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Bulletin 96



Guidelines for Submerged Floating Tube Bridges

Guide to good practice

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fib CEB-FIP Bulletin 96

Guidelines for Submerged Floating Tube Bridges

Guide to good practice

Working Party 1.2.2

October 2020

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Foreword

This bulletin is a guidelines document for "Submerged Floating Tube Bridges", that represents an innovation in Marine Concrete Structures. This theme is considered important for Commission 1 since in the future several applications are forecast in marine environments.

Submerged Floating Tube Bridges are a solution that can be proposed to solve different problems in passing water constrains as lakes and fiords, reducing the impact and allowing several economic advantages.

The guidelines certainly will boost the application of Submerged Floating Tube Bridges since the document is useful not only for designers but also for construction companies, owners and public administrations.

As guidelines, the bulletin gives wide information on the design, construction and management of these structures, allowing all the users to be confident in promoting the use of Submerged Floating Tube Bridges.

As Commission 1 Chair, I'm very grateful to Arianna Minoretti and to all members of WP 1.2.2 "Submerged Floating Tube Bridges", for having produced this document that I consider very interesting not only for the *fib* members but also for the concrete community.

Alberto Meda

Chair of Commission 1 *Concrete Structures*

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Acronyms

ADR	European Agreement concerning the International Carriage of Dangerous Goods by Road
AIS	Automatic Identification System
CH	Chapter
CoG	Center of Gravity
HAT	Highest Astronomical Tide
HAZID	Hazard identification exercises
HAZOP	Hazard and operability studies
HWL	Highest water level
LAT	Lowest Astronomical Tide
LWL	Lowest water level
MEDEVAC	Emergency Chartered Medical Evacuation Services
MEP	Mechanical, electrical and plumbing
MSL	Mean Sea Level
PGA	Peak ground acceleration
ROV	Remotely operated underwater vehicle
SACP	Sacrificial Anode Cathodic Protection
SFT	Submerged Floating Tunnel
SFTB	Submerged Floating Tube Bridge
SOLAS	International Convention for the Safety of Life at Sea
TLP	Tension leg platforms
VIV	Vortex Induced Vibrations

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1. Introduction

The aim of Working Party 1.2.2 is to provide guidelines for the engineering community on the design and construction of a Submerged Floating Tube Bridge (SFTB), based on relevant experience. A Submerged Floating Tube Bridge is a type of structure for general use, but this document focuses on concrete structures for traffic.

As shown in this guideline, the potential benefits of the SFTB are great, for crossing lakes and fiords, also in terms of economic and environmental impacts: imagine, for example, how this structure could solve traffic problems on lake shores, keeping the beauty of the surrounding landscape.

The state of the research and recent developments in the offshore field have made it possible to think of building this type of structure, conceived for the first time in the 19th century. In the last fifty years, several concepts have been developed in different countries; nevertheless, only one of those structures has been completed (see Ch. 14, Fig 14-5). The lack of regulations and rules has certainly contributed to the poor confidence of the designers and the authorities towards this type of structure, even if the studies carried out have shown its maturity in terms of feasibility.

The Working Party aims to improve the knowledge of the scientific community on SFTBs, helping them in the design process and strengthening the confidence of the community on a type of structure that has a great potential.

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2. The SFTB

2.1 The name

A Submerged Floating Tube Bridge is a structure that takes advantage of the uplifting force on a body in water (Archimedes' principle) to balance the gravity actions. Horizontal, transverse and longitudinal actions have to be carried by the SFTB.

This type of crossing is also called 'Archimedes' bridge'. The structure is also known as Submerged Floating Tunnel (SFT), for the shape similar to ordinary tunnels. Nevertheless, the behaviour of the structure is that of a bridge and for this reason the Norwegian name for it is 'rørbru', which means 'tube bridge'.

The name 'Submerged Floating Tube Bridge' describes both the main characteristics of the structure – Submerged Floating Tube – and the structural behaviour – Bridge.

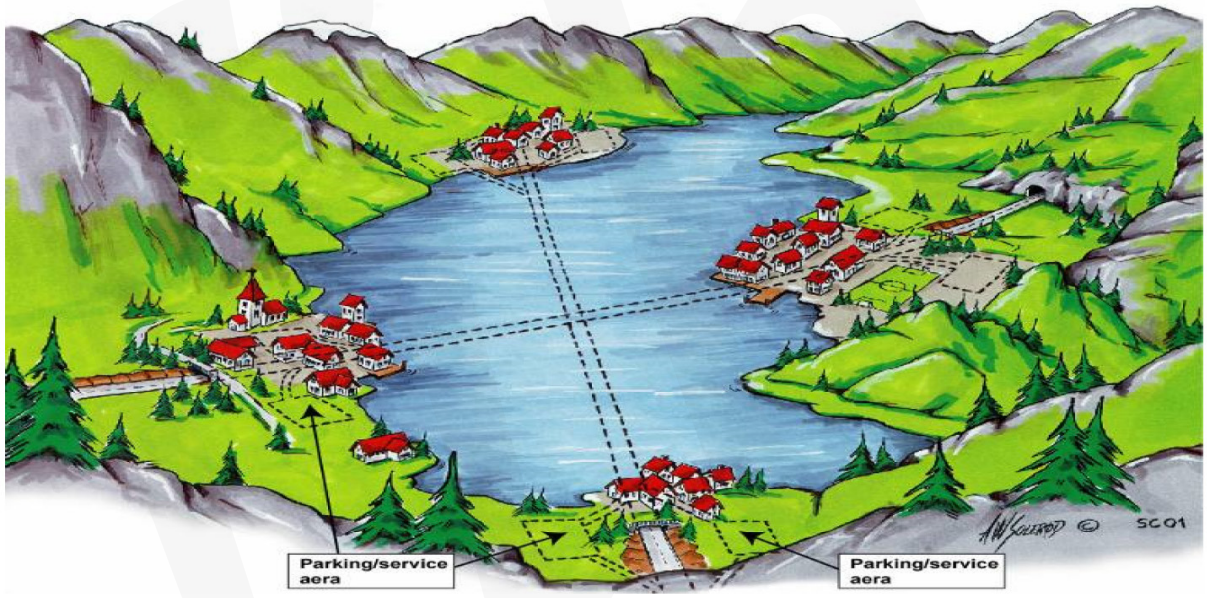


Fig. 2-1 Crossing with an SFTB, A.W.Solerød – Statens vegvesen

2.2 The concept

An SFTB is a floating bridge, submerged at a defined depth below the water surface (Fig. 2-2) with water surrounding the tube cross-section. The structure could be used for several purposes, but this document is focused on the use for transportation.

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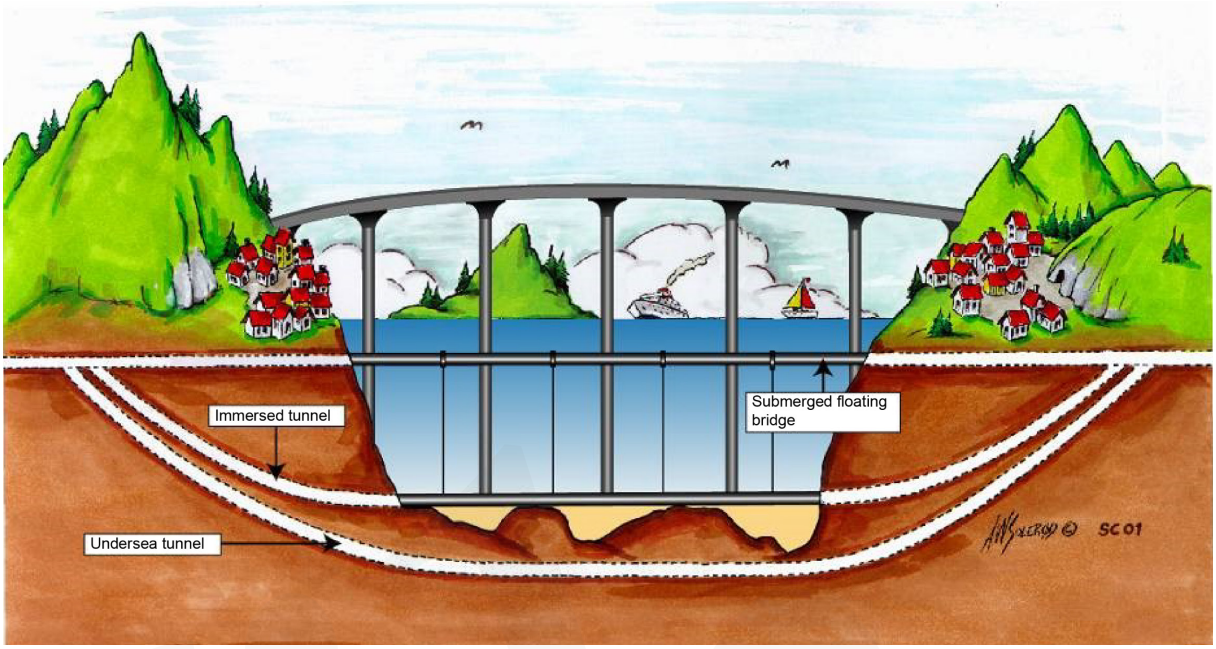


Fig. 2-2 Scheme for possible crossings, A.W.Solerød – Statens vegvesen

The geometry of the structure, having a closed cross-section (examples in Fig. 2-3), can be calibrated, together with the permanent weight, in order to achieve the desired buoyancy.

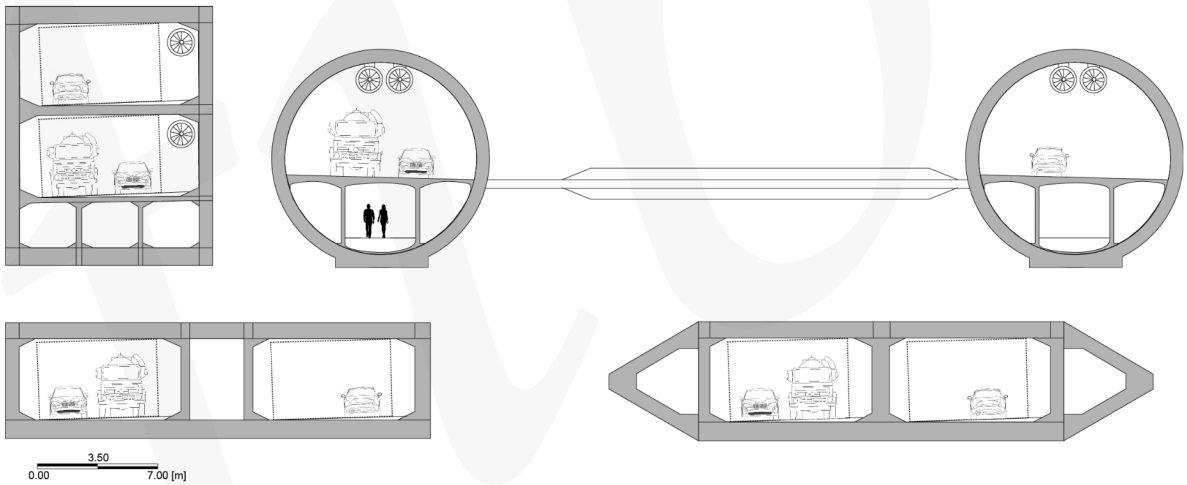


Fig. 2-3 Possible cross section for the SFTB, Dr. Techn. Olav Olsen AS - Statens vegvesen

The bridge can consequently be vertically stabilised by floating pontoons, columns or tethers, connecting the structure with the seabed (Fig. 2-4). For short crossings, the structure could rely just on the shore connections, without intermediate supports.

The bridge is laterally stabilised either by its own stiffness for short crossings, or by moorings or tethers to control lateral movements. Column-supported SFTBs are likely to rely on the bridge stiffness for lateral stability.

Several alignments can be possible, both in the horizontal and vertical directions.

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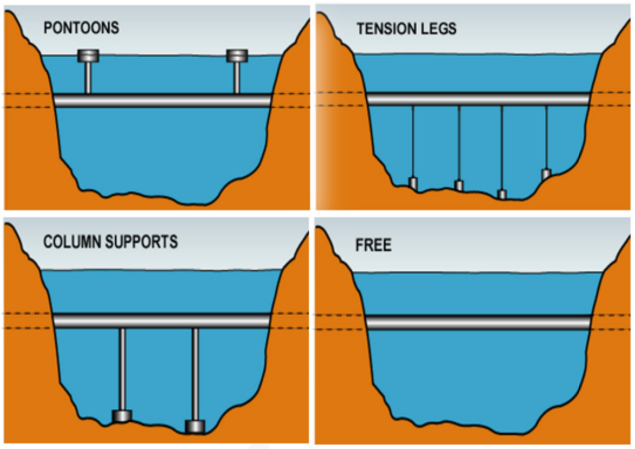


Fig. 2-4 Scheme for possible crossings, A.W.Solerød – Statens vegvesen

The submergence of the structure can naturally reduce the main environmental loads on it, making it competitive in exposed areas with respect to traditional over-water bridges and allowing the design of zero-downtime crossings. In addition, it gives the possibility to create a hidden crossing (except for the pontoons option), minimising the visual impact on the landscape and allowing a reduction in noise impact on the surroundings.

2.3 Benefits of the SFTB

The submergence of the SFTB is, first of all, a natural way to reduce the main sea action on the structure. This is a huge advantage in exposed areas, where traditional bridges would be exposed to extreme wind and sea loads.

Another important benefit of the SFTB is that it is invisible. This is a very positive aspect in fjords (Fig. 2-5) and in lakes (Fig. 2-6). Picturesque landscapes will not be affected by a new crossing and all the traffic will travel hidden, without being seen, smelled nor heard.



Fig. 2-5 Trolltunga, Norway – Dong Zhang on Unsplash

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Fig. 2-6 Bellagio, Italy – Fallon Travels on Unsplash

In comparison to subsea tunnels, a traditional way of strait crossing, the SFTB can reduce the length of the crossing. For example, if the strait is deep, a rock tunnel will have a considerable length (Fig. 2-7) and this will affect a large area onshore, whereas the SFTB is substantially shorter.

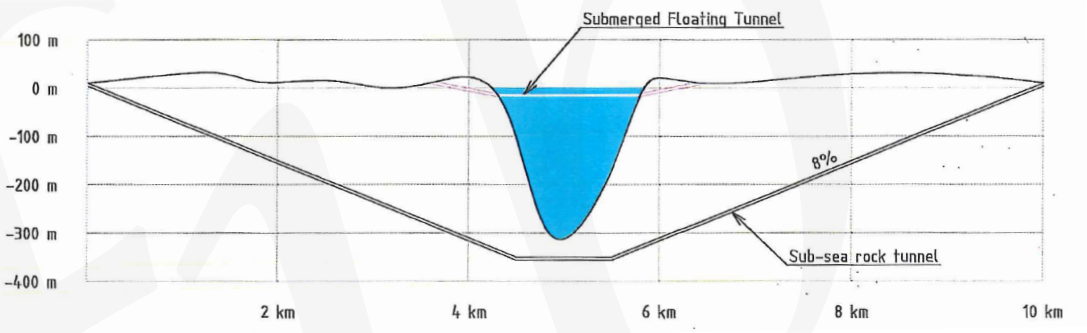


Fig. 2-7 Comparison SFTB and subsea tunnel – The Norwegian Submerged Floating Tunnel Company AS

Ferries are a common solution for crossing (Fig. 2-8). A ferry trip can be a nice part of a journey, for touristic purposes. For other transport purposes, it can be difficult to rely on ferries due to journey time and capacity to meet future traffic volumes. From an environmental point of view, electric ferries could have a lower impact than current vessels. Nevertheless, the impact of many boats on marine species has to be carefully evaluated.



Fig. 2-8 Drøbak, Norway – Vidar Nordli-Mathisen on Unsplash

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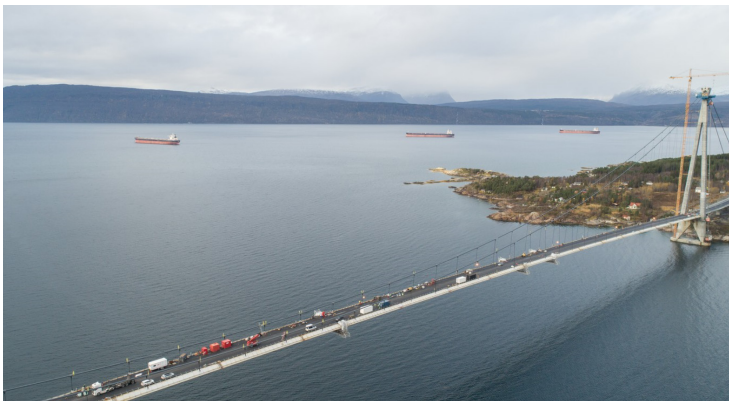


Fig. 2-9 Hålogalandsbrua, Norway – Tomas Rolland, Statens vegvesen

Traditional bridges (Fig. 2-9) have shown to be feasible up to lengths of approximately 2 km, while floating bridges (Fig. 2-10) and SFTBs do not have a length limit. Cost comparison between these solutions shows that the cost of a suspension bridge grows exponentially with length, whereas the cost of a floating bridge or SFTB, being modular, increases more linearly with the length, indicating that the cost per unit length is more or less constant.



Fig. 2-10 Nordhordalandsbrua, Norway – Statens vegvesen

2.4 Environmental issues

Among the environmental issues that need to be investigated, the 'underwater noise pollution' produced by the SFTB and its possible effects on marine life needs to be carefully evaluated in early design stages, in order to define the best mitigation choices and procedures.

The noise production has two separate stages: the construction period and the operational life of the structure. While the operations related to construction involve short time exposure to possibly high sound pressure levels, the sound produced by the operational life of the structure will be a low intensity noise that can be relatively continuous over time, depending on the characteristics of the road. This type of sound is similar to that produced by wind farms and recent studies[2-1] have demonstrated that, for long-term exposure, these types of sounds do not affect the stress level in the marine population.

Nevertheless, there are several aspects that need to be evaluated, from the interferences with the animals' communication system, to the influence on their growth. It seems fundamental to understand the 'threshold limits' of sounds that are acceptable to the specific

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marine living organisms in the area of intervention, to evaluate the potential impact. In this regard, the local characteristics of water depth and sea-bed composition and structure are also important, and these issues should be addressed in the early design, to better define the best mitigation possibilities for both construction and operational phases of the bridge.

Regarding climate changes, an SFTB will be less affected to changes like sea rise (at least the solutions anchored to the seabed), respect to a traditional bridge, but the impact of climate changes on the environmental actions must be carefully evaluated, as for any other type of bridge, and considered in the early design.

2.5 Reliability

The SFTB concept[2-2] was proposed for the first time in 1886 by the British naval architect Sir. Edward James Reed. In 1923 a design for an SFTB was patented in Norway by Trygve Olsen. In 1948 a proposal was presented (Erik Ødegård) for the Karmsund crossing, in Norway, but a traditional bridge was finally built, due to the cost evaluation.

In 1969 a committee was established in Norway to study the feasibility of the concept. The final report, in 1971, underlined that the reliability of the subsea anchorage and the uncertainties regarding the marine operations were the main technological challenges.

In 1982 F.Selmer A/S, a Norwegian company, built a floating submerged rectangular bridge in Kalstø (Norway) to protect and guide two gas pipeline on the coast. Even if the structure was not designed for the use we intend today (transport), it can be considered the precursor of the SFTB.

Several countries have proposed SFTBs for some crossings in the last decades[2-3], but it is only since detailed studies commenced for the crossings along the E39 in Norway (Sognefjord, 2011, Bjørnafjord, 2016) that the concept with tethers or pontoons has been proven to be really feasible[2-4]. As a result of the research carried out and to the developments and the experience gathered in the offshore and oil & gas field, the components of an SFTB have already reached a level of maturity such that their combination in an SFTB can now be considered reliable. The vertical stabilising elements proposed for a submerged bridge with pontoons are, in fact, the same as the pontoons used for the floating bridges (the first permanent floating bridge with concrete pontoons is from 1940 – Lacey Murrow Memorial Bridge, Seattle, USA) or, in case of an SFTB with tethers, as the tethers used for the Tension Leg Platforms (The first TLP structure was built in the early 1980's in Scotland; in Norway the first concrete TLP was Heidrun, installed in 1995).

Recent structures demonstrate in practice the feasibility of the SFTB. The Statpipe Shore Approach SFTB, was constructed in 1982 for protecting gas pipes and bridging subsea topography, and in 2015 the Söderströmstunnel was opened, both shown in Ch. 14.

The Söderströmstunnel is a 300 m long submerged railway bridge relying on pile foundations, and it has a static behaviour like that of an SFTB on column supports (Fig. 2-4).

Also shown in Fig. 2-2 is the immersed tunnel. The immersed tunnel is very similar to the SFTB: both allow invisible traffic through lakes and straits. The difference is simply that the SFTB bridges, while the immersed tunnel rest continually on the seabed. This is a purely structural difference, while all aspects related to construction, installation and operation are very similar. This influences the reliability of the SFTB positively: there are more than 150 immersed tunnels operating in the world.

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3. Rules and regulations

3.1 General

Although the SFTB concept has been extensively studied in several major projects around the world, no SFTB-specific codes have yet been developed. Specific, stand-alone rules for the SFTB would certainly assist in creating a broader acceptance of the SFTB concept, but the lack of codified provisions is not necessarily a major drawback for establishing an adequate basis for design, construction and operation for a specific crossing project. There are several international standards and codes, both land-based and offshore, that are applicable to the structural design of SFTBs. Some examples of relevant standards are briefly described below. All of the standards will require additional provisions to deal with the unique characteristics of the SFTB, although the extent of individual adaptation will vary since their scope and fields of application are different.

3.2 Relevant standards

3.2.1 Eurocodes

The Eurocodes comprise a series of standards providing a common approach for the structural design of buildings and other civil engineering structures. The pan-European structural design codes are also harmonized with execution, material and test standards as well as technical specifications for construction products.

The codes for concrete, steel and composite steel and concrete have a separate part explicitly covering the design of road and railway bridges. The traffic loads and bridge design provisions are particularly relevant for the SFTB design. Being primarily a land-based code, the Eurocode will however require additional formulations on floating structures. Collection and processing of metocean data, specification of wave and current actions and analysis of floating stability are examples missing in the Eurocode framework, and should be taken in from ship or offshore guidelines.

Since 2010 the Eurocodes are mandatory for the specification of public works within EU-EFTA. Future crossing projects in EU-EFTA countries will probably have to be carried out in compliance with the Eurocodes, despite some deficiencies. For example, in Norway, fjord crossing studies related to the E39 ferry replacement project have been conducted according to the Eurocodes since 2014.

Extending the applicability of the Eurocode 2 to also cover offshore structures has been part of the scope for the second generation of Eurocodes, expected after 2023. If implemented, the Eurocodes will certainly become a very suitable standard framework for the SFTBs, which may likely also be used outside Europe due to its international standing.

This chapter was mainly authored by Corres Peiretti and Haugerud.

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3.2.2 ISO standards

The ISO Offshore Structures suite of standards applies to offshore platforms for the oil and gas industries. The ISO 19900 series addresses the design, construction, transportation and installation of offshore structures, both gravity-based and floating, and is applicable to the design of complete structures including substructures foundations and mooring systems. The ISO standards cover many essential aspects of design for SFTBs, but are, like all other offshore standards, lacking bridge design provisions. Coupled with suitable national or regional standards for vehicle and/or rail traffic loads, the ISO standards can be adopted as a basis for design of SFTBs.

ISO 19900[3-1] provides general and specific principles for the design of offshore structures. The ISO 19901-1 series provides specific requirements for different aspects of design and operation such as metocean criteria, geotechnical and foundation design considerations, weight control and marine operations. ISO 19903[3-2], shortly to be published in a second edition covering fixed and floating concrete offshore structures, is intended to be flexible with respect to the choice of structural configurations, materials and techniques. The standard, giving guidance instead of detailed prescriptive rules, relies on a recognized reference standard for the structural concrete design, covering "all aspects" relevant to offshore structure. Except for the withdrawn NS 3473, not many concrete design standards meet this requirement. The new edition of ISO 19903 will refer to Eurocode 2 as a suitable reference standard. However, with adequate supplementary provisions, most modern concrete standards can be used. Some of the most essential aspects are described in Ch. 3.3.

3.2.3 DNV GL standards

DNV GL is an international classification body which has been a leading contributor to research on design requirements and in developing industry standards and best practices for the global oil and gas industry. The DNV GL Offshore Standards for structures, reflecting more than 40 years of experience with marine structures in the North Sea and other international waters, provides principles, technical requirements and guidance for the structural design, construction and in-service inspection.

DNVGL-OS-C502 (recently rebranded as DNVGL-ST-C502) provides, together with DNVGL-OS-C101, the general part of the Offshore Standards for structures, a thorough basis for the design comprising most topics relevant for the SFTB such as: specification of wind, wave, VIV and current actions as well as their correlation, design format, analysis approaches and foundation design premises.

Regarding the structural concrete design, DNVGL-OS-C502 offers, in contrast to ISO 19903, separate provisions. The detailed design provisions are mostly adopted from the former Norwegian concrete standard NS 3473. The use of an alternative detailed reference standard is still acceptable provided that specific requirements similar to those described in Ch. 3.3 are satisfied. The level of safety using a non-DNV GL code shall be as required by DNV GL standard.

The DNV GL standards address many aspects which can be applied to SFTB design. Similar to the ISO 19900 standards, additional provisions for vehicle and rail traffic loads are also needed for the DNV GL standards. It is noted that the former DNV rules have been applied in some previous SFTB studies.

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3.2.4 Other relevant standards

In general, every country can refer to national specific standards, as for example Korea refers to several documents (Korean Design Standards and Korean Construction Specification). In this chapter, the standards used as a reference in several countries are mentioned.

Without further gap analyses, the list of relevant standards may also include the AASHTO standards, British Standards and the API Offshore Structure Standards. The AASHTO LRFD Bridge Design and Construction Specifications[3-3] are widely used and accompanied with the FHWA tunnel manual[3-4] applicable for immersed tunnel design and construction. The British Standard BS 8110 is, like the Norwegian standard NS 3473, superseded by the Eurocodes but is still in use in a number of Asian countries. BS 8110 has been used as a reference standard in the detail design of two offshore concrete platforms, the Sakhalin II Gravity Base Structures, but with supplement of specific interpretations and additional provisions, including the NS 3473, to make it applicable to offshore structures.

The 2010 *fib* Model Code for Concrete Structures is also mentioned, in particular for the verification of limit states associated with durability, robustness and sustainability. In addition, the Model Code focuses on verification assisted by numerical methods and by testing, that is of particular importance in the case of the SFTB.

3.3 Supplementary SFTB provisions

Until codified SFTB provisions are in place, supplementary provisions shall be developed on a project-by-project basis. The additional provisions will to some extent depend on chosen reference standards. Some of the most relevant design aspects likely not covered are listed below, subdivided into general design aspects and concrete design aspects. The listed aspects are not exhaustive but can be used as a check-list when establishing project-specific design premises. Further guidance is also given in Ch. 6 and 7.

3.3.1 General SFTB design aspects

Supplementary provisions can be derived from recognised provisions for analogue structure types, e.g. immersed tunnels, fixed and floating offshore installations, or by direct reference to them. Perhaps the most relevant resource for supplementary SFTB provisions is the Norwegian bridge design provisions Handbook N400[3-5]. The special design rules for submerged floating tube bridge contained herein were originally developed for the Høgsfjord SFTB project and have been revised and updated for the E39 ferry replacement project, most recently in 2015.

Net buoyancy: An SFTB is partly a weight-buoyancy stabilised structure so that a robust regime for the net buoyancy will have to be established. The design should be based on upper and lower characteristic values for self-weight and buoyancy including geometric and density tolerances. Weight and ballast reserves for the tube bridge should be specified and coupled with a weight control programme. The design assumptions on permanent self-weight can be checked by the possibility of weight tuning while afloat, e.g. in dock. Parts of the variable self-weight to consider may be time-dependent such as marine growth and water absorption of concrete and solid ballast.

Accidental actions: Potential accidental scenarios have to be identified and their probability and characteristic value, as far as possible, be determined by risk analyses taking into account

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any risk mitigating measures. Examples of site-specific scenarios are typically ship impact, submarine impact, sinking ship, unintended dropped or dragged anchor, land/sea slide and tsunamies. For the generic scenarios such as fire, explosion and vehicle impact common and well accepted design criteria can normally be adopted from the mandatory tunnel regulations.

Floating stability: For a free-surface stabilised SFTB the minimum freeboard should be greater than zero for the most onerous combination of characteristic wave and tide condition.

Temporary phases: For temporary floating phases during construction, assembly or marine operations, sufficient positive stability and reserve buoyancy should be ensured. For pontoons and auxiliary buoyancy units stability should be considered both intact and in damaged condition.

Mooring system: Design criteria for tether slack control during dynamic motion of the SFTB will have to be defined in order to warrant global stability. Normally, the requirement on net buoyancy is that tension shall be maintained in all tethers for the most onerous combination of unfactored characteristic loads, whereas tether slack can be accepted in the ultimate limit state without causing structural failure or tether unlocking. The risk of unexpected loss of a tether or pontoon should be assessed as part of the risk analysis.

3.3.2 Concrete design aspects

Provided that the reference standard for concrete design covers the more general aspects of marine structures such as durability in a marine environment, post-tensioning including partial prestressing and discontinuity regions, additional provisions specifically required for the design of an SFTB include:

Design model for shell members: Independent of cross-sectional shape, the tube bridge will act as a shell structure subjected to in-plane and out-of-plane load effects. A consistent design method considering equilibrium and compatibility of the force components producing membrane stresses (N_x, N_y, N_{xy}, M_x, M_y and M_{xy}) will thus be needed. As a rigorous method for considering out-of-plane shear (V_x, V_y) may not exist in common practice yet, a consistent approach for the design for transverse shear will also be required in which the coexistence of membrane forces and directionality of the principal transverse shear are accounted for. A codified design method meeting such requirements can be found in[3-6] and further information may be found in the literature about the Modified Compression Field Theory (MCFT), developed by Veccio and Collins at the University of Toronto.

Requirements for tightness control: Control of water-tightness is essential for the performance and integrity of a concrete SFTB in in-place and temporary conditions. Strict requirements to tightness to avoid membrane tension and through-thickness cracks shall be considered under service conditions. Adequate water-tightness or leakage criteria need also to be established for ULS and ALS to ensure structural resistance and integrity. The tightness criteria in NS 3473[3-6] have been applied to many offshore concrete platforms worldwide. Corresponding requirements can be found in DNVGL-OS-C502[3-7] and EN 1992-3:2006[3-8].

Provisions related to water pressure in pores and cracks: Potential water pressure in pores and cracks may affect both action effects and resistance and the effect will also have to be reflected in the detailing of the reinforcement. Provisions and detailing rules for handling the effect of pore and water pressure may be adopted from NS 3473[3-6] and DNVGL-OS-C502[3-7].

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Design principles for fatigue: The fatigue loading for an SFTB will depend on the actual structural design and on the environmental site conditions (wave, current and tides). Although fatigue is generally not expected to govern the tube design, a thorough fatigue design check is something which should be considered. This will require a fatigue design approach applicable for elements subjected to in- and out-of-plane action effects and capable of predicting fatigue resistance for all relevant fatigue failure modes such as concrete in pure compression or compression/tension, transverse shear considering both shear tension and shear compression, reinforcement considering both main bars and stirrups including bond failure, and prestressing reinforcement. Fatigue strength formulas for these failure modes can be adopted from DNVGL-OS-C502[3-7] or NS 3473[3-6] in which also the inferior concrete fatigue performance under water is covered. Some guidance on capacity predictions for 2-dimensional members can also be found in the latter.

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4. Functional requirements

4.1 Introduction

Most functional requirements for submerged floating tube bridges will refer to regular tunnels or immersed tube tunnels as well. This chapter summarises these general functional requirements and covers the specific requirements or points of attention for SFTBs.

4.2 Design life

The SFTB has to fulfil its functions during the specified design life. The functional requirements should specify the design life for the tunnel system in general and for its components (main load-bearing system, replaceable parts, technical installations).

The typical design life for the main load-bearing system of tunnels is 100-120 years. For replaceable parts and technical installations shorter design life periods are common, e.g. 25 years for technical installations (pumps, switches), 50 years for railings, 10-15 years for pavement.

For essential non-replaceable parts, specific requirements are appropriate in order to guarantee its functional life under all circumstances. Waterstops can serve as an example: the material is vulnerable at elevated temperatures and with aggressive fluids. Therefore, during fire, the rubber temperature should stay well below critical values: 80 °C is a safe value. On the other hand, waterstops should be protected against very low temperatures (e.g. due to contact with pressurised gas) as well. They should be resistant to aggressive materials such as petrol, motor oil, diesel and seawater.

Although tethers, moorings and their connections are designed to be replaceable, it is recommended that a design life equal to that of the tunnel system is specified.

4.3 Traffic

Generally speaking, the tunnel should enable safe passage by traffic with a given design speed, volume and composition. This requirement specifies:

- Design speed
- Capacity
- Composition (road and/or rail, passenger and/or cargo traffic)
- Dangerous goods (related to e.g. ADR classes); restrictions are often placed on dangerous goods in tunnelling projects and this applies more so on SFTBs

These requirements relate to requirements for the layout and alignment, installations and systems and operation and maintenance.

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4.4 Layout and alignment

Requirements regarding layout and alignment specify the geometry of the tunnel in transverse and longitudinal directions. They define the space for traffic, galleries for equipment, forced ventilation, evacuation and maintenance. Most importantly, the alignment and the cross section strongly relate to the requirements regarding traffic.

The portals shall be at an elevation where the risk of accidental flooding is minimised.

The ventilation demand is strongly related to the tunnel length – as SFTBs are generally longer, the space demands for ventilation are higher. Consideration may need to be given to intermediate ventilation shafts, which can also be used for evacuation.

Depending on the requirement for the design, SFTBs will have one or several tubes. A twin-tube configuration offers a solution for safety purposes. In this case, interconnections should be guaranteed also for personal safety.

Requirements regarding the cross section specify:

- Number of lanes, clearance/traffic gauge and roadway geometry, shoulder width
- Space for galleries and other principal spaces

Requirements regarding the alignment specify:

- Clearance for vessel traffic
- Cross Slopes
- Vertical alignment gradient (related to transport modes)
- Reference lines, vertical and horizontal curve radii

Especially for SFTBs, horizontal and vertical curvatures enhance the lateral stability of the structure. The minimum radius will still be dictated by traffic alignment requirements. As SFTBs usually have gentler gradients, there will be less need for crawler lanes.

For SFTBs, additional requirements have to be given with respect to spacing of floaters, moorings and/or tethers, considering:

- Vessel clearance (both ships and submarines)
- Distance between pontoons

Regarding vessel clearance, some guidance on this topic can be found in e.g.[4-1]. As this document mainly focuses on the ship's safety and manoeuvrability, additional clearance has to provide sufficient reliability with respect to the structural components.

From the point of view of public perception or acceptance of the SFTB concept, visibility from the shore could, in some cases, be considered in relation to the depth of the SFTB.

4.5 Tunnel structure

Generally speaking, the structure should carry all operational, design and accidental actions showing appropriate response (i.e. accidental loads may result in damage, whereas operational loads may not) during the construction period and the specified service life.

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- Design working life (permanent structure, replaceable components)
- Carrying loads during construction and service life: For actions during construction, the metocean conditions are relevant. Actions during service life relate mainly to soil and water pressures (including surcharge), traffic loads and temperature cycles.
- Ballast chambers
- Withstanding accidental loads: Accidental loads referred to fire, explosion, vessel related risks like sinking ship or dragging anchor, collision or derailment.
- Watertightness: Crack width control, joint seals
- Durability
- Permanent structures and replaceable components; specification of life to first maintenance

By their nature, SFTBs are subject to actions from buoyancy, waves and currents. These actions are covered in more detail in Ch 5 and 6.

Dependent on the location of the SFTB, it might need to withstand additional accidental loads from:

- Ship and submarine impact
- Dropped anchor and vessel mooring lines
- Icebergs and sea ice
- Seismic actions and tsunamis – design level and accidental
- (Underwater) landslides and related hydrodynamical effects
- Movements on fault-lines
- Terrorism related actions

With respect to watertightness, greater control of compression zones and crack width limits may be appropriate.

With respect to durability, requirements may be given for fouling and corrosion protection in the marine environment. External parts with limited life such as tethers should be designed to be replaceable.

The shore connections shall either enable movement or transfer the end restraint loads. The connections shall be designed to facilitate and minimise maintenance.

4.6 Installations and systems

Requirements regarding installations and systems address all mechanical, electrical, plumbing and safety systems in the tunnel. In addition to direct specifications for the systems, the requirements should address reliability and availability of each system (e.g. by requiring redundancy).

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Typical installation and systems are:

- Power supply system
- Lighting
- Drainage system: Both drainage of rain/leakage and drainage of fire extinguishing water should be specified, in terms of flow rate, storage, pump capacity
- Smoke extraction system: This requirement is related to the cross-section design and the length of the tunnel
- Safety systems: Special attention should be paid to intercompatibility of all subsystems and components. Automatic warning systems
- Traffic management systems
- Security control
- Toll systems
- Supervisory Control And Data Acquisition (SCADA) system

For SFTBs, both drainage/pumping systems and smoke extraction/ventilation will probably need a higher capacity and fail-safe operation.

4.7 Safety

The risk of failure of a single component or of the complete system shall be defined.

The degree to which an SFTB is to withstand flooding shall be defined.

Redundancy should be applied into the structure, its connections and the seals and the stabilization system (e.g. surface pontoons, tethers, moorings and foundation anchors).

Active and/or passive systems may be included to mitigate the risk of flooding.

Essential safety systems such as pumping, ventilation and fire extinguishing systems shall be configured to remain operational during flooding events.

Safety studies may require intermediate surface access (for example a device to access to a large floating island that could serve as an emergency service landing point)

4.8 Environmental requirements

Environment-related requirements include e.g.:

- Impact on surroundings
- Sustainability
- Nuisance

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For SFTBs additional requirements may cover:

- additional sedimentation or scour
- shadowing
- influence on currents
- water-borne noise
- electro-magnetic fields
- the influence of corrosion protection systems on marine ecosystems

4.9 Operate and maintain

These requirements address normal operation, maintenance and emergencies. Aspects are:

- Reliability and availability of the transport system
- Traffic operations during maintenance and emergencies
- Inspectability and maintainability of the tunnel
- Facilities for emergency and rescue

As a tunnel is a confined space, special attention has to be paid to evacuation and to accessibility by emergency services. Additional restrictions will apply for maintenance situations in SFTBs.

Requirements should be defined for periodic inspection and maintenance of external parts of the stabilization system.

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5. Site conditions

The following conditions will be most influential for SFTB design.

5.1 Bathymetry and seabed condition

The choice of vertical stabilisation for SFTB design is dependent on the water depth. In order to establish a sufficient and satisfactory basis for the design of the anchor system a comprehensive seabed soils investigation will be required. The investigation should comprise a detailed geophysical survey including bathymetry, subsea seismic profiling and preliminary geotechnical borings. Further, advanced geological and geotechnical investigations including drilling with soil sampling, optional rock core sampling and laboratory testing of the retrieved soil and rock material should be conducted as the project progresses through detailed design.

Based on the results from the seabed investigations, a geological interpretation should be carried out in order to establish a general description of the seabed geological situation along the bridge corridor.

The outcome of the seabed survey work will form the basis for the geological and geotechnical evaluations required to establish a safe and optimum anchor system. The evaluations anticipated to be required in this respect comprise issues such as:

- Slope stability evaluations
- Geohazard identification
- Evaluation of different anchor concepts such as piles, gravity anchors, suction anchors or hybrid combinations of these foundation types
- Foundation capacity
- Foundation settlements
- Foundation installation methods

For early phase studies, data on bathymetry and distance can be obtained from publicly available sources.

5.2 Landfall geology and soil condition

For design of the landfall tunnels and the landside of the shore approaches engineering geology evaluations have to be performed. This includes:

- Mapping of rock types and soil overburden
- Mapping of weakness zones and faulting
- Evaluation of possibility for landslide and avalanches

Thereafter, methods for tunnel driving and the SFTB anchorage can be designed.

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5.3 Seismic conditions

The seismic activity at the bridge site should be evaluated. This includes acquiring data regarding:

- Previous earthquakes
- Faults
- Ground motions, based on seismic site effects, intensity scales and magnitude scales

These data are usually available through national geological survey organisations, or national standards in the case of ground motions.

5.4 Wind conditions

Wind conditions are mostly relevant for determining the wind generated sea movements, as the SFTB will typically have small surfaces exposed to wind loading, if any. Parameters in wind dynamics such as turbulence and coherence are thus less important for the SFTB than for a bridge. The direction of the wind, with the corresponding wind speed and storm duration are the most relevant factors to investigate. National engineering standards will give an indication of the dimensioning wind speeds, but site measurements for the crossing have to be performed.

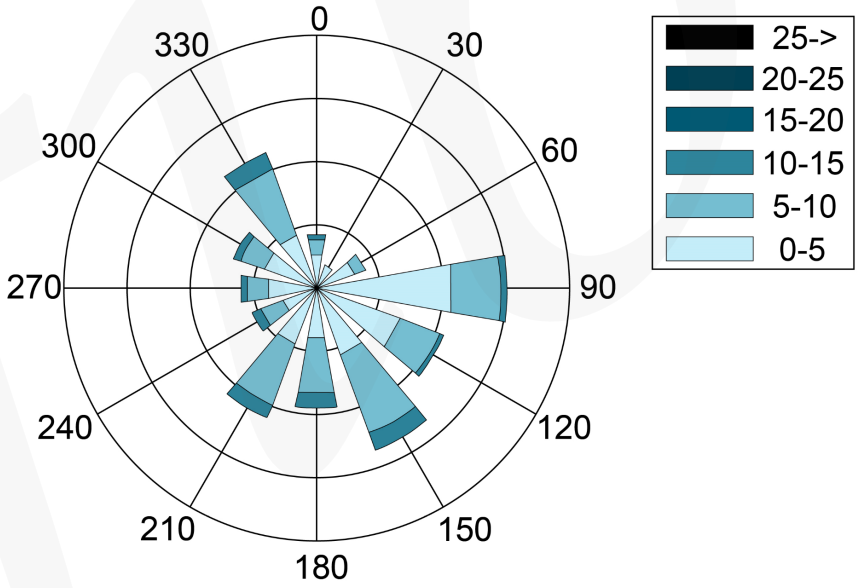


Fig. 5-1 example of wind speed rose from Bjørnafjorden floating bridge Metocean design basis - Statens vegvesen

5.5 Wave conditions

Wave loading is a key premise for design of an SFTB. The waves can be divided into two types: wind generated sea and ocean swell. Site wind speed measurements will have to be performed in the SFTB corridor to supplement historical data, together with correlation in time and position, in order to capture the wind sea contribution and ocean swell contribution to the total wave height. Nearshore wave transformation modelling can be undertaken using software such as Mike 21.

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Before data is available, the formulae such as the one below from Norwegian Standard NS9415 can give a reasonable estimate of the wind sea based on 10 min mean wind speed, U, and fetch length, F.

Adjusted wind speed U_A [m/s]:

$$U_A = 0.71U^{1.23} \tag{5-1}$$

Significant wave height, H_s [m], and corresponding wave period T_p [s]:

$$H_S = 5.112 \cdot 10^{-4} U_A \sqrt{F} \tag{5-2}$$

$$T_p = 6.238 \cdot 10^{-2} (U_A \cdot F)^{1/3} \tag{5-3}$$

By using a JONSWAP spectrum to calculate the wave forces, the peak shape parameter can be estimated as:

$$\gamma = 44(H_S / F)^{2/7} \tag{5-4}$$

An SFTB may be located in a position exposed to ocean swell waves. Swell waves can be estimated based on a wave model analysis with hindcast data for ocean waves.

5.6 Current

As for wave and wind, current has to be measured to obtain reliable data. Current can be divided into three main contributions:

- Wind generated current
- Tidal current
- Density difference driven current
- Presence of rivers

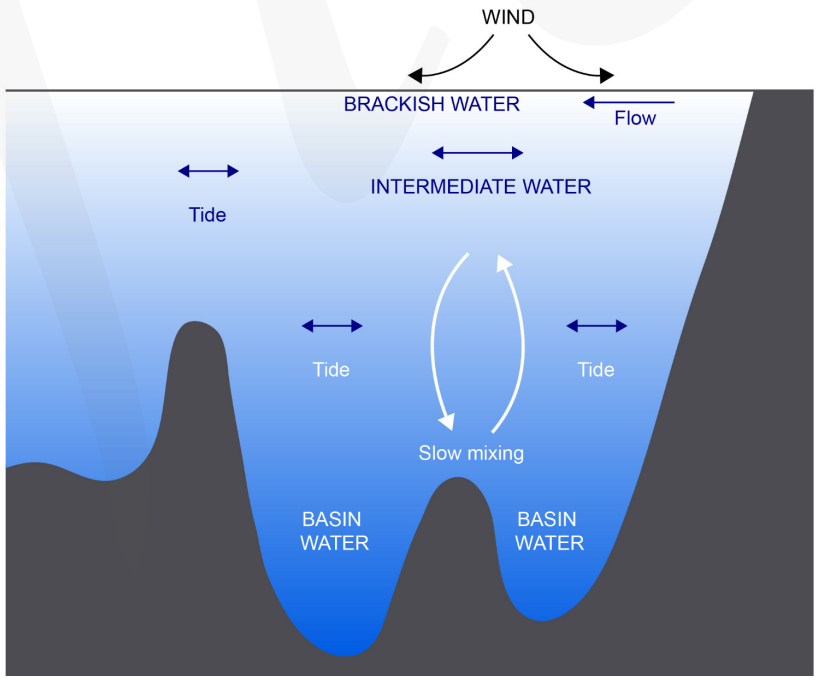


Fig. 5-2 Fjord current processes

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Wind generated current can, at the surface, be estimated at 2% of the wind speed according to NS9415. The distribution of current speed as a function of water depth is dependent on the layering of the water. With homogenous water, more of the energy will go to mixing, while layering in the water will cause more of the energy to be transferred into current in the upper layers, as shown for example in Fig. 5-2.

Current data can sometimes be provided by oceanographic service companies.

Strong tidal currents are mainly an issue for narrow straits. For fjords and straits tidal current can be estimated by the following formula:

$$\overline{u}_t = 2\pi \frac{A}{S}\frac{h_o}{T}\cos\left(2\pi\frac{t}{T}\right) \tag{5-5}$$

$$U_{max} = 2\pi \frac{A}{S}\frac{h_o}{T} \tag{5-6}$$

Where A is the water surface area 'inside' the crossing, S is the cross-sectional area at the crossing, h_0 is the tidal amplitude and T is the period, which can be assumed as 12,4 hours.

5.7 Temperature

The water temperature dependent on water depth should be included in the site measurements and should be established year-round.

5.8 Water level variations

The water level variation is defined as an astronomical component and a surge component combined. The surge component includes effects from low/high atmospheric pressure and storm surge, while the astronomical component is independent of the environmental conditions. Since the confidence on the two components varies, that is, astronomical tide values are much more predictable than the surge components are, appropriate safety factors can be applied separately to each component until accurate data is obtained. Tidal data give Lowest Astronomical Tide (LAT), Mean Sea Level (MSL) and Highest Astronomical Tide (HAT) and tidal data for coastal areas are published by national hydrographic services. When the surge component, based on site measurements, is added, the lowest water level (LWL) and highest water level (HWL) for different return periods can be determined.

5.9 Sea water density

The seawater density ρ_ω depends on sea water temperature T (°C) and salinity S (‰) according to the formula;

$$p_w = 1'027 - 0.6(T-10) + 0.75(S-35) \tag{5-7}$$

Combining a salinity range with a temperature range the low, high and mean value sea water density can be calculated. For sea water, the variation of density from temperature is roughly in one order of magnitude less than the variation in salinity ~1 ‰ vs 1%.

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For lakes, studies done for the Lugano lake (refer Ch. 14.2) have demonstrated that the temperature has a considerable seasonal variation until a depth of ~20 meters from the surface and becomes constant after that, as shown in Fig. 5-3.

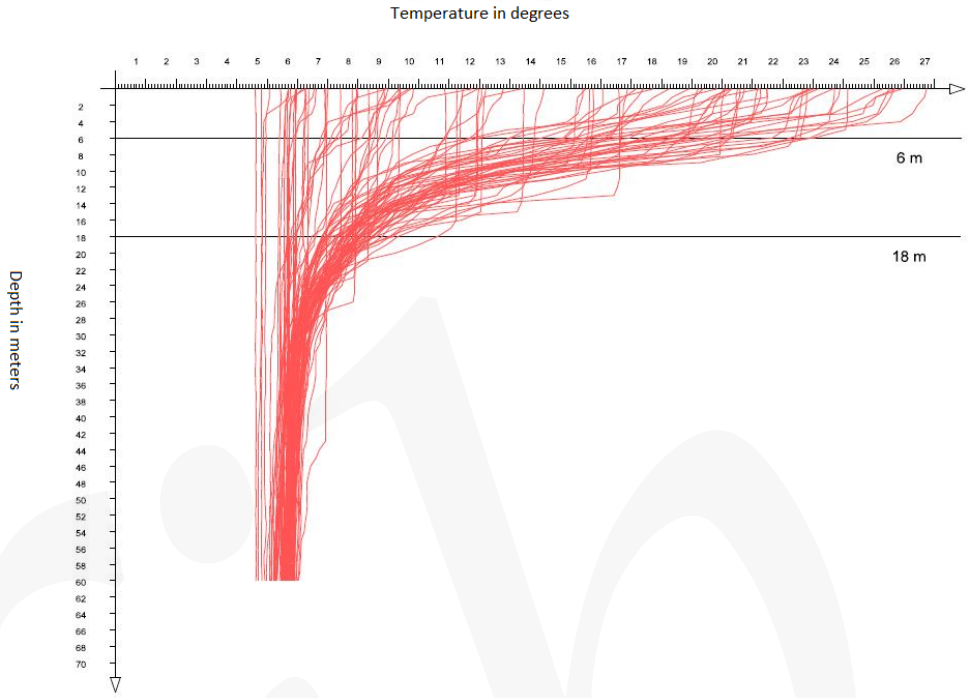


Fig. 5-3 Temperature seasonal variation with depth, study for the Lugano lake –Muttoni & partners, Dr. techn. Olav Olsen

5.10 Ship traffic

An SFTB can provide a crossing with minimal disturbance to ship traffic. Maritime authorities will set the vertical clearance and fairway width (see chapter 4 for references). Ship traffic can be evaluated based on Automatic Identification System (AIS) data, which all international voyaging ships with 300 or more gross tonnage and all passenger ships regardless of size are obliged to report. A risk analysis based on ship traffic data can determine the energy level for evaluating the design accidental load for ship impact with the SFTB.

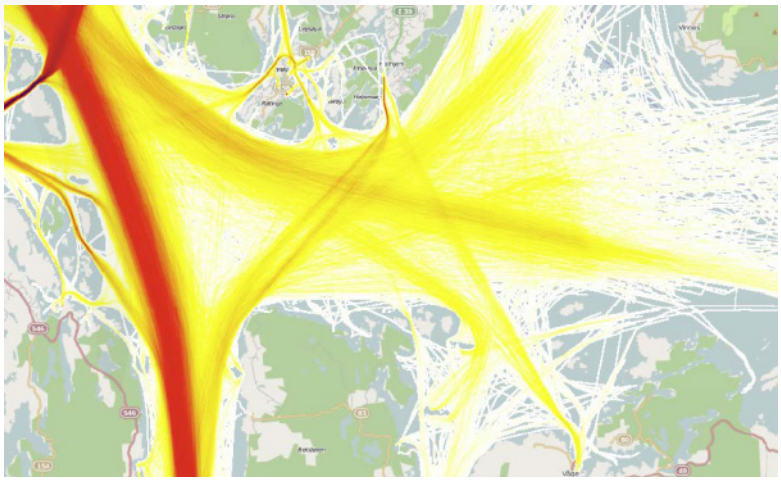


Fig. 5-4 AIS data example, Bjørnafjord – Statens vegvesen

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5.11 Other actions

The following loads dependent on site conditions are relevant to investigate, but are normally less important for design:

- Sea level rise (increased displacement can be compensated by ballasting down a pontoon supported SFTB. The effect of increased water pressure is dependent on cross-section)
- Snow and icing
- Marine growth
- Earthquakes

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6. Actions and action effects

6.1 Self-weight

The self-weight consists of three main components:

- Permanent self-weight, G_{perm}
- Buoyancy, B
- Variable self-weight, G_{var}

The tolerances in the calculated weights within each component will vary, and the mean, maximum and minimum values for each component should be established.

6.1.1 Permanent self-weight, G_{perm}

- Permanent self-weight consists of the following elements:
- calculated weight of structure (+/- 0,5 %)
- calculated weight of structural elements (+/- 0,5 %)
- weight of permanent ballast (+/- 0,5 %)
- weight of permanent asphalt (+/- 15 %)
- weight of permanent equipment (+/- 15 %)

The numbers in parenthesis are the tolerances (weight contingencies) proposed by the Norwegian handbook N400.[6-1] Another standard which can provide guidelines on design allowances and weighing procedure and tolerances is ISO 19901-5:2016 Specific requirements for offshore structures, Part 5: Weight control during engineering and construction.[6-2] The SFTB should be designated in a weight control class A: High definition of weight and CoG. Class A weight projects shall:

1. regardless of the source, have full traceability of all weight and CoG data;
2. record weight and CoG data using a relational database from the commencement of detail engineering, with integration of suppliers', fabricators' and weighing results into the system;
3. verify the calculated weight and CoG of assemblies, modules or topsides by means of physical weighing;
4. update weight data per weight item produced during the design phases to "as-built" status during the fabrication.

6.1.2 Buoyancy, B

Buoyancy is calculated based on mean sea water density and submerged volume. Tolerance in sea water density is dependent on the site measurements available as described in Ch 5.

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6.1.3 Sum permanent action, G

The resulting permanent action is the difference between permanent self-weight and buoyancy, $G_{perm} - B$, where positive value acts downwards. The most unfavourable value shall be used in load combinations. Maximum and minimum values for permanent self-weight and buoyancy are dependent on the tolerances as described in sections. 6.1.1 and 6.1.2.

$$G_{max} = G_{perm.max} - B_{min} \tag{6-1}$$

$$G_{min} = G_{perm.min} - B_{max} \tag{6-2}$$

6.1.4 Variable self-weight, G_{var}

Variable self-weight consists of the following elements:

- weight of marine growth. Marine growth, characterised by its thickness, roughness, density and variation with depth and time, will give a weight contribution, but may also influence hydrodynamic instability as a result of vortex shedding. Unless more accurate data is available, marine growth can be estimated by Table 6.1[6-3] in specific regions but may have to be established from local surveys in other parts of the world.
- weight of water absorption in structural concrete, taken as 1 % of concrete weight unless project specific values for the relevant concrete mix are provided
- weight of water absorption in solid ballast, taken as 1% unless more accurate data for the ballast material are provided
- weight of variable asphalt
- weight of variable equipment

Table 6-1 Thickness and weight of marine growth, NORSOK

Water depth	56° to 59° N			59° to 72° N		
	mm	In air kN/m^2	Submerged kN/m^2	mm	In air kN/m^2	Submerged kN/m^2
Above +2 m	0	0	0	0	0	0
+2 to -40 m	100	1.3	0.3	60	0.78	0.18
Below -40 m	50	0.65	0.15	30	0.39	0.09
Values are calculated with a specific weight in air of 13 kN/m^3 Roughness height may be taken as 20 mm						

6.1.5 Traffic loads

Traffic loads are given by the applicable regulations.

The effect of traffic loads can, in principle, be reduced to half compared to bridges in air, by adjusting the self-weight to result in approximately equal net downwards and upwards resulting vertical forces, i.e.:

$$\left|G_{perm.max} - B_{min} + G_{var.max} + T_{max}\right| \approx \left|G_{perm.min} - B_{max} + G_{var.min}\right| \tag{6-3}$$

For high traffic loads, typically for rail, it is important to ensure sufficient stiffness in the cross-section to avoid large rotations, as for the design of long-span bridges.

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6.2 Imposed deformations

6.2.1 Post-tensioning loads

Post-tensioning, both in the longitudinal and transverse direction, is a key element in design to meet water tightness criteria. It is recommended to keep the level of post-tensioning under 10 Mpa average compressive stress in the cross section. Post-tensioning loads may be neglected from the global analyses. Possible restraint forces and bending should be considered in a detailed design stage.

6.2.2 Creep and shrinkage

Creep deformation is a result of long-term deformations from permanent loading. These deformations may result in restraint forces. An SFTB is trimmed vertically in an almost neutral position, which leads to very small bending moments and deflections from permanent loading. Creep is therefore insignificant for these loads. Relevant creep loads are axial loads from post-tensioning and external water pressure. The creep deformation will lead to some restraint forces, but in early phases of the design, these forces are often neglected.

Shrinkage deformations are limited due to swelling on a wet surface. As for creep loads, shrinkage loads can usually be neglected in early phases of design.

6.3 Environmental actions

6.3.1 Quasi-static current actions

The distributed sectional drag force on the tube is:

$$F_c = \frac{1}{2} p_w \cdot C_D \cdot D \cdot U_c^2(z) \tag{6-4}$$

where ρ_ω is the water density, C_D is the current velocity, D is the diameter/characteristic width and U_c is the current velocity. Drag coefficients for different cross-sectional shapes can be found in DNVGL-RP-C205, analysed with CFD or measured experimentally in a wind tunnel or tow-tank.

With a twin-tube cross-section, a reduced inflow velocity u on the shielded tube can be estimated by Schlichting's wake formula using Blevins' approach with a virtual origin six diameters in front of the cylinder.[6-4]

$$\frac{u^2}{U_c^2} = \left(1 - 2 \cdot 0.95 \sqrt{\frac{C_D \cdot D}{CDBT + 6D}} \right) \tag{6-5}$$

where C_D is the drag coefficient for front tube and CDBT is the Centre Distance Between Tubes.

The sectional drag force on each pontoon is given by:

$$F_c = \frac{1}{2} \rho_w \cdot C_D \cdot S_w \cdot U_c^2(0) \tag{6-6}$$

where S_w is the projected under water area of the pontoon and $U_c(0)$ is the surface current velocity.

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6.3.2 Vortex Induced Vibrations (VIV)

Vortex shedding is caused because the flow is unable to follow along the surface of the structure and separates from its surface, as shown in Fig. 6-2. This flow separation creates a vortex which causes a local increase in rotational velocity. Higher flow velocity yields a reduction in dynamic pressure, which creates an attraction force by the vortex on the structure.

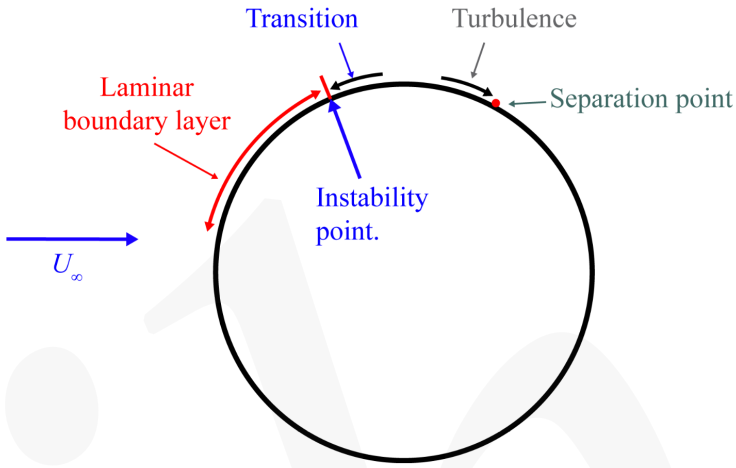


Fig. 6-1 Flow around a circular cylinder, Statens vegvesen

The position of the separation point alternates between the top and bottom of the cylinder, and creates a vortex pattern, as illustrated in Fig. 6-2. These vortices create a force fluctuation due to the previously mentioned pressure changes. Due to the spatial properties of the problem, the forces for the inline flow direction will have a period of half the shedding period, whereas the forces in cross flow will have a period identical to the shedding period.

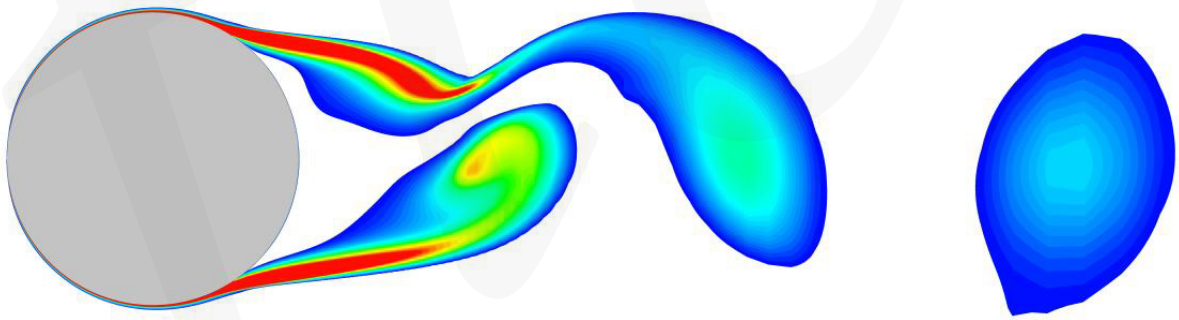


Fig. 6-2 Vortex pattern behind a circular cylinder, Statens vegvesen

For a fixed cylinder, the correlation length of the vortex shedding in the span direction is only 1-3 diameters, and will therefore not produce resonance issues. When the structure is free to oscillate, however, the vortex shedding frequency will change towards the natural frequency of the structure and create a lock-in effect between the structure motion and the vortex shedding. This lock-in will increase the correlation length significantly, and resonance will occur. The phenomenon of lock-in may only appear when the vortex shedding frequency is sufficiently close to the natural frequency of the structure. The most relevant design guideline is DNV-RP-F105 for subsea pipelines[6-5], where values for reduced velocities where lock-in may occur are given for one cylinder. Corrections in Strouhal number for two cylinders can be based on[6-6].

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6.3.3 Wave actions

The sea state can be expressed by a power spectrum, e.g. the JONSWAP spectrum, with a directional distribution. Hydrodynamic response analyses can be carried out using different methods, typically:

- Analytical method. Dynamic response analysis from wave loading is a modal analysis performed for an Euler- Bernoulli bending beam in the frequency domain. Potential flow theory can be used to express the load transfer function, which contains both first order wave forces and slowly varying drift forces. The spatial variations of the wave loading due to wave direction and from the curvature of the structure are applied by the use of a spatial correlation function. One of the main advantages of this method is its ability to account for the geometry of the structure, as well as the short-crested properties of the sea state, when evaluating the load characteristics[6-7, 6-8]. Correlation lengths for some action effects are illustrated in Fig. 6-3
- Hydrodynamic response analyses in the frequency domain
- Hydrodynamic response analyses in the time domain

Resonance issues are dominant for structures with the high slenderness that an SFTB typically has. As the wave loading, which is not a part of ordinary bridge design, is the most dominant action effect, this section elaborates on the different wave action effects.

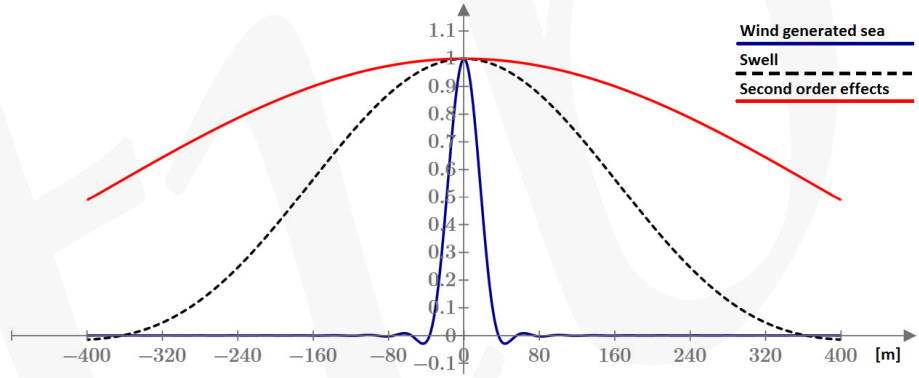


Fig. 6-3 Correlation effect of different load types in space, Dr. Techn. Olav Olsen

6.3.4 Sea states

The statistical properties of design sea-states are characterized by a model wave spectrum defined in terms of significant wave height H_S and wave spectrum peak period T_p. In an extreme sea state the wave spectrum can be assumed to be composed of two main wave systems, wind sea and swell.

Wind sea is mainly generated from local wind action where the sea state characteristics (H_S, T_p) are determined by the wind speed and the fetch length. Wind sea waves cause loads at periods close to the wave spectrum peak period as well as slowly varying low-frequency loads at higher periods. Extreme wind sea waves can also cause sum-frequency wave actions exciting higher vertical eigenmodes.

Swell can excite resonant response of the bridge. Swell may be due to wind induced waves generated by distant storms offshore and propagating to the nearshore SFTB without being affected by the local wind. Swell waves have long periods and large wavelengths. Swell

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usually has a narrow range of frequencies, typically with periods in the order of 12-20s. Swell has dispersed from its area of generation so the waves take on a more regular shape. This means that the swell wave spectrum is highly peaked. Swell sea states have less directional spread than wind sea states.

More details on calculation of a JONSWAP wave spectrum can be found in DNV-RP-C205[6-9].

6.3.5 First order wave actions

6.3.5.1 Wave force regimes

In general wave induced loads on tube and pontoons have contributions from i) inertia actions due to pressure gradients and unsteady flow, and ii) drag actions due to flow separation.

In the example in Fig. 6-4, the study for the Bjørnafjord of wave force regimes for tube and pontoons in extreme wind sea and swell conditions is reported. Red markers are for wind sea and green markers are for swell. Filled markers indicate pontoons and open markers indicate tube. Wind sea H=5 m. Swell H=1 m. Tube D=15 m. Pontoon L=80 m. For both, the wave force is inertia dominated. For swell sea conditions a simplified load model (Morison's equation) is applicable, while for extreme wind sea conditions wave diffraction effects are important, in particular for the pontoons.

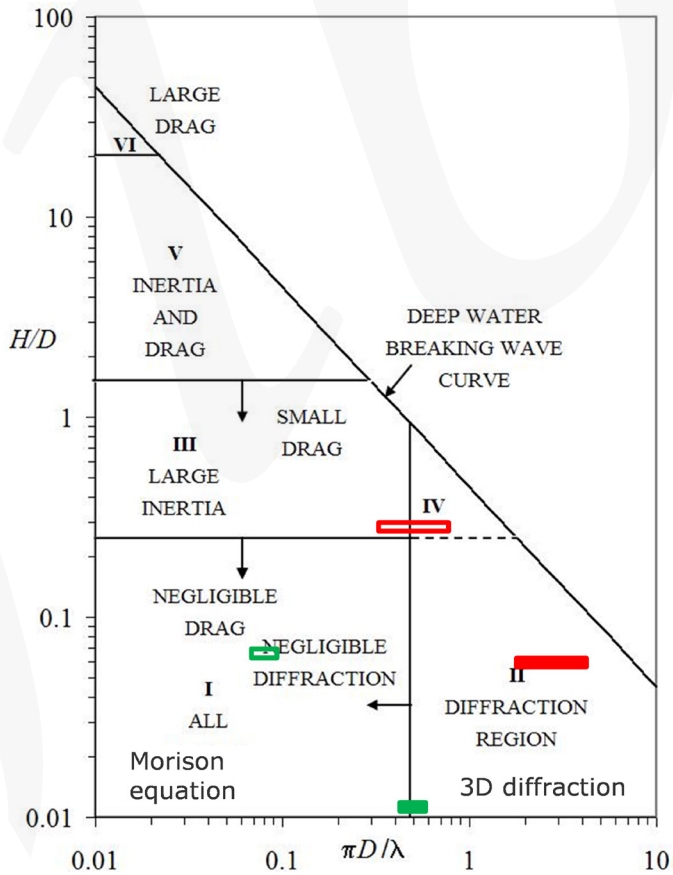


Fig. 6-4 Example from the study for a SFTB for the Bjørnafjord, DNV GL/Statens vegvesen

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6.3.5.2 Morison's equation

For slender objects where the cross-sectional dimensions are much smaller than the wave length, Morison's equation can be used to calculate wave actions. The wave action is described as a sum of an inertia force proportional to fluid particle acceleration ($\dot{v}$) and a drag force proportional to the square of fluid particle velocity (v^2). Morison's equation for the sectional load on a fixed slender object is:

$$F = (1+C_A)\rho_w \cdot A \cdot \dot{v} + \frac{1}{2}\rho_w \cdot C_D \cdot D \cdot v^2 \tag{6-7}$$

where A is the cross-sectional area. For a circular tube $A = \frac{\pi}{4}D^2$

Morison's equation for a moving object is

$$F = -\rho_w \cdot C_A \cdot A \cdot \ddot{r} + (1+C_A)\rho_w \cdot A \cdot \dot{v} + \frac{1}{2}\rho_w \cdot C_D \cdot D \cdot v_r^2 \tag{6-8}$$

where $\ddot{r}$ is the acceleration of the object and v_r is the relative velocity between wave particles and the object. The first term on the right-hand side is the added mass force acting in the opposite direction to the object acceleration. Linear Airy wave theory (DNV-RP-C205) may assumed to be valid and is the basis for calculating the wave kinematics v and $\dot{v}$ (although higher order wave theories may be appropriate).

6.3.5.3 Morison inertia force

The Morison inertia exciting force per unit tube length and per unit wave amplitude for a fixed tube can be written:

$$F_M = (1+C_A)\rho_w \cdot A \cdot \omega^2 \cdot e^{-k \cdot d} \qquad \left[\frac{\text{kN}}{\text{m}} \cdot \frac{1}{\text{m}}\right] \tag{6-9}$$

where ω is the wave angular frequency, $k=\omega^2/g$ is the wave number and d is the depth of the centre of the tube.

6.3.5.4 Morison drag force

The Morison drag force (for fixed tube) per unit length of the tube can be written as:

$$FD = \frac{1}{2}\rho_w \cdot C_D \cdot V^2 \cdot D \qquad \left[\frac{\text{kN}}{\text{m}}\right] \tag{6-10}$$

where C_D is the drag coefficient. The fluid particle velocity at the centre of the tube is $v = \omega \frac{H}{2} e^{-k \cdot d}$ where D is the outer diameter of the tube and H is the wave height.

The drag force is 90 degrees out of phase with the inertia force in linear wave theory. At the wave crest and wave trough there is only drag force contribution and when the wave surface elevation is zero (midway between crest and trough) there is only inertia contribution. The magnitude of the inertia force relative to the drag force for a structure close to the sea surface is given by the ratio 20D/H, which means that the wave induced force on the tube is inertia dominated.

Morison's equation can be applied for the tube, the Vierendeel structure and the connection columns between tube and pontoons. For added mass and wave damping on the pontoons, a slender body approximation can be made applying two-dimensional coefficients presented by Vugts [6-10].

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However, these approximations have to be verified against results from three-dimensional diffraction analyses.

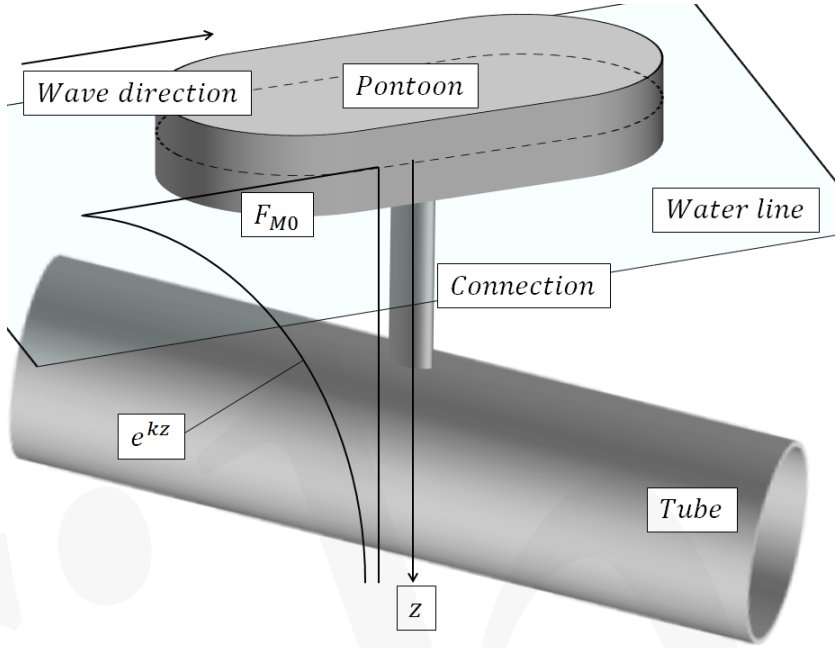


Fig. 6-5 Sketch of tube, pontoon and connection, illustrating the variation of the wave force with depth *z*, Statens vegvesen

6.3.5.4 Wave force on connection, Pontoon supported SFTB

Integrating over the height of the connection the wave action on each connection can be expressed as:

$$F = (1+C_A)\rho_w \cdot V_{Conn} \cdot \omega^2 (1-e^{-k \cdot LC})/k \tag{6-11}$$

where C_A is the added mass coefficient of the connection tube, V_{conn} is the volume and *LC* is the length of the connection. Similar simplification can be applied for Vierendeel braces between tubes.

6.3.5.5 Wave force on pontoons, Pontoon supported SFTB

For wind generated waves, pontoons shall be considered as large volume structures with horizontal dimensions on the same order as the wave length. Hence wave actions shall be calculated from three-dimensional diffraction theory. In general, the wave force on a large volume structure can be written as:

$$q(x,t) = q_{exc}(x,t) + q_{rad}(x,t) \tag{6-12}$$

where q_{exc} is the wave excitation force and q_{rad} is the wave radiation force. Coordinate *x* indicates the location of the pontoon along the tube bridge. *q* includes three components, heave force, sway force and roll moment, where sway is motion normal to the tube bridge and roll is the rotation about the tube axis.

The wave radiation force can be written:

$$q_{rad}(x,t) = -A(x,\omega)\frac{\partial^2 \eta(x,t)}{\partial t^2} - B(x,\omega)\frac{\partial \eta(x,t)}{\partial t} \tag{6-13}$$

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where A is the pontoon added mass, B is the pontoon wave radiation damping and η is the motion of the tube bridge at the location of the pontoon. In general A and B are 3x3 matrices, but for bodies with two planes of symmetry only the diagonal terms are non-zero. Added mass and damping matrices are calculated by three-dimensional wave diffraction analysis (for example using Sesam: Hydro-D). A simplified assessment of added mass and wave damping can be made using the charts due to Vugts[6-10].

6.3.5.6 Wave excitation force

The first order wave excitation force is the sum of the Froude-Krylov force (FK) and the diffraction force (D).

$$q_{exc,k} = q_{FK,K} + q_{D,k} \tag{6-14}$$

Assuming long waves relative to the tube and pontoons, the following approximations can be made:

- The Froude-Krylov force in direction k (sway = 2, heave = 3) may be approximated to be $F_{FK,k}(t) = \rho_\omega Va$ where $\rho_\omega V$ is the mass of displaced volume and a is the fluid acceleration
- The diffraction force is the sum of two terms, one proportional to the added mass and one proportional to wave radiation damping

The sectional Froude-Krylov force for one tube is

$$q_{FK,k}(\omega) = \overline{a}_{0,k}(\omega) \cdot \rho_w \cdot A_{tube} \tag{6-15}$$

where $\overline{a}_{0,k}(\omega)$ is the fluid particle acceleration in the incident wave at the centre of the cylinder, ρ_ω is the density of water and A_{tube} is the area of the cylinder.

The diffraction forces for one cylinder is:

$$q_{DK}(\omega) = \overline{a}_{0,k}(\omega) A_k^{2D} + \overline{u}_{0,k}(\omega) B_k^{2D} \tag{6-16}$$

where $\overline{u}_{0,k}(\omega)$ is the fluid particle velocity of the incident wave at the centre of the tube, A_k^{2D} is the sectional added mass of the tube and B_k^{2D} is the sectional wave potential damping. The correlation factor being a function of the wavelength λ and Centre Distance Between Tubes (CDBT), used for summing up the forces acting on the two tubes, is

$$\rho_c(\omega) = 1 + \cos\left(2\pi \frac{CDBT}{\lambda}\right) = 1 + \cos\left(CDBT \frac{\omega^2}{g}\right) \tag{6-17}$$

The total sectional excitation force on the tube is therefore:

$$q_{exc,k}(\omega) = \left[\overline{a}_{0,k}(\omega)\left(A_K^{2D} + \rho_w \cdot A_{tube}\right) + \overline{u}_{0,k}(\omega) B_k^{2D}\right] \rho_c(\omega) \tag{6-18}$$

Assuming linear Airy wave theory, the fluid particle acceleration and the velocity of the undisturbed wave are expressed as:

$$\overline{a}_{0,k}(\omega) = \omega^2 \cdot \zeta_a(\omega)\, e^{-\frac{\omega^2}{g}d} \tag{6-19}$$

$$\overline{u}_{0,k}(\omega) = \omega \cdot \zeta_a \cdot e^{-\frac{\omega^2}{g}d} \tag{6-20}$$

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Implementing eq. (6-19) and (6-20) in eq. (6-18) one obtains:

$$q_k(\omega) = \left[\omega^2\left(A_k^{2D} + \rho_w \cdot A_{tube}\right) + \omega \cdot B_k^{2D}\right]\zeta_a(\omega)\, e^{-\frac{\omega^2}{g}d} \cdot \rho_c(\omega) \tag{6-21}$$

The load spectra can therefore be expressed as:

$$S_{qk}(\omega) = \left\{\left[\omega^2\left(A_k^{2D} + \rho_w \cdot A_{tube}\right) + \omega \cdot B_k^{2D}\right]e^{-\frac{\omega^2}{g}d} \cdot \rho_c(\omega)\right\}^2 S_{\zeta a}(\omega) \tag{6-22}$$

Due to the assumption of distributing the effects from the pontoons, the total sectional A_k^{2D} and B_k^{2D} are a sum of added mass and damping from the tubes (t) and pontoons (p).

$$A_k^{2D} = A_{t,k}^{2D} + \frac{N_p}{L} A_{p,k}^{3,D} \tag{6-23}$$

$$B_k^{2D} = B_{t,k}^{2D} + \frac{N_p}{L} B_{p,k}^{3,D} \tag{6-24}$$

where N_p is the number of pontoons.

The added mass of fully submerged circular tubes is approximated by

$$A_{tubes,k}^{2D} = 2\rho_\omega \cdot A_{tube} \tag{6-25}$$

The added mass coefficient for a tube with rectangular cross-section can be found from Fig. 6-6. For L/H = 4, C_{Ax} = 0.4 for horizontal motion and C_{Ay} = 4.3 for vertical motion. The sectional added mass is:

$$A_{t,k}^{2D} = \rho_\omega \cdot C_A \cdot A_{tube} \tag{6-26}$$

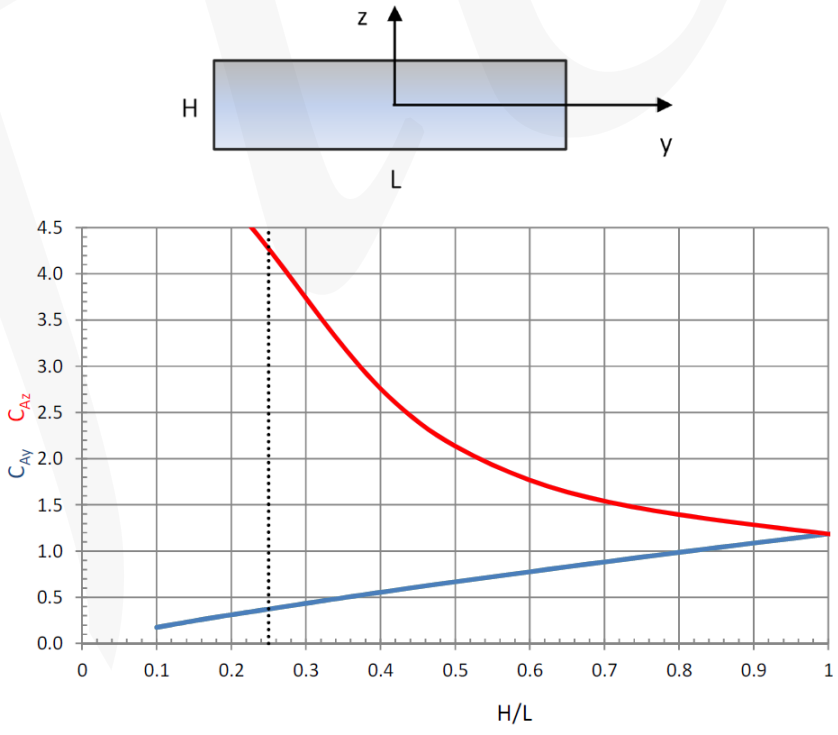


Fig. 6-6 Sectional added mass on tube with rectangular cross-section with height H and width L. Added mass coefficient for horizontal motion (blue curve) and for vertical motion (red curve), Statens vegvesen

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The added mass for the pontoons can be estimated by the use of Vugts[6-10] or calculated accurately by 3D diffraction analysis. Wave radiation damping for the tubes may be neglected.

For swell sea states a long wave approximation can be made to estimate the excitation force on the pontoons. Simplified formulae can be derived for a barge-like pontoon with length L (in direction normal to the tube), breadth B (in direction of tube axis) and draught T.

Excitation force in sway (y-direction) per unit wave amplitude:

$$F_y = 2\rho_\omega \cdot g \cdot B \cdot T \ \frac{1-e^{-k\cdot T}}{k\cdot T} \sin\beta \frac{\sin\alpha}{\alpha} \tag{6-27}$$

Excitation force in heave (z-direction) per unit wave amplitude:

$$F_z = \rho_\omega \cdot g \cdot B \cdot L \cdot e^{-k\cdot T} \ \frac{\sin\beta}{\beta}\frac{\sin\alpha}{\alpha} \tag{6-28}$$

where

$$\alpha = \frac{k\cdot B}{2}\cos\varphi \tag{6-29}$$

$$\beta = \frac{k\cdot L}{2}\sin\varphi \tag{6-30}$$

φ is the angle between the length axis of the bridge and the wave direction.

6.3.6 Second order wave actions

In a general sea state the SFTB will mainly be excited by first order wave actions at periods close to the wave spectrum peak period. However, due to non-linearities in the ocean surface waves and non-linear wave-structure interactions, the SFTB will also be excited by second order wave actions proportional to the square of the wave amplitude. These are slowly varying low frequency wave actions and sum (high) frequency wave actions.

6.3.6.1 Slowly varying wave forces

The second order slowly varying wave force (also termed drift force) on the SFTB can be divided into forces on the submerged tube sections, forces on the connections and forces on the pontoons. The forces on the tube sections are distributed actions along the length of the tube, while the forces on the pontoon are discrete loads at the location of the pontoons.

Long crested waves and deep water are assumed along the length of the bridge. The effect of a tube arc configuration, causing a local incident wave direction, is not included when second order wave forces are evaluated, hence the wave forces are equal along the whole bridge. The effect of short-crested waves and finite correlation length can be taken into account when estimating the response of the tube bridge due to second order slowly varying wave forces.

The spectral density of the global slowly varying force on the tube in the lateral (j=2) and vertical (j=3) directions for long-crested waves normal to the tube bridge can be written as:

$$S_{Fj}(\mu) = \frac{2\left(\rho\cdot C_M \cdot A_{tube}\cdot L\right)^2}{g^2} e^{\left(\frac{-2\mu^2\cdot d}{g}\right)} \mu^4 \int_0^\infty \left(\omega+\mu\right)^2 \left(2\omega+\mu\right)^2 S(\omega)\ S\left(\omega+\mu\right) e^{\left(\frac{-4\mu\cdot\omega\cdot d}{g}\right)} d\omega \tag{6-31}$$

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where

A_{tube} = total tube cross sectional area

L = length of tube

d = vertical distance between water level and the centre of tube

S(ω) = wave spectrum (JONSWAP)

If hydrodynamic interactions between the tubes and the effect of the free surface effects are neglected, the added mass can be approximated by $A_{jj} = \rho\pi D^2 L/2$.

For the pontoons, the force spectral density for the slowly varying force in the lateral (longitudinal) (j=2) and vertical (j=3) directions of the pontoons are expressed as:

$$S_{Fj}(\mu) = 8\int_0^\infty S(\omega)S\left(\omega+\mu\right)\left[\frac{\overline{F}_j(\omega+\mu/2)}{\zeta_a^2}\right]^2 d\omega \tag{6-32}$$

Here $\overline{F}_j(\omega+0.5\mu)$ is the mean wave force on the pontoon in direction j=1 and j=3 in regular waves with frequency $(\omega+\mu/2)$ and wave amplitude ζ_j. Depending on the frequency content in the wave spectrum, the horizontal mean wave force may be approximated as $\overline{F}_1(\omega+0.5\mu)$ with the high-frequency asymptotic value. However, the vertical mean wave force $\overline{F}_3(\omega+0.5\mu)$ needs to be calculated by a wave diffraction program such as, for example, Sesam:WADAM.

6.3.7 Sum-frequency wave actions

For a tether supported tube bridge the natural period T_n for vertical modes may be in the order of 6-8 sec and lower (for higher modes). These modes are mainly excited by 1st order wave actions. Due to non-linear wave effects and non-linear wave-structure interaction effects, the tube bridge will also be excited by waves with energy at $2T_n$. These waves with frequency $f \sim 1/(2T_n)$ will give second order sum-frequency wave actions at frequency $2f \sim 1/T_n$ close to the frequency of vertical modes for the bridge.

Since the sum-frequency dynamic pressure decays only algebraically with depth, and not exponentially as the first order pressure, this non-linear wave action contribution may be of importance for a tether supported tube bridge. The sum-frequency wave action is given in terms of Quadratic force Transfer Functions (QTFs) which can be calculated, for example, by Sesam:WADAM or using hydrodynamic analysis software for calculating wave-structure interaction.

6.3.8 Other wave loading effects

Other wave loading effects which may be considered, but most likely will not be governing:

- Internal waves
- Waves from passing ships
- Underwater landslide generated waves
- Tsunami

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6.3.9 Water level variations

The water level variation is defined as an astronomical component and a surge component combined. The surge component includes effects from low/high atmospheric pressure and storm surge. The astronomical component is independent of the environmental conditions, whereas the surge component is governed by the atmospheric conditions. Since the confidence in the two components vary, that is, astronomical tide values are much more predictable than the surge components, appropriate safety factors can be applied separately on each component during further design.

Water level variations contribute to both global and local forces and bending moments in a pontoon tube bridge. This can be considered as a forced displacement at the bridge end supports. Water level variations are less important for an SFTB without structural elements in the sea surface, as only the increase in water pressure should be considered.

6.3.10 Wind actions – pontoon type SFTB

The pontoons have a limited surface exposed to wind pressure. Wind actions for development phase accuracy level are well defined in engineering standards. To run coupled wind and wave actions in the time-domain is something which may be considered for the detail design phase. Further, detailed derivation of data on wind direction, mean wind speed distribution and extreme wind speed distribution with corresponding return periods may be required if the SFTB has a significant area exposed above water.

6.3.11 Temperature actions

An SFTB is presumably located at a water depth where there is unrestricted ship passage and thereby is in stable surroundings and quite small temperature variations. The action effect is less significant compared to a bridge in air, where there are high maximum, low minimums and also large differential temperatures. Temperature actions give axial forces in the SFTB and are mainly relevant when considering the vertical and horizontal alignment of the SFTB. Giving the SFTB a curved alignment can reduce the influence of temperature actions. In the case of a straight alignment, tensile stresses from minimum temperature shall be considered when evaluating post-tensioning levels for maintaining water tightness criteria.

6.3.12 Water pressure

Water pressure is a positive action effect in the case of a round tube-shaped cross-section, which gives compression in the circumferential direction. In the case of flat plates in the cross-section, water pressure can give significant bending moments.

6.3.13 Snow and ice actions

Snow and ice actions on pontoons influence the draught of the pontoons and contribute to both global and local forces and bending moments in the SFTB.

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6.4 Ship impact on pontoon supported SFTB

Early in the project development, a ship impact design philosophy should be established.

The design of offshore platforms for ship impact, and related requirements and practice, could be a useful reference[6-11]. For example, the vertical shafts of offshore concrete structures are similar to concrete SFTBs, except they are vertical. They are designed to resist impact from tankers as well as smaller ships, such as supply vessels. This seemingly surprising possibility is due to the fact that the impact action, in terms of load per unit area, will never be greater than the strength of the ship.

If the ship impact energies are large, a "weak link" that limits the transfer of forces from the pontoon to the traffic tubes could be considered. Below is a suggestion for a general design procedure for developing weak link concepts, which consists of the following steps:

1. Establish the design impact energy based on the risk analysis. In absence of a risk analysis, an estimate of the impact energy can be made based on selecting a design ship from the largest ships that navigate the waters of the crossing (obtained by the Automatic Identification System data), and applying a design speed.
2. Establish a design criterion for the weak link and the tube bridge, respectively.
3. Perform a quasi-static and dynamic non-linear analysis of an isolated weak link including material and geometric effects.
4. Perform global time-history impulse analyses to predict the dynamic response in the weak link connection for different impact locations and impact angles (head-on and glancing) taking into account energy released from impact impulse and dissipation in hull and pontoon.
5. Perform a preliminary design of the pontoon – tube connection including the weak link component(s).
6. Conduct design verification of the concrete tube bridge for the residual impact loading. The tunnel shall remain intact after the event. Design checks are performed in the ultimate and serviceability limit states.
7. Evaluate the weak link concept in relation to key performance parameters such as overall structural implications, durability, inspection and post-collision replacement.

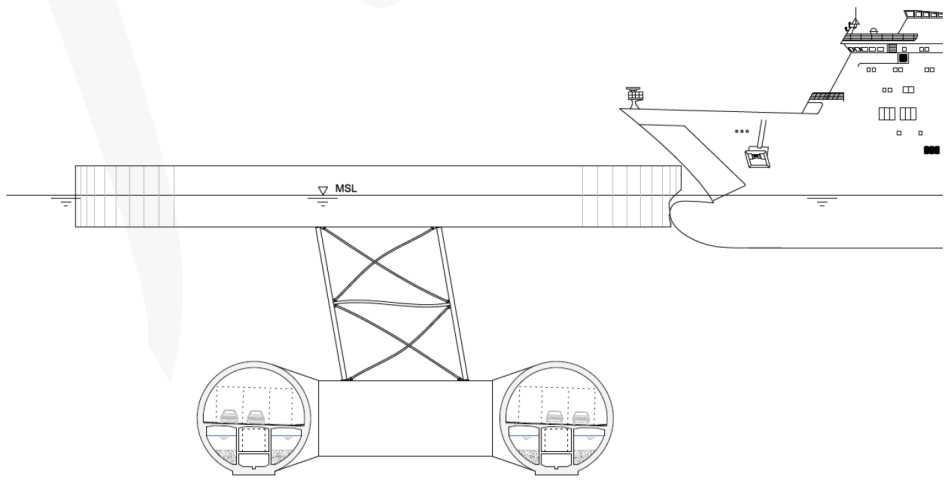


Fig. 6-7 Example truss integrated weak-link from Bjørnafjorden SFTB - Dr.techn. Olav Olsen

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In Fig. 6-8, the reaction force time history at the base of pontoon after a 350 MJ ship impact is shown, for the case study on the Bjørnafjord crossing.

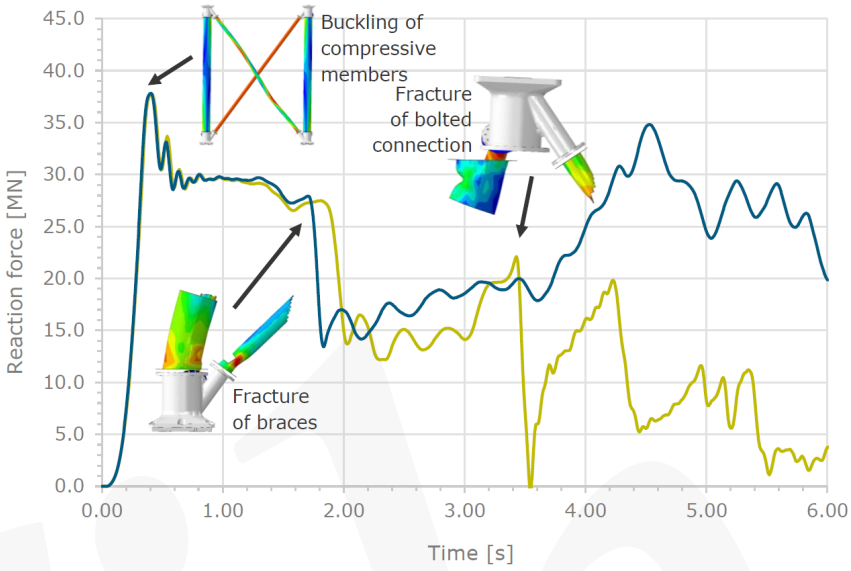


Fig. 6-8 Example Bjørnafjorden SFTB weak link connection (the green line shows the response from a 40 000-ton ship, while the blue line shows the response from a 25 000-ton ship[6-8]), Dr. Techn.Olav Olsen.

6.5 Other accidental actions

There are other accidental action effects which can influence the design besides ship impact, most importantly:

- Earthquake load. Important when shore approach tunnels are located in a weakness zone or if the SFTB is short and large differential movements are possible.
- Loss of tether. It will affect the design of vertical load carrying capacity. The risk should be evaluated in a geohazard study.
- Fire and explosion in tunnel. They will affect concrete cover and design of hoop post-tensioning for internal overpressure.

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7. Design principles

7.1 Structural configuration

7.1.1 General

The approach to the design of an SFTB starts with the development of a series of rational crossing schemes. The preparation of preliminary crossing layouts involves the determination of the crossing trajectory, general dimensions of the superstructure, vertical support system and span configuration. The choice of the structural arrangement will, in addition to fulfilling structural safety requirements, be influenced by both functional demands and site-specific parameters. Mode(s) of transport, traffic envelope and other space requirements will set the premises for the tube cross-section. Road and railway will moreover impose different restrictions on the alignment of the tube. Beyond this, finding the most suitable scheme for the local conditions will concern interpretation of the hydrographic parameters such as wave and current data as well as the bathymetry.

To facilitate the option development process, some general considerations related to the key concept parameters are presented in the following sections.

7.1.2 Horizontal alignment

In the design of an SFTB location, alignment and length are not usually strictly defined parameters. This means that various crossing trajectories will have to be explored to find the most effective alignment. Possible landfall locations and routing of access tunnels will represent typical constraints for the horizontal alignment. For a moored SFTB the bathymetry, sea bed conditions and possible anchor locations will have to be considered when developing the plan alignment. Usually, such assessments shall be made on an uncertain basis, since data from field surveys are not always available in the early stages.

For a pontoon-stabilised tube bridge being independent of the seabed conditions, location and width of the navigation channel will affect the pontoon or span distribution. The width of the channel will, depending on the waterway traffic, be typically from 250 to 400 m (detailed information is given in Ch. 4).

Regarding the geometric design, the shortest route between the shore connections is a natural first choice, but not always the better option. As described in Ch. 7.2.3 the horizontal geometry can effectively be utilised to tune the dynamic behaviour of the tube bridge. If a curved horizontal layout is found favourable, one should be aware that a horizontal curvature may require extra allowance in the traffic envelope due to sight requirements for stopping, and super-elevation of roadway and tracks.

7.1.3 Vertical alignment

In the vertical plane the SFTB offers alignment flexibility and allows a shallow vertical profile. In lakes, where there is normally little or no boat traffic, a draught just below

This chapter was mainly authored by
Corres Peiretti, Haugerud, and Minoretti.

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the influence of water level fluctuations and alternatively also the seasonal temperature variations would be adequate. In waters with marine activity, the vertical profile has only to accommodate a safe sailing depth. The net under-keel clearance is determined for the design vessel reflecting the future maritime traffic development and should account for tidal and meteorological water level variations, maximum static draught and allowance for change in water density, ship squat and dynamic heel and wave response. The net minimum under-keel clearance, i.e. the safety margin, will be set by the navigation authorities, typically as a fraction (e.g. 10 – 15 %) of the total draught. A more detailed assessment of the safety margin will be required if the navigation channel is exposed to long-period swell waves. Pending detailed under-keel clearance predictions, a free clearance of 20 m over the tube bridge is normally considered to be well below the reach of ship keels and thus adequate to avoid direct ramming from surface vessels. If submarines are expected to sail in the waters, this clearance is also adequate for submarines transiting in surface mode whereas a deeper draught will be required to allow transit when submerged. In the latter case, until further agreement with the Navy and coastal authorities, a free water clearance of 30 m above the tube bridge can be regarded as a base reference.

With the navigational channel defining the shallowest elevation of the tube bridge, a deeper draught may be utilised to reduce the hydrodynamic response in the superstructure. As shown in Fig. 7-1 (ref. Ch. 6.3.5) the wave energy decreases (in deep water) exponentially with the water depth. Also evident from the diagram is that the effect is strongly dependent on the wave period (T_p). Ocean swells typically of period over 10 s have long wavelengths and can transfer more energy than shorter, wind-generated waves (T_p < 6 s). Whereas it would be impractical to avoid loading from long-period swell waves by increasing draught, a moderate lowering of the tube bridge will significantly reduce the excitation from wind seas. As an example, for a predominant wave period of 5.5 s, increasing the water depth from, for example, 20 to 30 m will imply about 75 % reduction in wind wave loading. It should be noted that, when a structurally efficient cross-sectional shape is chosen (refer Ch. 7.1.4), the associated increase in external hydrostatic pressure will have little impact on the design of the tube structure.

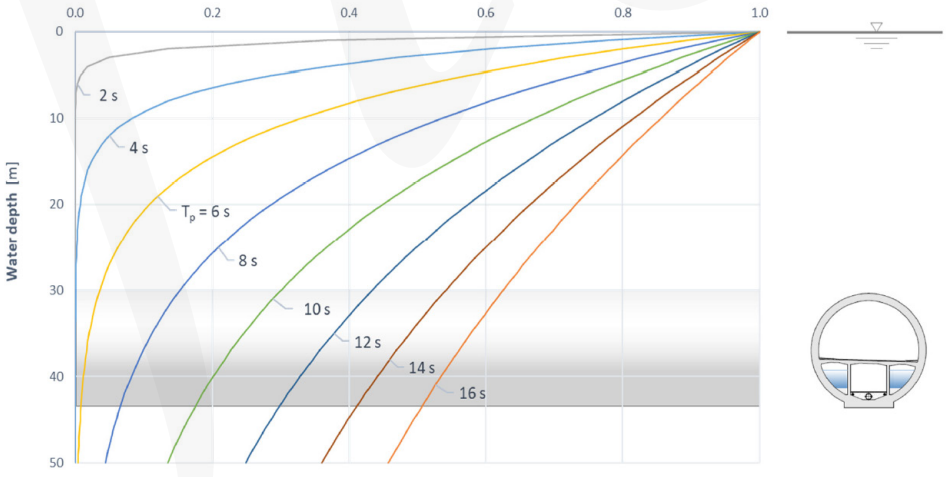


Fig. 7-1 Penetration of wave load (normalized) with water depth – Dr. techn. Olav Olsen

The vertical position will thus be a trade-off between the drawbacks of deeper approach tunnels and more demanding marine operations versus lower hydrodynamic loading on the SFTB. A smaller navigational clearance near the shores can normally be justified, allowing

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shallower approach tunnels and shore connections. However, this will introduce a curved vertical alignment.

Due to constructability reasons, i.e. jointing of the tube elements afloat, a straight vertical alignment of a submerged floating tunnel is clearly preferable. Sticking to this premise, a vertical curvature will still be possible for end-anchored SFTBs utilising the so-called 'bucket handle' technique where the arc is assembled in a horizontal position and subsequently tilted about its chord/end points during installation. The method, first proposed for the Høgsfjord SFTB crossing (Norway, 1998) and later proposed also for the Sognefjord crossing (Norway, 2011), is compatible with a central navigation channel and will create a rational alignment. The global rotation has, however, implications for the construction in dock.

In most cases the gradient limitations for road and railway alignments will not be a challenge for the vertical profile of an SFTB, but rather determine the length of the approach tunnels. Road and railway will have different gradient limits. Road tunnel gradients are generally limited to 5 %, whereas trains are limited to 1.25 – 2.5 % depending on design speed, train types and track layout. For a dual-mode crossing, a separation of railway and highway into different bores in the approaches will minimize the length of the approach tunnels and provide flexibility for individual alignments and connection with existing infrastructure.

7.1.4 Tube cross section

For submerged floating tube bridges a wide variety of cross section demands can be accommodated. The external shape of the tube can be circular, elliptical, polygonal or rectangular, depending on water depth, stiffness demands and environmental loading. In addition, construction methods as well as the safety philosophy for escape routes come into consideration.

The cross-sectional shape derives from the necessary traffic envelopes, escape corridors and the space required for ballast, drainage and ventilation systems. In determining the arrangement of the tube cross-section, a compact packing of the different space requirements is imperative as dead space will imply extra permanent weight, more drag and increased hydrodynamic added mass. The most effective weight-displacement ratio for the rectangular road and rail envelopes is generally obtained with a rectangular cross-sectional shape of the tube. The rectangular form is however less favourable in terms of hydrodynamic behaviour. Compared to a round(ed) form, the box shape will, as described in Ch. 6.3, attract significantly higher current forces. It will also increase the vertical component of the wave forces. For marine crossings exposed to high currents and swell waves, tuning the cross-sectional configuration with respect to hydrodynamic performance should thus take precedence over optimising the weight-displacement ratio.

Rectangular cross sections will, regardless of tube material, result in the most economical fabrication. Nevertheless, a curved or circular shaped cross section possesses distinct characteristics which can still have a considerable cost advantage over a rectangular or polygonal cross section. Besides the superior dynamic behaviour as already mentioned, a cylindrical section features a rational structural behaviour, with respect to pressure actions. A circular shell will carry the external water pressure primarily by hoop compression without adverse bending moments and shear and thus is better suited for larger water

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depths. It will also provide higher resistance towards concentrated loading such as dropped anchor or submarine impact. A superior structural behaviour has recently also been demonstrated for internal blast loading[7-1].

Fig. 7-2 shows a typical cross section layout which has been favoured in recent Norwegian studies, both in mono and twin tube arrangement. Despite an area ratio between the tube inner profile and the traffic envelope of nearly 2.5, an effective utilisation of the cross section is still obtained. The free headroom offers ample room for ventilation fans, traffic signals and signboards. Below the traffic deck, the tube is divided into three separate compartments. The outer ones are used for solid and wet ballasting, whereas the central one serves as a service gallery. Head-stocks for temporary and permanent water ballasting, water pipes for drainage and firefighting and utility lines such as communication and electricity are all routed through the service gallery.

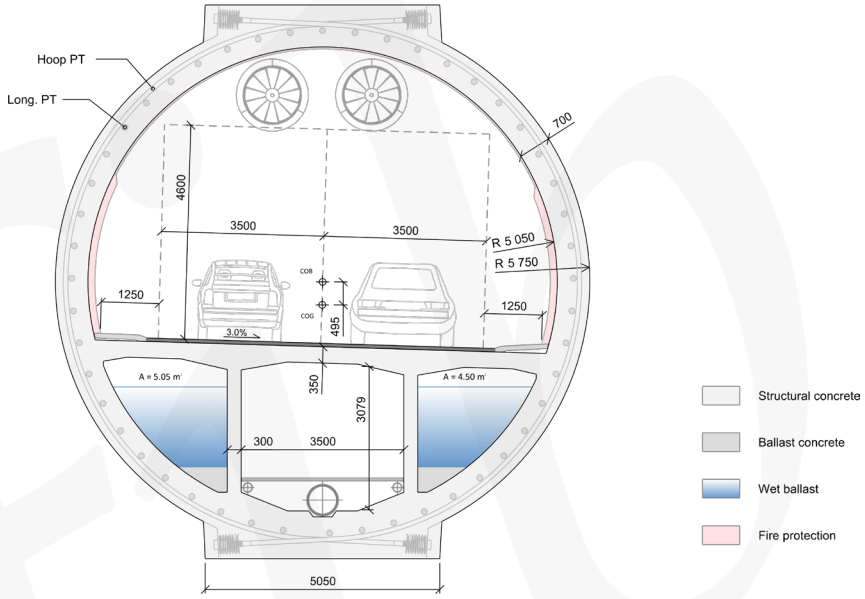


Fig. 7-2 Concrete cross section developed the E39 ferry replacement project (E39 Sulafjord SFTB) – Statens vegvesen

7.1.4.1 Tube material

The tube may be constructed in concrete or steel. Each material has its own advantages and limitations as well as offering different efficient structural configurations. The common criteria for selecting tube material are initial costs, durability, site conditions and local construction practice. Among the studies listed in Ch. 14.2 there is a clear trend towards concrete tubes. The preference for concrete can be attributed to various factors, of which costs, durability and maintenance are perhaps the most decisive.

One of the biggest advantages of steel is weight savings. For a structure with excessive buoyancy though, the advantageous strength-to-weight ratio is less profitable. A steel tube will in fact require nearly the same amount of concrete (80 – 90 %) as in a concrete tube of the same shape to achieve the same net buoyancy. Given suitable site conditions, the direct fabrication costs for a concrete tube are likely to be significantly lower than for a steel shell tube. A steel tube may, under certain site constrains still be a more economical option, e.g. in cases where there is no access to a suitable dry dock or where the towing draught is limited. Steel may also offer more flexibility when it comes to tailoring the dynamic characteristics of the main tube.

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The choice of tube material may also be influenced by the prevailing construction practices, especially within the related immersed tunnel industry. Among the approx. 130 immersed tunnels built for road and rail traffic around the world, there is a distinct geographical pattern in the choice of tunnel typology[7-2]. In North America most of the immersed tunnels are constructed in steel making use of the shipyard fabrication techniques. In Europe, Southeast Asia, and Australia the concrete tunnel technique has been the standard, whereas Japan has a mixed use of concrete, steel and composite tunnels. Although there is a growing interest in the latter variant, concrete is currently the prevalent immersed tunnel option and this is likely to be reflected in the future projects for SFTBs.

In Norway, concrete tube design has been the preferred solution in all the SFTB studies performed, mainly as a result of the extensive experience with gravity and floating offshore concrete platforms in the North Sea (Fig. 7-3). These platforms representing ~1.8 mill. m^3 high strength/high performance concrete have demonstrated excellent durability over 40 years of service. The well documented in-service durability performance[7-3] is relevant in view of the minimum design lifetime of 100 years for an SFTB.

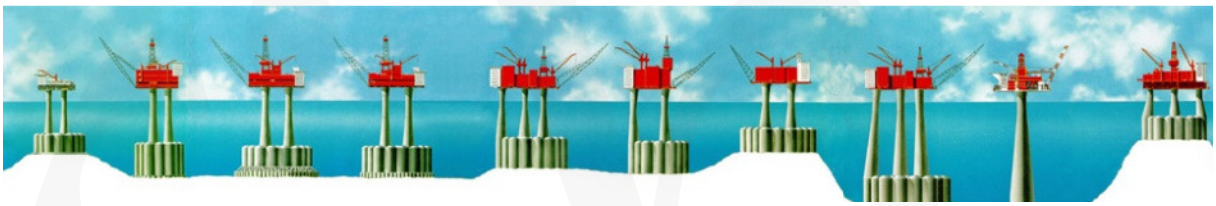


Fig. 7-3 Line-up of the Condeep platforms installed in the North Sea (1975 - 1995) – Dr. Techn. Olav Olsen

Fig. 7-4 shows examples of a concrete and steel cross section developed for a 2-lane main tube (E39 Sulafjord SFTB). As mentioned initially, the central focus of this Bulletin is on concrete submerged floating tubes. Nevertheless, for the sake of generality, a description of the structural characteristics of both the principal tube technologies is given below.

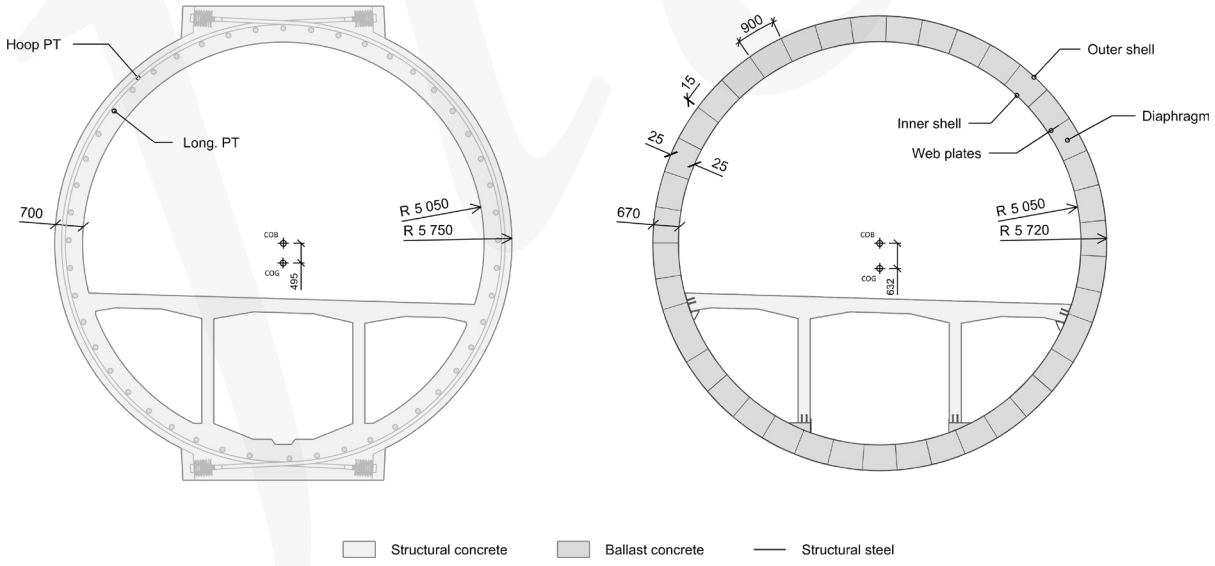


Fig. 7-4 Example of an all-concrete (left) and double-shell steel cross section (right) developed for a 2-lane main tube (E39 Sulafjord SFTB – Statens vegvesen

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7.1.4.2 Steel shell tube

As for an immersed tunnel, different cross-sectional variants can be envisaged also for a steel SFTB. The basic configuration categories are single shell, double shell and sandwich tube.

A single shell tube is formed by an outer shell plate acting compositely with an inner lining of reinforced concrete providing the primary strength of the tube. The structural steel shell is acting as a waterproofing membrane and serves as a floating formwork. It is formed as a hull plating with transverse stiffening frames and longitudinal stiffeners with local scantlings determined by the temporary floating condition. The steel-concrete composite action is ensured by steel studs. The thickness of the concrete liner is governed by the net buoyancy demand rather than strength and will typically be 0.8 – 0.9 m.

A double shell tube has, as the name suggests, two complete layers of watertight hull skins. Both the inner and outer shells are load carrying and together they form the pressure hull. The skin plates are connected by longitudinal web plates and transverse steel diaphragms as indicated in Fig. 7-4. The voids are filled with low shrinkage self-compacting concrete to counteract excess buoyancy and protect the shell plates against corrosion. The infill concrete provides local stiffening for the skin and web plates and may also be activated in hoop compression. A composite action in the tube's longitudinal direction should, as further explained for the sandwich variant below, be omitted for SFTBs exposed to significant hydrodynamic loading to avoid uncertainties regarding dynamic behaviour and response. Due to higher steel weight, the double shell structure will require less concrete ballast than a single shell and hence a reduced hull thickness. To facilitate proper access for welding of the double hull cells, the distance between the skin plates should be a minimum of 0.6 m.

The sandwich variant has similar structural configuration as the double hull, except for a somewhat finer cell grid and shell plates internally stiffened with flat and L-shaped ribs for full composite action with the concrete core. Recently, for offshore applications, other and more efficient mechanical shear connectors, rather than angle connectors have been developed. Compared to a double shell, a sandwich shell provides higher stiffness, energy absorption and impact resistance. However, the prediction of the structural behaviour in a sandwich shell under hydrodynamic loading is complex. For an SFTB where the eigenperiods and mode shapes of certain motions are tailored to avoid resonant response, uncertainties related to the effective stiffness of the superstructure is undesirable. This limits the applicability of the steel shell variants relying on composite action for 'high-performance' SFTBs.

Provision of passive fire protection is required for all 3 tube variants to maintain structural integrity during the specified fire exposure conditions (refer Ch. 6.5). Adequate space must be reserved in the cross section for thermal barriers on soffits and walls. Double shell and sandwich tubes are especially exposed to fire and will need larger application thicknesses than a single shell tube with concrete lining, which for the latter typically will range from 30 – 50 mm in the soffit.

Being permanently exposed to (sea)water, corrosion protection of the exterior shell is of great importance. Various protection systems can be used to delay and mitigate corrosion initiation. In seawater, sacrificial anode cathodic protection (SACP) in combination with protective coating is the principal measure for external corrosion control of primary steel structures. When properly designed, applied and monitored, it can provide adequate

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corrosion resistance for very long service life. The anodes are typically stand-off sled-mounted anodes that can be replaced by a ROV for maintenance. For added protection, preparations for retrofitting of an impressed current cathodic system may also be considered. Such considerations will also apply to the tethers and mooring system.

As already mentioned, the main advantage of steel shell tubes lies in the fabrication and construction method. Steel tube elements can be fabricated in a shipyard or on a greenfield site. Due to their relatively light weight, steel tube elements may be lifted or launched into the sea and can be designed for transportation without the structural concrete. The ballasting and outfitting with interior concrete can thus follow near the crossing site whilst afloat. Whereas a concrete tube will have a draught of almost the full tube height, steel tubes can allow a shallow initial draught less than 3 m even for a circular shaped double shell tube such as the one shown in Fig. 7-4. Casting afloat is however a delicate operation which will require careful planning and monitoring of the casting sequences to avoid misalignments and imposed deformations. A single shell tube element is particularly flexible and sensitive to distortion until the composite action is mobilised. Temporary longitudinal and transverse bracings will thus be needed to stiffen the tube element for such operations.

A recent example of efficient application of a steel shell tube is the Söderströmstunnel in Stockholm (refer Ch. 14.2) opened in 2015. The 300 m immersed railway tunnel under the river estuary Söderström consists of 3 tunnel elements (2 × 107.5 + 85 m) constructed by the floating single steel shell method[7-4]. In fact, as a continuous tube supported on discrete pile foundations, the Söderströmstunnel can by definition be characterised as a submerged floating tube bridge, from the static point of view.

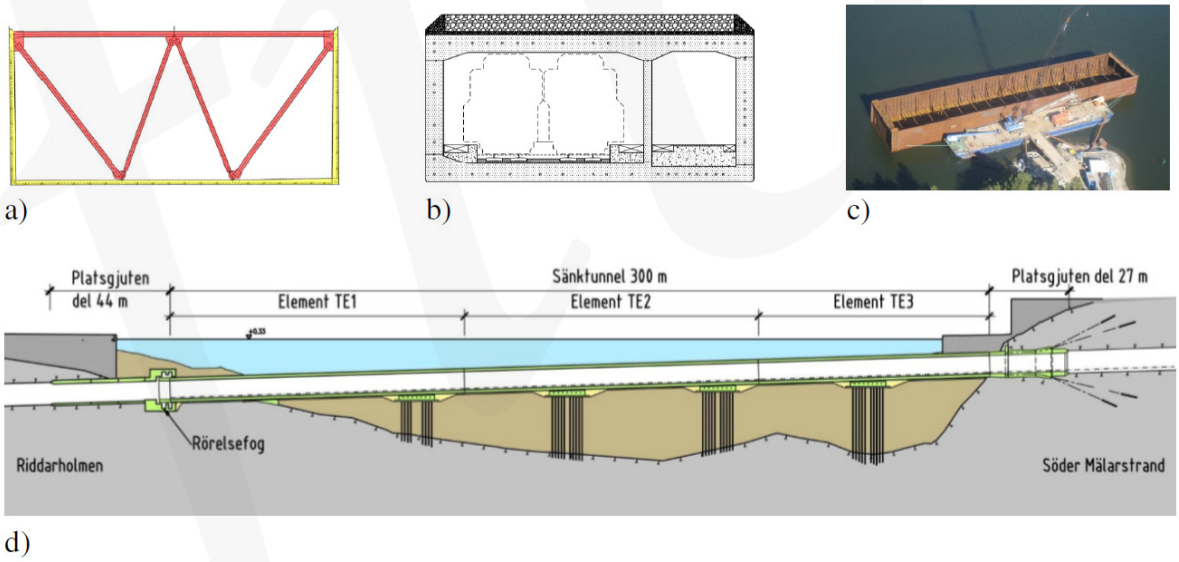


Fig. 7-5 Söderströmstunnel: (a) floating steel shell, (b) cross section, (c) concrete casting afloat and (d) longitudinal section - Trafikverket

7.1.4.3 Concrete tube

While different structural typologies are possible for a steel tube, the configuration of an all-concrete tube design is less diverse and typically consists of a concrete single shell constituting the pressure hull. Water tightness of the monolithic tube wall is ensured by a combination of strict design requirements and measures during construction to control cracking. Load-induced cracking is controlled through structural design and appropriate

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limits to cracking, which is described in more detail in Ch.7.5.2. The tube wall will be subjected to both in-plane and out-of-plane action effects and it is post-tensioned in the longitudinal and hoop direction to prevent through-wall cracks. Because of the large wall thicknesses, a relatively high level of prestress is required to maintain a significant compression across the section.

Experience from offshore concrete platforms (Fig. 7-3) and recent European immersed tunnels is that an external water proofing membrane is not necessary, provided that a proper mix design, good workmanship and some actions are taken to minimize early-age cracking. Longitudinal construction joints are avoided using the full-face casting technique, which was first employed for the Øresund immersed tunnel and since then has become standard practice. For the vertical construction joints between the casting segments, a preparation of the joint surface by water jetting or sandblasting to remove laitance and expose the aggregates is considered sufficient to achieve water tightness. Since the cold joints are prestressed, bonding agents are no longer considered necessary. The same goes for water stops which tend to impede proper concrete placement and create a potential water path.

Curing is critical for the concrete tube. Drying shrinkage will be marginal since the concrete itself is relatively impermeable. However, too high heat of hydration may lead to thermal through-cracking in the massive sections. Heat of hydration may be reduced by using coarser grind cement, such as blast furnace slag cement or by replacing a part of the cement with fly ash. The use of low heat concrete is a trade-off between heat rate and low early strength, but for a concrete tube built in a dry dock a slow strength development is not critical.

Simulation of stress development and crack risk is now commonly used to determine additional preventive actions (e.g. pre-cooling of mix, external insulation, postponed formwork removal) to avoid through-cracking during the hardening phase. In the thermal simulations the crack risk is commonly limited to 0.7, i.e. the tensile stresses shall not exceed 70 % of the developed mean tensile strength of the curing concrete.

As most of the concrete section will be completed prior to float-out from the dock, an all-concrete tube does not have the same draught flexibility as a steel tube. A concrete tube will have a freeboard of only 1-2 m and require a sill draught closed to the full section height, which obviously excludes conventional ship docks. Due to the large draught, in particular for circular shaped tubes, construction of concrete tube elements requires access to an offshore dock or casting basin. Although this may imply remote construction, sea transport is not very expensive and will have little impact on the overall construction costs. Besides, owing to the significant longitudinal prestress, the concrete tube elements are well suited for offshore towing.

As to the cross-sectional configuration, the flexibility in form and shape is clearly demonstrated in Fig. 7-6 by the variety of different cross-sectional shapes developed in previous studies (refer Ch. 14.2). The size of the cross-section derives generally from the required clearance gauge and the necessary space for wet/movable ballast. The contingency on net buoyancy, as described in Ch. 6.1, occupies considerable space in the cross section. As it can be seen in Fig. 7-6, the ballast compartments can typically take up 10-15 % of the interior space. The ballast compartment should be designed for the full hydrostatic head if they are non-inspectable.

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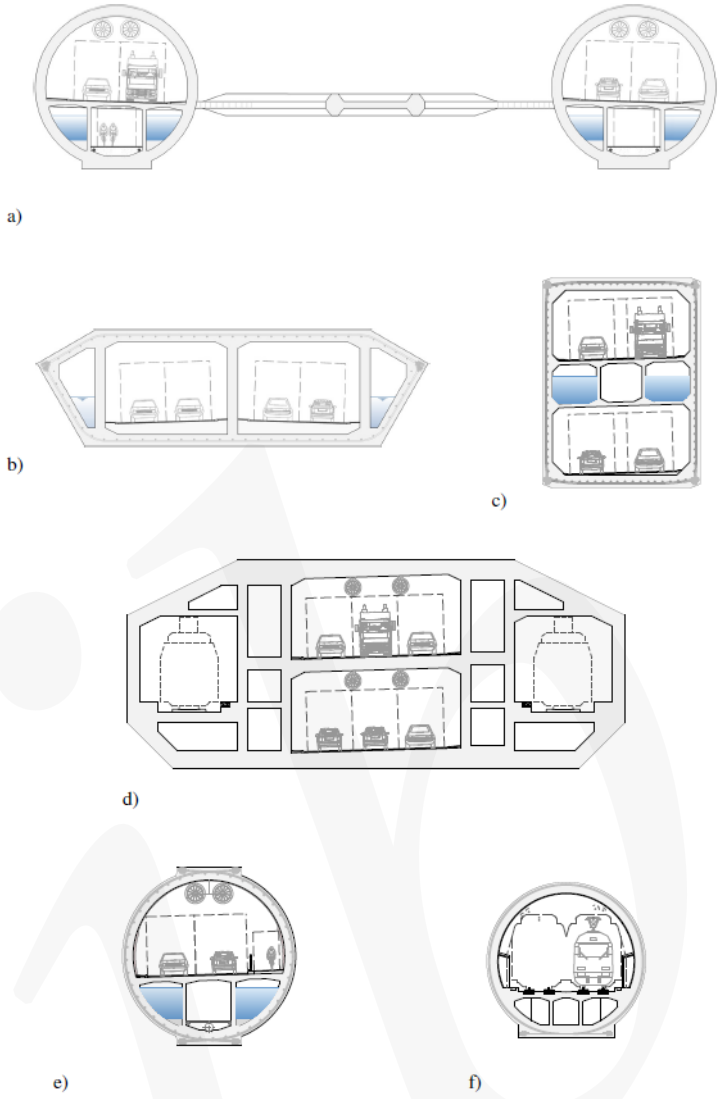


Fig. 7-6 Gallery of cross-sectional shapes from previous studies – Dr. techn. Olav Olsen, Statens vegvesen

The thickness of the tube wall is determined by the desired net tube buoyancy and typically ranges from 0.7-0.9 m. Although concrete strength is normally not governing for the tube's structural resistance, a concrete with characteristic compressive strength of 55 MPa or higher will still be beneficial to enhance the admissible compressive stresses and improve the resistance to cracking. Furthermore, the concrete mix design should meet the workability and performance criteria relevant for marine structures. The addition of polypropylene fibres in the cast of the inner face of the soffit/roof slab to inhibit explosive spalling in the event of a fire is largely recognized, even if alternative methods are recommended to avoid the use of plastic. The need for additional passive fire protection will depend on the specified fire exposure conditions but will probably be required.

The durability strategy for an all-concrete tube in saline waters will typically be the same as has been proven effective for the North Sea platforms (Fig. 7-3): protection of the reinforcement and embedded steel with sacrificial anodes installed during construction. The galvanic anodes effectively protect the reinforcement even if chloride contaminates the concrete. They eventually require retrofitting if the design service life is exceeded. Preparations for impressed current systems by ensuring galvanic connection of the reinforcement may be included for added protection.

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7.1.4.4 Tube arrangement

Historically, SFTBs accommodating road traffic have predominantly been proposed as singular tubes with two-way traffic as shown in Fig. 7-6 (e) due to low traffic demand. Modern tunnels standards and safety requirements combined with increasing traffic volumes makes the configuration with bidirectional traffic in a single bore less relevant for future crossings. Multi-lane carriageways can be accommodated either in a single tube with two separate traffic compartments as shown in Fig. 7-6 (b) or in two individual tubes as shown in Fig. 7-6 (a). For the twin tube arrangement, the tube spacing is a key parameter that affects the horizontal flexural rigidity and also influences the bridge's sway characteristics and in-line Vortex Induced Vibration (VIV)-response.

For the E39 Sognefjord and Bjørnafjord crossings in Norway (refer Ch. 14.2), a twin tube arrangement has been selected over the full rectangular box section based on a comparative study of structural behaviour and efficiency. Besides an effective weight/displacement ratio and high resistance towards concentrated loading, the most distinct advantage of the cylindrical section is seen in the hydrodynamic performance due to significantly less added hydrodynamic mass. The added mass in vertical mode results in inertia actions in the range of 2-5 times higher for the box cross section compared to a twin tube cross section[7-5]. In tests a twin tube cross section has shown stable behaviour under current and wave action, eliminating the need for extra design remedies to eliminate uncontrolled motion. Wind tunnel tests[7-6] also underlined the twin tube as the preferable cross-section rather than a rectangular box. The wind tunnel tests showed the in-line VIV amplitudes to be unaffected of the tube spacing L for a ratio $L/D > 2.0$.

In a twin tube arrangement, the main tubes are interconnected by cross tubes at supports, see Fig. 7-7. The cross tubes, rectangular in cross section, are rigidly connected with the tubes and fulfil several functions. Besides transferring vertical support forces, the cross tube contributes to composite action between the main tubes in the horizontal plane (Vierendeel action), accommodates escape ways, room for technical installations and water ballast tanks for temporary and permanent ballasting. The overall size is governed by the ballast and buoyancy demands, which is fundamentally different for the vertical support systems as described below.

Where there is significant wave loading, horizontal bracing of the main tubes between the cross tubes may be required to limit the lateral, wave induced flexural response in the main tubes to an acceptable level. From the assessment of alternative bracing configurations, diagonal bracing has been identified to be the most efficient system to obtain structural interaction of the tubes. It should be noted, though, that the complexity of the structural coupling between the tubes should not be underestimated, as safeguarding water tightness in the cylindrical main tube shell is a major challenge.

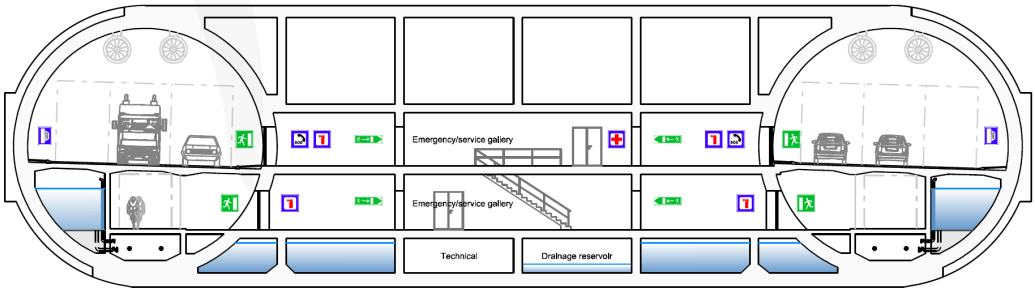


Fig. 7-7 Typical layout of cross tube with lay-by arrangement at supports (E39 Bjørnafjord) – Statens vegvesen

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7.1.5 Structural joints

An SFTB is a multi-span structure that acts as a girder bridge. It is made up of pre-fabricated tube elements and rigidly jointed to a continuous superstructure. The construction method may resemble that of an immersed tunnel, which may be the reason why the temporary joints of an SFTB are often confused with the flexible joint scheme of immersed tunnels. However, flexible joints in a continuous-span tube make little sense and will only compromise structural efficiency and water tightness. Hence the tube elements in an all-concrete tube are monolithically connected by a cast in place joint infill with reinforcement and prestress continuity across the joints. Fig. 7-8 shows an example of monolithic jointing of tube elements in which the elements are coupled by continuity of cables crossing the joint in two layers.

Casting the in-situ continuity is a delicate operation that requires careful detailing and planning. As the tube elements are floating, any movement in the joint needs to be prevented during hardening of the infilled concrete. The jointing method shown in Fig. 7-8 will require temporary sealing with the aid of a cofferdam or a temporary proprietary sealing joint and is thus mainly suited for floating position. A solution developed for submerged jointing is shown in Fig. 7-9, featuring a flanged steel sandwich connector designed for robot welded connection.

The tube is also fully restrained by the landfall structures. The overall structural system should provide adequate flexibility to absorb imposed deformations due to thermal variations and shrinkage. Only in special cases, e.g. if located in high seismicity zones, arrangement of terminal joints at the shore connections as outlined in Ch. 7.1.6 may be required.

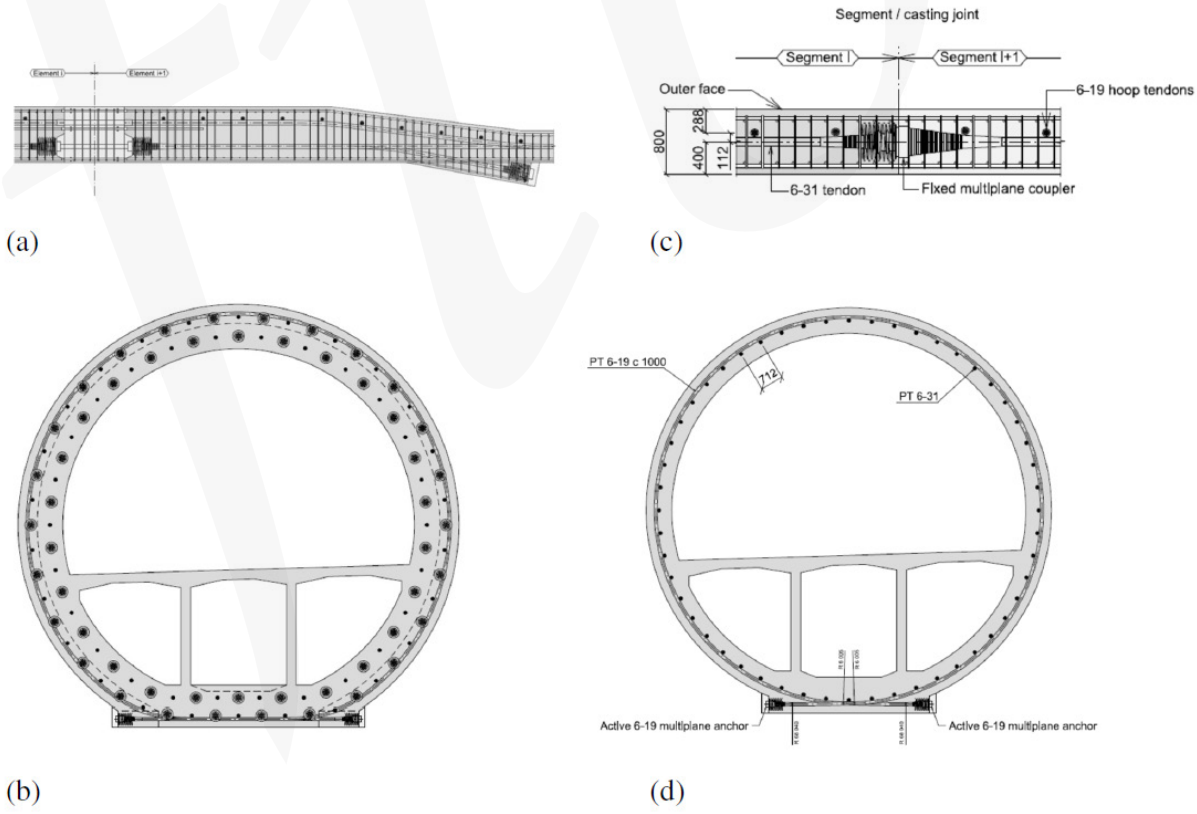


Fig. 7-8 Detail and PT arrangement of an element joint, (a) – (b), and a segment/construction joint, (c) – (d), (E39 Sognefjord SFTB) – Statens vegvesen

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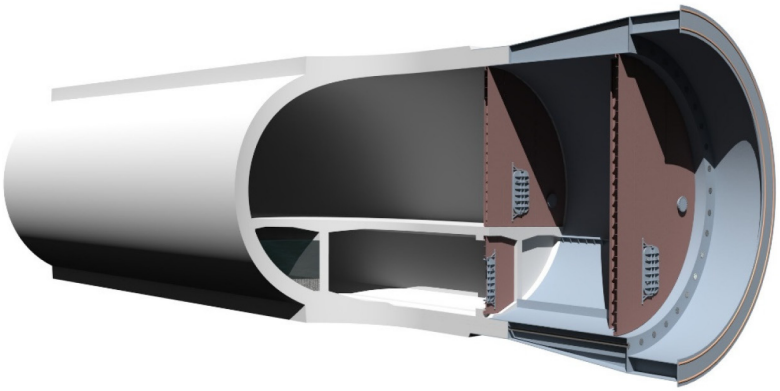


Fig. 7-9 Flanged steel connector with integrated bulkheads for submerged element jointing (E39 Sulafjord SFTB)- Dr. techn. Olav Olsen, Statens vegvesen

7.1.6 Shore connections

The interfaces between a submerged floating tube and the approach tunnels comprise landfall structures which will provide end supports for the tube bridge and secure a watertight connection between them. The structural configuration of the landfall structures, which may also accommodate room(s) for technical equipment and installations, will largely depend on the sea bed slope, ground conditions and form of the approaches. Since the landfall structures may have to transfer significant forces from the bridge, ground conditions and location should be considered early in the assessment of potential crossing alignments. Steep slopes with hard rock conditions and little sediment overburden are considered ideal conditions for landfall locations. The method for connecting an SFTB to a hard rock tunnel can be based on the underwater tunnel piercing technique used for hydropower intake tunnels and gas pipe tie-ins. With over 70 reservoir intakes constructed by underwater tunnel piercing since 1980 in Norway, down to water depths of 160 m, this is a well proven method which has been refined for SFTBs through several Norwegian studies. The principle is shown in Fig. 7-10 for a Norwegian SFTB crossing project currently being studied.

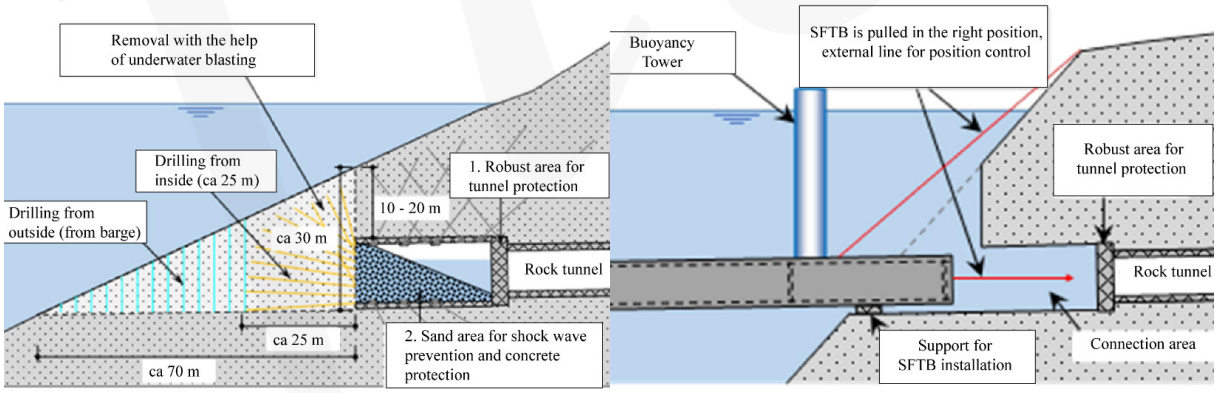


Fig. 7-10 SFTB landfall approach in hard rock conditions (E39 Sulafjord SFTB) - Dr. techn. Olav Olsen, Statens vegvesen

In soft ground conditions, the SFTB may be linked to a cut-and-cover tunnel. Depending on the vertical alignment, the transition element can be constructed in-situ within a dewatered excavation or installed wet by Immersed Tube techniques in a trenched canal, and subsequently fixed by solid ballasting as shown for the Statpipe shore approach (refer Ch. 14.2) in Fig. 7-11.

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Fig. 7-11 Statpipe shore approach (1982, Norway) -Selmer/Skanska

In seismic regions, the interfaces between the tube and landfall structure represent a general challenge and may require shore connections with dissipative capabilities. Preliminary designs of suitable seismic terminal joints have been developed for significant Peak Ground Acceleration (PGA) levels, e.g. for the Messina SFT crossing and the Transbay Tunnel in San Francisco. The triaxial seismic joint for the latter was put to test and performed successfully during the 1989 earthquake. As for many immersed tunnels, a less complex seismic terminal joint will be applicable also for a submerged floating tube bridge, including longitudinal articulation capability.

7.2 Global stabilization

7.2.1 General

Unlike a bridge in air, the superstructure of an SFTB is practically 'weightless' and inherently very stiff as the cross section required by the traffic envelope exceeds that for strength. As a result of the favourable stiffness-to-weight ratio, relatively large span lengths can be obtained. For predominantly quasi-static load conditions, e.g. for crossing lakes and sheltered waterways, a tube bridge can be stabilized by virtue of its own structural capability. The Rovdefjord and Digernessund crossings in Norway, described in Ch. 14.2, spanning over 230 m and 515 m respectively, represent recent examples of fixed single span tube bridges. For a given cross section, the maximum length for a single span configuration is typically governed by the water tightness criteria and can easily be predicted. Beyond this length, supports will have to be introduced for the stabilization of the tube. The various methods for controlling vertical and horizontal stability of an SFTB are described in more detail below.

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A tube bridge represents a large structural mass together with about the same magnitude of added mass from surrounding water. Structure damping is low (generally 0,5 - 1,0 %) and moderate deflections also limit the hydrodynamic damping[7-7]. The SFTB concept is thus sensitive to wave action periods within the range of the natural periods. Tuning of the dynamic characteristics of the bridge to avoid resonant behaviour is therefore an essential part of determining the overall support configuration.

7.2.2 Vertical stability

As indicated by its name, an SFTB carries a large part of the vertical actions by buoyancy. The difference between vertical actions and buoyancy results in a buoyancy surplus or deficit which will have to be carried to the supports and landfalls by beam action. The supports can be designed either as pontoons floating at the surface, tethers anchored at the seabed or underwater columns. The support concepts will imply different buoyancy strategies for the tube[7-8].

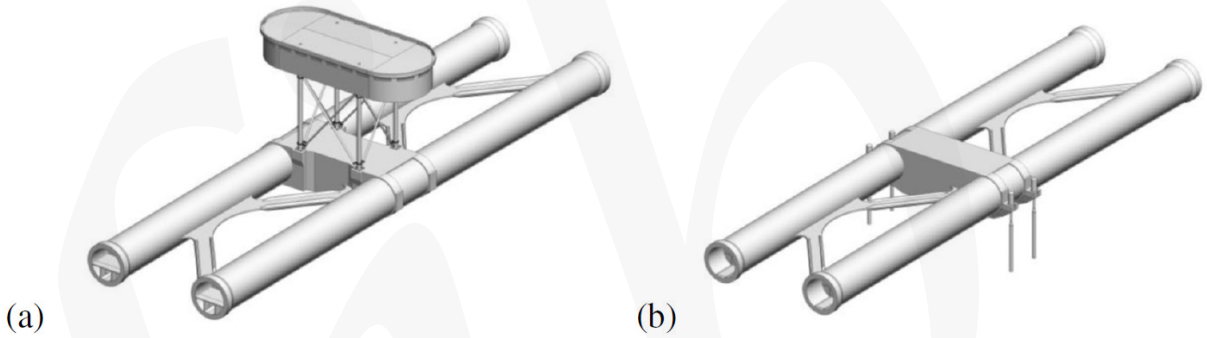


Fig. 7-12 Tube bridge elements with (a) pontoon and (b) tether stabilisation (E39 Bjørnafjord SFTB) – Statens vegvesen

7.2.2.1 Pontoon stabilisation

A pontoon-stabilized SFTB is supported by discrete pontoons controlling the vertical position and motions of the tube. Vertically, a pontoon supported tube bridge acts like a multi-span continuous beam on elastic supports, where the mass, the tube stiffness and the water plane stiffness of the surface floating pontoons are governing for the structural behaviour. Variations in weight, buoyancy and live action will be counteracted by vertical displacements of the pontoons. Variations in water level and water density will induce restraint forces in the tube towards the ends of the bridge. Horizontally, a pontoon bridge can be curved or straight.

The main advantage of the pontoon concept is that it is independent from the seabed. Potential uncertainties related to seabed conditions and geohazards can thus be avoided. Whereas a pontoon supported SFTB has the benefit of also being independent of water depth, it shall cope with the interaction with ships as the pontoons restrict free passage. The concept is however flexible with respect to ship passage location and the pontoons can be rearranged to fit different fairway arrangements, e.g. in the middle or at the ends of the crossing.

The span arrangement can normally be decided with emphasis on the static loading. As the heave excitation tends to be governed by 'global' heave modes rather than higher order modes within the span, the hydrodynamic response in the tube is not significantly

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influenced by span length. In order to standardize the construction, the span/element length is preferably kept constant for the whole length of the crossing, except at the ends where a larger distance can be utilized to relieve restraining moments from water level variation, if relevant. Increasing the end span is a trade-off between reduced flexural restraint from tide and increased bending moments from live loading for which an optimum balance can be found as indicated by the example in Fig. 7-13. If the tube has to be designed to tolerate the loss of one pontoon, the spacing between the 1st and 2nd pontoon shall be kept small to limit the enlarged end span in case the 1st pontoon is lost. The same applies also for the pontoons next to the navigation channel.

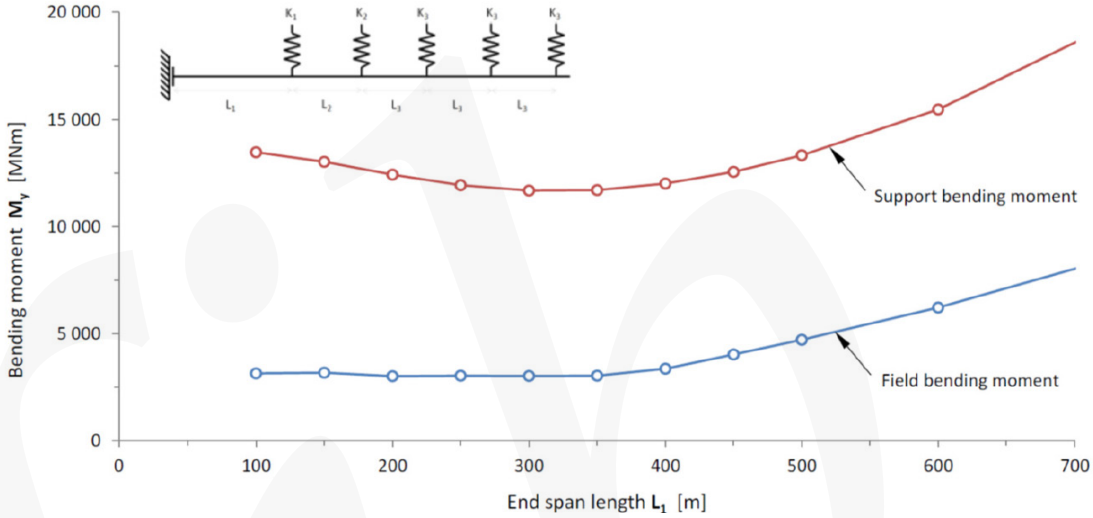


Fig. 7-13 Extremal range bending moments at support and in mid-span as function of end span length (L1) – Dr. techn. Olav Olsen

Notably, the heave response in a pontoon-stabilized bridge is significantly higher than in a tether anchored bridge. Although hydrodynamic damping from pontoons contributes to the vertical damping, pontoons will also attract loads from surface waves. Whereas resonance may be avoided by tuning the mooring stiffness in a tether-stabilized tube bridge, a pontoon supported SFTB will not offer the same flexibility. With a vertical stiffness of about an order of magnitude less than a tether mooring system and limited possibility for increasing the draught of the tube to duck away from wind sea loading, the pontoon system is likely to be in resonance in heave both for swell and wind induced waves. As hydrodynamic excitation forces on pontoons in heave tend to be in the same order of magnitude as on the tube, the sum of excitation forces in heave is about the double as for the tether concept.

A pontoon supported tube may be either positively or negatively buoyant. However, trimming the tube with solid ballast so that the net buoyancy surplus under permanent loading approximately corresponds with the net buoyancy deficit during maximum live loading is a common strategy for optimizing the tube's structural behaviour in operation.

For the sizing of the pontoons, the water plane area is largely governed by the vertical stiffness required to control deflections under traffic loading. The shape, i.e. the water plane moment of inertia, can be utilized to control the wave roll behaviour and potential adverse torque from ship impact. The draught should be deep enough to avoid wave slamming on the underside of the pontoon and the freeboard high enough to prevent green sea or water loading on the deck accounting for draught variations due to traffic loading, wave motions, marine growth and water absorption.

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Pontoons can be of steel or concrete. Steel pontoon design is a standard ship type construction with internal bulkheads and stiffeners suited for fabrication in shipyards. Steel pontoons can be fabricated on land, on a slipway or in a dock and may be preferred from a fabrication and installation point of view. However, in a marine environment, splash zone corrosion is a major problem which requires special considerations as cathodic protection is not reliable and in-service maintenance is costly. The application of super duplex stainless skin plates as a substitute for high-build epoxy coatings within the splash zone belt is currently being investigated in the E39 project in Norway, with the aim to obtain a maintenance-free structure. Concrete, on the other hand, offers distinct advantages over steel in terms of construction costs, robustness and durability, but it is likely to produce increased heave response as a consequence of higher pontoon masses. Concrete pontoons can be built in ship docks and have a typical configuration consisting of an outer hull stiffened by regularly spaced bulkheads and diaphragm walls. The hull is biaxially post-tensioned to warrant water tightness and to withstand hydrostatic and dynamic water pressures.

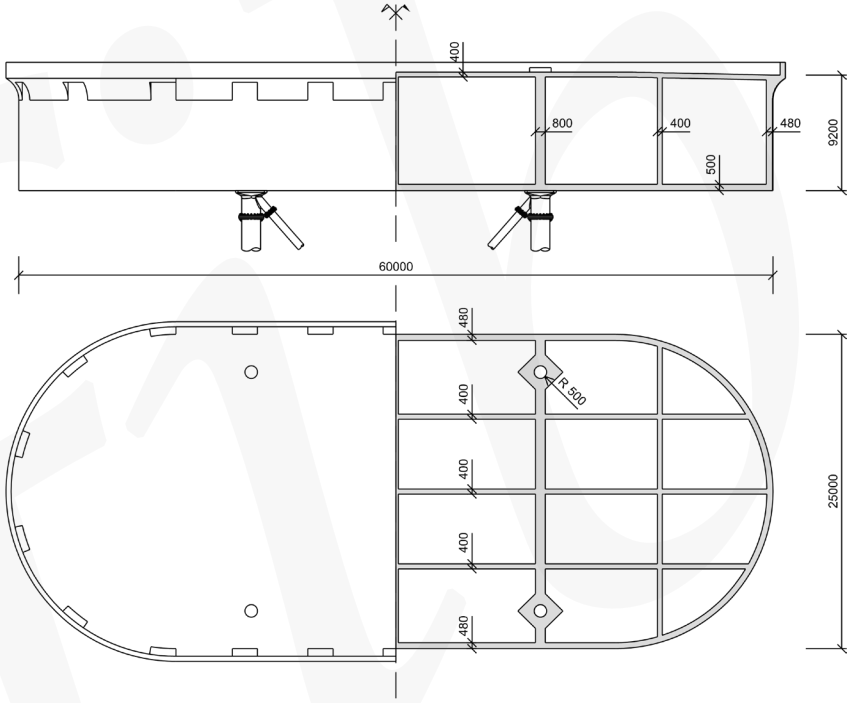


Fig. 7-14 Example of concrete pontoon configuration; plan and elevation (E39 Bjørnafjord SFTB) – Statens vegvesen

For the connection with the tube, different structural typologies can be envisaged, e.g. cylindrical steel shaft and steel tubular truss. The capability of resisting a severe impact from surface vessels brings special challenges for the design of the rigid pontoon-tube connection. A severe impact to one of the pontoons will likely cause extensive deformation to the shaft and some deformation to the pontoon itself. If the dissipated energy is not enough to prevent overload to the tube structure, the integration of a 'fail-safe' mechanism to the connection may be a mitigating measure to preserve the structural integrity. A structural fuse concept was first proposed in the Høgsfjord crossing project in the 90's and has later been developed further for both the E39 Sognefjord and Bjørnafjord crossing, in Norway. From the investigation of different weak link concepts following the design procedure described in Ch. 6.4, a steel truss system, made up of four vertical columns carrying axial action and a cross bracing resisting horizontal loading (Fig. 7-15),

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has been proven to be the best solution. In this connection, the cast steel joints are designed to remain elastic during a ship impact and the bolted connections are designed to break, and release the pontoon, after the braces have fractured.

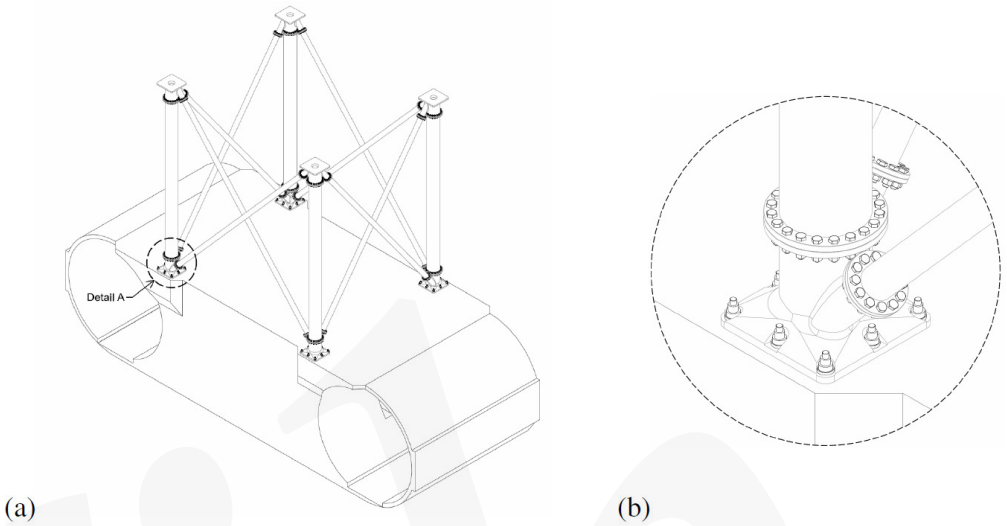


Fig. 7-15 Braced column system designed as 'fail-safe' connection: (a) isometric view and (b) detail of cast steel joints – Statens vegvesen

7.2.2.2 Tether stabilization

A tether-stabilized SFTB is positively buoyant and is tied down by tension in the tethers. The buoyancy surplus can be distributed along the length of the tube bridge, and/or have concentrated buoyancy near the tether supports. The excess buoyancy, mobilized under installation by de-ballasting the tube, is utilized as pre-tension of the tethers to prevent slack in operation, see Fig. 7-21. Vertically, a tether supported tube bridge acts like a continuous beam on rigid supports. Since it is entirely submerged, a tether-stabilized tube bridge is not influenced by tides and water level variations. Besides being invisible, another significant advantage of the tether concept is that it represents no restrictions to ship traffic and thus no risk for ship collision.

A drawback of the tether option is that its foundations are dependent on seabed soil, rock and slope parameters which, despite extensive seabed mapping and marine geotechnical surveys, will always represent uncertainty in the design and in the execution of the foundations. Being anchored to the seabed, a tether-stabilized SFTB may also be vulnerable to submarines, landslides and earthquake.

In Norway, comparative studies of pontoon- and tether-stabilized tube bridge alternatives have recently been carried out for several fjord crossings. The trend has been that the tether alterative is generally the preferred solution even for water depths down to 500 m and partly demanding seabed conditions. The bias towards tethers is mainly attributed to the significant lower excitation to 1st order wave loading, elimination of slowly varying dynamic response as well as no risk of ship impact. An additional positive aspect is the possibility for deeper submergence of the tube to avoid loading from wind generated sea (Fig. 7-1).

For vertical tethers, a tether system has about the same dynamic characteristics in sway as a pontoon system. However, their behaviour in the vertical direction is different. A tether stabilized bridge system involves a minimum of structure and hydrodynamic damping so that natural periods of the bridge shall thus be tuned out of the period

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range of wind generated sea to avoid vertical resonant behaviour. Given the mass and stiffness of the superstructure, the key parameters for tuning the dynamic performance are essentially mooring spacing and group configuration of the tethers. The mooring spacing should be sufficiently small to eliminate higher order modes of vibration within the span. The mooring stiffness, mainly controlled by the aggregated tether cross-sectional area, is increased until the 1st natural periods of heave are supressed below the period domain of wind generated waves.

The tether assembly can be based on available systems developed and proven in offshore TLP applications and comprises a top connector, the tether string, bottom connector and the receptacle that provides the interface between the tether and the foundation. The tether string is fabricated into a single steel pipe close to neutrally buoyant with diameter/wall thickness ratio D/T of ~29.5. The tether pipe is designed to remain void (or dry) for the entire design life. Whereas a TLP will be subjected to sway deformations in the order of 1/10 of the water depth, an SFTB will have significantly smaller lateral movements. There is hence a potential to simplify several of these components (eg. Flex-joints and top connector). Key hardware components of the tether assembly are shown in Fig. 7-16.

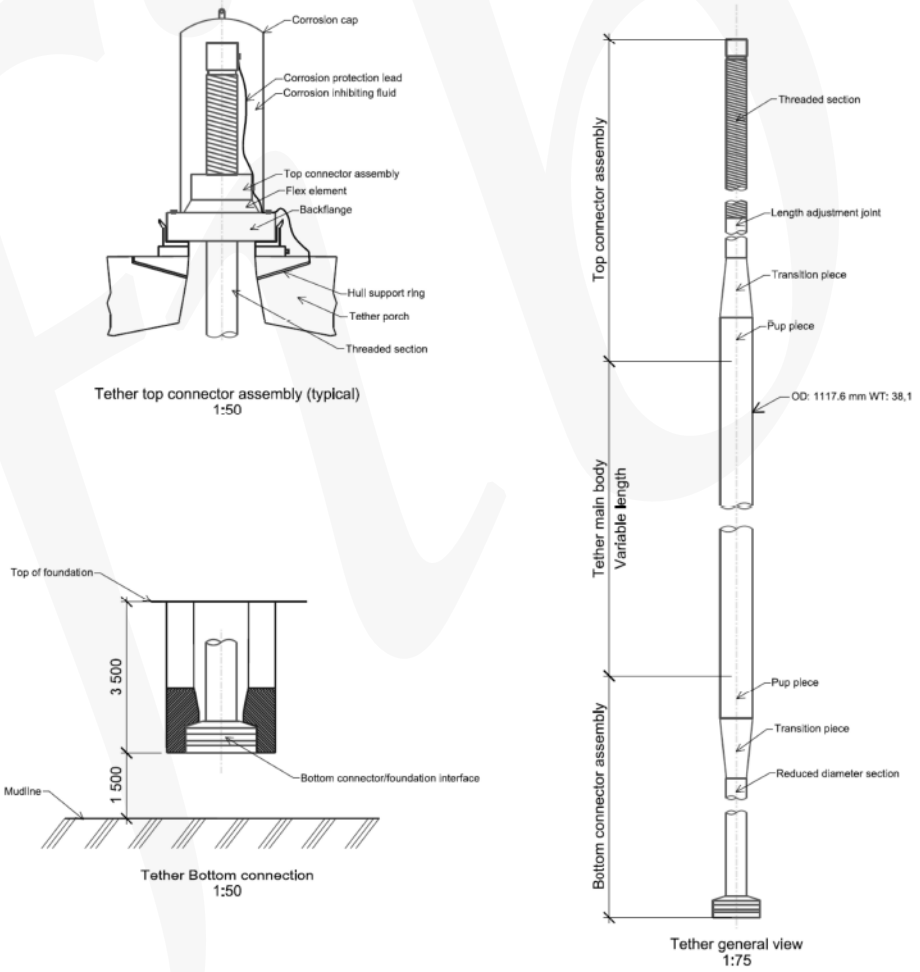


Fig. 7-16 Example of complete tether assembly – Statens vegvesen

The foundation is the key component of a tether-stabilized tube bridge. Therefore, the subsea soil properties are essential in the selection of the type of foundation. Seabed conditions often vary over the crossing trajectory, typically with soft soil in the mid and deeper part,

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turning to harder soil and rock towards the ends, calling for a flexible foundation design. Different types of foundations have been developed for the use in the offshore industry. The most common types are gravity foundation, suction anchors, hybrid suction anchors and friction pile anchorages. Rock pile anchorages are currently at a less advanced phase of development. For the case of gravity/suction anchors, the tether connector receptacle is incorporated into the anchor structure. For the case of the single pile foundation, the receptacle is welded to and installed with the tether pile.

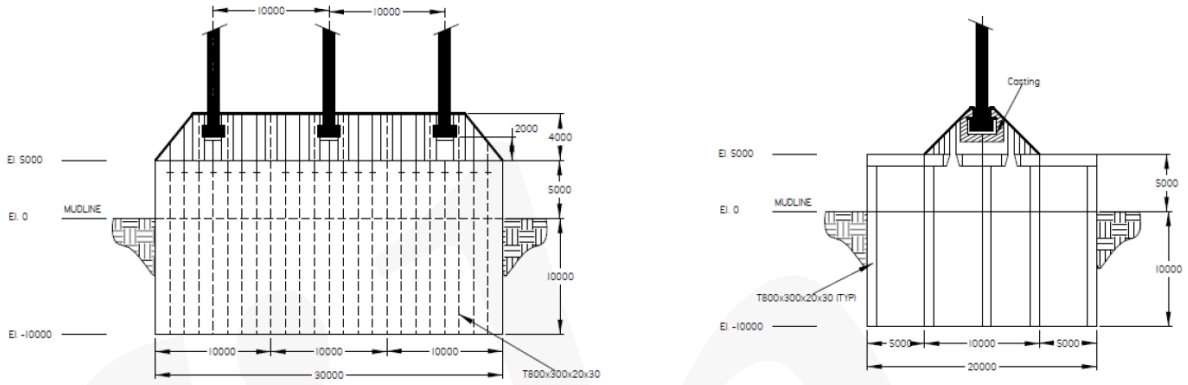


Fig. 7-17 Gravity based foundation for soft clay conditions (E39 Bjørnafjord) – Statens vegvesen

7.2.2.3 Column-supported SFTB

A substructure consisting of fixed tube supports is a somewhat less common, even if, for moderate water depths, it is a highly efficient support concept. Support on columns, which has for example been proposed for the Lake Lugano and Lake Geneva crossings (refer Ch. 14.2), is applicable to water depths down to 100 – 150 m. The columns will restrain horizontal and vertical displacement in the tube, and with reasonable choice of span length all natural eigenperiods of the SFTB can effectively be tuned below the wave energy regime.

To facilitate the construction, the column supports are typically made up of hollow tubular concrete shafts designed to be constructed afloat and each installed on its own keel. Here again, proven foundation techniques applicable for different soil conditions can be adopted from the offshore industry. Firm soils may allow the use of a gravity base concept whereas deeper waters and/or soft soils will require a piled caisson foundation as shown in Fig. 7-18.

A monolithic connection between the tube and the column is preferable with respect to costs and maintenance. An alternative connection based on adjustable, replaceable bearings has been developed in the study for the Lake Lugano AlpTransit crossing[7-9], to improve the concept's flexibility to accommodate changes to the assumptions of the geotechnical parameters.

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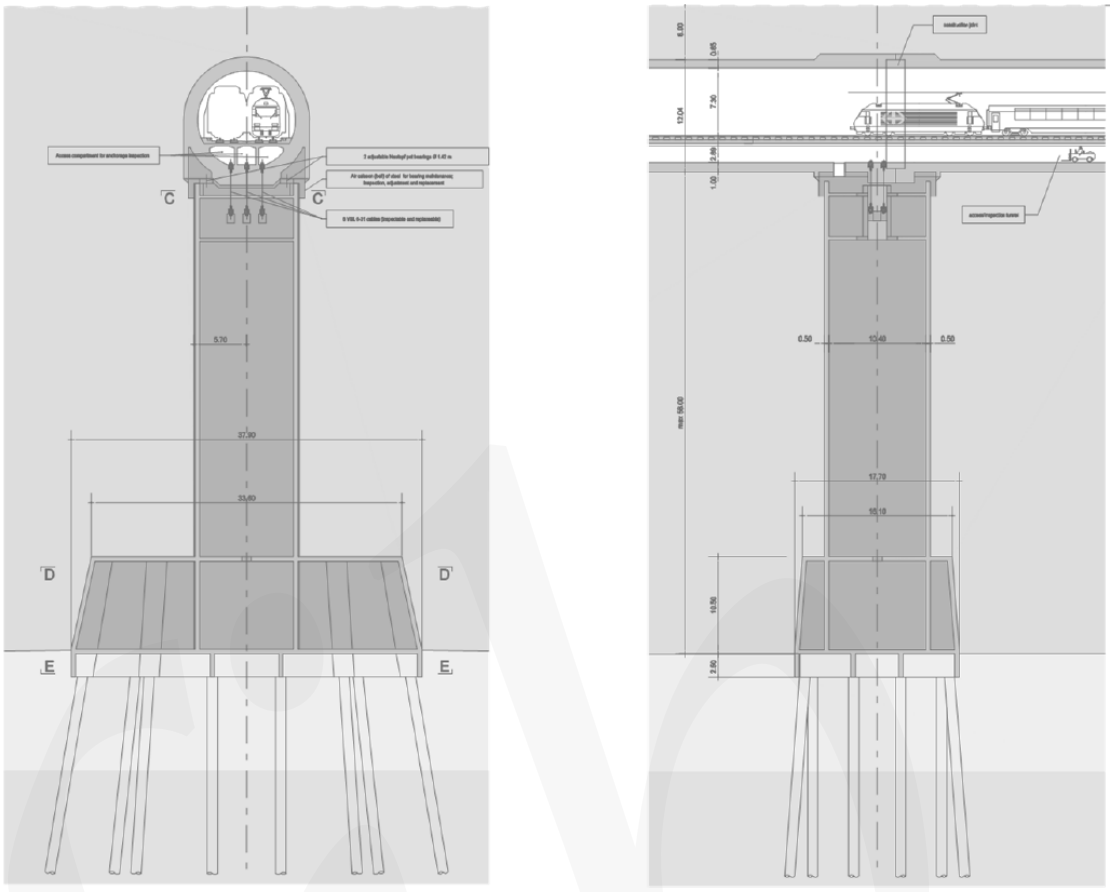


Fig. 7-18 Column support developed for the Lake Lugano Alp Transit crossing[7-9] - Grignoli Muttoni Partner/Selmer/Dr.techn.Olav Olsen

7.2.3 Horizontal stability

In chapter 7.2.2 it is described how an SFTB carries the vertical actions. There are of course also horizontal actions from wave, currents, wind, braking etc. that need to be transferred through the structural system and into the foundations. Pontoons and vertical tethers provide virtually no contribution to the horizontal stiffness of a tube bridge and therefore the structure needs stabilization in sway to control the horizontal displacement. Frequently this is obtained by arch action through curved horizontal alignment. The arch-shaped placement of the tube bridge, commonly referred to as 'end anchorage', enables greater resistance against horizontal actions, provides flexibility for thermal expansion and eliminates the first mode with one half wave between the end supports. Whereas the horizontal arch stabilization may provide adequate stiffness up to crossing lengths of about 2 000 m for single tube SFTBs, it has been proven feasible up to 5 500 m for a twin tube configuration in the E39 Bjørnafjord crossing study in Norway (see Ch. 14.2).

For a tether-stabilized tube bridge, additional horizontal stiffness may be obtained by inclined tethers as seen in several of the crossing studies outlined in Ch. 14.2. Among the various mooring configurations proposed, a system of 4 inclined tethers as shown in Fig. 7-19(a) has been proven especially effective as it also provides significant torsional restraint. Horizontal stiffening by means of inclined tethers will, however, require a significant mooring stiffness and due to limited experience from installation and real-life performance of inclined tethers, most recent studies with tether-stabilized SFTBs tend to explore other means to enhance horizontal stiffness.

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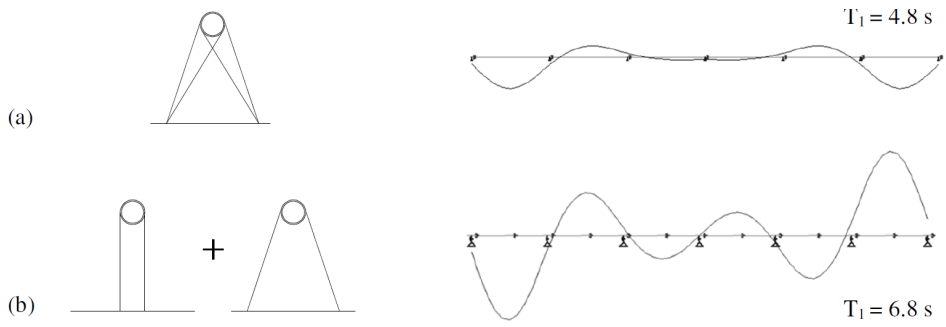


Fig. 7-19 Evaluation of possible mooring configurations based on their fundamental sway vibration – Dr. techn. Olav Olsen

Relative to the horizontal eigenperiods of the SFTB, the wave action is a narrow-banded process as can be seen in Fig. 7-20 (a). This allows the tuning of the eigenperiods, with sufficient gap margin, to not coincide with the peak wave energy periods. Changes to the cross section will typically move all eigenperiods, whereas geometric stiffness might be used to move eigenperiods of specific modes only. For a twin tube bridge the main parameters would be the shape of the horizontal alignment and the tube spacing as illustrated in Fig. 7-20 (b). It will not be possible to totally avoid resonant behaviour, but implications will be minimized.

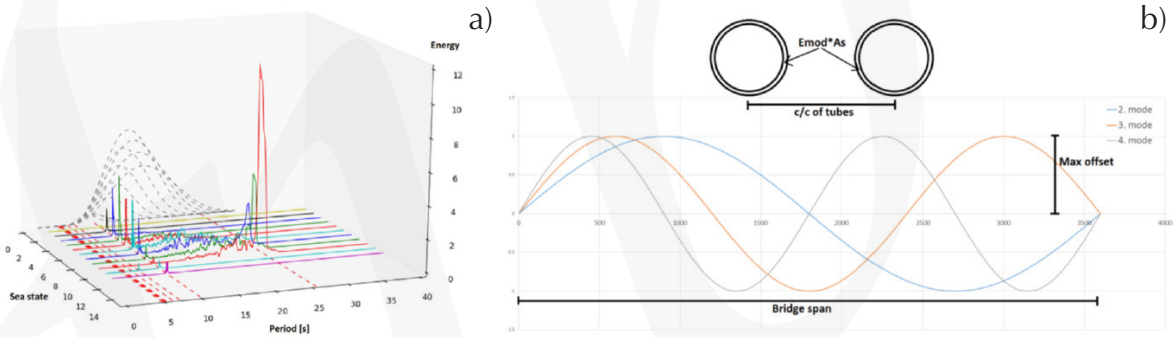


Fig. 7-20 (a) Wave power spectrum density plot for relevant sea states with eigenperiods indicated by red dashed lines and (b) example of tuning parameters for the dynamic behaviour of an SFTB – Dr. techn. Olav Olsen

7.3 Design conditions

7.3.1 Temporary design conditions

Construction and installation shall be planned in compliance with recognized provisions and regulations for marine operations (refer Ch. 3), as the transportation will have to be insured and consequently approved by a marine surveyor. Depending on the methods chosen, certain stages of tube construction, assembly and installation are more critical than others and will need to be considered.

Owing to the significant longitudinal prestress, the concrete tube elements are well suited for transportation and offshore towing will normally not govern the design of the tube elements. During immersion and mooring stages, the actions on the elements, e.g. from bulkheads, temporary ballast tanks, buoyancy and access shafts, are considerable and their arrangement should be specified. During the assembly, the floating bridge structure may be susceptible to resonant behaviour from sea states with various periods.

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The temporary design conditions should be specified at the early design stage and the structure should be designed and checked that it can withstand all the conditions implied by the temporary stages.

7.3.2 In-place condition

For in-place conditions we mean the final configuration of the structure, with all the elements connected in the final scheme.

For this condition, the analyses described in the following chapter should be performed, taking into consideration the design life defined for the structure that will influence the actions that should be taken into account in the analyses.

7.4 Structural analysis

7.4.1 General

The structural analysis of an SFTB can be divided into a quasi-static response analysis and a dynamic response analysis. The results of the analyses are then combined and used as input for the structural design of the structure.

Due to the amount of data processed for the input and output, and considering the large number of variables and necessary iterations to achieve an optimal solution for the design of the structure, automation algorithms to handle the input and output data are strongly recommended, to minimize the possibility of errors. For the study developed for the Bjørnafjord crossing, in Norway, a GreenBox-system was utilized for this purpose[7-7].

Among the design parameters, the size of the tube is generally governed by global functional requirements and this is also partly valid for the road alignment. The thickness of the cross-section walls is generally defined by the required space to locate the reinforcement, the pre-tensioning system and the concrete cover. This means that the mass of the structure is mostly defined and the optimisation of the structure should be based on the curvature, on the vertical stabilising system and on other elements that have usually a lesser effect on the general structural behaviour (typically in the range of 10-20% of the response). The possibility of tuning the response of the structure with the choice of the material (e.g. light-weight aggregate concrete) is an additional possibility for optimisation of the design.

Accuracy of the design loads, especially regarding the extreme values, is of a fundamental importance for the design of this type of structure. This implies investigation of a significant number of load cases, both static and dynamic, and consequently running a consistent number of simulations.

Additional considerations are necessary for the analysis for an SFTB: for long-span bridges the aerodynamic actions are the governing ones in the design and the analysis is normally solved by frequency-domain approach. This is based on the possibility of linearize loads and responses. But for an SFTB the correlation between loads acting on different parts of the structure is very important, so they should be combined in both time and space. This implies the need of a time-domain analysis, that has high computational costs.

A simplified approach is therefore necessary to start the design of such a complex structure, to identify the potential influence of changes of the design parameters, before

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running more advanced analyses, and to create a sensitivity analysis so that the designer can keep control of the results produced by the computational tools.

7.4.2 Simplified approach

The global behaviour of an SFTB could, for example, be evaluated applying a modal analysis procedure[7-10]: assuming shape functions of displacements, the eigen periods of a system can be calculated from modal masses and modal stiffnesses. In this way the modal frequency response function can be found and with it the magnitude of the expected response of the system. In addition, the displacement spectrum can be described, and the maximum displacement and the associated moments can be found.

Describing the whole structure with the Euler-Bernoulli beam, the shape function for mode n (variable of the length x) is:

$$\phi(n,x) = e^{-\beta(n)x} - \cos(\beta(n)x) + \alpha(n)\sin(\beta(n)x) - (-1)^n \frac{e^{\beta(n)(x-L)} - e^{-\beta(n)(x+L)}}{1+e^{-2k(n)}} \tag{7-1}$$

The modal mass $M_m(n)$ and modal stiffness $K_m(n)$ are calculated with each shape function Φ and, with them, it is possible to calculate the circular frequency:

$$\omega(n) = \sqrt{((Km(n)) / (Mm(n)))} \tag{7-2}$$

And the natural period:

$$T(n) = 2\pi / \omega(n) \tag{7-3}$$

The modal damping $C_m(n)$ is expressed by total damping ratio ξ and the modal mass and stiffness for each mode:

$$C_m(n) = \xi(n)\sqrt{((Mm(n)) / (Km(n)))} \tag{7-4}$$

Knowing these values, the modal frequency response function can be calculated, that is the magnitude of the circular frequency. This shows for which frequencies the structure will become excited.

The simplified method will give an idea of the global response of the structure that, compared with the exciting load applied to the specific case, could help the designers to understand if the main geometrical parameters chosen could be a viable solution for the specific study or to change them before starting detailed and time-cost-demanding analyses.

A detailed description of the method and a case-application is shown in[7-10].

7.4.3 Quasi-static response analyses

Quasi-static analysis can be carried out with a general finite element program.

In the analysis, all permanent and static actions are applied to the structure. These actions are, for example, self-weight of the structure, structural elements, equipment, asphalt, ballast and so on, including the weight of the relocatable elements (e.g. asphalt and equipment).

These actions, together with the traffic load (which is a quasi-static load because it is time-dependent but slow enough so that the inertia effect can be ignored), are the governing loads in the analysis.

For a long crossing with several traffic lanes, the traffic loads can generate a large number of load cases to consider and for this reason specific programs developed for bridges are

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recommended, in order to handle traffic loads and combinations of a large number of load cases in an easier way.

Comparing the pontoon and tether solutions, we can generally expect larger loads on the pontoon SFTB, which can require a higher cross sectional stiffness to withstand the response from the loads. In addition, depending on the location of the bridge, tidal actions could introduce large moments in the structure.

Other actions can also be added directly in the design check, such as post-tensioning and outer-water pressure.

The accidental loads of explosions, flooding and collisions, like ship impact, can also be performed in a dedicated analysis and then combined with other actions.

Load combinations should follow the applicable regulations (reference Ch.3).

7.4.4 Dynamic response analyses

For global dynamic response analyses of an SFTB, the time domain approach is suggested, as explained in Ch. 7.4.1.

The global dynamic response of the structure is caused by fluctuating environmental loads from wind, waves and currents, that shall be combined following the procedure described in the applied rules (reference Ch.3). For the initial feasibility evaluation of the chosen solution, a minimum return period of 100 years is suggested for all the environmental loads, to ensure a robust design that could be optimized in a further design step. This differs from what is usually applied for offshore structures (reference, for example, NORSOK and DNV codes), which assumes a non-concurrent action of the environmental loads. Nevertheless, the conditions that create the extreme values for the different environmental loads can vary, depending on the position of the crossing. In addition, it has to be considered that the structure can be very sensitive to the environmental loads and the extent of knowledge of them is not always complete at the beginning of a feasibility study. Therefore, it is strongly suggested that a full correlation of the environmental loads is considered at the beginning the feasibility of the structure.

The evaluation of vortex induced vibration (VIV) can be, for an early stage screening analysis, evaluated with a simplified method based on the DNV-GL-RP-F105, to understand if just in-line VIV or also cross-flow VIV can occur and it can be further analysed in a detailed design stage. The validity of this method has been demonstrated with specific analyses in the wind tunnel test[7-11].

The wave actions on the main structure and on tethers can be represented by Morison formula (ref. Ch. 6.3.5), while the wave actions *s* on the pontoons shall be calculated from a three-dimensional wave diffraction analysis.

The global dynamic analysis shows that it is possible to limit the global response when working on the design, for example changing the horizontal shape of the crossing that could influence specific modes.

Working on the cross-section, in terms of elements and connections, it is possible to reduce the stress level, for example in the main tubes, so as, in general, to control the stress level of individual parts of the structure. In the cross-section design, particular attention has to be paid to the optimisation of the dimensions of the structure, due to the fact that a larger cross section has a larger added mass to take into consideration and this could result in worse performance in respect of environmental loads, especially for long crossings.

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To limit the dynamic response of an SFTB, the depth of submergence can be increased, thus reducing the wave action on the structure. For a tether stabilised SFTB, the stiffness in the tether system can influence the vertical eigen periods. In terms of cost, it is efficient to increase the tether size rather than the number of tethers, due to the high cost of the sensor equipment and connectors necessary for each tether. For a pontoon structure, the required in-plan water area is largely governed by the required vertical stiffness, to avoid deformations during the quasi-static load conditions. The required draught of a pontoon is governed by the lowest wave passing through in combination with the maximal vertical translation, in order to avoid slamming on the bottom plate.

The sensitivity to damping is dependent on how close the eigen period is to the spectral peak period of the excitation, from the environmental loads. An increase in damping results in a less dynamic response and this is more influential for a tether SFTB than for a pontoon SFTB, because the pontoons introduce the main damping to the structure. To have a conservative setup, a damping level lower than 0.8% is recommended in the analyses. Higher values can be considered but shall be properly demonstrated.

The complexity of the analyses requires a procedure of control and validation that should be planned from the start, taking into account:

- The experience of the designers with the calculation tools
- The possibility to check the calculation using different computational tools

The importance of performing a third-party verification of the calculation, to be sure that the hypotheses used are not affecting the validity of the results

7.5 Structural design

7.5.1 General

Adhering to the Eurocode design format (reference Ch.3), the structural design of an SFTB involves in principle three control stages at ultimate:

1. Global stability (EQU), checking that the system configuration is maintained throughout the Ultimate Limit State (ULS) combinations. The control goes on slack in tethers for the bottom anchored concept, and on allowable free board for the pontoon concept.
2. Member strength (STR), involving design control of structural elements in ULS: tubes, cross bars, pontoons and towers and, for the moored concept, the tethers with foundations.
3. Foundation control (GEO), checking soil capacity during installation and operation. The critical stage is the foundations with dead weight after installation before unloading by tether tension. Settlement calculations are also crucial for the structural integrity.

The serviceability limit state design will mainly concern control of water tightness in the main tube and conformance with deformations and acceleration criteria in order to ensure an adequate level of user comfort. Water tightness of the tube is crucial for the integrity of the SFTB. Special requirements are therefore included for the SFTB design to ensure that the structure is watertight for all design stages and events. For an all-concrete tube, complete water tightness in SLS is ensured by maintaining residual membrane compression under the

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most adverse combination of characteristic loads, requiring the application of longitudinal and hoop prestress. For accidental events and ultimate limit states, the criterion for water tightness is a strain limitation in the reinforcement ensuring no plastic yielding to occur in the rebars.

It is an indispensable requirement for an SFTB that the level of safety is at least as good as for other bridges or tunnel crossings. Consequently, the effects of structural loads arising from potential accidental events such as ship impact, explosion, fire, loss of tether/pontoon and earthquake must have the fullest consideration.

The results of the static and dynamic analyses are combined and used as input for the design of the structure.

7.5.2 Mooring system

As described in Ch. 7.2.2, the tethers are pretensioned and the choice of nominal pretensioning level is highly dependent on the hydrodynamic actions: a high hydrodynamic tether force results in a high value for nominal pretension. This then influences the foundation weight requirement.

The tether response is governed by a combination of static forces and environmental forces typically from waves. The tethers are the governing elements for the natural periods in the heave direction. A high axial stiffness and therefore a high steel area are required to control the heave and roll natural periods. The choice of tether cross sectional area is therefore based on stiffness properties rather than tether capacity. Vertical tethers will not contribute significantly to the stiffness in sway.

The variation of the static loads from the main structure acts on the tethers, giving a maximum and minimum tension that shall be combined with the effect of the actions that are acting directly on the tethers, like marine growth, tether weight, drag forces and so on. The variable static actions acting on the tunnel also influence the tether design, as for example the traffic loads, water absorption and others. A scenario where the tunnel is rehabilitated and the weight of asphalt, structural elements and equipment are totally removed should be carefully evaluated because it can represent the maximum tension in tethers.

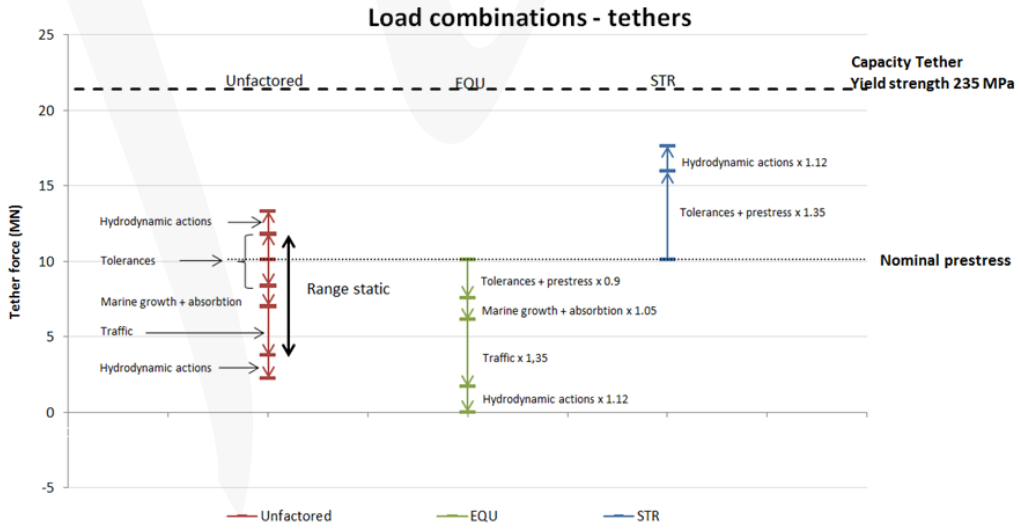


Fig. 7-21 Tether design criteria with response breakdown – Statens vegvesen

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Misalignment of foundations and settlements should be evaluated for a single element or considering the effect for the whole bridge.

The loads from the quasi-static analyses; both dynamic one and the permanent loads shall be combined in order to find the worst case for the tethers that will govern the design.

The EQU combination determines the value for static tension for avoiding slack in tethers and the EQU load case is also the governing case for determining the foundation submerged weight to avoid uplift of the foundation element.

Consideration of fatigue and design life of tethers is recommended, to minimize the maintenance of the structure. A design life equal to the one of the main structure is recommended for the tethers (reference Ch. 11).

Accidental states such as submarine impact, loss of one tether and similar, shall be taken into account in the design.

7.5.3 Pontoons

For the tethers, the loads from the quasi-static analysis, dynamic analysis and permanent loads are combined to find the governing case for the design of the pontoon elements.

The pontoons experience variation of draught from static variable loads acting on the main bridge and on the pontoons itself. These loads are, for example, marine growth, water absorption, variation of self-weight, asphalt variation, traffic and so on. The maximum draught shall be then evaluated to ensure the vertical stability of the structure as described in Ch.7.2.2.

The design loads for the pontoons walls and plates usually are:

- Wind
- Current
- Water pressure
- Wave pressure
- Added mass pressure
- Snow and ice

Accidental states like the filling of pontoons compartments in case of a ship collision should be taken into account and the draught of the pontoons shall be checked.

7.6 Safety concepts

As an SFTB represents a novel concept without previous service history, safety and robustness is a highly important issue when developing new crossing concepts.

7.6.1 Robustness

Robustness is normally meant as the structure's ability to sustain unexpected exposures during operation without undergoing global collapse or overall damage that is out of proportion to the event. The requirement on maintaining global stability in the case of a single element failure is a means to incorporate robustness in design, which is a normal requirement for offshore structures.

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For SFTB development a wider definition is suggested: concept robustness is the ability to withstand unexpected situations, both in upcoming design and in operation, without the concept undergoing major changes. Modified input parameters in the form of wave height, current velocity or seabed parameters outside the values used in the design, should be taken into account as additional "unexpected" events.

A common way to achieve robustness is to perform sensitivity studies on relevant parameters. During the design stages of an SFTB several key parameters for the crossing should be investigated:

- Diameter of tubes
- Alignment
- SFTB length
- Depth profile
- Environmental conditions

The requirement can be set out in the form of local and global robustness, respectively:

- **Local robustness**: The ability of the cross section, including vertical stabilizing system, to prevent a damage event leading to progressive collapse. Fatigue stoppers, ductile connections, twin beams and pontoon compartmentation are relevant remedies. The unforeseen events will be local; thus the problem is initially kept within the cross section.
- **Global robustness**: The ability of the structural system to prevent progressive development leading to system failure caused by a local cross section damage. For example, the global system is the bridge beam which in the case of a local damaged cross section shall survive environmental conditions.

A way to design for robustness is to implement a certain degree of redundancy in the system by securing global stability with single element failure. This can be successful for sudden accidental actions. Redundancy within tether or pontoons, in case of a loss of one element, is an example of this.

Another way to implement robustness is to design a structural system that can be modified in the event of an unforeseen situation not covered by the design, and this can be a solution for longer term changes in the governing actions. Adjusting the ballast, in case the unforeseen action is creating slack in the tethers, is an example of applying this method. This also shows the importance of a permanent monitoring system on tether forces. The pontoon robustness lies in the reserve of free board capacity and ballast capacity inside the pontoon. The measuring of the draught of the pontoons can be used to monitor the loading in the main structure.

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8. Construction

Submerged Floating Tube Bridges (SFTBs) have been proposed as alternative strait crossings for more than a century. Available technical studies highlight design challenges, possible locations and anticipated construction methods and installation procedures. The experience from construction of SFTB's is not very large, and the construction and installation insights are hence based on extrapolation of knowledge of related construction works, as immersed tunnels, bridges and offshore structures.

As is the case with many other civil engineering structures the design principles and construction methods of SFTBs are strongly comparable and the construction method may dictate the design configuration to be developed, and vice versa.

8.1 Comparison with existing structures

Although the experience with constructing SFTB's is limited, the various components of the SFTB can fully rely on proven technology. Principles of the construction of SFTBs show similarities to the construction principles of immersed tunnel structures and of caisson and tower construction of large gravity foundations of offshore platforms. Principles of the tensioned cable foundations can be derived from floating offshore platform moorings. Example structures are therefore discussed below.

8.1.1 Comparison with immersed tunnel construction

In several studies, it is emphasised that SFTB construction does not necessarily require completely new tunnel technology, rather technology from immersed tunnel construction can be used. Though deformation and stresses in the tunnel tubes may be different, the tunnel sections of SFTBs and immersed tunnels are made of similar materials, have a similar size and can be interconnected using the same principles. Tunnel elements can be made of concrete or steel, using steel shell or sandwich structures. Large elements can be constructed in dry docks and then be transported in a floating condition after flooding of the construction docks. Steel shell tunnels can be constructed above the waterline and can be launched from ramps like ship hulls. A submerged dry dock system for concrete tubes could avoid the problems related to the connections of the elements, but attention shall be given to the ballasting procedure.

Immersed tunnel construction is based on the method in which temporarily closed elements are transported floating (with typically 5 or 10% buoyancy) and when arrived at their exact location are ballasted and immersed and carefully lowered to their final position in a trench in the seabed. The elements are connected utilising the water pressure on the bulkheads as axial pre-load. The joint typically consists of two rubber profiles, one for temporary and one for permanent sealing. Reinforced concrete joints or welded steel joints can also be made as a permanent joint.

The design of immersed, sub-seabed tunnels, including their joints, is based on the principle that the tunnel elements rest on the soil. They are articulated to be able to 'follow' soil settlements. Although the joints are flexible, large deformations do not occur because the soil around the tunnel section embeds the structure and restrains motion.

This chapter was mainly authored by Peters.

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The SFTB is surrounded by water. The water does not restrain movement of the tunnel elements. On the contrary it will – through the flow patterns and pressure gradients caused by currents and waves – exert motion of the tunnel elements. It is hence a question whether construction joints in an SFTB should provide flexibility or should be stiff and strong. The design criteria that indirectly dictate decisions on strength and flexibility are reliability of components (risk of overloading and risk of too small a resistance and/or material strength), and motion comfort criteria.

Principles of how to construct the SFTB concepts have a close link with the basic design considerations.

There are three concepts of providing lateral resistance (refer to Fig. 8-1). In the first concept the inertia and the damping of the system provides resistance against fast movements and significant accelerations. Using that concept, an SFTB anchored to deep water foundations at 500 m depth, could have a lateral motion response of e.g. 5 m to tidal current, and in the same time will have negligible response to surface waves. This is caused by the very low natural frequency and hydraulic damping of the system. This system requires a sufficiently flexible tube structure and might require flexible joints and a flexible shore connection. The system can be constructed and installed element by element.

The second concept uses lateral mooring lines, like offshore platform and anchored ships. The system does not require flexible joints. For long tunnels element by element installation is preferred. The third concept utilises shape to obtain horizontal stability of the SFTB, giving the tunnel alignment an arch shaped curve. Uniform lateral loads will be transferred into axial forces and bending moments in the tunnel tube which can be designed for that. The first (full length) lateral bending Eigenmode is omitted. An important advantage is that thermal expansion can be treated as lateral deflection. The tendons are vertical. The construction method can be based on installation as one piece. In the case of jointed construction temporary lateral supports may be needed.

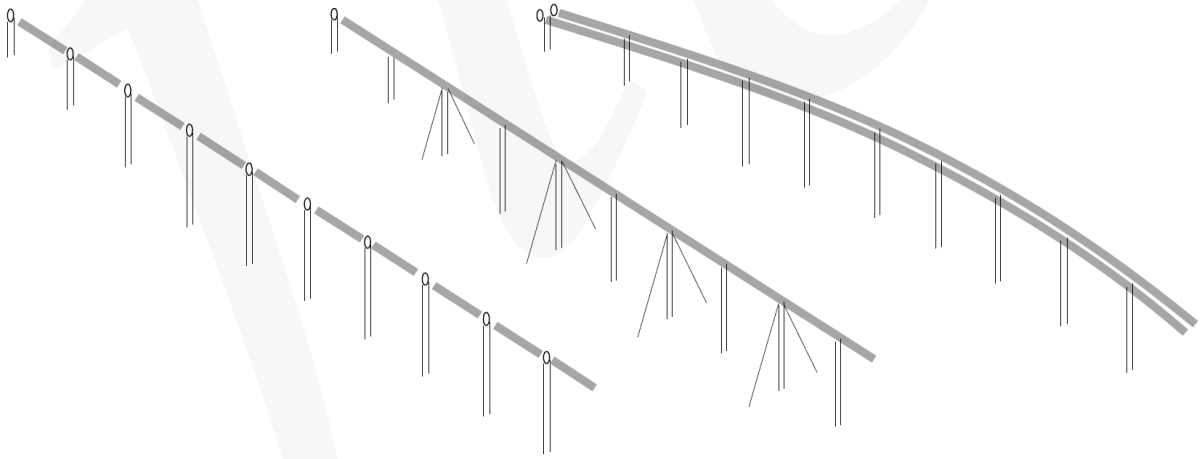


Fig. 8-1 SFTB lateral stability concepts, a) stable through inertia, b) stable through mooring lines, c) stable by shape

The elements need to be jointed to create one continuous tube. In the case that the joints are expected to remain flexible, a solution with a permanent rubber gasket and seal and bolt that compress the joints can be used. For the case where the joints require similar (or larger) resistance and stiffness to the tube sections the connection can, for example, be equipped with high-strength pre-loaded anchor bolts and can be concreted. Underwater joint construction can be done using a temporary cofferdam.

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8.1.2 Vertical stabilisation of floating offshore platform construction

The SFTB could be vertically stabilised with tethers, connecting the structure with the seabed. The tethers are well-known structures, used for offshore platform construction since 1980.

Floating offshore platforms exist in various forms. Tension leg platforms have foundation concepts that are used for SFTB planning as well. Tension leg mooring systems[8-1] are used for position keeping of Tension Leg Platforms (TLP's). Fundamentally, a TLP consists of a buoyant hull, which supports the topside and well system, anchored by rigid tendons to a seabed foundation. The tendons allow lateral movements (sway, surge and yaw) while retraining vertical movements (heave, pitch and roll). The excess buoyancy of the hull, greater than the platform weight, pretensions the tendons and as such limits the horizontal offset to a prescribed watch circle.

The first TLP was the Hutton TLP in the North Sea, at only 145 m water depth. The deepest application of a TLP to date (2013) is the Big Foot platform in the Gulf of Mexico in about 1750 m water depth, which is considered as the current commercial limit with existing tendon technology. New tendon technologies will be required to extend their application beyond this depth.

The design of the tendons is closely related to the design of the TLP. The tendons provide axial stiffness which is defined by the cross-sectional area. Also, the tendon needs to be able to allow rotation at its top and bottom connectors and the tensile force should always be positive. The maximum stress in design wind, wave and current conditions shall be less than the design allowable stress.

Pre-tension forces and first order wave induced tension are the largest components of total tendon tension. Other stress components are induced by second order wave loads and by Vortex Induced Motions (VIM) in hulls and Vortex Induced Vibration (VIV) of tendons excited by currents that also may create fatigue stresses in the tendons.

The tendon diameter to wall thickness (d/t) ratio, is designed so that each tendon is neutrally or positively buoyant, so as not to increase hull buoyancy requirements or tendon axial stresses. A d/t-ratio greater than 32 will achieve this requirement for steel tubular tendons and is possible in water depths to about 1,000 m. Beyond this depth, stepped outer diameter tendons will achieve this objective.

8.1.3 Horizontal stabilisation of floating offshore platform construction

Horizontal stabilisation of an SFTB could be provided by the horizontal alignment or by external elements. It is also a need that could occur, for example, during the construction phases. This technology is taken from that used for floating offshore platform construction.

For horizontal positioning, catenary or taut wire systems are used.

The catenary system is the most common type of mooring system employed in shallow water. The catenary refers to the shape that a free hanging line assumes under the influence of gravity. The catenary system provides restoring forces through the suspended weight of the mooring lines and its change in configuration arising from vessel motion. In other words, under environmental loadings the moored vessel tries to lift the mooring lines, which creates a restoring force. The mooring line terminates at the seabed horizontally. Hence the anchor point at the seabed is only subjected to horizontal forces. This requires

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that the mooring lines should be relatively long compared to the water depth. With the increase of the water depth the weight and the length of the mooring line start to increase rapidly. In deep water the weight of the mooring lines becomes excessive and the mooring lines tend to hang directly down from the structure. The excessive weight of the line reduces the working 'payload' of the floating offshore structure. To overcome this problem synthetic ropes are used as mooring lines.

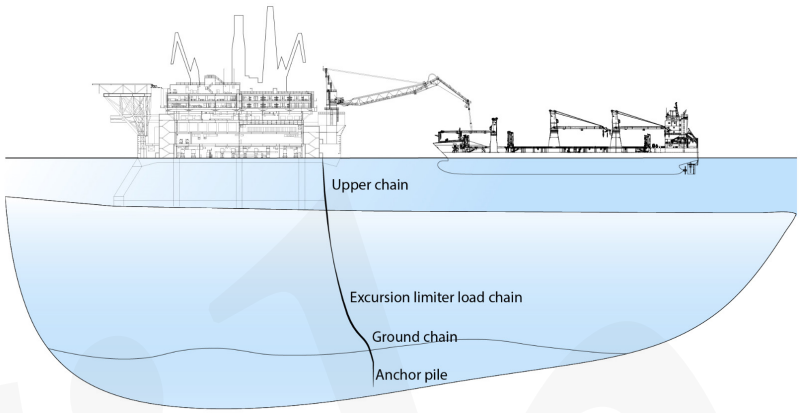


Fig. 8-2 Catenary wire principle

As water depth increases, conventional catenary systems become less and less economical. For permanent systems a cost-effective, synthetic alternative has been developed - the Taut Leg Mooring.

In this system each line is strung out directly to the seabed typically at an angle of 45°. As the platform drifts horizontally with wind or current, the lines stretch and this generates an opposing force. Polyester mooring lines have the right restoring characteristics and thus are the preferred choice for this application. Because of the angle at which the rope arrives at the seabed the anchor will be loaded vertically as well. Suction piles or special anchors, so-called Vertical Loaded Anchors or VLAs have to be used. The figure below shows a typical make-up of a taut-leg.

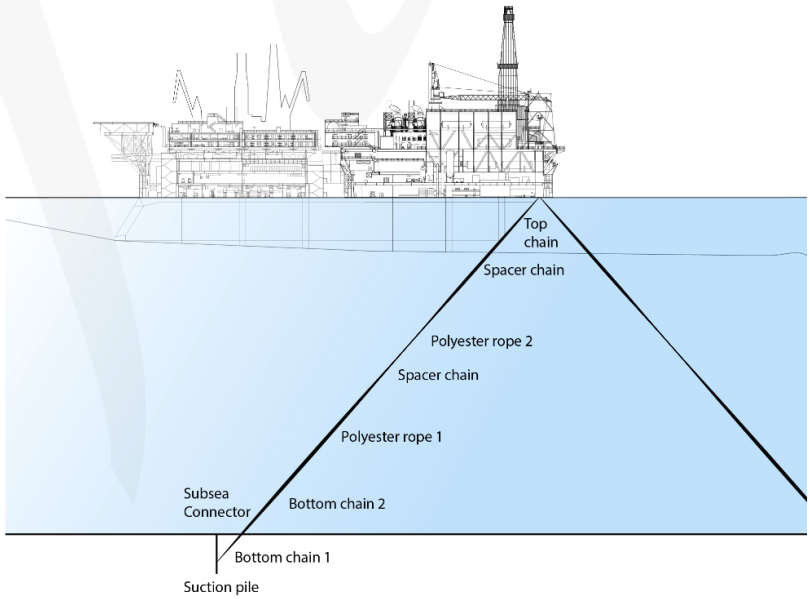


Fig. 8-3 Taut wire principle

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The foundation of the seabed anchor point of taut and catenary wires can consist of ballasted caissons or piles.

The drag embedment anchor[8-2] is the most popular type of anchoring point available today. The drag embedment anchor has been designed to penetrate into the seabed, either partly or fully. The holding capacity of the drag embedment anchor is generated by the resistance of the soil in front of the anchor. The drag embedment anchor is very well suited for resisting large horizontal actions, but not for large vertical actions, although there are some drag embedment anchors available on the market today that can resist significant vertical loads. Traditional drag anchors are unlikely to be used for SFTBs.

The pile is a hollow steel pipe that is installed into the seabed by means of a piling hammer or vibrator. The holding capacity of the pile is generated by a combination of the friction of the soil along the pile and lateral soil resistance. Generally, the pile has to be installed at a great depth below the seabed to obtain the required holding capacity. The pile is capable of resisting both horizontal and vertical loads.

Like the pile, the suction anchor is a hollow steel pipe. But unlike the pile the suction anchor is closed at the top and generally has a much larger diameter than that of the pile. The suction anchor is forced into the seabed by means of a pump connected to the top of the pipe. When the water is pumped out of the suction anchor this creates a pressure difference between the outside of the pipe and the inside forcing the anchor into the seabed. After installation the pump is removed. The holding capacity of the suction anchor is generated by a combination of suction and friction of the soil along the suction anchor and lateral soil resistance. The suction anchor is capable of withstanding both horizontal and transient uplift loads.

8.2 Submerged Floating Tube Bridge types

SFTB types can be distinguished based on their support system. Negatively buoyant SFTBs can be suspended by a series of pontoons or bear on a series of pylons built on the sea floor. Positively buoyant SFTBs are tied down to the seabed using tendons, catenary anchorages or taut wires.

For all three types, the elements can be fabricated elsewhere and transported to their final position. Negatively buoyant SFTB elements require temporary ballasting to pass the waterline, after which they need to be held in vertical position by the pontoons or the piers. After that, permanent ballast material can be applied to replace the temporary ballast.

Positively buoyant tunnel sections need to be brought into their vertical position first. Connecting and tensioning of the tendons and wires can be done with simultaneous de-ballasting of the element.

Longitudinal tunnel construction sequencing can be carried out as follows:

1. Element-by-element fabrication, transportation and installation, anchoring and securing the elements in their final position
2. Construction of elements or section at the shore and incremental launching or the tunnel
3. Assembling of the elements into a completed tunnel structure, transported and installed as one completed structure

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Element-by-element fabrication and installation is promoted as the common method in many studies. This method is closest to known technology for immersed tunnelling, cable stayed bridges and cantilevering bridges. Launching of a tunnel with increasing length has similarities with launching of bridges. The most important advantage is that the construction site of elements and making of joints is at a fixed location.

An example of the first construction method is the study done for the Lake Lugano Crossing. Plan and vertical views are shown below.

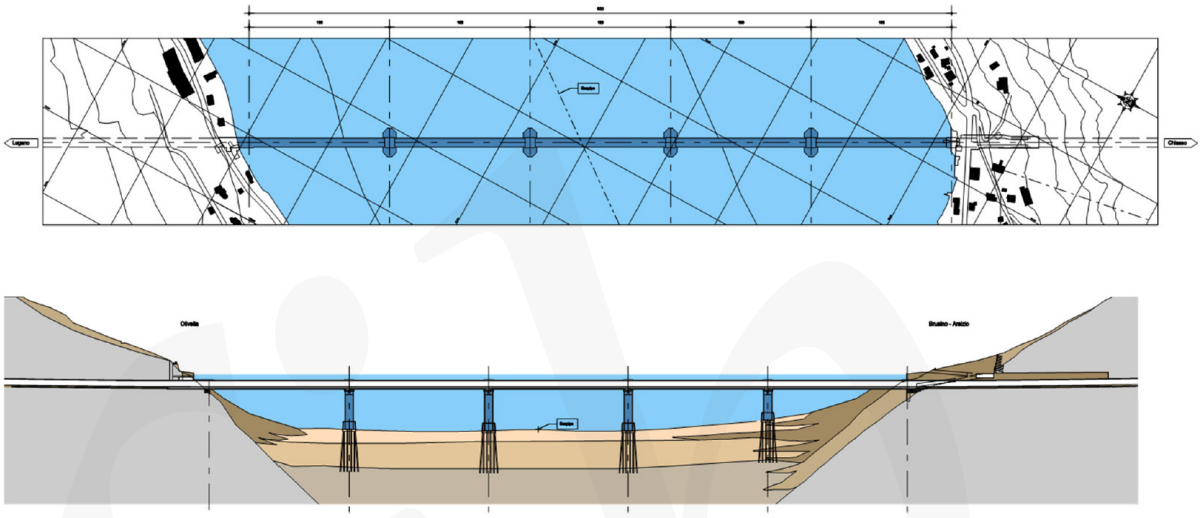


Fig. 8-4 Plan and vertical views of the proposed Lugano SFTB crossing, The Lake Lugano Crossing – Grignoli&Muttoni & partners , Dr. techn. Olav Olsen

Both the vertical supports and the tunnel elements were planned to be prefabricated in a dry dock integrated in the eastern landfall excavation, shown in the following picture.

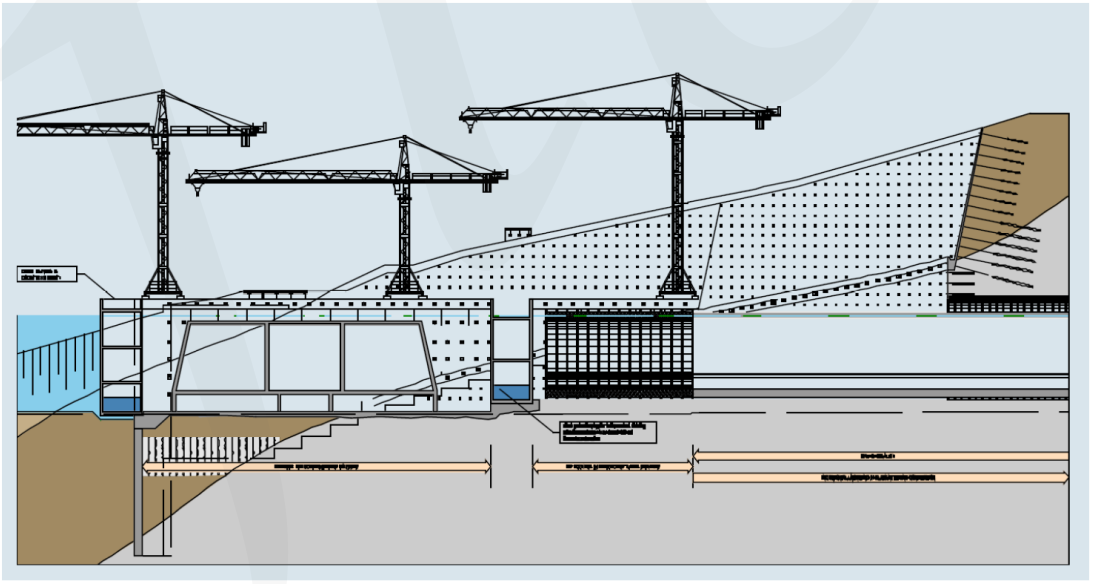


Fig. 8-5 Longitudinal section dry dock, The Lake Lugano Crossing – Grignoli&Muttoni & partners, Dr. techn. Olav Olsen

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The elements are then towed to the site and the connection is completed, following a pre-defined installation sequence partially shown in the following picture.

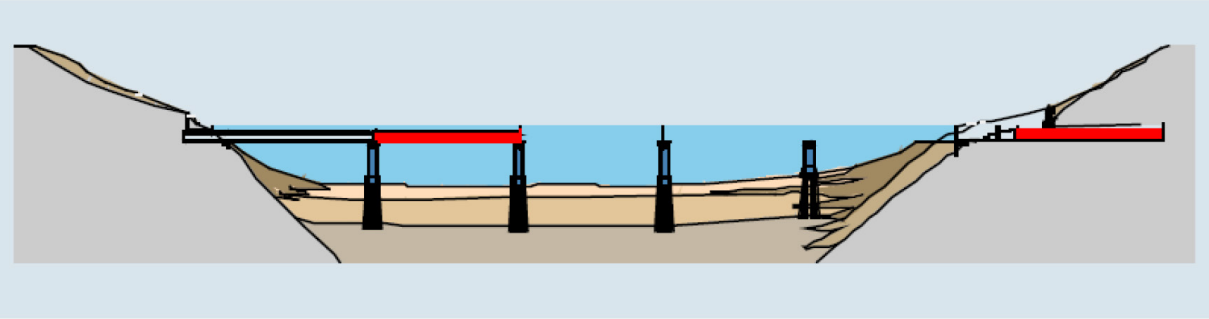


Fig. 8-6 Installation sequence, The Lake Lugano Crossing – Grignoli&Muttoni & partners, Dr. techn. Olav Olsen

Assembling elements into a completed tunnel structure elsewhere was, for example, considered as the preferred method for the design of the Bjørnafjord.

The choice for methods depends on the design. For Bjørnafjord the curved alignment was an important design feature, which can only be utilised if the arch is complete from one shore to the other. The advantage of this method is that temporary stages with a tunnel protruding from the coast into the water are avoided. Ideally the support system or the global force distribution in the temporary situation should be the same as in the final situation. This principle avoids the cost of temporary structures and reduces risks during the construction period.

8.3 SFTB cable anchors

For the tether supported tunnel types anchors need to be constructed. The anchor lines span between a connection point at the sea floor and the tunnel. They need to be adjustable in length. Ideally their force is reasonably constant and, if the force varies, there should be a minimum tension force.

The cable system is connected to anchor points. The anchor point may consist of driven or drilled piles or a suction anchor or (partly buried) gravity structures. The piles and suction anchors are installed with underwater installation technology. The gravity structures can be installed by lowering them to the seabed. Choice of the suitable type of anchor structure depends on the soil type of the sea bed. Sometime rock anchorage is required. All types of anchorages come from traditional and known technology.

Tendons or taut wires are connected to the anchor points. The tendons can be assembled out of sections and lowered vertically from specialist installation ships. Alternatively, they can be assembled and transported in a horizontal, floating condition and rotated and lowered into position. They are temporarily held in position with floaters.

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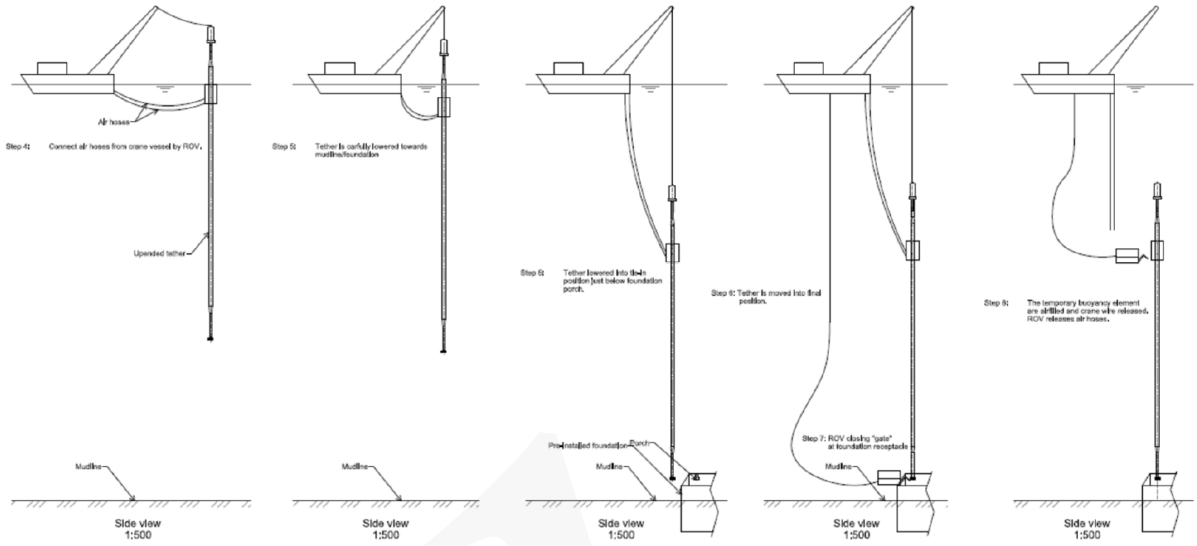


Fig. 8-7 Installation sequence, The Lake Lugano Crossing – Grignoli&Muttoni & partners, Dr. techn. Olav Olsen

As the design life of tendons and wires can be shorter than that of the tunnel, the design should allow for easy replacement of the cable supports, in part or full length.

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9. Installation

9.1 Introduction

The transportation and installation of SFTBs can draw from the considerable experience gained in the offshore oil & gas industry[9-1, 9-2] surface floating bridge and immersed tube tunnel technology. SFTBs differ from these technologies in terms of their size, their exposure to potentially more severe metocean conditions over an extended installation programme and the need to integrate a floating structure with fixed shoreline infrastructure to create a complete structure.

All marine operations require comprehensive planning and sequencing in relation to the logistical, topographical, bathymetric and metocean constraints. Established procedures are available for conducting hazard identification exercises (HAZIDs), hazard and operability studies (HAZOPs) leading to method statement preparation and contingency planning. Weather windows can be established for time-critical transportation and installation stages that require an understanding of the behaviour of the floating object and the marine equipment being employed.

The main aspects of the planning and execution of marine operations are examined below with an emphasis on where the characteristics of SFTBs, or the locations where they are likely to be deployed, require additional consideration in comparison to more established technology.

9.2 Data

Most of the data required for developing installation plans and procedures will have been gathered at the SFTB design stage but some additional data or increased definition may be required for the installation works including:

1. water depth – a detailed 3d survey of the SFTB alignment plus the anchor locations is required;
2. tidal range – is most important for the shoreline connection works;
3. investigation on bottom visibility – some activities may require good visibility such as ROV operations to secure tethers or to confirm that latching or connection operations have been successful;
4. accuracy of survey equipment – establishing water depth to a high accuracy becomes more difficult in greater depths of water and where the seabed is extremely soft;
5. available current data – spatial variation in currents, locations of eddies or swirls, variation of direction and magnitude at various depths;
6. temperature – air and sea-water temperatures need to be known to assess likely thermal movements over long distances such as tethers and sections of the SFTB;
7. salinity – should be mapped seasonally and with depth to understand buoyancy variations on tethers and the SFTBs.

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The following data are already included in the design data, but a further investigation, taking into consideration the installation paths and procedures, should be performed for the installation phase:

8. wave/wind statistics for area – detailed hindcast models in conjunction with study of near-shore influences to be carried out to give a clear picture of conditions along the whole SFTB alignment;
9. presence of internal gravity waves such as solitons to be understood. Local conditions such as squalls, sea breezes may be relevant;
10. soil conditions and soil properties – shall be gathered at all anchor locations with sufficient detail to ensure risks associated with foundation installation can be mitigated as far as is practicable;
11. seabed topography – any obstacles, sea-bed ordnance, zones of special scientific interest, fishing areas will have to be established prior to any marine works.

9.3 Planning

SFTBs cross significant bodies of water that can present difficult exposed conditions for marine operations. Also, the large size of units being installed calls for increased attention to be placed on the planning of the marine operations. Indeed, what is possible and practical in terms of transportation and installation is likely to have an influence on the overall design solution or the connection of component parts.

Marine operations in high current regimes such as straits or island-to-island crossings are challenging in terms of equipment performance and the limited periods of slack water.

The complexity of the towing procedures increases with the length of the towed structure. A towing procedure detailed plan, including the training of the entire towing crew and the test of all the installed control systems, is needed.

The following considerations apply to all marine operations planning, but SFTB installation will require more comprehensive study of the following:

1. operation reference period – this is needed for assessing weather downtime and weather windows for critical transportation and installation steps;
2. expected time to reverse the operation – some ability to reverse an installation step is required;
3. type of operation – the range of operations is likely to be greater than other marine installation works with each operation requiring detailed planning;
4. HAZID/HAZOP – should be conducted at an early stage in developing the construction and installation plans, preferably with all the stakeholders present who will be involved in the marine operations;
5. weather forecast and monitoring uncertainties – the hindcast data should be planned to be verified with offshore weather stations, current meters and wave-rider buoy data;
6. Metocean planning – detailed studies of weather downtime, weather window assessment will be required;

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7. contingency procedures, e.g. retrieval or abandonment of component - there shall be plans to be able to hold or moor SFTB sections during periods or adverse weather, tethers may need to be laid on the sea-bed whilst other operations are suspended and the ability to de-ballast the floating units is likely to be required;
8. installation trials – if there are substantial risks emerging from the HAZID/HAZOP process, these may need to be de-risked by conducting a trial installation prior to commencing construction of the whole SFTB.

9.4 Logistics

Where an SFTB solution is preferred, there may be difficult shoreline access or less-established shoreline infrastructure such that construction may need to take place some distance from the final installation site. In addition to the construction considerations, the following logistics considerations will need to be taken into account for transportation and installation:

1. A shoreline or inshore assembly site may be required with its own moorings/quays/jetties and subject to different metocean conditions to the installation site;
2. A safe harbour or anchorage location may be required during tow from construction site to installation site if the SFTB sections or vessels cannot withstand possible storm conditions en-route;
3. Metocean holding criteria will need to be established for open water tow durations that are longer than a typical 3-day forecast weather window;
4. Temporary moorings may be needed at the installation site if the SFTB section cannot be immediately installed;
5. A changeover from towing mode to installation mode may require different equipment and personnel;
6. Final commissioning checks may need to be made prior to commencing installation works.

9.5 Marine Characteristics of SFTB

The marine characteristics of the SFTB sections shall be established to ensure a safe, successful installation. An SFTB that is positively buoyant in its permanent condition with tethers may need to be self-stable when ballasted down during installation to limit the need for substantial temporary buoyancy, such as the installation barges typically associated with immersed tube tunnel construction. Nonetheless, some means of controlling the transfer of load to the tethers will still be required and this may only be possible with installation barges. Such barges may use winch lines or strand jacks to draw up the SFTB to engage the tethers rather than relying on deballasting. SFTBs supported by surface pontoons may utilise these pontoons for buoyancy and stability during installation.

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The following marine stability characteristics should be determined for the SFTB in combination with any additional installation equipment for all steps in the transportation and installation process:

1. hydrostatic and hydrodynamic effects (air-filled structure or not) – the weight, buoyancy and centre of gravity positions need careful monitoring and management;
2. metacentric height – needs to be positive for the SFTB both afloat and submerged where no other installation equipment is being relied upon;
3. tether-supported SFTBs will require auxiliary buoyancy to allow them to be lowered below the water surface
4. intact stability – the sections of SFTB shall be able to demonstrate an adequate range of stability and shall have no risk of down-flooding;
5. entrapped air – any air should preferably be in a fixed compartment to avoid undesired compressibility effects and reduced floating stability;
6. ballasting – detailed knowledge of the water ballast and solid ballast to be deployed at each stage of deployment shall be known;
7. motion response – the effects of open-water transportation of segments of the SFTB shall be understood as these could be more critical than for the completed structure;
8. slow-drift response (heave, surge and sway) and effects of vortex-shedding should be evaluated during transportation of sections of the SFTB;
9. damage tolerance – the design should be able to withstand accidental flooding through (temporary) water-tight bulkheads or reserve buoyancy so that the effects of any damage can be controlled;
10. if providing damaged stability is impractical, the marine operations procedures should ensure that the risk of vessel collision or of mal-operation of water ballasting equipment can be mitigated as far as is practicable.

9.6 Foundation Installation

Foundation installation for the SFTBs can represent a considerable scope of work, perhaps larger than that of the SFTB installation itself. Some considerations on the foundation installation are as follows:

1. Conventional driven pile installation may be currently limited to around 2000m of water depth using an underwater power pack to operate the pile hammer;
2. Gravity foundations can be deployed with controlled ballasting to neutral buoyancy and either a series of floats on a line or a surface barge with winches or jacks for the controlled lowering of the foundation to the sea-bed;
3. If the seabed is rocky, driven and suction piles may not be feasible and a prepared bed of gravel may have to be placed prior to gravity base installation. In this case, drilled and grouted rock anchors could be a viable solution;

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4. Suction buckets/caissons are the most common type of foundation for deep offshore moorings as positional accuracy is readily achieved and there are established caisson pumping and monitoring systems to ensure the caisson successfully penetrates;
5. Fluke or drag anchors may not be suitable due to the difficulty in applying the pull-in force, controlling the pull-in distance and extent to which the anchor self-penetrates below the sea-bed.

9.7 Mooring and Tether installation

The foundations are linked to the SFTBs using mooring lines and/or tethers. Depending on the design, these can include vertical or inclined moorings, or possibly a combination of the two. Some considerations on the selection and deployment of moorings are given below:

1. Mooring lines are likely to be taut or semi-taut using a mix of synthetic elements, wire rope and chain;
2. Care should be taken to avoid damage to synthetic ropes due to snagging or tears;
3. Torsion control may be required on wire ropes, such as a torque connector to avoid damage to the wire lays;
4. Mooring lines can be installed using messenger lines attached to the foundation and pulled in and then anchored off on the SFTB using a vessel to tension the line;
5. Some technical development may be required to devise straightforward means of locking off moorings as there may be no surface structure for pull-in or lock-off;
6. Catenary moorings are likely to have too low a lateral stiffness to be suitable for SFTBs;
7. Tethers, as used on TLPs, are near neutrally-buoyant tubes that can assembled at shore, towed horizontally and upended to the vertical before latching into porches on the foundation and SFTB (see Fig. 9-1 and Fig. 9-2);
8. Alternatively, tethers can be assembled vertically offshore using proprietary connectors between the tube sections;
9. The tether can have connectors capable of adjustment by ROV tools to achieve the final required elevation of the SFTB;
10. Final tensioning of the tethers will require the SFTB to be de-ballasted from its neutrally-buoyant or suspended condition;
11. Subsea adjustment of tether connectors to take up tolerances may benefit from further technical development, given the number to be deployed compared to an oil & gas platform;
12. The design of tether connections on the SFTB should be configured to facilitate possible replacement through-life;
13. Tendon tension-monitoring equipment can be incorporated for in-service monitoring.

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Fig. 9-1 Heidrun TLP platform (North Sea) – Fabrication (left) and sea launching and tow of tethers (right) - AkerSolutions/Norwegian Public Roads Administration



Fig. 9-2 Heidrun TLP platform (North Sea) – Upending - AkerSolutions/Norwegian Public Roads Administration

9.8 Jointing Operations

SFTB jointing operations may be required between relatively short modular units or larger pre-assembled sections. As described in Ch 8, SFTBs can be constructed from shore into considerable lengths, thereby reducing or eliminating joints at the offshore but it is likely that some joints will still be required.

The considerations of conducting jointing operations offshore in marine conditions include:

1. Holding and manoeuvring the sections becomes less straightforward as their size and displacement increases;
2. Guide structures are likely to be required to allow sections to be close-connected;
3. The offshore jointing is likely to require manual assistance with the workforce located in the SFTBs;
4. Sections shall be rigidly held in place during grouting operations;
5. The shoreline connection could be grouted in place or articulated depending on the SFTB design.

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9.9 Marine Equipment

The tow and installation of the SFTB units could be designed around the use of equipment that is used for similar operations in a marine environment or could be specially designed for the particular operations. In selecting the equipment, the following aspects should be evaluated:

1. Size of installation support barges where required;
2. type of lifting and lowering gear;
3. crane capacity and specifications including weight of crane wire for deep water installation;
4. crane capacity de-rating in relation to metocean conditions;
5. crane hoisting/lowering speed;
6. whether the crane sheave block can be submerged;
7. vessel pull-in, towing and holding capacity;
8. method of tugger line disconnection;
9. load reducing systems (heave compensation capacities);
10. dynamic positioning (DP) capability / position keeping systems;
11. Remote operating vehicle (ROV) station keeping capability;
12. ROV working range;
13. complexity of ROV tasks (e.g. ROV Interfaces on structure);
14. guide wire forces and winch speed limitations;
15. strand jack capacity.

9.10 Safety

The safety of the offshore workforce as well as the safety of all stakeholders in the project is of paramount importance. The particular nature of the SFTB marine operations will increase the exposure of the workforce to potentially hazardous conditions compared to a single offshore installation such as an oil & gas platform. Underwater jointing and completion operations, before the full water-tight integrity of the SFTB is assured, will involve many workers for a considerable duration. The work shall be conducted to international and local safety regulations in the country of construction as well as addressing the following safety aspects:

1. procedures should be underpinned by international standards such as SOLAS;
2. control of 'island' assembly where there is no exit route to shore;
3. control of multiple work sites over considerable distances where an incident at one location could affect other work locations;
4. a robust hierarchy of command and control is established;
5. a target to eliminate divers from all offshore construction works;

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6. patrol and guard vessels are used to ensure there is effective control of leisure and commercial vessel movements;
7. permit to work systems are implemented;
8. temporary entry and exit-ways are specified for personnel including pressure-locks;
9. confined space working and vessel entry procedures are put in place;
10. the safety of personnel in transit to the offshore construction site is ensured;
11. medical evacuation provisions are made from all offshore worksites.

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10. MEP - systems

With MEP- systems we mean the mechanical, electrical and plumbing system typical of the tunnel structures. The systems addressed in this chapter are meant to include systems for ventilation, for access and compartmentation, for drainage water collection and pumping, for signage, traffic control, tele-surveillance and communication, for lighting and for firefighting. It includes instrumentation, control, automation and electrical power supply for those systems.

The requirements for operability and safe use of tunnels enabled by these systems are increasingly important for the safety of use of tunnel infrastructure. For submerged floating tube bridges (SFTBs) this will not be different. Hence, any normally applicable requirement for immersed or bored road and/or rail tunnel systems shall be considered also valid for an SFTB. The focus of this chapter on mechanical and electrical systems of SFTBs is only on systems or characteristics or requirements that are specific for SFTBs.

10.1 Traffic management / CCTV surveillance

SFTBs require a traffic management system and a surveillance camera system like other tunnels, and should be seen in connection with the adjoining road and/or tunnel system. The traffic management should be aimed at avoiding too many vehicles in the SFTB. Accumulation of traffic will increase the congestion in case of evacuation, and specific for floating tube bridges accumulation of traffic will influence the vertical equilibrium.

Thermographic control of heavy vehicles at the entrance of the approach tunnels for the SFTB can prevent the entry of overheated vehicles into the tunnel.

10.2 Air quality and smoke ventilation

Ventilation systems are used to avoid high concentrations of toxic gas generated by fossil fuel car engines. Mechanical longitudinal ventilation is often used. The required fresh air volumes depend on the traffic density and the tunnel length. The air speed should stay within limits (e.g. 10 m/s). The use of intermediate ventilation towers is a common solution that improves the system and reduces the fan capacities. Like immersed tunnels and long subsea rock tunnels, possibilities for ventilation towers at mid-sea locations are limited. If technically feasible they will be very expensive, since they require the construction of a (floating) island or towers founded on the seabed. Ventilation systems that do not require intermediate ventilation shafts and towers are therefore preferred. This might mean that additional longitudinal air channels are needed. Tunnel lengths up to 20 km (in Japan) and 30 km (in Denmark / Germany) without intermediate ventilation openings have been considered feasible. A traffic management system will relieve the ventilation requirements and contribute to the feasibility of long(er) continuous tunnel tubes. Besides that, an increase of the market share of electrical cars may have a beneficial effect on the toxic gas load.

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Mechanical ventilation is also used to create safe, non-toxic zones in the event of fire. Conventional longitudinal ventilation in the event of fire operates at low speed and the direction shall be in the traffic direction, coordinated with the evacuation plan. Smoke ventilation in fire can also be achieved using a separate horizontal exhaust shaft, which opens itself above the location of the fire and extracts smoke by under pressure in the exhaust shaft, supported by forced longitudinal ventilation from two sides towards the fire location.

10.3 Automatic fire extinguishing

Fire can be a threat to the safety of the public, but also for structural integrity of the tunnel. The consequence of a large local fire in terms of damage to the strength and the water tightness of compartment walls and the exterior hull structure of the SFTB can potentially be severe and be a cause of unrepairable damage. Therefore, even if passive fire protection is provided, automated fire extinguishing needs to be installed as well.

Fire extinguishing water must not accumulate in the tube. It shall be drained and pumped away simultaneously with the fire water supply.

10.4 Emergency lighting / exit signs / radio / telephone / emergency power supply

Electric power supply for lighting and for safety systems, and the safety systems themselves shall be made redundant and invulnerable for local problems like collisions, traffic accidents and/or leakages, or global power cut-off.

10.5 Drainage / water collection / bilge pump system

Rain water load, water transported with the traffic, cleaning water and expected leakage water shall be collected and drained like in other tunnels. Leakage water shall be collected and eliminated from the floating tube system, preferably by longitudinal pressure pipe, or by underwater non-return valve systems. Application of valve systems as applied in ships needs further study. At present they are not preferred for safety purposes, but they might be unavoidable in long tunnels. The systems shall be over-designed and shall be inspectable and easy to clean/remove sediment and clogging. The materials shall be saline water resistant. Any excess water from adjoining rock or bored tunnels should not be allowed to enter the SFTB. Cumulation of water caused by strong gradients in the vertical alignment shall be prevented.

Fire extinguishing water reserve tanks shall be included in the design and the relocation of the associated weight shall be included in the design scenarios.

If an automated bilge pumping system is part of an inherent safe design, it shall be equipped with separate, independent power supply and an off-line control system. It shall be subject to periodic inspections, test runs and a certification programme. Pump engines and control systems shall be located where the risk of flooding is the lowest.

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10.6 Indication of level and/or flooding of ballast tanks

Since the weight balance of the floating tube is of utmost importance to the safety of the system, automated sensor, monitor and control systems of water levels in ballast tanks and compartments is recommended, supplemented with reliable double-check systems using mechanical floats and visual indicators that allow easy human inspection.

10.7 Force measurement on tethers

As an extra means of control of the SFTBs structural integrity, and to collect data on the time variant processes that influence the safety margins and the design life, it is necessary to include permanent force and displacement measurement systems. Forces in the tethers should be measured to detect over-load, tendon failure, or a tendency towards slack of the tethers. This system can, in addition, detect eventual abnormal behaviour of the structure as a consequence of collisions, for example with submarines.

10.8 Collision prevention

If a sonar reflector system is installed on the structure, to enhance its detection by floating or submerged bodies (submarines) which could collide with the structure, a passive system is recommended instead of an active one, due to the proven negative effect that the active system has on the marine mammals.

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11. Operation & Maintenance

This chapter is written based on an SFTB with traffic separated in two tubes, as this is assumed to be the preferred solution with regards to both safety and operability.

11.1 Design for minimum maintenance

Operation and maintenance (O&M) should be considered already in the early design phases in order to ensure that O&M related costs are reduced to a minimum. Ideally, no maintenance should be required, e.g. by choosing materials which are not susceptible to deterioration, but in many cases this can be challenging. Structural elements in steel need corrosion protection, for instance by replaceable sacrificial anodes (cathodic protection) on tethers and a combination of sacrificial anodes and corrosion allowance on anchors. If sufficient design life cannot be achieved with great confidence, replacement of tethers and their end connectors have to be planned for in the design.

Consideration of O&M typically regards the following elements:

- Choice of materials in general: reinforced concrete solutions are preferred to structural steel for better durability, in particular for elements in splash zone and above where the use of cathodic protection has no effect;
- Guideline for design of concrete mix: concrete mix needs to be optimised for high density (= low permeability); use of slag cement based on Dutch experience or microsilica additions is recommended[11-1];
- Design of ballast system: components which require relatively intense maintenance such as pumps are not installed permanently in peripheral locations but stored in an operation centre with easy access;
- Design of top anchorage of tethers: in view of necessary access with ROV (inspection, repair), the accessibility of critical elements is important and inaccessible corners / gaps need to be avoided;
- Provisions for destructive testing: for destructive testing (extraction of concrete samples), inactive concrete elements are added that can be collected during the service life;
- Selection of fire protection: durability and maintenance are considered in the design of the fire protection on the inner surface of the concrete shell.

11.2 Continuous maintenance

Continuous planned maintenance can be performed with a minimum of one tube open for two-way traffic. An example of normal tunnel maintenance operations is given in the tables below for maintenance with one lane closed and one tube closed.

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The tables provide an overview of planned maintenance inside the tunnel. Maintenance on structural elements outside the tunnel can be performed without interruption of traffic.

Several of the activities in the tables below can be performed in parallel operations. Durations are based on empirical figures and local variations will occur.

Table 11-1 Example: Planned maintenance with one lane in one tube closed

Activity	Frequency [#/year]	Duration [hours]	Comment
Simple inspection	1	1 x 6	1 night, 6 hours per night per tube
Main inspection	1/5	3 x 6	3 nights, 6 hours per night per tube
Electrical inspection	2	2 x 6	2 nights, 6 hours per night per tube
Installations inspection	2	2 x 6	2 nights, 6 hours per night per tube
Technical cleaning	5	2 x 6	Lights, Road sign, SOS cabinets 2 nights, 6 hours per night per tube

Table 11-2 Example: Planned maintenance with one tube closed, two-way traffic in the other tube

Activity	Frequency [#/year]	Duration [hours]	Comment
Cleansing	1/5	3 x 6	Including sand trap, foam and water drain through the whole tunnel 3 nights, 6 hours per night per tube
Cleaning	2	2 x 6	2 nights, 6 hours per night per tube
Partial cleaning	3	1 x 6	1 night, 6 hours per night per tube

11.3 Rehabilitation

Structural steel should be protected by a combination of coating and cathodic protection systems. Sacrificial anodes should be designed to be replaced with ROV access if necessary. This is a relatively simple procedure and can be performed with the SFTB in normal operation. For the SFTB alternatives with tethers, replacement of tethers is not expected due to the cathodic protection system. This is assuming that sufficient fatigue life has been allowed for in the design.

Piles and foundations (embedded parts) are not accessible for inspection, however, the holding capacity can be monitored through the tension in the tethers.

Structural concrete is not expected to need rehabilitation during the design life. Performance studies on the offshore concrete platforms in the North Sea show excellent durability after almost 40 years in service[11-2]. For comparison, the concrete elements for Fehmarn immersed tunnel, currently under construction in Germany, have a design life of 120 years. Extensive testing of different concrete mixes is being carried out to ensure that the requirements for the contractors will ensure the intended durability. Results so far confirm that the concrete behaviour is as expected.

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Fig. 11-1 Concrete testing for Fehmarn immersed tunnel, Femern A/S

An SFTB does not rely on any active systems in operation which are dissimilar from other normal tunnel technical systems. Technical installations in the running tubes can be rehabilitated/replaced successively one tube at a time.

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12. Risk and reliability

12.1 Definitions and framework

Submerged floating tube bridges are a special category of civil engineering structures regarding their exposure to environmental conditions. Floating structures are being considered as being more vulnerable and the consequences of loss of stability and failure may be large. Hence risks and reliability of the structure are a topic of interest.

In the planning and design stage, tendering and contracting stage, as well as during construction of bridges and tunnels, risk appraisals are common practice. Guidance on the format of identification and assessment of risks is given in e.g. the ITA Guidelines for Tunnelling Risk Management[12-1], or in ISO 2394 General principles on reliability for structures (2015)[12-2].

Risks are defined as exposure to undesired consequences of a hazard. Examples of hazards are: natural circumstances such as extreme weather or earthquakes, or uncontrolled actions caused by human error such as collisions, fires, explosions, etcetera. Within a systematic risk analysis, the risks are quantifiable. Their predicted frequency (or probability) of occurrence, as well as the potential consequences, can be estimated and analysed. This produces defined risks. In the ITA guideline, risk analysis is performed according to the process depicted in Fig. 12-1.

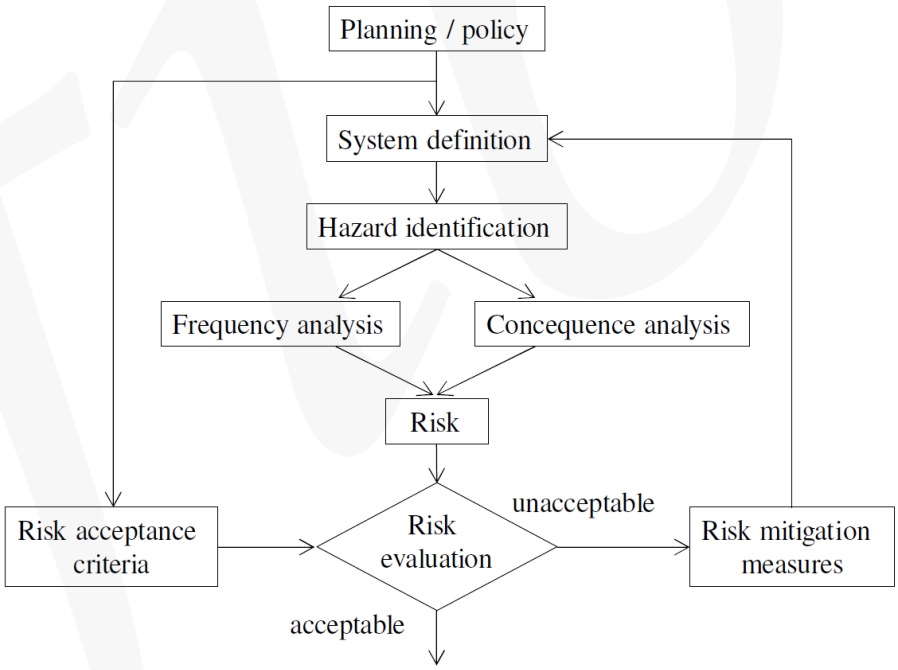


Fig. 12-1 Risk evaluation system definition in ITA guideline reproduced from ITA-AITES presentation, Seoul, 2006

The usual order of preference of risk reduction measures is: a) inherently safe design, b) prevention, c) detection, d) control, e) mitigation, and f) emergency response[12-3].

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For example, in the feasibility study for an SFTB crossing the Sognefjord, in Norway, the following levels for the evaluated risks during construction, installation and operation of the structure were established:

- Risk not relevant or counteracted by simple or trivial measures
- Risk to be eliminated by recognised measures
- Risks eliminated by measures investigated by advanced theoretical or experimental analyses
- Risk unlikely to be avoided by reasonable measures

After that, for every specific identified risk, a mitigation procedure has been described and a general evaluation of the risk analysis has been done to be able to recommend the chosen solutions for the further design steps.

It is generally accepted practice that known, identified risks shall be reduced as much as possible, by means of above-mentioned strategies a) and b). For structures used by the general public, like SFTBs, risks shall be below a certain value, or shall be optimised based on cost benefit type criteria based on individual risk and societal risk of fatalities and on economic optimisation. The marginal lifesaving principle and the principle of balance between the cost of mitigation and the economic loss while accepting a risk are applicable and recommended for this evaluation. ISO 2394 introduces the criteria and gives guidance on how to perform a risk analysis.

For tunnel projects the scope of the risk assessment may (according to ITA) include risks:

- to the health and safety of workers, including personal injury and loss of life
- to the health and safety of third-party people
- to third-party property
- to the environment including pollution, and damage to flora and fauna
- of delay of the completion date
- of cost increase and associated financial loss
- of loss of goodwill

Loss of goodwill is a relevant consequence category to assess for projects that are politically, economically or environmentally sensitive and where public opinion can be expected to have a large influence on the project development. Tunnel types with new technology, like the submerged floating tube bridge fall in this category.

The general hazards considered include contractual disputes, insolvency and institutional problems, (untimely) authorities' interference, third party interference, labour disputes etcetera. Focussing on the process up to completion, specific hazards considered are: accidental occurrences, unforeseen adverse conditions (unexpected ground and groundwater conditions), inadequate designs, specifications or programmes, failure of major equipment, and substandard construction works.

The ITA does not focus on risk assessments during the operational phase of the project. The design is considered as an outcome of a risk assessment that includes hazards that may occur during the lifetime of the structure.

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12.2 Specific Hazards for SFTB

Submerged Floating Tube Bridges have not yet been constructed. Hence assessment of risks is hampered by lack of knowledge of how complicated construction can be. Designers and contractors will need to go through a learning curve before projects are successful and to the full detail of what is technically manageable. Complexity and immaturity of the applied technology is generally considered a cause of hazard.

Focussing on design, hazards are often managed by an analysis of the probability of occurrence, and if a certain probability needs to be accepted the consequences are calculated. The rare case of an extremely hazardous condition is not ruled out but is accepted. And, in practice, design engineers do not fear excessively the quantitative exceedance of known hazards, because the consequences are 'in line' with the known scenarios for which a tunnel or bridge is designed for. Examples of these exceedances are: effect of a higher wind speed on a bridge deck, stay cables and pylons, effect of slightly weaker soil leading to a somewhat larger overall settlement on a tunnel, or the effect of higher water levels on a bridge pier in a river. Structures generally have redundancy for known scenarios. Or structures may, as a principle, be designed as invulnerable for certain overload scenarios. An example of this is the limitation of water depth in front of a wave wall, which is designed to withstand a certain wave height. If the oceanic and meteorological conditions lead to higher waves, these waves would break in front of the structure since they cannot propagate in too shallow water.

For Submerged floating tube bridges (SFTBs) it may be more challenging to apply the same principles to manage the risks.

Specific hazards for SFTB are:

- Extreme weather and high waves and currents
- Extreme surges and water levels
- Increased water levels due to global warming
- Extreme hydrological conditions like tsunamis or seismically induced changes in water level in estuaries and fjords
- Vessel and submarine collision risk
- Sinking vessel risk
- Dropping or trailing anchors
- Fish trawls
- Drifting ice
- Marine growth and associated weight increase
- Material fatigue
- Consequences of static overload, fire and explosion
- Effects of motions and public perception of movement of the structure
- Consequences of leakage
- Consequential damage, or the probability that component failure leads to system failure

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- Tsunamis
- Underwater landslides
- Explosions and fires

For some hazards the probability is very low, similar to normal tunnels, but the consequences are high(er). Leakage is, for instance, a potential threat for loss of buoyancy and loss of equilibrium. For other hazards the principle of invulnerability may rule them out, e.g. a tether-supported SFTB will not be affected by high tides, surges and sea level rise. Giving the SFTB a sufficiently deep position may decrease the risk of vulnerability to surface waves. Collision risks may be mitigated with nautical measures and guide structures; similar to mitigating the collision risk between large vessels and low bridges. The vessels may be forced to pass under a high bridge section, or through a moveable bridge section and navigation of ships close to other parts of the structure is made impossible.

12.3 Stability of floating structures

The redundancy of stable equilibrium for floating structures is often assured by making watertight compartments in the structure. The designer's assessment shall include an intact stability assessment with sufficient redundancy for extreme actions, and shall include the assumption of certain compartments to be damaged and flooded. The so-called damaged stability needs to be assured, for which freeboard and righting angle criteria apply (e.g. refer to DNV-OS-C301). This analysis is commonly done using defined scenarios treated in a deterministic way. Probability of occurrence of the scenarios and the probability of the analysis of consequences can also be examined using probabilistic methods.

For pontoon supported SFTBs this method may be used to design reliable pontoons. For a tether supported SFTB any damaged and partly flooded scenario implies a less buoyant tunnel, which exerts less upward force in the mooring lines.

12.4 Ultimate limit state design

Limit state design is widely accepted and is also recommended for design of an SFTB. Definition of limit states include:

1. Exceedance of static equilibrium (including buoyancy loss, and cable slack) EQU
2. Exceedance of strength/strain limits STR
3. Exceedance of fatigue strength FAT
4. Exceedance of geotechnical equilibrium and deformation limits GEO

Deformation criteria and water tightness criteria are generally considered as serviceability limit states.

Precise definition of what is considered as the limit state is to some extent a free choice. For an SFTB this shall be done with great care. For example, using plastic design in reinforced or post-tensioned concrete structural components may implicitly lead to acceptance of cracking and leakage.

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Leakage water shall be treated differently to that in normal tunnels, since it may affect the limit state of equilibrium. It may be necessary to install pumping systems to avoid loss of equilibrium in case of excessive water quantities. Those pumping systems will in that case be part of the reliability assessment of the tunnel structure.

Like other structures, the system shall be robust. Component failure is not supposed to lead to propagation of problems ultimately leading to system failure. For a pontoon supported structure, as an example, the failure of one pontoon may need to be assumed. This scenario should not lead to sinking of the neighbouring pontoons.

SFTBs are very long structures. This introduces the need to consider 'length effects'. If it is accepted that a certain cross section may fail under a certain condition, the probability for which this is acceptable will be different if a structure consists of a few or of hundreds of these cross-sections.

12.5 Target reliability

SFTB structures are a special type of structure to be designed to withstand a variety of hazards and scenarios but need to be as safe as other infrastructure. Target reliability values for infrastructure as well as other structures and vessels are presented below. It should be realised that, to meet the same target reliability, SFTBs might thus need higher requirements and higher safety margins compared to normal infrastructure, since they will be prone to more and different risks.

Table 12-1 Summary of recommended target reliability values

	P_f	β	
Road tunnels in national road grid	10^{-5}	4.3	CC3 in Eurocode system
Offshore wind turbines unmanned	10^{-4}	3.7	DNV guidelines for offshore wind
Offshore wind turbines with maintenance personnel under extreme conditions	10^{-5}	4.3	
Shipping	10^{-3} (crew) 10^{-4} (passengers)		Study[12-4]

Table 12-2 Recommended values from DNV report 95-3203 Guideline of offshore structural reliability: application to jacket structures

	Small consequences	Serious consequences
Redundant structure	$P_f = 10^{-3}$ $\beta = 3.1$	$P_f = 10^{-4}$ $\beta = 3.7$
Non-redundant structure, with significant warning prior to occurrence of failure	$P_f = 10^{-4}$ $\beta = 3.7$	$P_f = 10^{-5}$ $\beta = 4.3$
Non-redundant structure, without significant warning prior to occurrence of failure	$P_f = 10^{-5}$ $\beta = 4.3$	$P_f = 10^{-6}$ $\beta = 4.75$

The SFTBs in Norway (in 2012 and 2016) were designed under Eurocode and the Norwegian National Annex, using category CC3, and a 100-year design life.

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12.6 Verification methods

Methods that verify if a design meets the target probability are based on the following process[12-4, 12-5] of possible failure mechanisms.

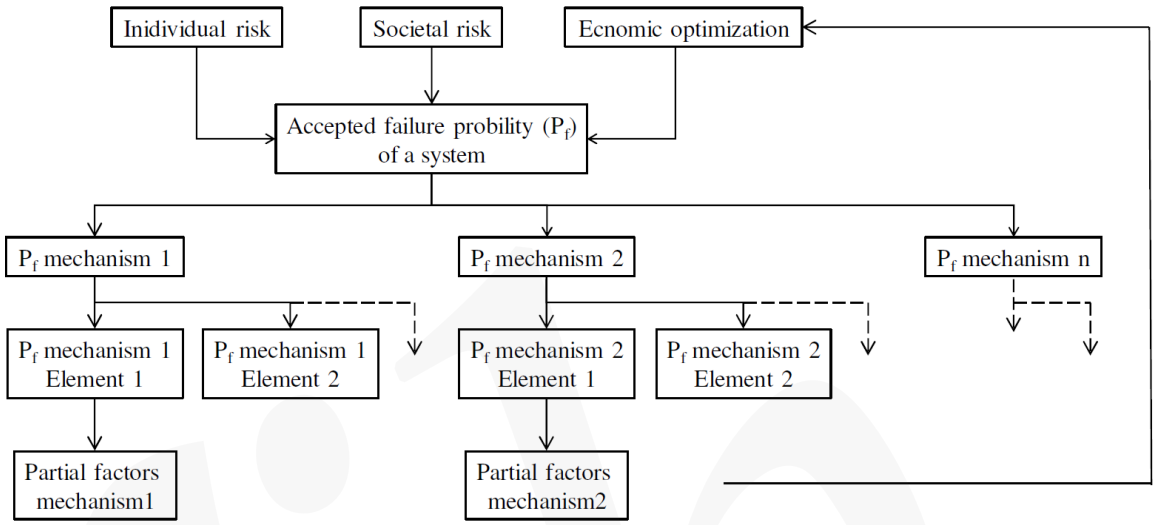


Fig. 12-2 Schematic summary of failure mechanism addressing multiple mechanisms and multiple identical structural components

In most codes for structural design the required reliability level can be analysed by using the load and resistance factor method. For this method combinations of simultaneously occurring actions shall be known and defined in a matrix of combination factors.

As an alternative, the DNV GL code system allows a design by the partial factor method with direct simulation of the combined effect of simultaneous load processes.

Also (partial) verifications of the design using experiments are allowed. This can be especially interesting since the effects of wave and current actions (and their randomness and spatial variability) on 'long' structures may be investigated best using realistic simulations and measuring overall effects.

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13. Costs

13.1 Cost competitiveness

Construction cost is one of the main factors when choosing between alternatives for crossing waterways. Although a submerged floating tube bridge is expensive, it may be a cost-efficient option for deep and wide crossings. As shown in Fig. 13-1, the costs of an SFTB per unit length tend to be little influenced by crossing length, and this has been demonstrated in several studies. An SFTB can be competitive over a suspension bridge beyond 1'000 m beyond which the unit costs for the latter will rise steeply.

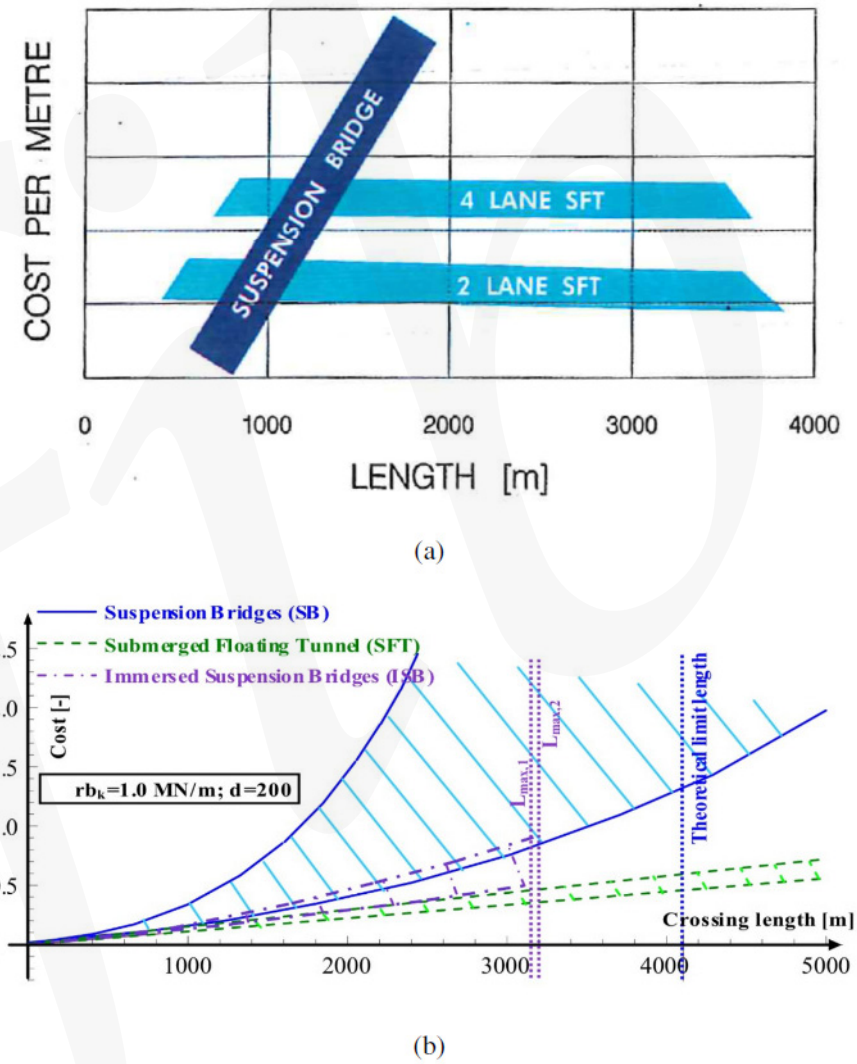


Fig. 13-1 (a) Dr. Techn. Olav Olsen; (b) study[13-1]

Fig. 13-1 shows the large range of costs for suspension bridges (an Immersed Suspension Bridge is a suspension bridge where the pylons sit on a floating island, typically shown anchored with tension legs). The longest span for a traditional suspension bridge today, in 2019, is approximately 2'000 m.

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For sub-sea rock tunnels the depth of the crossing is the major factor determining the length and hence the cost. The low cost per metre is partly or fully offset by the length, which, for a steep gradient of 8%, can be 5 to 10 times the width of the fjord. For more gentle and comfortable gradients, the length and cost of a sub-sea rock tunnel would increase significantly.

Construction costs are one thing, running costs are another. In fact, running and maintenance costs can be quite different from one structure to another and each case deserves a closer look. For example, lighting, drainage, ventilation, cleaning and especially pumping, represent important costs for long sub-sea tunnels. In a general study, taking into consideration also the crossing time and the fuel, an SFTB would be more economic compared to a sub-sea tunnel, due to the absence of high gradients.

13.2 Cost comparison

A suitable basis for comparison of costs for different crossing types is the Norwegian fjord crossing campaign currently underway, partially described in the example given in Ch. 14. Studies related to the fjord crossing project comprise floating bridges, floating suspension bridges and submerged floating tube bridges. Fig.13-2 shows a normalised comparison of the predicted unit construction costs (best estimate) for some of the most developed crossings, all including a four-lane bridge and a bicycle/pedestrian lane. Fig. 13-2 also includes the two existing floating bridges in Norway, the Bergsøysund bridge (1992) and Nordhordland bridge (1994), both with two traffic lanes.

As shown in Fig. 13-2, the floating bridges constructed (1992/1994) have a considerably lower cost with respect to the evaluated costs for new structures. This is partly due to the functional requirements (four lanes instead of two), to the exposure of the new crossings to environmental actions and to rules and regulations.

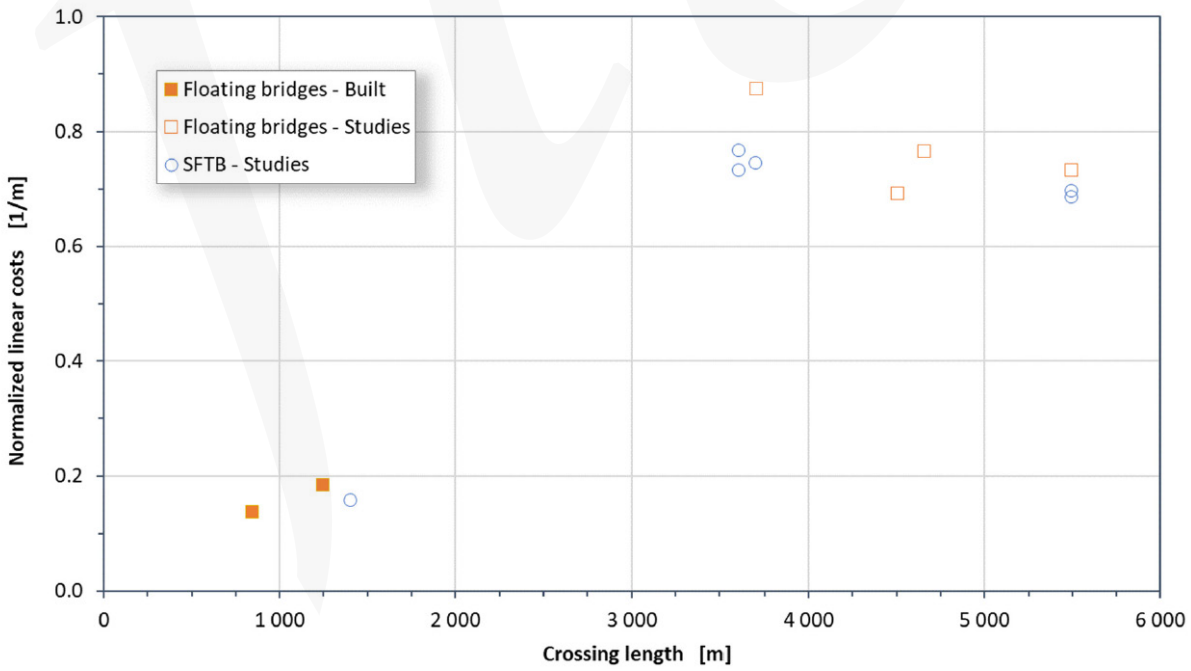


Fig. 13-2 Normalized comparison of unit construction costs (2018) in Norwegian fjord crossing projects

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13.3 Cost estimates

Proper cost estimates are important, both for comparison of various methods of strait crossings, and for the cost itself. As mentioned, cost should include all elements of construction costs as well as operational costs.

For the cost evaluation of an SFTB, there is not a large cost database of constructed work to draw from. Instead, cost estimates should be prepared from first principles and experience from other and similar structures. One type of similar structure is the family of platforms for the oil and gas industry.

The concrete platforms built in Norway and elsewhere are sufficiently numerous to form a useful database.

The construction of the Condeep platforms in Norway, by Norwegian Contractors, took place almost exclusively at the Hinna construction yard, south of Stavanger. Close cooperation between contractor and designer for more than 20 years, in the same construction yard, gave very good results in terms of experience and quality. The structures were relatively similar, and had similar mechanical outfitting required for construction, mooring, towing and installation. Then it was possible to establish simple cost/cubic metre of concrete estimates, valuable for conceptual work.

When constructing concrete platforms elsewhere, the experience of cost may be very different. There are many reasons for this and one is the experience of the contractor. As these structures must float, permanently or temporarily, weight is very important, much more important than for land-based construction. Because weight is important, structural elements, although often heavily loaded, are commonly slender, and the reinforcement content is high. Weight importance and high reinforcement content requires good construction planning. Errors due to inexperience could be very costly, compared to the initial evaluation of the cost of the structure.

Based on the experience of the designers, there are several ways to avoid high costs. As an example, during the feasibility studies for the SFTB crossing for the Norwegian fjords, Dr. Techn. Olav Olsen, with experience in designing more than 3 million cubic metres of concrete marine structures, gave the following suggestions:

- Concept is important, and conceptual design;
- Understanding the real use of the structure is important to optimize it;
- The level of knowledge of the loads is crucial;
- The needs of the specific environment where the structure is installed is important;
- Weight control is important;
- Design and construction should be well integrated;
- Construction management is important;
- Construction method is important;
- Design and design tools are important.

These are elements that should be evaluated when planning the contracting and execution scheme.

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However, experience shows that costs can vary considerably, with the location of construction work, client, contractor and designer all being factors in this. Also, one should bear in mind that oil and gas are expensive commodities that may sometimes make it tempting to increase cost to reduce construction time (for example when the Troll A platform was designed and built, and the gas to be delivered was already sold and the delivery date set, this meant that the construction price was not the main concern).

Not all the cost variations may be explained by physical aspects; national habits, national laws, union issues, national regulations and competition aspects are some aspects that can also influence the costs.

The cost of dry docks may vary considerably over time, for market reasons. Smaller marine structures may preferably be built on a floating dock or barge.

An SFTB is not a particularly complex structure. It could be considered similar to the shafts of an offshore platform, although built horizontally rather than vertically.

An SFTB has approach tunnels, for which the costs are relatively easy to cost-predict. The technology of coupling a rock tunnel to a structure under water is also relatively well known. The SFTB itself can, to a large extent, gain from the experience from offshore structures, and so may the anchorages of the structure.

13.4 Cost drivers

Some interesting observations can be made from the above-mentioned cost comparisons for the Norwegian fjords applications. For this specific case, due to the width and the depth of the fjords, SFTBs vertically stabilised with floating pontoons or tethers connecting the structure with the seabed were analysed.

A cost evaluation of the different solutions allowed the designers to understand how the costs were distributed and to evaluate the factors that could reduce the general cost.

In Fig. 13-3 the cost breakdown for a pontoon stabilised and a tether stabilised SFTB are shown. As the diagrams show, the tube construction comprises nearly half the total cost.

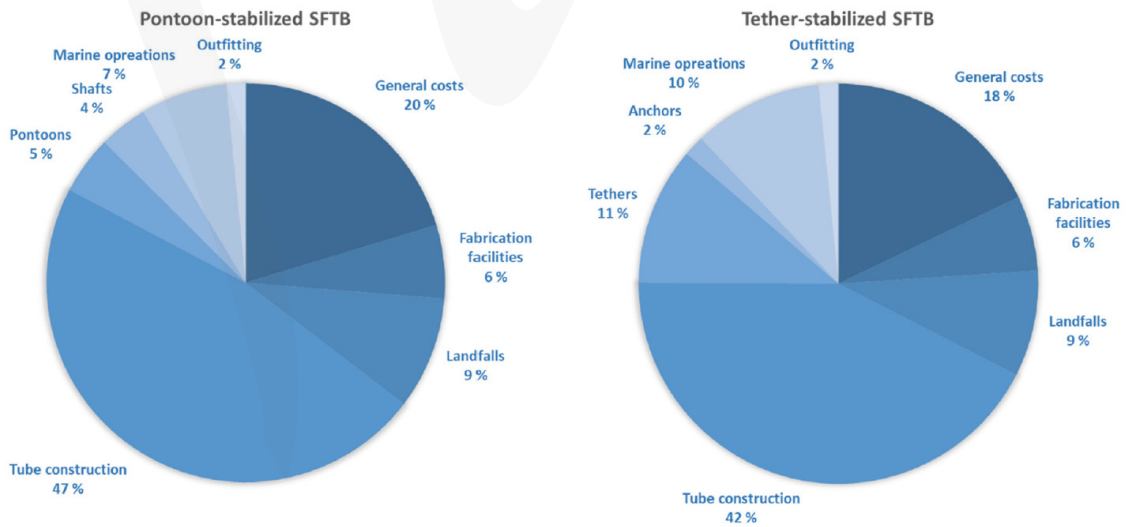


Fig. 13-3 Cost breakdown for a pontoon-stabilised (left) and a tether-stabilised (right) SFTB alternative developed for the E39 Bjørnafjord crossing

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Due to the modularity of the tube-structure, precasting could potentially reduce the fabrication cost of the main structure, increasing the competitiveness of the structure considerably, even for short crossings.

Further general observations can be made, comparing the two diagrams.

For example, it would be more expensive to add a tether than to enlarge the thickness of the designed ones. This is mainly due to the cost of the monitoring system that should be supplied for every tether element.

Furthermore, the additional experience gathered during the execution of the marine operations, together with technology development, will lower the related cost.

13.5 Cost uncertainties and sensitivities

When dealing with new structures there are many uncertainties to take into consideration for the cost evaluation. Experience has also shown that for structures composed of known parts, the uncertainty level is higher than for single structures, because the interface between the single parts is a new field of application. This is, for example, the case for the SFTB.

In the following diagram, the cost uncertainty drivers evaluated for the crossing of the Norwegian fjords are shown.

The graph has been developed after several analyses performed during the feasibility phase of the projects and it can give a qualitative idea of the cost uncertainty drivers that need to be taken into consideration.

The spreading of the colour on the horizontal axis gives an idea on how that factor could influence the final effective cost, with an increase or a decrease in costs.

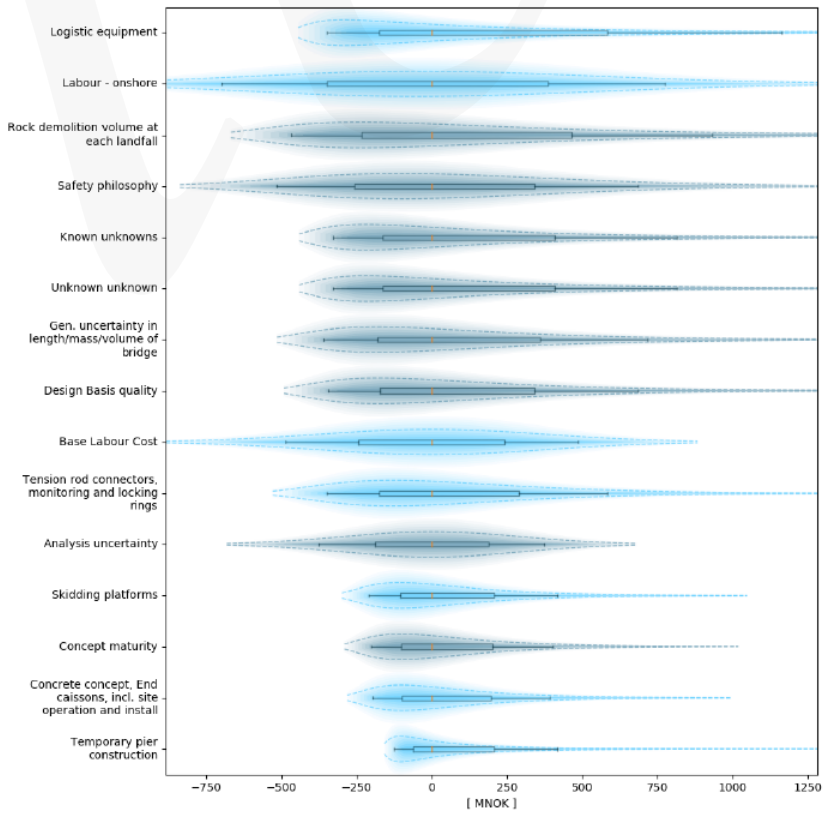


Fig. 13-4 Cost uncertainty drivers

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14. Studies on the SFTB

14.1 General

For sub-sea rock tunnels, the depth of the crossing is the major factor determining the length and hence the cost. The low cost per metre is partly or fully offset by the length, which for a steep gradient of 8 %, can be 5 to 10 times the width of the fjord. For more gentle and comfortable gradients the length and cost of the sub-sea rock tunnel would increase significantly.

The concept of the Submerged Floating Tube Bridge was conceived by Sir. Edward James Reed[14-1], an English naval architect. The concept was regarding a railway in a steel or iron submerged tubular structure and it was patented in 1886.

In 1890 a similar concept was patented in the USA[14-2], with multiple tubes and inclined connections with the seabed (H. Moerser, n.447735). In the beginning of 1900 other patents were registered in USA and France (M.F. Hennebique, n.357983, 1905), and in this last one concrete was proposed for the first time as the material.

A Norwegian patent was registered in 1923 by Trygve Olsen, under the name of 'dykket pontonbro', 'submerged pontoon bridge'. The idea was proposed for a crossing (Karmsundet) in 1948 by Erik Ødegård, a Norwegian engineer. Ødegård tried, without success, to register a patent for the SFTB using reinforced concrete as material.

In the beginning of the 1960's, two other patents were registered (M.F. Cristaldi, France 1960, n.1262386, and C. Lombardi, Italy, 1962).

In 1969 an English engineer, Alan Grant, proposed, for a competition, an SFTB to cross the Messina Strait. The project was re-proposed later by a society composed by Saipem, Snamprogetti, Tecnomar and Spea.

The largest challenge for the construction of the SFTB in the 1950's was the lack of technology, especially for the tether structures connecting the bridge with the seabed for deep crossings, and for the marine operations. These aspects were later to be studied in Norway as part of an official commission (SINTEF was the research centre involved) instructed by the Norwegian Public Road Administration in 1969 to evaluate the feasibility of the SFTB structure.

In 1969, when the commission started evaluating the feasibility of the SFTB, the first oil was found in Norway (Ekofisk). The technology related to the construction of the oil & gas platforms grew rapidly and in the early 1980's the first TLP platform was constructed in Scotland. In 1990's the first Norwegian concrete TLP (Heidrun) was installed.

Interest around the SFTB grew in those years, together with the technological development.

In 1982 the company Selmer AS designed and realized a submerged floating rectangular bridge with the length of 600 meters at Kalstø, for Statpipe, to protect two gas pipelines and bridge the subsea topography. This structure can be considered the precursor of the SFTB.

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In 1979 a study was made in Norway for the Eidfjord. In 1984 an idea for an SFTB in the Como lake was developed by an Italian engineer, Gianfranco Magrini. In 1986 the first Strait Crossing conference took place in Norway. In 1995 a study was made for the Lugano lake in Switzerland, in 1998 a project and some tests were performed in a study for the Høgsfjord, in Norway and, in 2004, a study was done for the Qiandao lake in China. More proposals were made around the world, but none of these structures were ever built.

In 2011 the feasibility study to cross the Sognefjord, was completed. This is the deepest fjord along the E39 and the solutions evaluated for this fjord could also be feasible for other fjords along the western coast of Norway.

The study on the Bjørnafjord, one of the first of the long and deep crossings along the E39, concluded in 2016, brings a step forward in the knowledge of the SFTB, vertically stabilized with pontoons or tethers. This was possible thanks to the investment in research, with collaborations with universities, researchers and consultants. At the same time, the experience gathered in more than 40 years of operation of the offshore structures, has helped the concept to become, from an idea, to a real possibility for a crossing.

In 2015 a 300 meter long submerged bridge founded on group of piles was constructed in Sweden (Söderströmstunnel, Stockholm). This structure, not relying on the soil under the structure but working as a bridge between the group of piles, has a static behaviour similar to an SFTB on column support.

In 2015-2017 contract preparations took place for a column supported SFTB crossing the Golden Horn in Istanbul. The project has been put on hold due to changing political and economic priorities.

In recent years, several studies are ongoing on the SFTB, especially for long and deep crossings. Some of them are evaluations for a use of the structure in a specific crossing (Norwegian Public Road Administration, ARUP, and others), others are related to research projects (KAIST, Korea).

The future will see an increased use of the sea for reasons of, among others, space, environment, food and recreation. For this reason, the SFTB will have a key role in the future traffic system.

14.2 Short descriptions of concepts

This section illustrates some 21 examples of SFTB proposals. Some of them are the result of detailed studies, some are not. The intention of showing them is to illustrate the variety of the concepts and to illustrate how many countries are interested in this technology.

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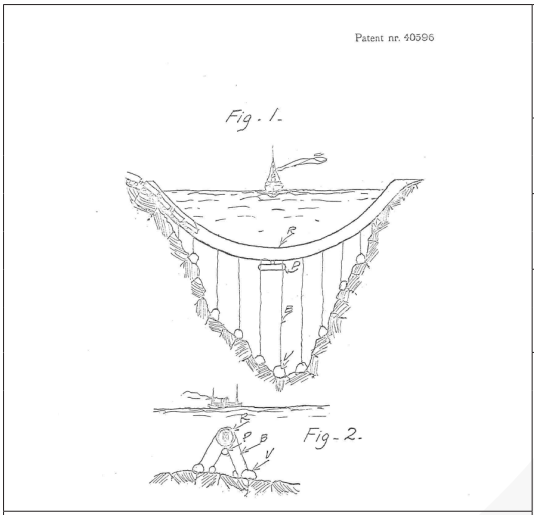


Fig. 14-1 Trygve Olsen

Name:		Year:
Dykket pontonbro (Submerged pontoon bridge)		1923
Location:		Country:
-		Norway
Total length [m]:	Water depth [m]:	Tube OD [m]:
-	-	-
Span lengths [m]:	Horizontal alignment:	Elevation [m]:
-	straight	-
Characteristics: Norwegian patent nr. 40596: arched submerged bridge, connected with tension structures to the seabed and with a pontoon at the deepest point of the structure, under the bridge, where all the pumping system is located.		

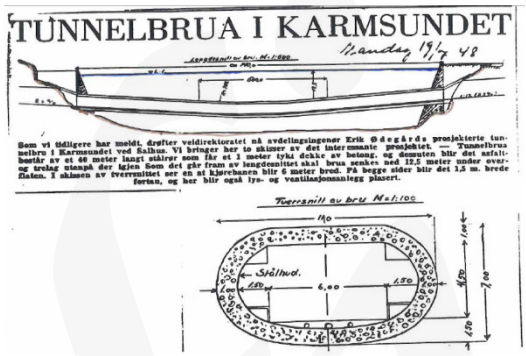


Fig. 14-2 Erik Ødegård – Statens vegvesen

Name:		Year:
Karmsund tube bridge		1948
Location:		Country:
Haugesund		Norway
Total length [m]:	Water depth [m]:	Tube OD [m]:
140	-	oval 11x7
Span lengths [m]:	Horizontal alignment:	Elevation [m]:
140	straight	-12.5
Characteristics: Free spanning tube bridge with concrete filled, double steel shell tube, shell thickness 1.0 m.		

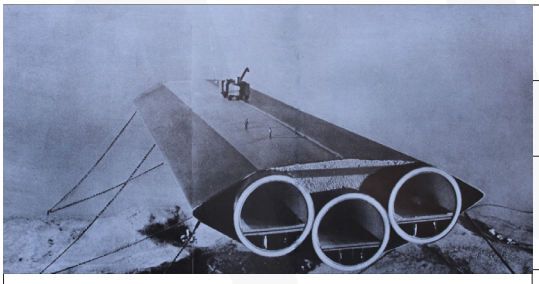


Fig. 14-3 Alan Grant & Partners

Name:		Year:
Messina SFTB		1969
Location:		Country:
Messina strait		Italy
Total length [m]:	Water depth [m]:	Tube OD [m]:
3 700	350	combined cross section
Span lengths [m]:	Horizontal alignment:	Elevation [m]:
-	-	-55
Characteristics: Concept proposed in a competition; Multi-mode SFTB anchored by inclined tethers @ 100-150 m. Cross section of 3 concrete tubes enveloped by a hydraulic steel shell. Politically on hold.		

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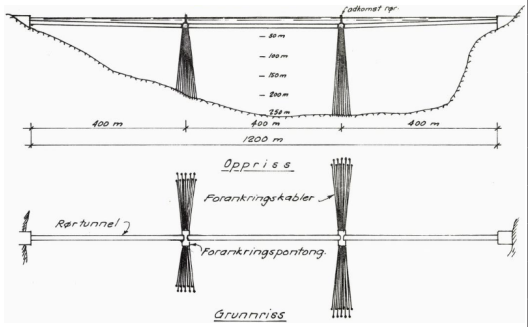


Fig. 14-4 SINTEF's rørbruutvalg – Statens vegvesen

Name: Eidfjord		Year: 1979
Location: Hordaland		Country: Norway
Total length [m]: 1'200	Water depth [m]: 250	Tube OD [m]: 11.4
Span lengths [m]: 400	Horizontal alignment: straight	Elevation [m]: 0/-16.5
Characteristics: Tube bridge vertically stabilized by group of inclined tethers		

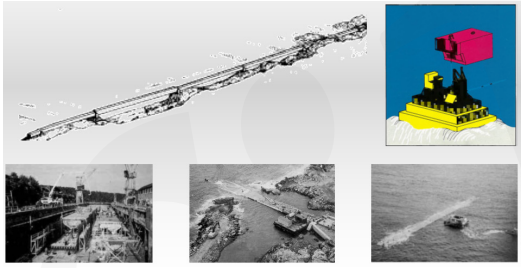


Fig. 14-5 Selmer AS - Skanska

Name: Statpipe Shore Approach SFTB		Year: 1982
Location: Kalstø		Country: Norway
Total length [m]: 670	Water depth [m]: 0 - 30	Tube OD [m]: rectangular 30 – 45 m^2
Span lengths [m]: 90 - 150	Horizontal alignment: straight	Elevation [m]: 0/-30
Characteristics: Constructed to protect and guide tie-in of gas pipelines. Rest on foundations, prefabricated and cast in-situ. Construction time 9 months.		

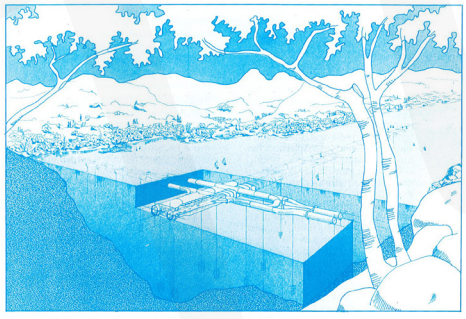


Fig. 14-6 Gianfranco Magrini – Edixxon.com

Name: Tubolario		Year: 1984
Location: Lake Como		Country: Italy
Total length [m]: ~70'000	Water depth [m]: 418	Tube OD [m]: 15.5
Span lengths [m]: -	Horizontal alignment: straight	Elevation [m]: -20
Characteristics: Twin tube bridge, for road and railway, anchored to the seabed with tethers		

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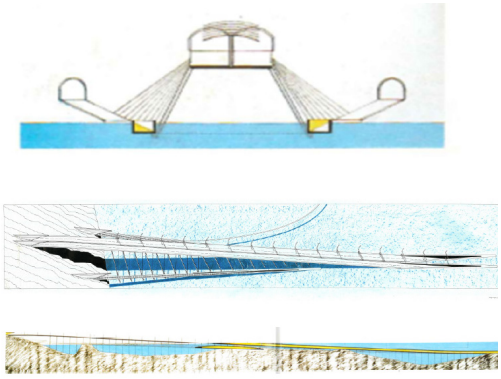


Fig. 14-7 Snøhetta

Name: Drøbaksundet hybrid bridge		Year: 1989
Location: Drøbak, Oslo fjord		Country: Norway
Total length [m]: -	Water depth [m]: varies	Tube OD [m]: -
Span lengths [m]: -	Horizontal alignment: -	Elevation [m]: -

Characteristics:
Proposal for combined rail and car bridge, with different inclination for the two types. Submerged on the east side where a bridge was unwanted, raising to a tall bridge on the west side where no one lives.

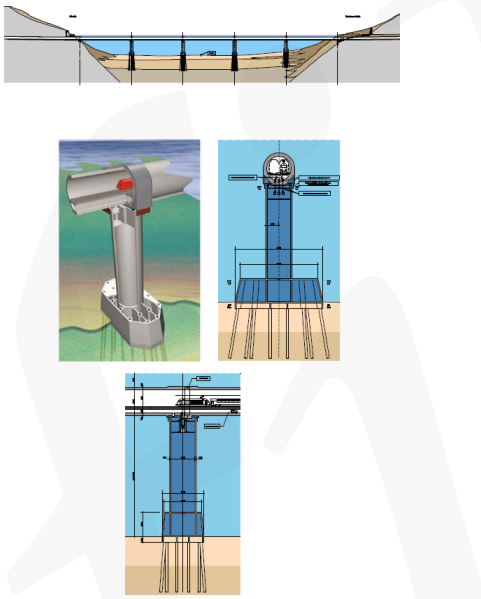


Fig. 14-8 Grignoli Muttoni Partner/Selmer/ Dr.techn.Olav Olsen

Name: Lake Lugano SFTB crossing		Year: 1995
Location: Lugano lake		Country: Switzerland
Total length [m]: 930	Water depth [m]: 70	Tube OD [m]: 12.3/13.6
Span lengths [m]: 186	Horizontal alignment: straight	Elevation [m]: -6

Characteristics:
Column supported straight SFTB. Project for the continuation of the new AlpTransit through Tessin. Conceptual idea was to avoid noise and vision of high-speed trains coming out of rock tunnels through the Alps, by simply lowering the route to a bit under the water level in the Lake Lugano.

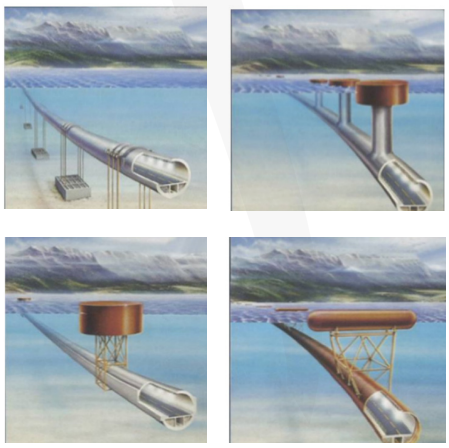


Fig. 14-9 Selmer, Kværner Rosemberg as, Aker Norwegian Contractors, EEG-Henriksen – Statens vegvesen

Name: Høgsfjorden SFTB		Year: 1998
Location: Høgsfjord		Country: Norway
Total length [m]: 1'400	Water depth [m]: 150	Tube OD [m]: 11.4
Span lengths [m]: -	Horizontal alignment: straight	Elevation [m]: -20

Characteristics:
Four different solutions, from four contractors, invited by the Norwegian Road Administration. All alternatives deemed feasible for all the road locations investigated.

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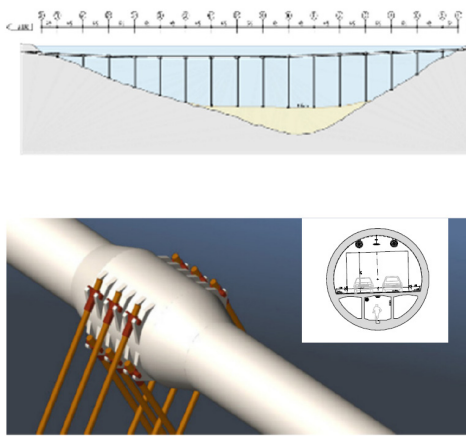


Fig. 14-10 Dr.techn. Olav Olsen

Name: Breisundet SFTB crossing		Year: 1999
Location: Sulafjord		Country: Norway
Total length [m]: 4'200	Water depth [m]: 450	Tube OD [m]: 11.6 / 17.4
Span lengths [m]: 250	Horizontal alignment: straight	Elevation [m]: - 35
Characteristics: Conceptual study for a crossing directly exposed to storm waves and swells from the North Sea. Concrete tube for 2-way carriageway with variable cross section (lay-bys). Anchoring system of two pairs of inclined tether groups connected to gravity anchors.		

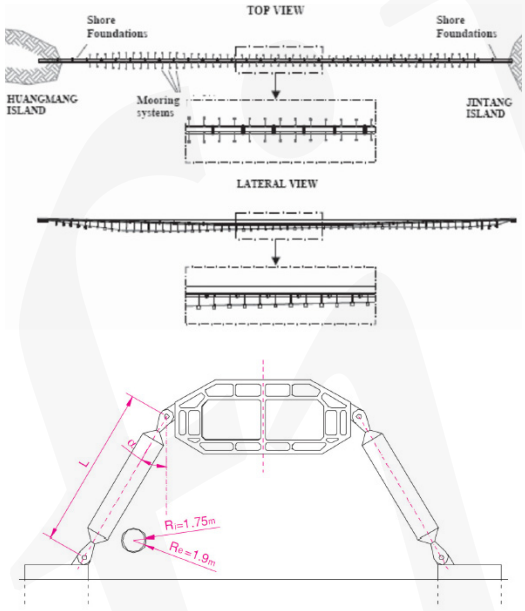


Fig. 14-11[14-3]

Name: Jintang strait		Year: 1999
Location: Jintang		Country: China
Total length [m]: 3'200	Water depth [m]: -	Tube OD [m]: combined 27.9×12.1
Span lengths [m]: 100	Horizontal alignment: straight	Elevation [m]: 0/-25
Characteristics: Concrete section connected to the seabed with inclined steel tubular members located 50 m apart along the length of the tunnel.		

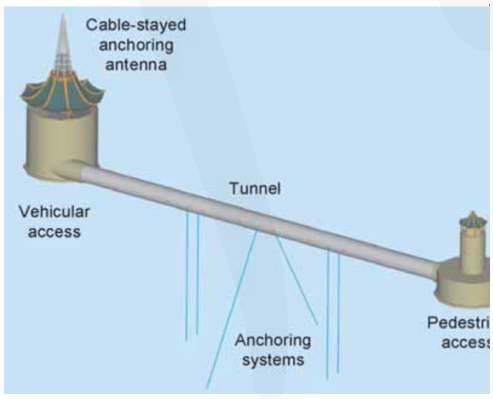


Fig. 14-12[14-4]

Name: Qiandao lake bridge		Year: 2004
Location: Chun'an County		Country: China
Total length [m]: 100	Water depth [m]: -	Tube OD [m]: 4.39
Span lengths [m]: 20	Horizontal alignment: straight	Elevation [m]: -2/-10
Characteristics: Composite cross section (aluminum – external-, concrete and steel -internal-)		

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Japan

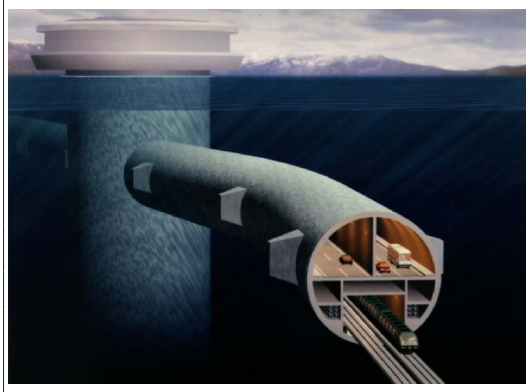


Fig. 14-13 Funka Bay Crossing with an air ventilation tower

Name:		Year:
Funka Bay Crossing		2010
Location:		Country:
Funka Bay		Japan
Total length [m]:	Water depth [m]:	Tube OD [m]:
30'000	120	4.39
Span lengths [m]:	Horizontal alignment:	Elevation [m]:

Characteristics:
One of several studies performed in Japan[14-5]

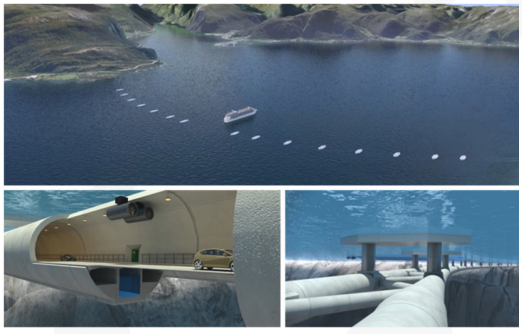


Fig. 14-14 Dr.techn. Olav Olsen & Reinertsen Eng. et al. – Statens vegvesen

Name:		Year:
E39 Sognefjord SFTB		2012
Location:		Country:
Sogn og Fjordane county		Norway
Total length [m]:	Water depth [m]:	Tube OD [m]:
3'700	1'200	12.6
Span lengths [m]:	Horizontal alignment:	Elevation [m]:
205	R = 2'682 m	-12 / -20

Characteristics:
Pontoon stabilized, end-anchored SFTB, twin tube arrangement. SFTB designed to survive the loss of one pontoon.

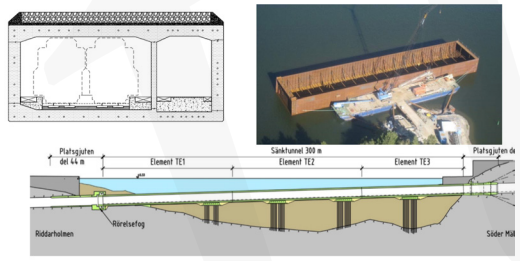


Fig. 14-15 Cross section, concrete casting afloat and longitudinal section - Trafikverket

Name:		Year:
Söderströmstunnel		2015
Location:		Country:
Stockholm		Sweden
Total length [m]:	Water depth [m]:	Tube OD [m]:
300	-	-
Span lengths [m]:	Horizontal alignment:	Elevation [m]:
107.5 max	-	-

Characteristics:
Subsea tunnel realized as a continuous steel single shell tube supported on discrete pile foundations, having a static behaviour like that of an SFTB on column support.

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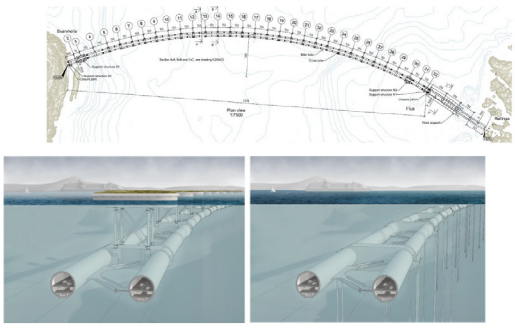


Fig. 14-16 Dr.techn. Olav Olsen & Reinertsen Eng. et al. – Statens vegvesen

Name:		Year:
E39 Bjørnafjord SFTB		2016
Location:		Country:
Bjørnafjord		Norway
Total length [m]:	Water depth [m]:	Tube OD [m]:
5'350/5'495	580	9.5/12.5
Span lengths [m]:	Horizontal alignment:	Elevation [m]:
200	R = 2'350	-30
Characteristics: Pontoon and tether stabilized arched SFTB, twin tube cross section. The structure is designed to withstand the loss of a pontoon or a tether, and to survive several design conditions, for example impact with submarines, fires and explosions.		

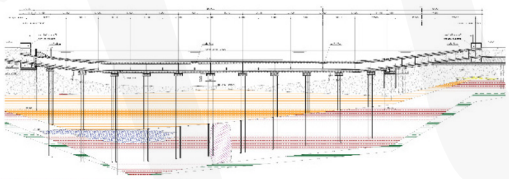






Fig. 14-17 Dr.techn. Olav Olsen & Reinertsen Eng. et al. – Statens vegvesen

Name:		Year:
Rovdefjord crossing		2017
Location:		Country:
Rovdefjord		Norway
Total length [m]:	Water depth [m]:	Tube OD [m]:
230	70	13.45
Span lengths [m]:	Horizontal alignment:	Elevation [m]:
230	R = 300 m	-14
Characteristics: A 3 500 m long combined crossing consisting of a SFTB, a helical culvert, a floating bridge and a viaduct/filling. The tube bridge spans 230 m freely between shore and Saudeholmen islet (navigation channel). The transition between tube and floating bridge is achieved by a spiraling culvert founded on the islet fulfilling the slope requirement of 5 %.		



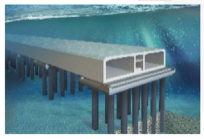


Fig. 14-18 Long section and artist impression, Arcadis/IBB

Name:		Year:
Golden Horn Unkapanı Highway Tube Tunnel		2017
Location:		Country:
Istanbul		Turkey
Total length [m]:	Water depth [m]:	Tube OD [m]:
660	35	-
Span lengths [m]:	Horizontal alignment:	Elevation [m]:
60	straight	10
Characteristics: Prestressed 60 m sections immersed on pile caps halfway between river bed and water surface. Local seismic activity demanded flexible solutions (joint house, relatively flexible columns). Project was put on hold after awarding the construction contract.		

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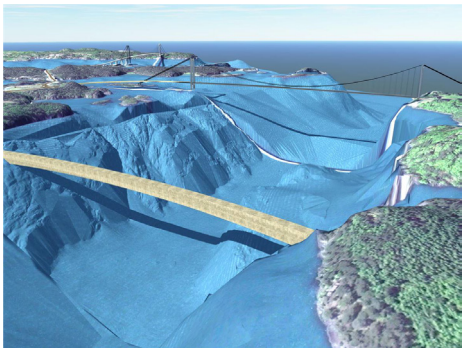


Fig. 14-19 Dr.techn. Olav Olsen – Statens vegvesen

Name: Digernessundet SFTB		Year: 2017
Location: Bømlafjord		Country: Norway
Total length [m]: 515	Water depth [m]: 200	Tube OD [m]: rectangular 12.2×16.3/18.8
Span lengths [m]: 515	Horizontal alignment: R = 1867 m	Elevation [m]: -10
Characteristics: Several SFTB configurations have been evaluated, considering a single span or a multi-span bridge. The chosen design is a free spanning rectangular tube bridge.		

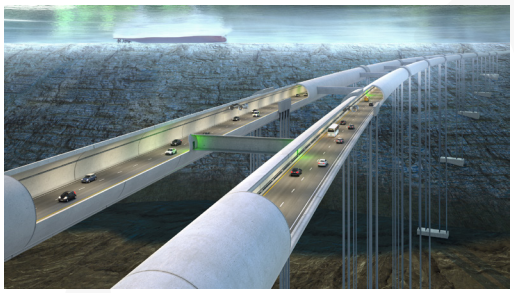


Fig. 14-20 Dr.techn. Olav Olsen – Statens vegvesen

Name: E39 Sulafjord SFTB		Year: 2018
Location: Sulafjord		Country: Norway
Total length [m]: 3 640	Water depth [m]: 440	Tube OD [m]: 11,5
Span lengths [m]: 183	Horizontal alignment: 3 half waves	Elevation [m]: -57,5
Characteristics: Tether stabilized SFTB, twin tube cross section. The higher submergence, compared to the proposals for the other fjords, allows reduction of the swell wave loading on the tubes and is necessary for the high exposure of the crossing position.		

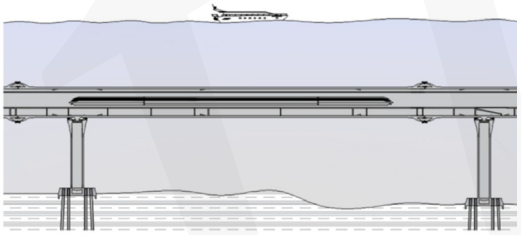


Fig. 14-21 Elia Notari, Prof. Aurelio Muttoni, Ing. Francesco Moccia

Name: Pont submergé dans le lac Léman		Year: 2018
Location: Geneva lake		Country: Switzerland
Total length [m]: 37 810	Water depth [m]: 50	Tube OD [m]: 14,5
Span lengths [m]: 190	Horizontal alignment:	Elevation [m]: -30
Characteristics: SFTB accommodating a high-speed MAGLEV line along north shore of Lake Geneva. Prestressed concrete tube assembled of 199 tube elements and supported on pile-founded tubular columns dia. 6 m.		

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