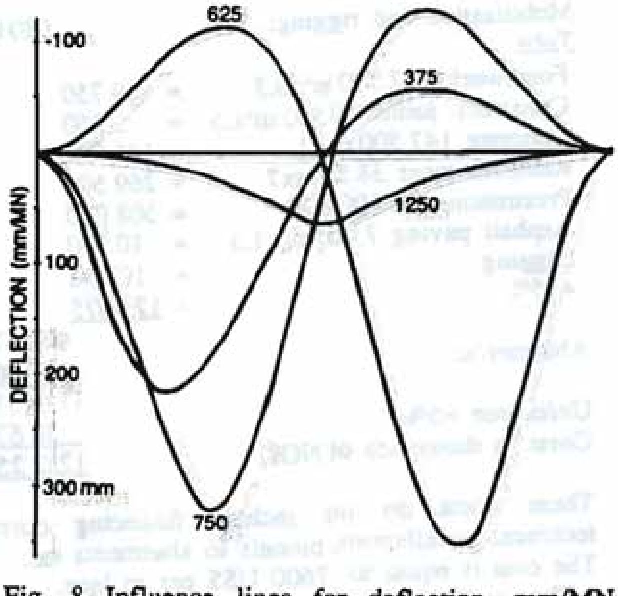

Fig. 8 Influence lines for deflection. mm/MN. Distances from the end of tube is written for each influence line.

10 DEFLECTIONS

The traditional limits on deflections in bridges have been imposed mainly to avoid the unpleasant and harmful effects of vibrations. Influence lines for deflections, see fig. 8, are presented mainly to give an idea of acceleration due to changes in loading. From these influence lines it can also be seen that a load on one half of the tube makes the other half of the tube move upwards. This is important for the design of self-adjusting stays suggested in the next section.

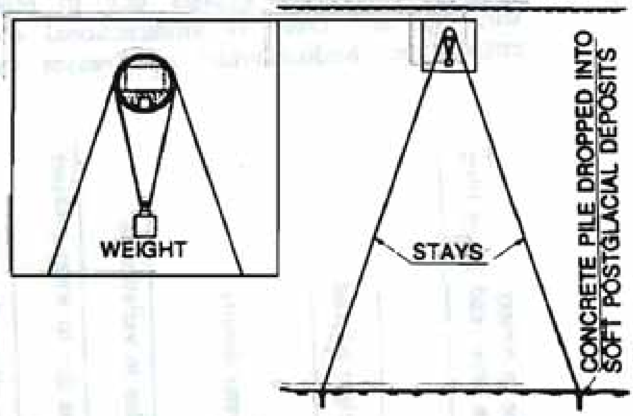

Fig. 9 A new anchoring and staying system

11 A NEW ANCHORING AND STAYING SYSTEM

Deep fiords gouged out by glacial ice often contain soft postglacial deposits. Concrete piles can be dropped from the surface to penetrate these deposits. The piles may have fins at their upper end to make them go down straight and to help take up the horizontal components of the tensile forces in the stays.

The upper end of the stays should be swung over the concrete tube as shown in fig. 9. The stays are kept in tension by the weight of a concrete bucket fastened to a pulley that sits on the stay under the tube. The arrangement would make the stay self-adjusting.

The weight of the bucket can be adjusted by filling it with sand. The concrete tube should be made lighter where the stays are fastened to compensate for the weight of the concrete bucket under the tube as well as for the tension in the stays.

Since the friction between the concrete tube and the stays is uncertain, it is important that the weight of the concrete bucket be adjustable, at least in initial applications of this design.

The self-adjusting stays would restrain the vertical movements of the tube. When one half of the tube is loaded it moves down and the stays in that area acquire minimum tension. Simultaneously the other half of the tube moves up and in this part of the tube the stays develop maximum tension. This reduces the bending moments in the tube considerably. When there is an evenly distributed load over the entire tube, all stays acquires minimum tension. Thus this load case has not been worsened by the use of self-adjusting stays.

The structure is a nonlinear system due to flexibility, considerable deflections and axial forces. It is also discontinuous. Stresses depend on load history because the top of the stays slide over the tube. The design entails complicated calculations.

The author is uncertain as to how much of the total load the self-adjusting stays should support and how much of the total load the tube should be able to carry without the help of these stays.

Friction, present when there is movement between the tube and the stays, would contribute to a reduction in amplitudes of hydroelastic vibrations. In order to resist sideways buckling better, the pulley carrying the concrete bucket could be prevented from moving. If the stays are not meant to restrain hydroelastic vibration or sideways buckling, they should be vertical.

A few of the stays, those nearest the abutments, can be fastened to the tube after they have found their adjusted length, but most of them should be able to glide over the tube to absorb movement due to changes in temperature.

If some concrete piles are taken by an underwater landslide, the stays would most likely pull them out of the surrounding deposits. Since the piles would be moved downwards, their weight would still give tension in the stays. Careful design could probably prevent damage to the tube due to this accidental situation.

It is obvious that these stays would be fairly simple to install and not very costly. An additional cost would be due to wear on that part of the stay that moves to and fro on the tube. It is too early to say how replacement of that part of the stay would influence costs. The author believes the structure to be cost-competitive for two lane tubes spanning 1.5 to 3 km.

12 ACKNOWLEDGEMENTS

The author would like to express his thanks and appreciation to the following institution and engineers: To Norges teknisk-naturvitenskape-lige forskningsråd for a grant to study optimal vertical curvature of downward arched tubes. To professor Geir Moe for interesting discussions about in-line vibrations. To Tomas Einstabland and Torbjørn Kjoberg for informative discussions.

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DOWNWARD ARCHED IMMERSED TUBES FOR ROAD TRAFFIC SPANNING BETWEEN SHORES OF DEEP FIORDS.

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1. Introduction

What is new in this paper is the result of a calculation of a downward arched tube for road traffic spanning 1,250 m between shores of a deep fiord. See fig. 7 p.6. The paper will also tell about a horizontally arched tube /3/ /5/ /6/ /7/ /8/ stayed by nearly vertical cables. A description of this type of tunnel, see fig. 1, has not previously appeared in English. Both tunnels are without expansion joints and were first described in a lecture in 1969 /5/.

2. Forces on immersed floating tubes

The horizontal forces are mainly drag forces due to the speed of current and forces due to colliding ships. In Norwegian fiords these forces can be very much reduced by placing the tubes more than 10 metres below the water level.

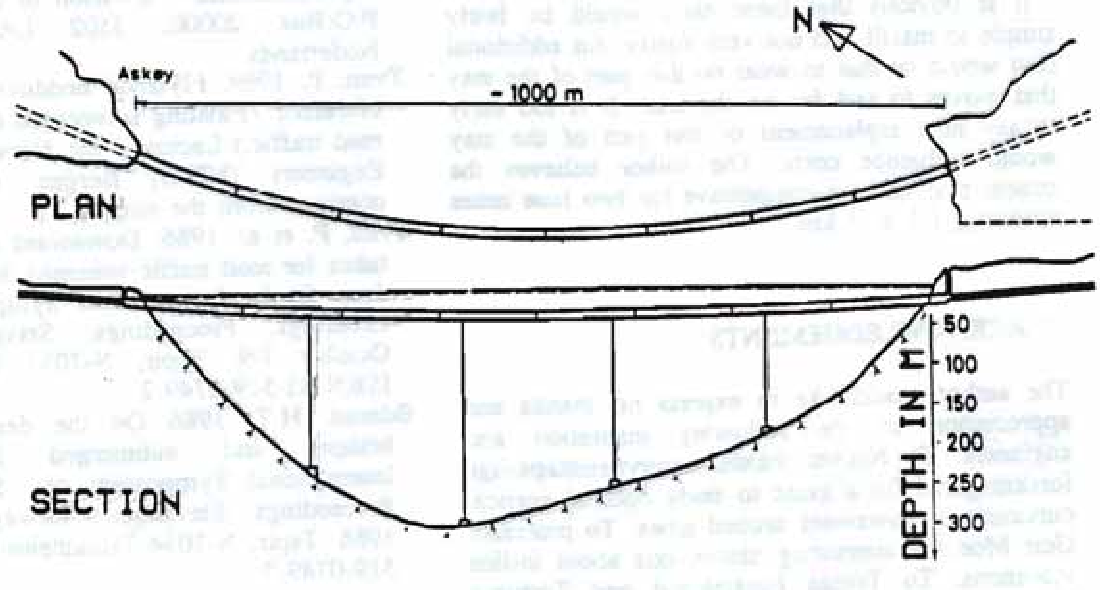

Fig. 1. Immersed tube for road traffic to Askøy by Bergen suggested in 1969 /5/

*AID = Agder Regional College of Engineering.

The vertical loads are: Traffic load, buoyancy, weight of marine growth, variation in the weight of the tube and of the ambient water.

If a natural frequency of a tube is near the frequency of the eddy shedding, selfexcited hydroelastic vibrations occur /10/. In /7/ it is shown that selfexcited hydroelastic vibrations will not occur even in the erection phase of the immersed tube in fig. 1

3. The horizontally arched tube

The statics of the horizontally arched tube is best explained in /3/. Here is only room for a very general description. The forces due to speed of current are mainly carried by axial forces in the arched tube. Bending moments due to creep and variation in temperature are very small. For sites which requires spans under 3 km strong current forces can be taken by the horizontally arched tube. Where big waves, for instance due to landslides, can be expected, the horizontally arched tube seems most suitable.

The authors assume that the vertical stays and their lower anchors are so costly that it does not pay to reduce the weight of the tube to save materials. Instead the tube is given a buoyancy that will give tension in the stays for all normal loads. Thus the bending moments due to vertical loads can be calculated like in a beam on elastic foundations. Long distances between the points of support increase the bending moments. These bending moments can be reduced by concentrating some of the buoyancy near the top of the vertical stays. Ways of fastening the vertical stays to the tube and adjusting their length are best described in /7/ p. 8 to 10.

4. Choice of cross-section and materials in the concrete tube

In the immersed tubes for road traffic hitherto examined by the authors, the strength of a tube of watertight concrete has been ample for carrying the occuring forces. The need for sufficiently small buoyancy decides the weight of the tube. The authors prefer a round cross-section with concrete walls of even thickness. This gives small internal stresses due to temperature gradients during hardening. Less concrete, but more transverse reinforcement, would be needed for a rectangular cross-section.

Longitudinal ventilation with fans above the lane allows using the space below the lane for service duct and ballast. The service duct will be used for water ballast when the concrete tube is floated into place. The ballast can either be stones and very coarse gravel or plain concrete. Two cross-sections are shown in fig. 2. The concrete is a costlier ballast material, but it has a higher specific gravity. Thus the cross-section with the concrete ballast has a thinner outside wall and needs less prestressing steel. Furthermore, stone and coarse gravel is cheaper than concrete ballast, but it needs prefabricated elements for the service duct. If the concrete ballast could be pumped, it would be easy to place.

The asphalt layer between the outer wall and the concrete ballast prevents the longitudinal prestressing of the ballast. The split in the middle of the concrete ballast has been made to prevent transverse bending stresses in the tube due to increased water pressure after the ballast has been cast.

The optimal deviation from a completely circular form has not yet been examined for either of the two cross-sections in fig. 2. The authors are not able to say which of the two cross-sections are best.

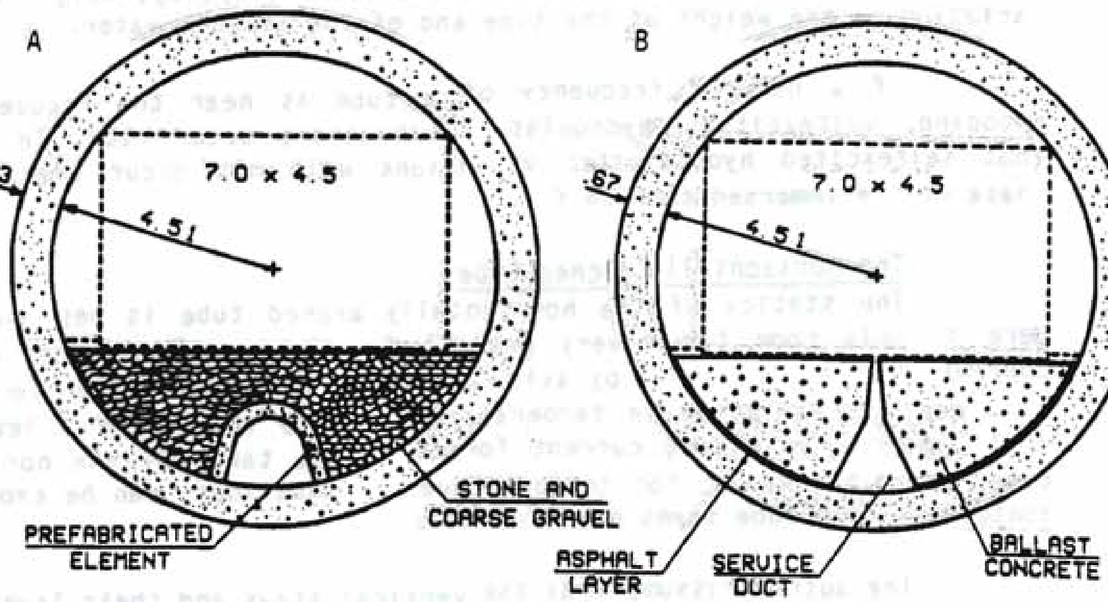

Fig. 2. Two cross-sections of immersed tubes for road traffic

5. Casting of concrete tubes for immersed tunnels for road traffic

Often a concrete beam bridge is cast section after section and then pushed on to the piers. Similarly great lengths of concrete tube could be cast section by section from a floating drydock. By this method the ballast could be put in place shortly after each section was cast. A good continuous control of the tube's buoyancy would be almost automatic.

Slipforming the tube while it is standing vertically in the water give better control of the casting /5/. By this method lengths would be limited to around hundred metres before they were turned over to horizontal position and joined together.

Slipforming would give the best control of the placing of the concrete in the form, but it is the authors opinion that sufficient concrete quality can be produced by casting in a floating drydock. The quality of concrete will be as good as the quality obtained by the method of production used for immersed tunnels placed in river beds /1/, but the floating tube has the advantage that a local leak can be stopped by gluing a membrane to the outside of the concrete wall.

6. On the erection of immersed tubes in Norwegian fiords

6.1 Building of the abutments

The Norwegian fiords have been excavated by the ice. Thus they are deep, and more often than not there will be rock at both sides of the fiord. The approaches to the submerged tubes through rock could be built as indicated by fig. 3. It is an adaptation of the method used for connecting a water reservoir to the tunnel leading the water to a hydroelectric power plant /5/. Near the surface of the rock a pit is made for collecting the stones from the final blast. In this way underwater rockblasting can be avoided.

As can be seen from fig. 3, the rock tunnel is lined with concrete and shut by a spherical concrete shell. Watertightness of the rock can be improved by injecting a cement mix. A circular Gina seal is put up to receive the concrete tube to be inserted.

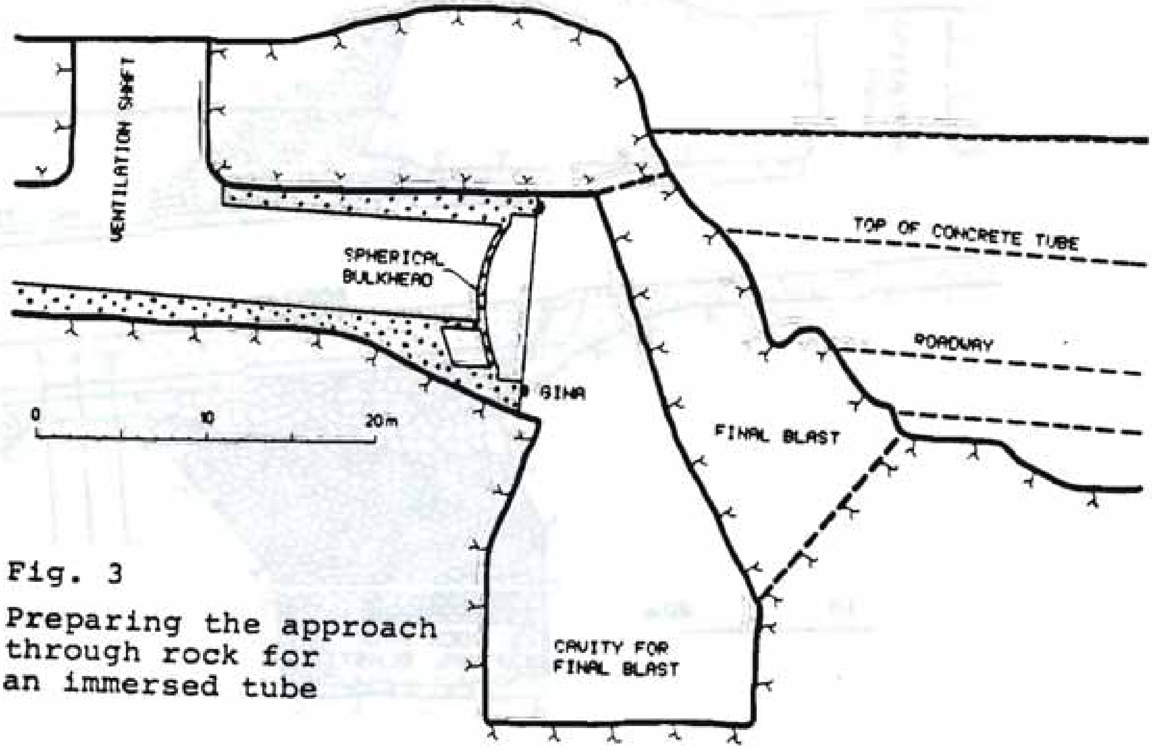

Fig. 3
Preparing the approach through rock for an immersed tube

Before the final blast the rock tunnel is temporarily filled with water, but an air pocket is left in front of the concrete shell to help absorb the shock. The shock from the final blast will be reduced by setting off the charges with small intervals.

The concrete tube can now be moved up to contact with the Gina. See fig. 4. Water is then let out of the cavity on the inside of the Gina, and the outside

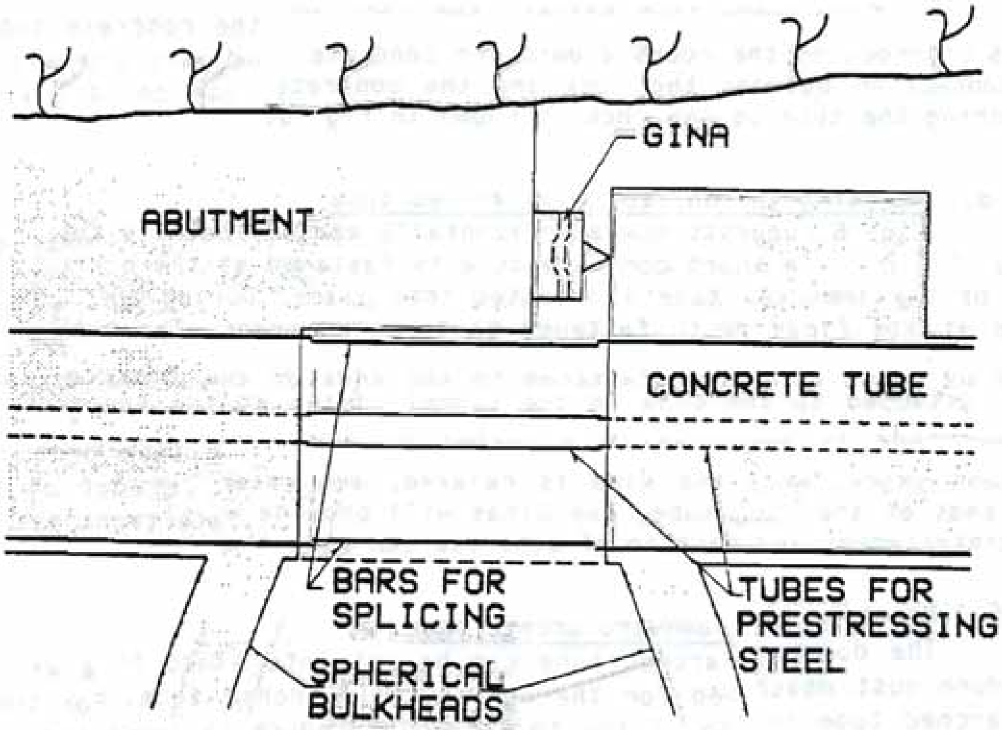

Fig. 4. Contact between abutment and immersed tube prior to casting of the joint between them

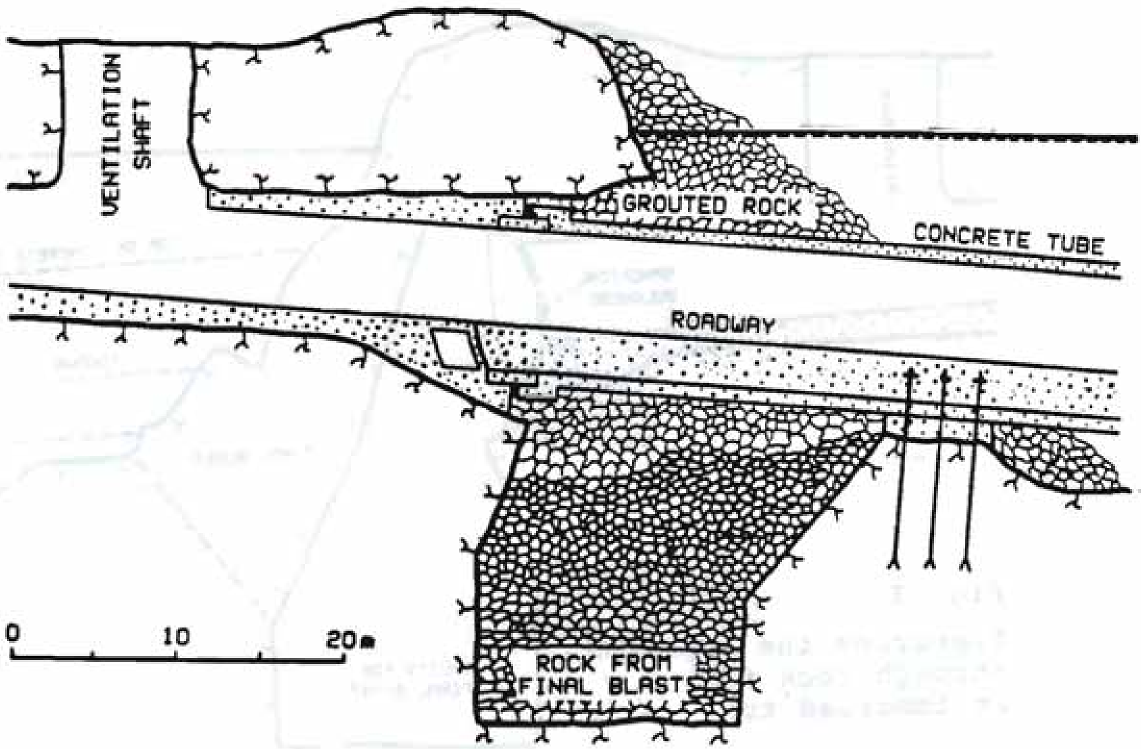

Fig. 5. Finished abutment with immersed tube

pressure will keep the concrete tube in place. The compressed Gina ensures watertightness for welding the reinforcement. See fig. 4. Concrete can now be cast to make a monolitic connection between the tunnel lining and the concrete tube. The technique of joining tunnel elements by means of Ginas is standard practice for immersed tunnels /1/.

A firm connection between the rock and the concrete tube is ensured by means of grouting the rocks around the concrete tube with a concrete mix. See fig. 5. Connection between the rock and the concrete tube could also be achieved by anchoring the tube to the rock as shown in fig. 5.

6.2 Mounting the horizontally arched tube

Fig. 6 suggests how a horizontally arched concrete tube could be put into place /7/. First a short concrete tube is fastened at the northern shore. Then the rest of the immersed tube is floated into place. During the transport it is floating at its final depth fastened to three pontoons. The tube is bent and shortened by means of a wire fastened to the ends of the floating tube. The tube is first attached to the Gina on the tunnel lining at the southern shore. Then the northern end is swung up to a corbel on the short tube protruding from the northern shore. When the wire is relaxed, and water let out of the cavities at both ends of the long tube, the Ginas will provide watertightness so that welding of reinforcement and casting of concrete can make the tube monolitic.

6.3 Mounting the downward arched tube

The downward arched tube can be put into place in a way similar to the procedure just described for the horizontally arched tube. For the 1,250 m downward arched tube in fig. 7 the following procedure is suggested: A concrete tube of 125 m is fastened to one of the shores by Gina, welding, casting and grouting. Then extra ballast water can be applied to the tip of the tube to make the shorter concrete tube deflect to make room for the longer concrete tube.

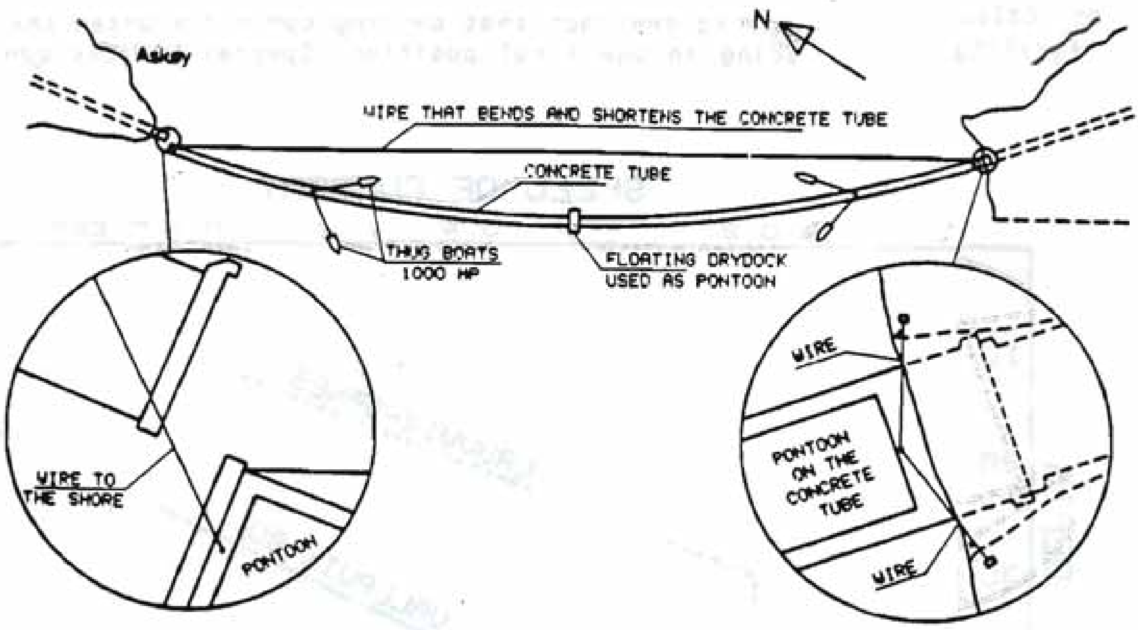

Fig. 6. Stage in putting a horizontally curved immersed tube into a rock tunnel. At the north end a cable for pulling the tube into place is tight

Now the longer concrete tube is floated into place suspended from a pontoon or a ship at each end. The tube is bent and shortened by putting the ballast water in the middle third of the tube. The deeper end of the longer tube floats above the lower end of the tube already mounted. Thus the two ends can not do damage to each other. Two wires are now fastened to each end of the longer tube to control the horizontal movements. Then the tube is pulled towards the Gina on the lining of the rock tunnel. After the water pressure has been removed from the cavity between the abutment and the end of the longer tube, the longer tube is straightened a little by moving the ballast water towards the deeper end of longer tube. At the same time some of the ballast water is removed.

After this the deeper end of the longer tube is lowered gently to a corbel on the lower edge of the already mounted shorter tube. Both tubes are now straightened by removing ballast water, and strong pressure on the Gina is established by taking the water pressure off the cavity between them.

Strong currents in Norwegian fiords are caused mainly by high winds. Thus a reliable longtime weather forcast can secure a low speed of the current while the immersed tube is being put in place.

The downward arched tube is best put in place in the early spring when the coldest sea water gives the tube a minimum length. For downward arched tubes spanning less than 900 m, the operation of putting the tube into place may be more

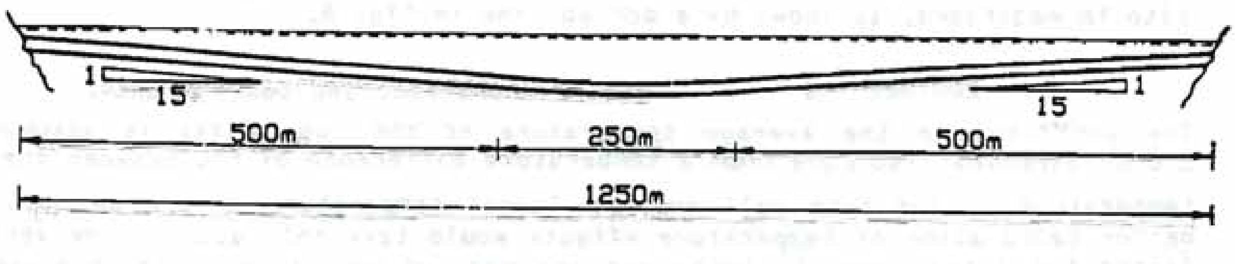

Fig. 7. Downward arched immersed tube for Vallavik - Bu

complicated. This is due to the fact that bending can not shorten the tube enough to facilitate its placing in the final position. Special devices can be used to solve this problem.

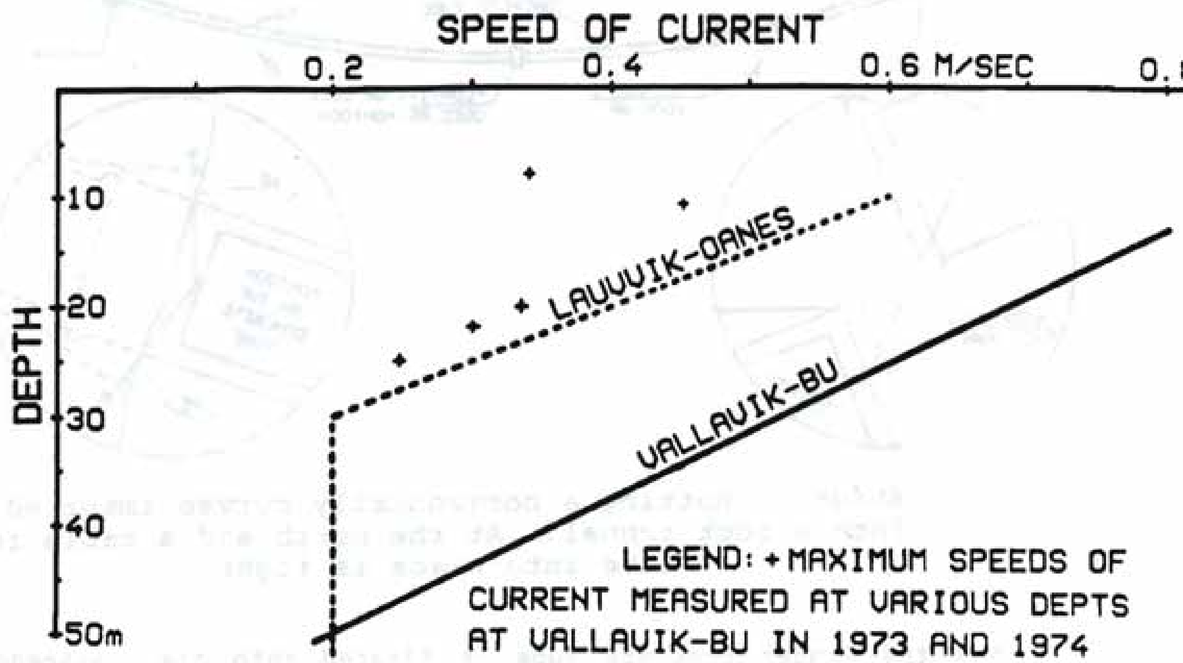

Fig. 8. Variation in speed of current with water depth for Vallavik - Bu and Lauvvik - Oanes

7. <u>Downward arched immersed tube spanning 1,250 m</u>

7.1 <u>Design and strength</u>

In the spring of this year the authors designed a downward arched immersed tube spanning 1,250 m for the site Vallavik-Bu in the Hardangerfjord. See fig. 7. The cross-section of the tube is shown in fig. 2A. Due to previous designs ofimmersed tubes for this site /9/ /3/ /6/ design data /2/ and other interesting material /10/ were available.

Based on /2/ the maximum currents have been assumed to be as shown by a full line in fig. 8. Ref. /4/ gives the maximum current for the bridge site at Høgsfjord near Stavanger, as shown by a dotted line. The drag force is proportional to the square of the speed of the current. As can be seen from table 2, the reduction of speed due to water depth is not very important because the bending moments due to current are small compared to other moments.

Based on /2/ the max. min. and usual weight of water has been assumed to be as shown in fig. 9. Ref. /4/ gives max. and min. water weight for the bridge site in Høgsfjord, as shown by a dotted line in fig. 9.

The domineering load is the road traffic. See table 1 and 2. p. 8 and 9. The variation in the average temperature of the tube walls is assumed to be ± 6°C. Stresses also come from a temperature difference of 5°C between the average temperature of the tube wall and the lowest temperature on the wall surface. A better calculation of temperature effects would take into account the ventilation needed for future traffic loads and the reduced variation in temperature in the walls below the roadway.

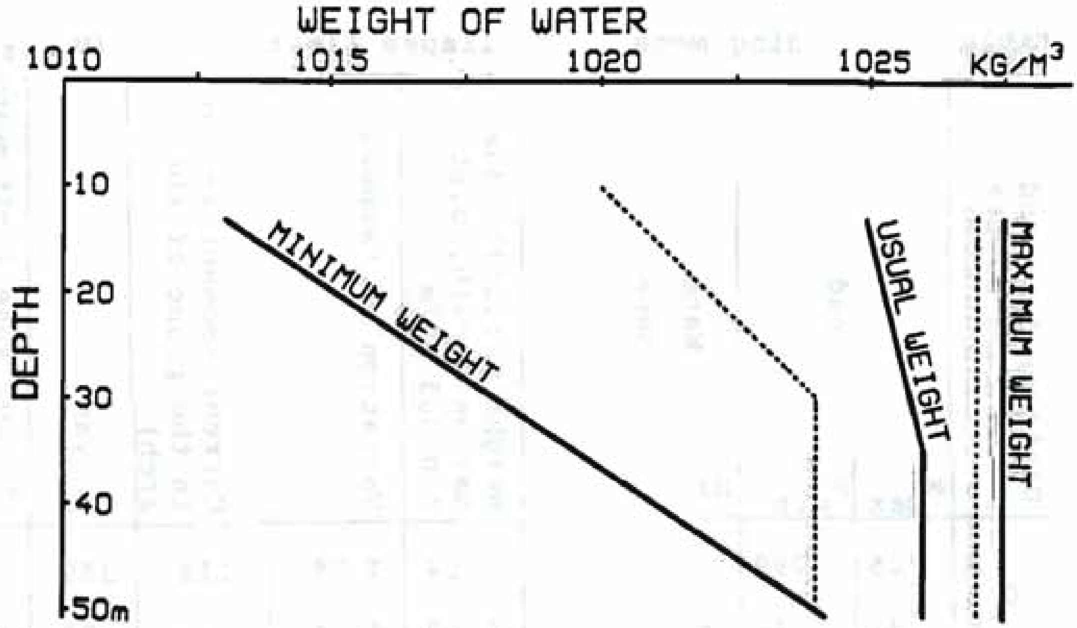

Fig. 9. Variation in weight of water with water depth for Vallavik - Bu and Lauvvik - Oanes

Table 1. Stresses in the servicability limit state (Units MN/m^2)

Distance from end of tube in meters	Maximum tensile stress in upper and lower fibre	Due to traffic load	Due to variation in water density High	Low	Due to weight variation due to marine growth, dirt etc. ±0.003 MN/m	Due to variation in temperature ±6⁰	Due to buoyancy	Maximum tensions due to (1) to (6)	Stress due to water pressure	Prestress needed to suppress all tensile stresses*
0	σ_u	15.6	-1.4	4.6	∓2.5	±1.3	-7.5	9.3	-0.3	9.8
	σ_1	3.4	0.9	-3.3	±1.4	∓1.7	+4.8	9.5	-0.5	9.7
125	σ_u	5.4	-0.3	0.9	±0.8	±0.7	-2.5	3.0	-0.5	3.2
	σ_1	2.9	-0.2	0.5	∓0.3	∓1.1	-0.3	2.9	-0.9	2.8
250	σ_u	2.2	0.1	-0.7	±0.1	±0.1	1.4	3.3	-0.8	3.2
	σ_1	7.1	-0.6	2.1	∓1.2	∓0.5	-4.2	3.3	-1.2	2.9
375	σ_u	3.9	0.1	-0.5	±0.2	∓0.5	+1.0	4.5	-1.1	4.2
	σ_1	8.3	-0.6	1.9	∓1.2	±0.2	-3.9	4.4	-1.5	3.7
500	σ_u	5.2	-0.3	0.9	∓0.5	∓1.1	-1.7	3.8	-1.4	3.2
	σ_1	5.2	-0.2	0.4	∓0.5	±0.8	-1.2	3.8	-1.8	2.8
625	σ_u	6.9	-0.6	1.9	∓1.1	∓1.4	-4.1	3.6	-1.5	2.9
	σ_1	2.1	0.1	-0.6	±0.0	±1.1	-1.2	3.5	-1.9	2.4
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)

* 0.8 MPa has been added on account of temperature difference in the tube wall.

Table 2. Bending moments, collapse limit state. (Units MN and m)

Distance from end of tube in meters		Traffic load M_{max}	Traffic load M_{min}	Water density High	Water density Low	Weight variation due to marine growth, dirt etc. ± 0.003 MN/m	Variation in temperature ± 6°	Current (Moment vector in the plane of the arch)	Buoyancy	Collapse limit state (1 or 2)·1.2+ (3 or 4)·0.8+ ±(5)·0.8 ±(6)·0.8 (8)·1.0 M_{max}	M_{min}
0	M	105	-799	70	-231	∓114	∓ 88	119	359	631	-875
	N	37	45	- 6	17	± 13	∓ 4	-	- 37	- 9	41
125	M	73	-162	4	- 11	∓ 15	∓ 52	38	64	167	-152
	N	31	51	- 6	17	± 13	∓ 4	-	- 37	- 17	48
250	M	280	- 51	-22	82	± 37	∓ 16	- 7	-164	267	-273
	N	33	49	- 6	17	± 13	∓ 4	-	- 37	27	6
375	M	383	-124	-21	72	± 42	± 19	-28	-145	405	-344
	N	41	41	- 6	17	± 13	∓ 4	-	- 37	36	- 39
500	M	200	-200	2	- 15	∓ 1	± 54	35	14	256	-238
	N	41	41	- 6	17	± 13	∓ 4	-	- 37	- 4	37
625	M	57	-256	19	- 73	∓ 33	± 72	37	157	268	-235
	N	25	58	- 6	17	± 14	∓ 4	-	- 38	-24	57
		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)

The buoyancy of the tube for normal weight of water is shown in fig. 10. The buoyancy has been chosen to make the maximum tensile stresses in the upper and lower fibre more even in the serviceability limit state. See table 1. The maximum stresses due to traffic loads have been found by influence lines for tensile stresses in the upper and lower fibres of the tube. See fig. 11 and 12 on the next pages. The various loads are combined according to the rules in the Norwegian code for load on road bridges and ferry quais. These rules are very much the same as the general Norwegian code for loads.

Prestress has been choosen to prevent tensile stresses in the concrete in the serviceability limit state. The need for prestress can be halved by increasing

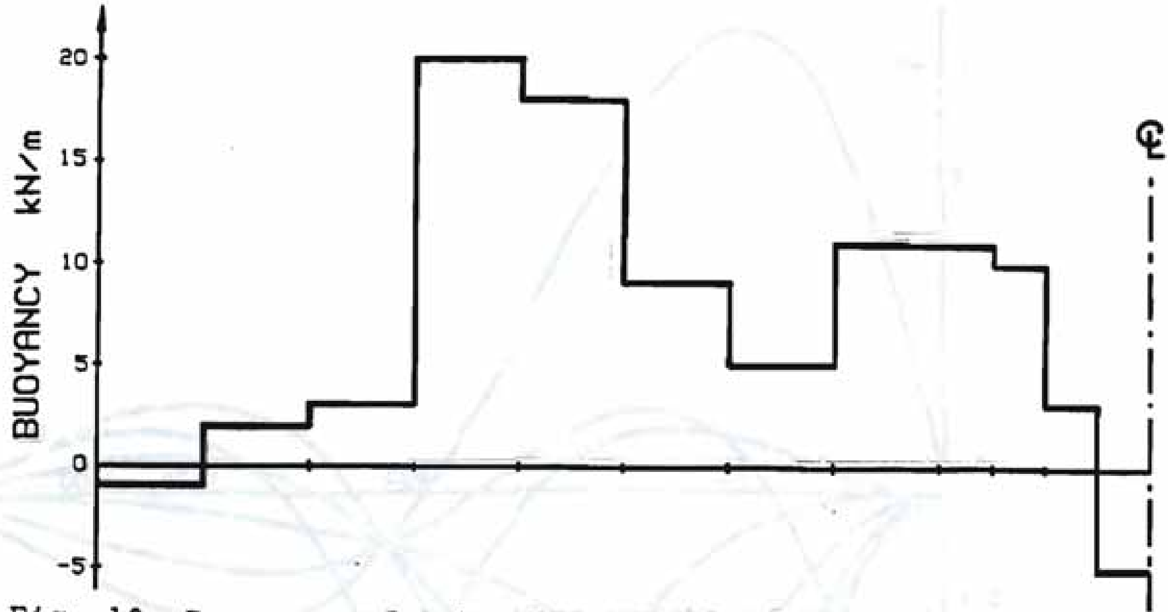

Fig. 10. Buoyancy of tube when weight of water is as assumed usual.

the buoyancy on the curved part of the tube and introducing compensating length and bending moments so that the resulting longitudinal bending moments in the tube equal zero. This would make it more difficult to bend the tube sufficiently during mounting. Calculations of the effects of shrinkage and creep would give small extra stresses in comparison to the stresses shown in table 1.

The maximum forces in the ultimate limit state are given in table 2. The influence lines used for finding bending moments due to traffic loads are shown in fig. 13.

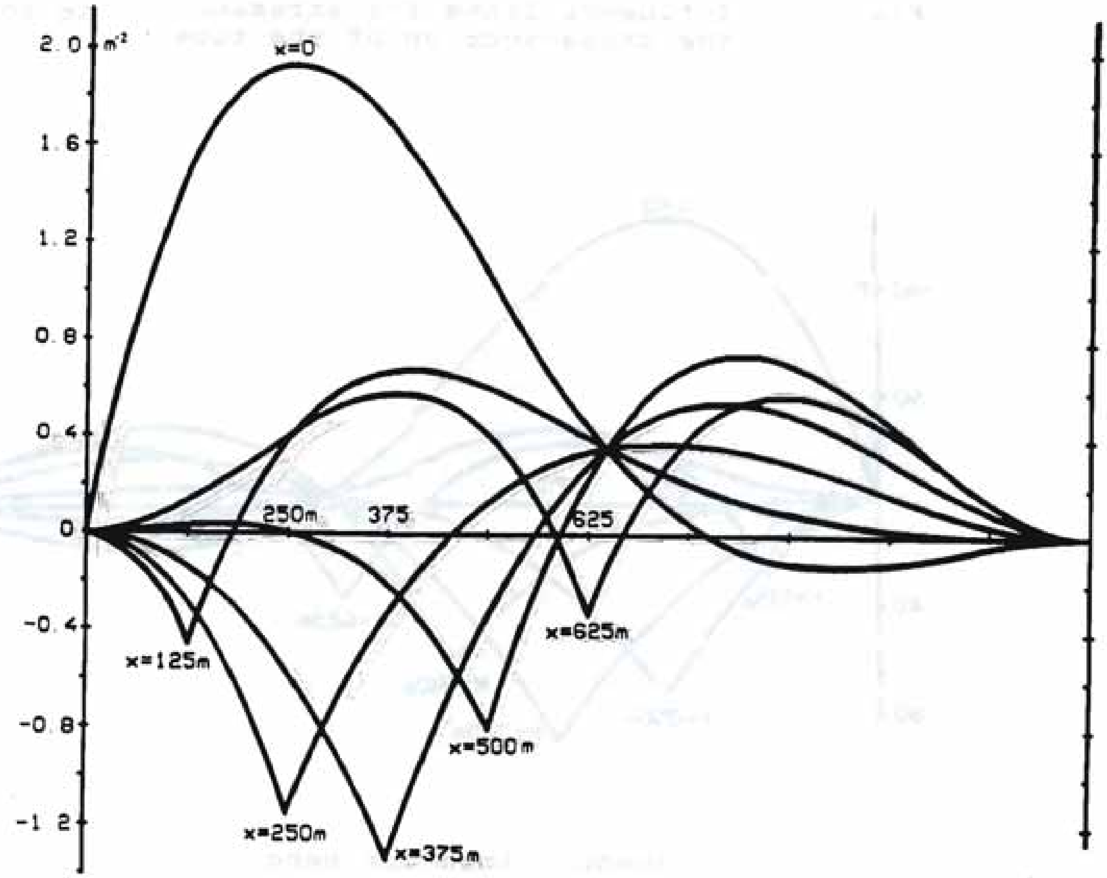

Fig. 11. Influence lines for stresses in the topmost fibre of the cross-section of the tube

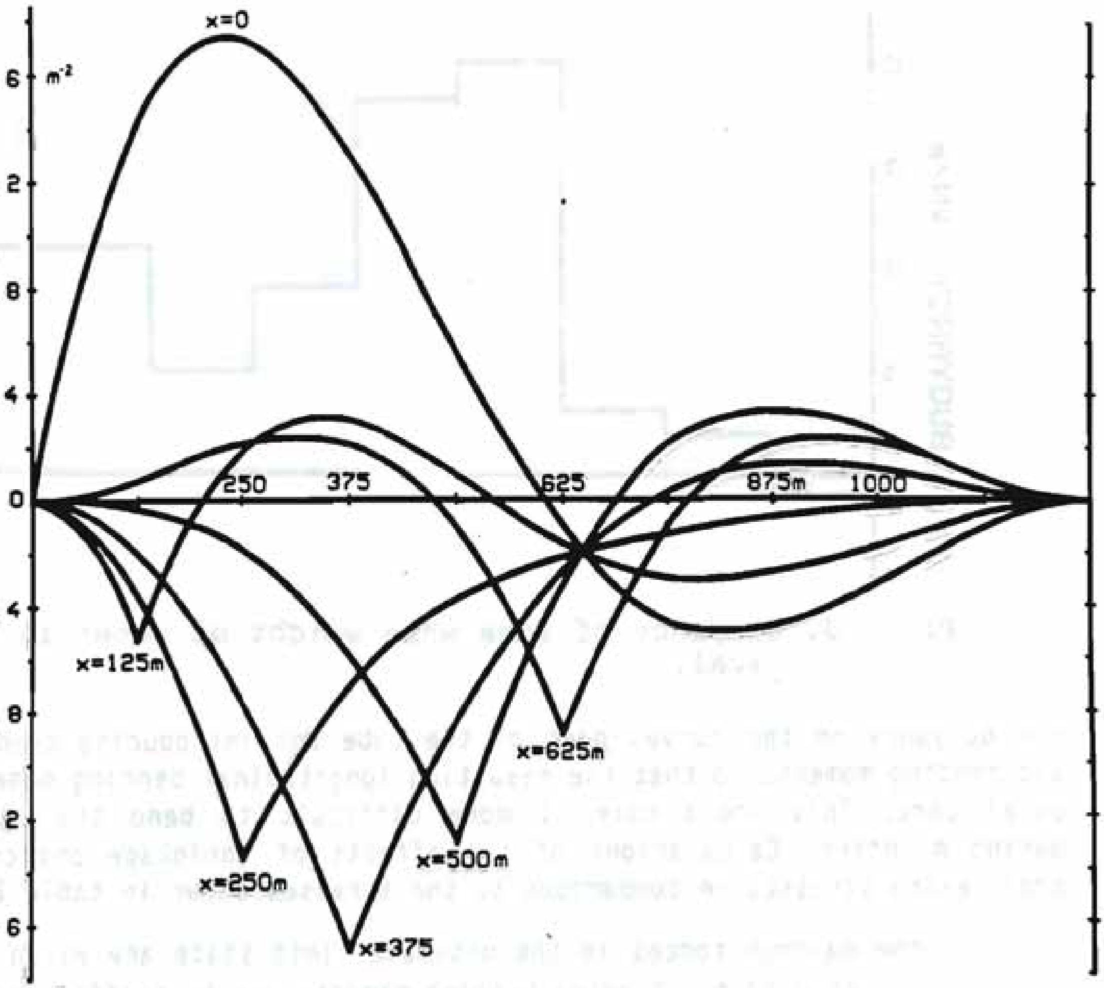

Fig. 12. Influence lines for stresses in the lowest fibre of the cross-section of the tube

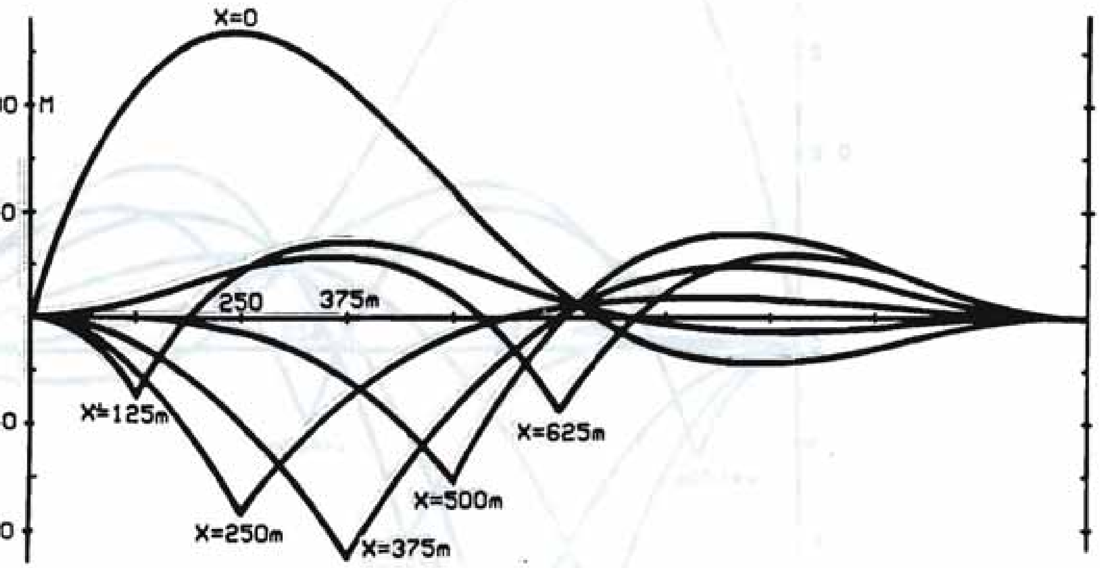

Fig. 13. Influence lines for bending moments in the tube

At every surface of the tube there is a high bond reinforcement in two directions perpendicular to each other. Yield strength is 600 MPa. The bars have a diameter of 12 mm, and they are placed 80 mm apart. This is a minimum reinforcement. According to unpublished tests carried out by the senior author, this reinforcement should make sure that there would be extensive cracking in the tube wall prior to any yield in the high bond reinforcement.

The fully drawn lines in fig. 14 show part of the interaction diagrams for the concrete tube made from three concrete qualities and reinforced by the minimum high bond reinforcement. The crosses in the diagram show combinations of M and N taken from table 1 and 2. The N values for the crosses have been found by

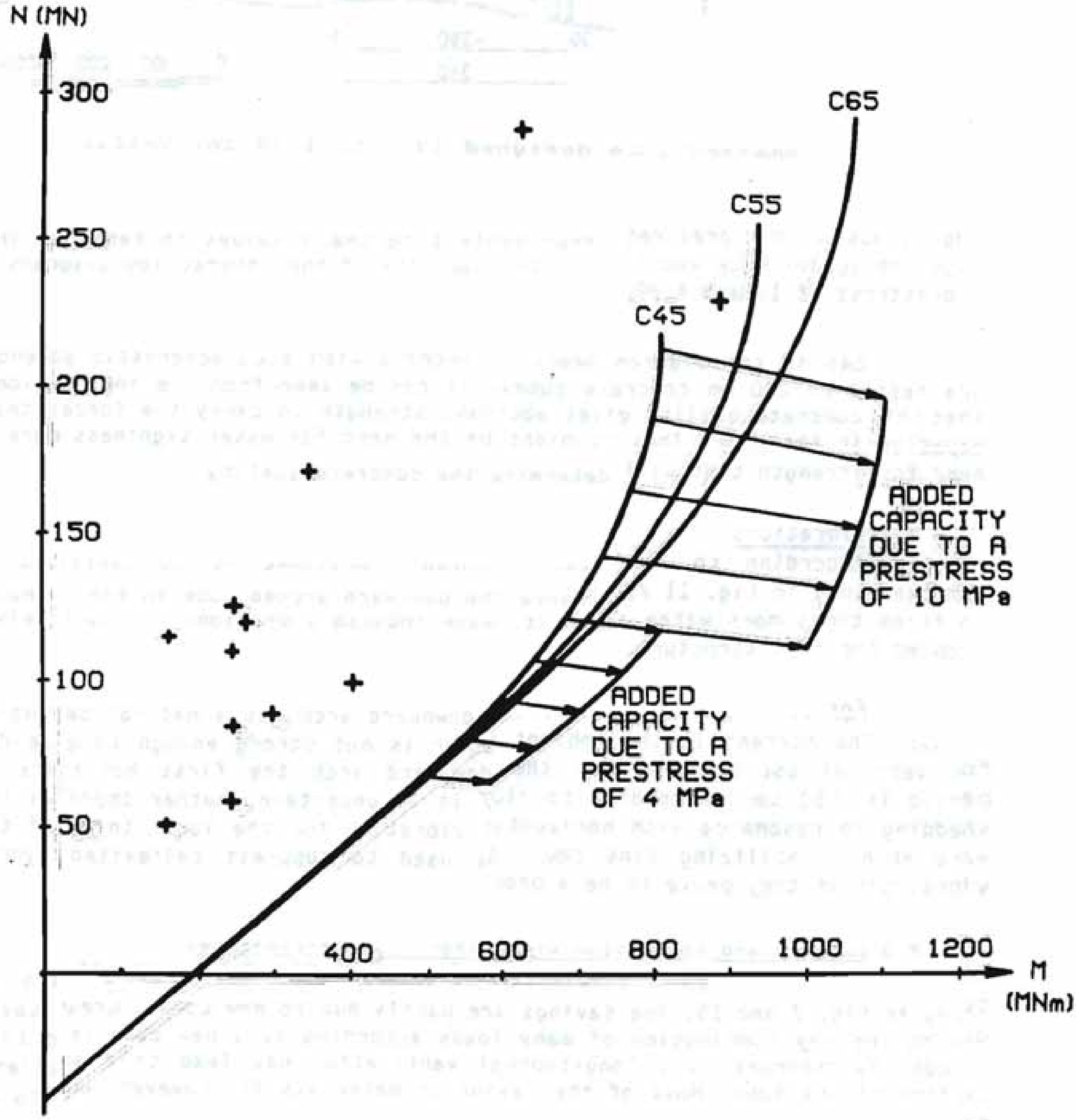

Fig. 14. Parts of interaction diagrams for reinforced tube

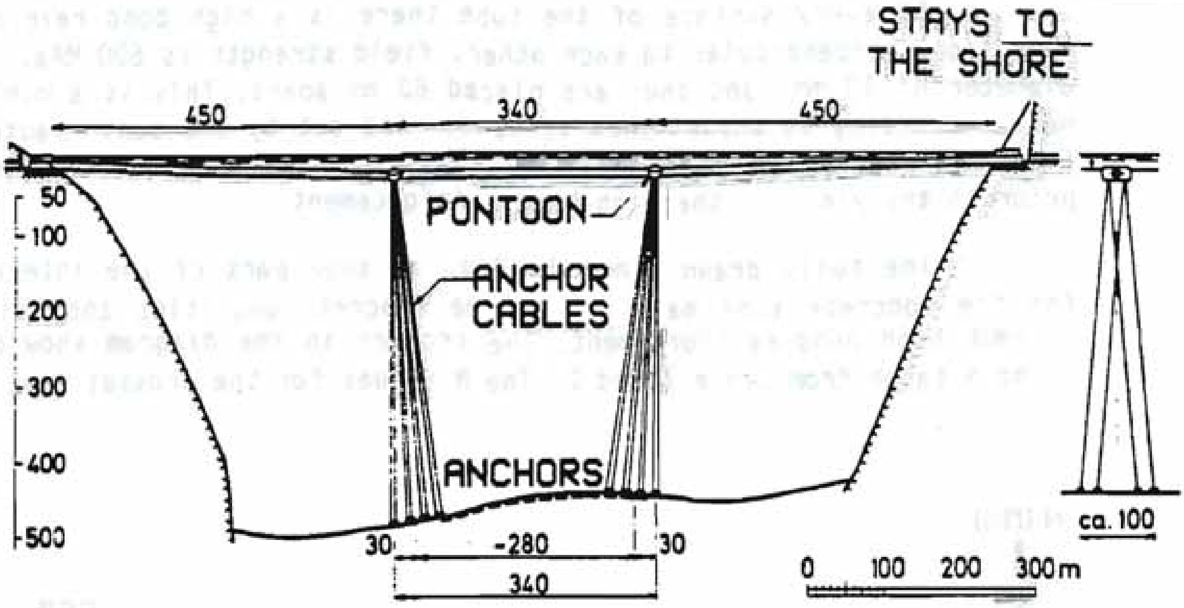

Fig. 15. Immersed tube designed 1974 to 1979 for Vallavik - Bu /9/

adding 90% of the prestress from table 1 to the N values in table 2. The arrows show the approximate addition to the capacity of the interaction diagrams due to a a prestress of 10 and 4 MPa.

C45 in the diagram means a concrete with a caracteristic strength of 45 MPa tested on 200 mm concrete cubes. It can be seen from the interaction diagram that this concrete quality gives abundant strength to carry the forces that can be expected in the tube. Thus it might be the need for water tightness more than the need for strength that will determine the concrete quality.

7.2 Vibrations

According to /10/ wave induced vibrations is no certainty for the immersed tube in fig. 11 /9/. Since the downward arched tube in fig. 7 has got two to three times more water above it, wave induced vibrations seem unlikely to be a problem for this structure.

For vertical oscilations the downward arch has a natural period of about 25 sec. The current in the ambient water is not strong enough to give resonance for vertical oscilations. For the downward arch the first horizontal natural period is ~53 sec. According to /10/ it is uncertain whether there will be eddy shedding in resonance with horizontal vibration for the lower third of the downward arch. Stabilizing fins could be used to suppress selfexited hydroelastic vibrations if they prove to be a problem.

7.3 Economy and comparison with other immersed tubes

Table 3 gives a comparison of some of the quantities in the immersed tubes in fig. 7 and 15. The savings are partly due to new codes. Great savings are due to the way combination of many loads according to a new code lead to smaller forces. Furthermore, the longitudinal ventilation has lead to a smaller cross-section of the tube. Most of the saving of materials is, however, due to a better concept.

The authors do not have the time and the know-how to calculate the price of the two alternatives, but would like to cooperate with anyone who wants to do so.

Table 3. Incomplete comparison between two immersed tubes between Vallavik and Bu in the Hardangerfjord.

	1979	1986
Structural concrete in tubes and pontoons	55.100 m^3	33.410 m^3
Concrete in anchors	6.240 m^3	0
High bond reinforcement	5.745 t	2.300 t
Prestressing steel	2.175 t	1.000 t
Vertical stays	500 t	0
Horizontal stays	290 t	0

To end the paper the authors would like to state why they think that the immersed tube in fig. 7 p. 6 is the cheapest solution possible for spans under 1,5 km in fiords where the speed of current is moderate and good rock is found at both shores.

The abutment in fig. 5 p. 5 is about the cheapest solution possible. The corresponding structure in fig. 15 is much costlier. The amount of concrete and ballast for the tube is decided by the area of the cross-section, thus a cheaper tube than the ones shown in fig. 2 is hardly possible, as long as the tube is circular and made from watertight concrete. The high bond reinforcement is a minimum reinforcement. If this reinforcement can be reduced for other immersed tubes, it can propably be reduced for the immersed tube in fig. 7 as well.

For the downward arched tube in fig. 7 the 1000 t of prestressing steel can propably be reduced to 500 t by increasing the buoyancy of the curved part of the tube. If we assume that more prestressing steel was saved by introducing many vertical stays, the saving in money would be ~25,000.- Norwegian kroner per tonn. Even 500 x 25,000 would not pay for many vertical stays.

Summing up: The downward arched immersed tube consists of abutments and tube. The authors can not see how saving in abutments and tube would pay for the extra structural elements in other alternatives. Thus the authors are led to believe that the downward arch is the cheapest immersed tube possible.

There may be a handout for the symposium where items from this paper are explained at greater length. Queries and advance orders would be appreciated.

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International Conference on Submerged Floating Tunnels

Sandnes, Norway, 29 - 30 May, 1996

15. A submerged floating tunnel across the Høgsfjord - a pilot project Dream or reality

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(Handout)

A submerged floating tunnel across Høgsfjorden - a pilot project Dream or reality

Once again the authorities and politicians are to consider a submerged floating tunnel across Høgsfjorden. The people of Ryfylke and the Public Roads Administration are eagerly awaiting the decision. For we have waited for so long!

The first submerged floating tunnel concept we know of is a British patent from 1886. The patent documents describe the concept in detail. It is surprising how this patent resembles the submerged floating tunnel concept we are planning today.

In Norway the first submerged floating tunnel patent dates from 1923. In the Sixties a committee was appointed by the Directorate of Public Roads to report on the possibility of submerged floating tunnels. The final report appeared in 1971. The report concluded that the use of submerged floating tunnels was feasible, and that it offered an interesting and, in many cases, economic alternative to conventional bridges.

In the next 15 years, from 1970 to 1985, many proposals were introduced on how submerged floating tunnels could be used. But it was not until 1985 that the Bridge Department of the Directorate of Public Roads proposed Høgsfjorden as the site of a pilot project. This proposal was subsequently approved by the Norwegian Parliament.

The Høgsfjord Project is now the only relevant submerged floating tunnel project that can enhance our expertise in such projects. Norwegian industry feels the project is important for both the development of technology and for the export of consultancy and contracting services.

There are many proposals on future submerged floating tunnels, both in Norway and abroad, but they are dependent on the implementation of the Høgsfjord Project.

Local and national trade and industry interests

Local and national industry will also benefit from the Høgsfjord development. Four Norwegian firms have submitted proposals for a submerged floating tunnel solution. Two of them are the contractors Selmer A/S and Eeg-Henriksen Anlegg AS, who have both chosen solutions using concrete. The two local companies, Norwegian Contractors (NC) and Kværner Rosenberg, have chosen individual solutions. NC, like the two others, wants to build the submerged floating tunnel of concrete, while Kværner wants to build it of steel. Both the local companies are of the opinion that their facilities in Stavanger will give them a competitive edge, and both need work to offset the downturn in offshore investments. Whether it is built of concrete or steel, the submerged floating tunnel will provide great technical prestige.

Is the Høgsfjord Project merely a prestige project?

There are many who «suspect« the Public Roads Administration of wanting a ferryless connection across Høgsfjorden merely so they can implement a test/pilot project with a submerged floating tunnel. But this is only part of the overall picture. Better conditions for Ryfylke are the main objective.

For generations Ryfylke had a good transport system based on the fjord as a transport artery, while Jæren was worse off. Now it is the other way round and the recognition that permanent connections are a sustaining element for development and stability in a district has become stronger with the passing of time.

These circumstances, together with the technological development and the expanded financing opportunities, have led the development of transport facilities into a new age, where the wish is to have as many ferryless connections as possible.

Rønnaug Foss Alsvik, popular singer and manager of the advertising agency Bindestreken in Strand puts it this way: **«It is soon time to create an alternative to ferries, give Strand and Ryfylke a new artery, a new road connection. A submerged floating tunnel across Høgsfjorden. I wouldn't hesitate to call such a road connection the opportunity of the century for Strand and Ryfylke. But I am anxious that we do not waste this unique opportunity to be more closely connected to Stavanger, Sandnes and the rest of northern Jæren.»**

The purpose of a ferryless connection

Even though the construction of a submerged floating tunnel has received the greatest attention, it is not the submerged floating tunnel that is the most important goal of the Høgsfjord Project. The goal of the local community is to give the Ryfylke municipalities Forsand, Strand and Hjelmeland, with their approximately 13,000 inhabitant, a permanent road connection to northern Jæren. A submerged floating tunnel across Høgsfjorden would be the first stage of a ferryless Ryfylke road. The tunnel will increase the accessibility of Ryfylke for all travellers and improve transport conditions for existing and new industry in the region. Industry and the people who live in Ryfylke would have the same transport conditions as the rest of the county.

The Høgsfjord Project will give Ryfylke the fastest start to a positive development in trade and industry and settlement. Today's commuters can obtain new work alternatives and new communications enable the people of Ryfylke to travel when they want to.

The Høgsfjord Project, environment and public transport service

It is evident that a ferryless connection across Høgsfjorden will create new activity and thus lead to an increase in vehicle traffic. This follows as a natural consequence of shutting down two ferry connections. In addition to new traffic there will be a transfer from the ferries. And people will probably drive their cars over longer distances.

But this doesn't mean that the project is detrimental to the environment. On the contrary, the Høgsfjord Project will lead to a substantial improvement of the public transport service between Ryfylke and northern Jæren. An expanded express boat service means shorter travel time than by ferry, and, in addition, the bus service will be better than it is today. A better bus service across Høgsfjorden will make it easier to travel to the large job and commercial areas around Sandnes and Forus.

If we look at the public transport service in connection with a ferryless connection across Høgsfjorden, the project is special in relation to toll projects around the country. Other toll projects do not usually permit parallel «competing» public transport services. Nor is it reckoned that passengers will pay toll, so that the travel costs for bus passengers will be the same as for any other bus trip.

Environmental impact

The natural environment will not be significantly affected by the submerged floating tunnel. With today's ferry and vehicle technology, a submerged floating tunnel will lead to a considerably lower discharge level of the gases NOx, CO_2 and SO_2 than that of today's ferry service and, not least, of an expanded ferry service (cf. Faktarapport & miljøanalyse (Fact report and environmental analysis)). The environmental analysis shows that the best alternative for the environment with today's technology is a submerged floating tunnel. This means inter alia that it is **not** environmentally friendly to expand transport services by an increased ferry service. Nor will air pollution be substantially less by continued use of ferries as opposed to a submerged floating tunnel, even if we use new technology, including gas-driven ferries.

Outdoor life is also part of the environment and surveys show that relevant outdoor areas will hardly be affected by a submerged floating tunnel and what this involves in the way of increased traffic and road repairs. On the contrary, new and attractive picnic areas will give more people the chance to experience the beautiful Ryfylke countryside. Driving through the world's first submerged floating tunnel, a drive that takes a few minutes, will in itself be a big experience for tourists at the same time as the tunnel in itself is a tourist attraction.

Pedestrians and cyclists

For many of us walking and cycling are an important part of our life quality and experience of the environment. A submerged floating tunnel across Høgsfjorden will be adapted to cyclists and pedestrians, with regard both to ventilation and lighting. Cyclists will take about five minutes and pedestrians about 15 minutes to pass the 1,500-metre long tunnel.

How safe is a submerged floating tunnel?

Many people are sceptical about crossing a fjord through a tunnel that «floats» 25 metres below the surface of the fjord. In the past there were many people who didn't like the idea of building underwater tunnels, but they use them daily after they acquired experience. The same will undoubtedly apply to a submerged floating tunnel.

In reality, submerged floating tunnels are based on known technology and we can obtain the same safety as we do when crossing the fjord by underwater tunnel or ordinary bridge. The submerged floating tunnel will be dimensioned to withstand accidents and overloading, i.e. that necessary measures will be implemented to reduce the probability and consequence of major accidents to absolute minimal and down to an acceptable level. But we should nevertheless realise that there will be a risk tied to ferries as well as submerged floating tunnels with regard to collisions and danger of explosion.

All that remains now is yet another political consideration before we know whether Norway and Rogaland are to lead the way, also regarding submerged floating tunnels!

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Objectives

bearings

settlement

torsion

crack ing

joints

hysteresis

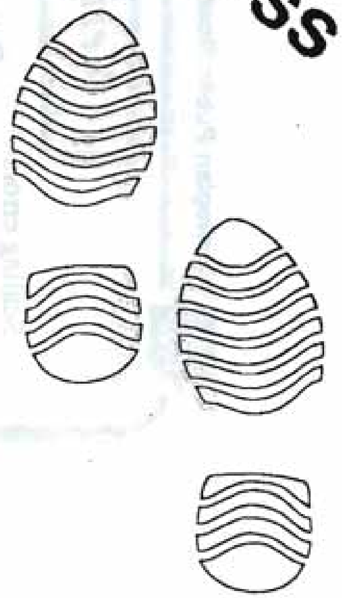
reference point

stiffness

contin u ity

admissible deflection

abnormal behaviour



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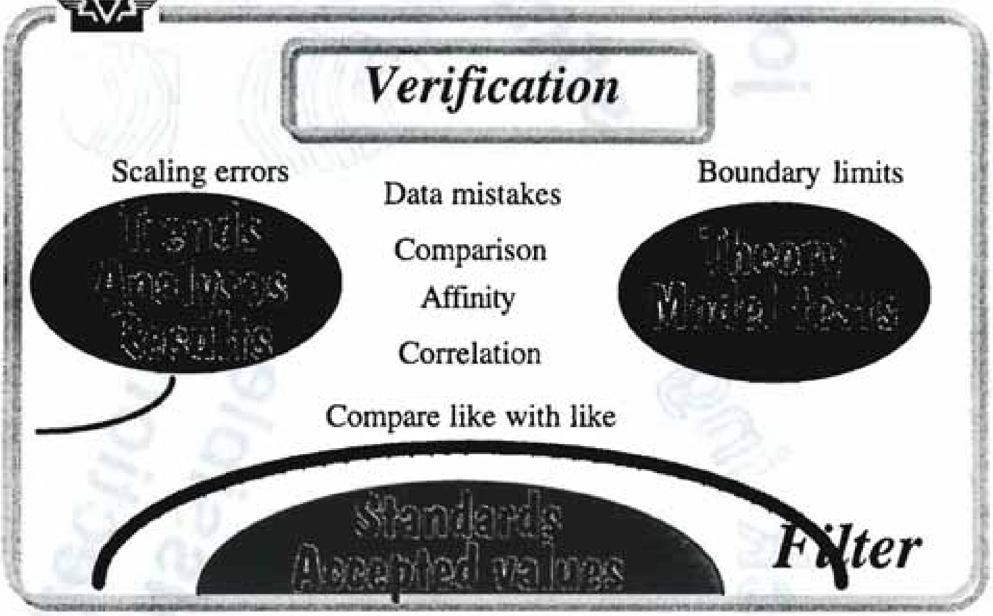

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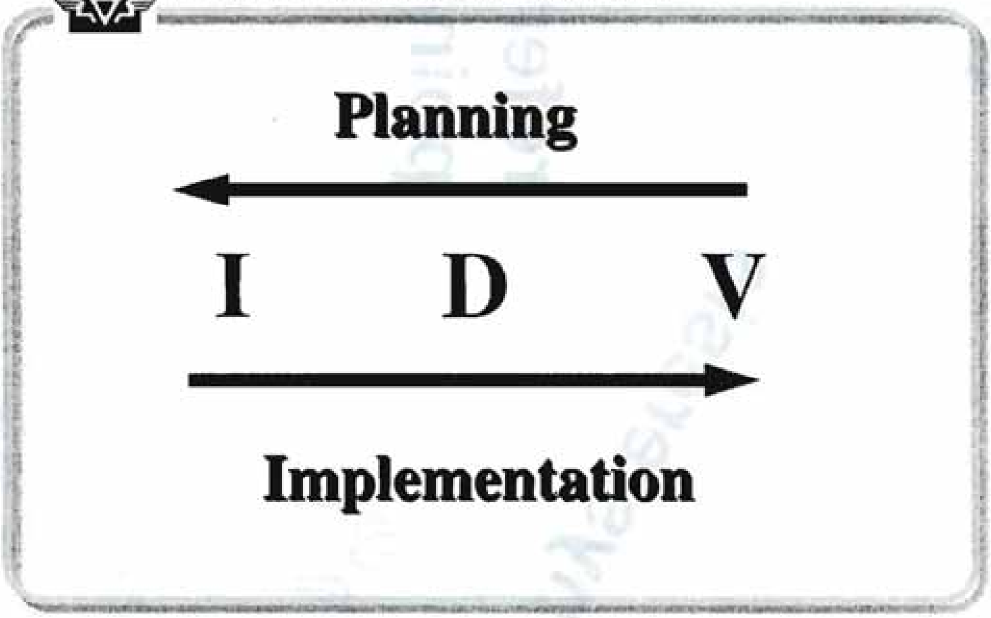

SFT-CONFERENCE IN SANDNES, MAY 29-30, 1996

INSTRUMENTATION, DOCUMENTATION AND VERIFICATION

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Norwegian Public Roads Administration

1. INTRODUCTION

Knowledge and experience are two key elements of competence. Their obtainment is difficult and their transfer to others even more so. In addition to this comes the problem of distinguishing between knowledge that is fact and knowledge that is supposition. In an effort to improve this situation, the IDV-system was developed in order to improve our knowledge and understanding of the **real behaviour** of structures. IDV stands for instrumentation, documentation and verification. The system allows us to identify what knowledge is good, what knowledge is bad and what knowledge is only average. From that point, we can discard the bad, improve the average and promote the good.

A key word in all this is "reliable". At each stage, I, D and V, the results produced must be reliable. This requires not only resources but also competent personnel. They have the difficult task of obtaining and judging quality. They must also have the ultimate responsibility of what is reliable and what is not. It must not be forgotten at this point that documenting "bad knowledge" is just as important as documenting "good knowledge". But of course, both are worthless if the documentation is not reliable.

2. CONCEPT OR SYSTEM

A distinction should be made between the IDV-system and IDV-concept. They are closely linked but a simplified division can be found by saying that the system refers to the actions (planned or carried out) whilst the concept refers to the governing principles. In this way, the concept defines and guides the system to a reliable result. This is the real strength of IDV as its structure is never defined beforehand but is adapted to each new construction/problem. In fact, the IDV system evolves as it progresses. For example, as results flow in, additional measurement points may be necessary to corroborate the initial and preliminary findings.

3. IDV

This conference is about submerged floating tunnels and as such this contribution should present an IDV-system for SFTs. However, as previously stated without an actual design or defined problem, an IDV-system can only be outlined by working through the IDV-concept. The first thing to be done is to establish objectives; what needs to be verified and what can be verified? Safety and performance of the structure top the list here. Research needs may also be included. Identification and quantification of all aspects associated with the objectives is important at this stage as the available competence and resources must be sufficient to obtain these goals. The role of the designer, instrumentation company and contractor will vary from project to project but their early participation will often lead to better team work and a more reliable result. The designer can play a large role in the IDV-system but it is very important that he views it as an occasion to learn and not as a control of his work.

Thorough planning is needed if confusion and doubtful results are to be avoided, the enclosed figures give an indication of the type of questions that must be asked and answered/solved during the planning, implementation and analyses phases of the IDV process. It is by having a certain mistrust of the IDV-system that an increased sense of responsibility will come about and with that, reliable results.

4. IDV FOR SFT

What will be measured on an SFT? The following is by no means a comprehensive list but will serve as an indication of the extent of such a system.

Safety. Monitoring of ; water infiltration, change of ballast, stresses and strains, fire, CO gases, submarines, dropped anchors, earthquakes, rock slides, etc.

Loading. Monitoring of ; wind, waves, currents, water density, traffic, settlement, additional self weight, ballast, etc.

Structural behaviour. Monitoring of ; joint movement, structural displacement, bearing reaction, acceleration, velocity, tension leg force, pontoon buoyancy, etc.

Material performance. Composition, production method, curing regime, temperature gradient, chloride concentration, permeability, water content, electrical/electrochemical properties, nitrogen content, weld properties, joint material properties, uniformity, etc.

Durability. Monitoring of ; corrosion, chloride levels, pH, RH, carbonation, marine growth, changes in dynamic behaviour, stiffness.

With many of the above, it is important to establish a starting point or reference point. Without this, variation is very hard to detect. In addition, IDV helps us to maintain the construction properly by indicating the right time to initiate repairs/maintenance.

5. CONCLUSION

For small or simple IDV projects, many problems are overcome by the fact that it is the same person at each stage; I, D and V. However for large/complex projects, many experts are involved and frequently they are not experienced with IDV. To avoid misunderstandings, planning, team work and good documentation are required. Documentation must always be of top quality and written with the reader in mind. This is particularly important if documents are to maintain their value/usefulness for say twenty years.

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Statens vegvesen

Transfer of knowledge and experience

Civil Engineering

Degrees
Books
Conferences
Articles
Journals
Courses
Traineeships
Videos
:
etc.

Does this

lead to?

Better structures
Improved design
Increased durability
Reduced maintenance
:
Increased knowledge of the real behaviour of structures

Norwegian Public Roads Administration

IDV- SYSTEM

INSTRUMENTATION
DOCUMENTATION
VERIFICATION

By verification of construction methods and practice, establish knowledge of what is bad, average and good.

Then, discard the bad, improve the average and bring the good with us into the future.

London 3-5 July -95 H. Østlid

Norwegian Public Roads Administration

IDV - SYSTEM

INSTRUMENTATION:

* Arrange for instruments to overlap and confirm each others measured values.
* Use more than one type of sensor to measure similar values.

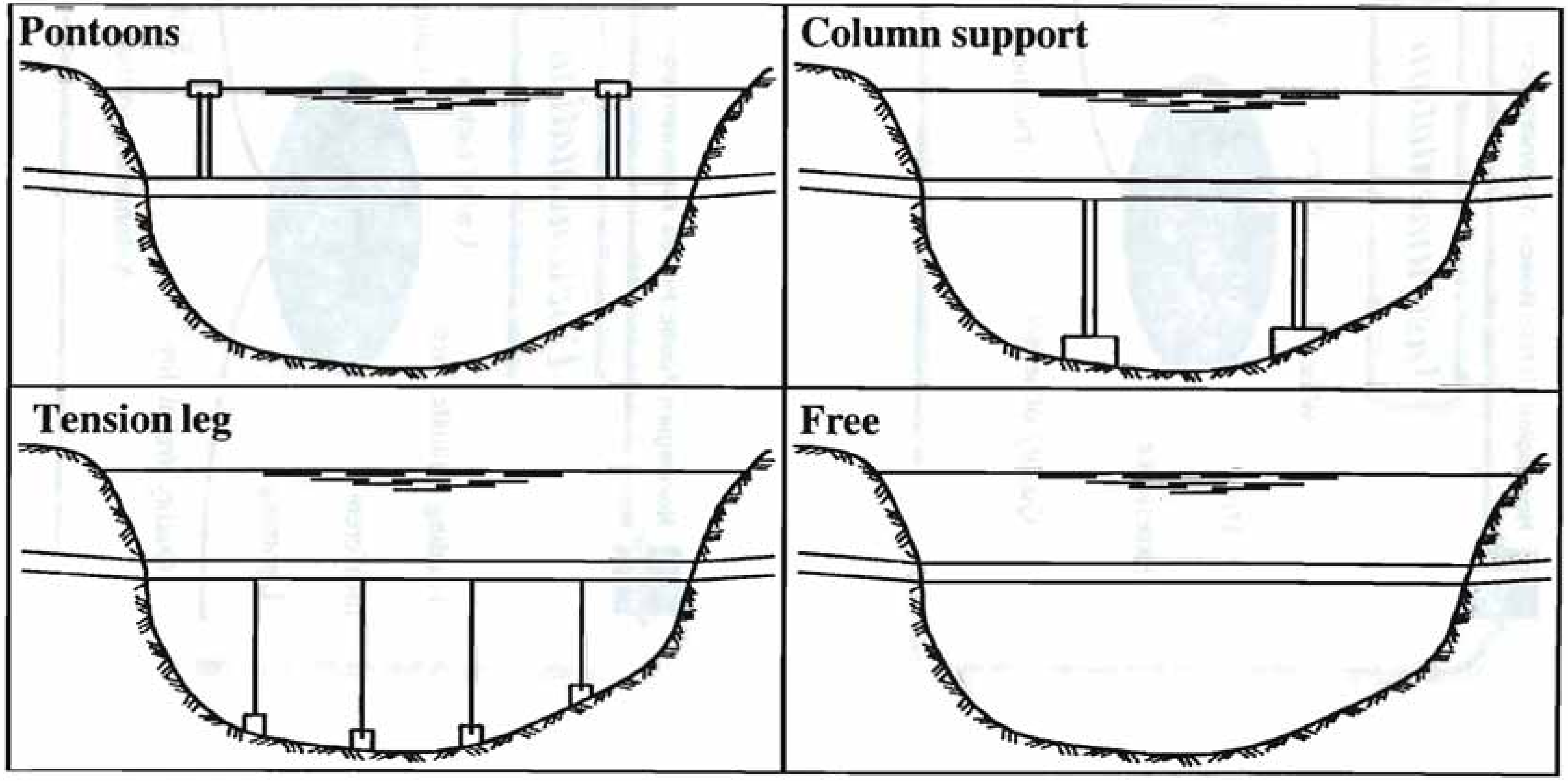
DOCUMENTATION:

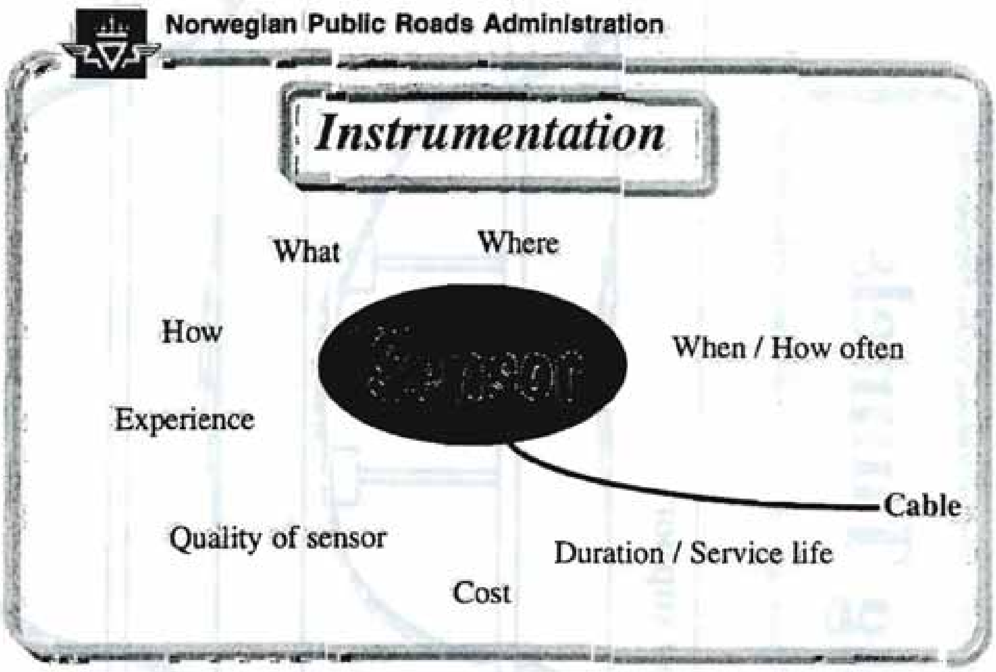
* In some way or other, check the reliability of data going into documentation.
* Make sure that documented results may be used for verification purposes as directly as possible.

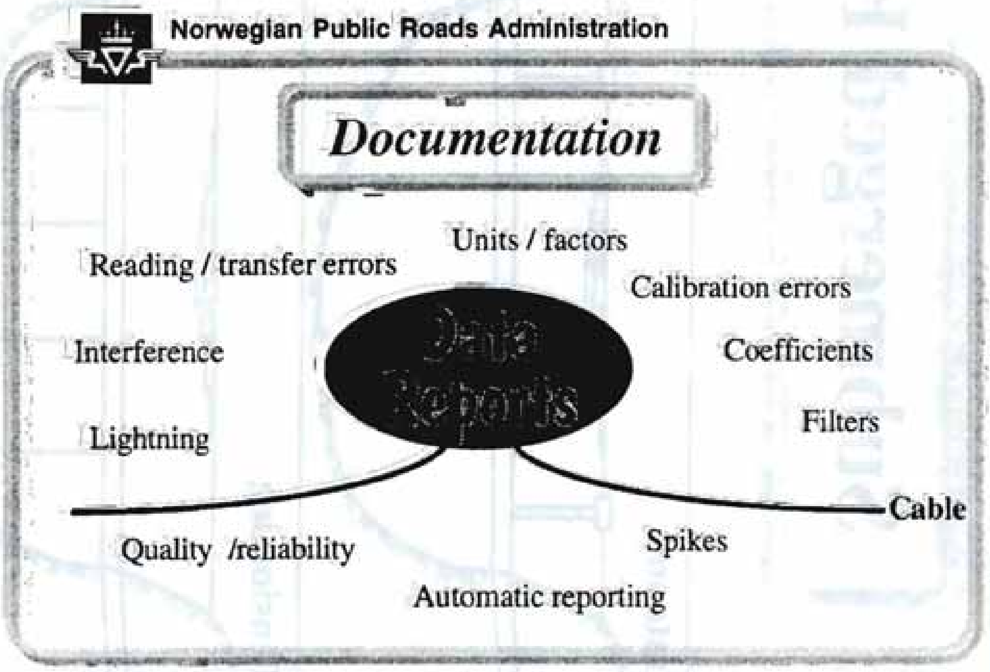
VERIFICATION:

* Make sure that results from verifications are presented and understood by all involved parties.

Submerged Floating Tunnels




Norwegian Public Roads Administration
Instrumentation
What
Where
How
When / How often
Experience
Cable
Quality of sensor
Duration / Service life
Cost


Norwegian Public Roads Administration
Documentation
Units / factors
Reading / transfer errors
Calibration errors
Interference
Coefficients
Filters
Lightning
Cable
Quality /reliability
Spikes
Automatic reporting