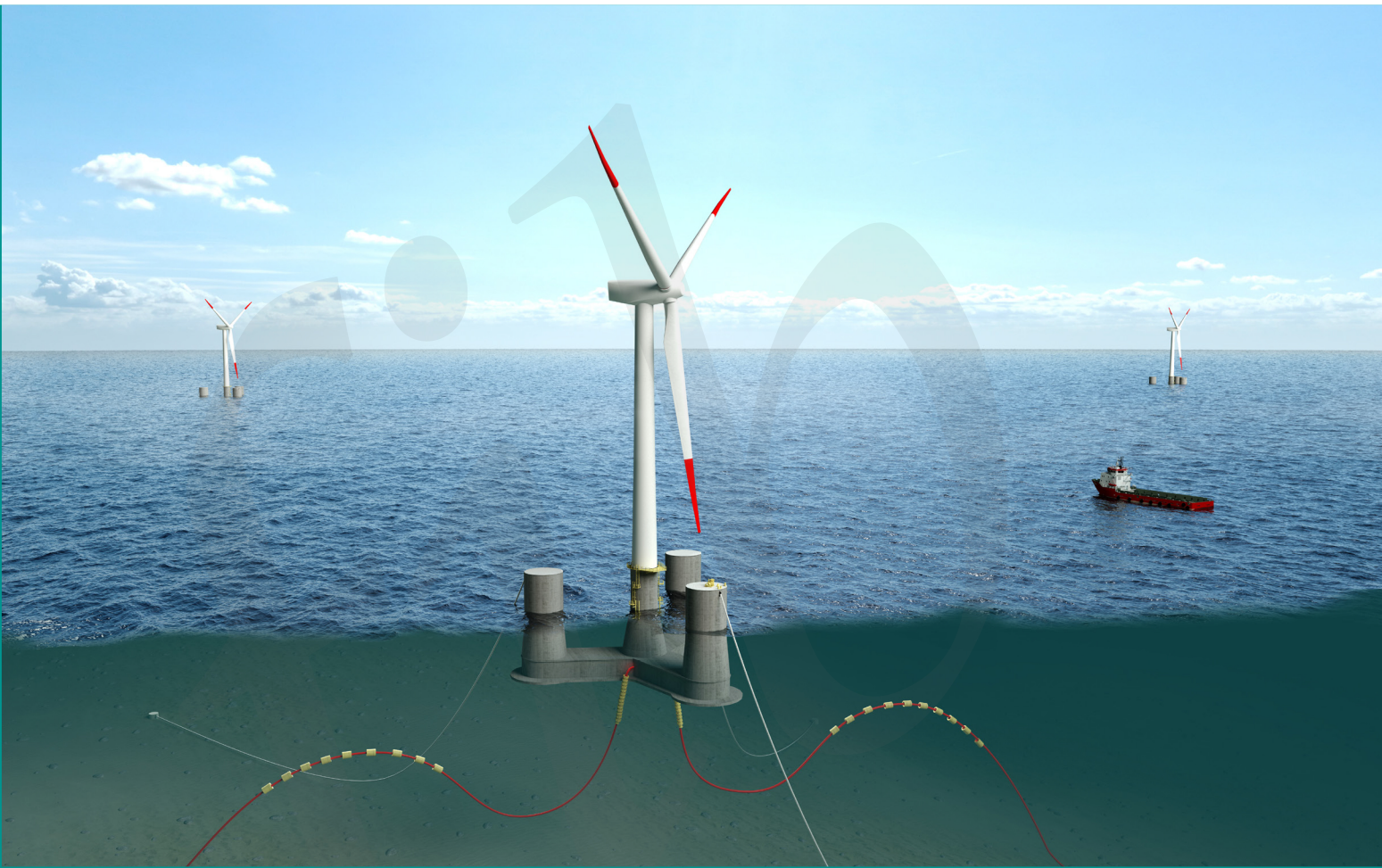
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Bulletin 91



Floating concrete structures

State-of-the-art report

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fib CEB-FIP Bulletin 91

Floating concrete structures

State-of-the-art report

Working Party 1.2.1

August 2019

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Approval for this bulletin

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Technical report	Task group and chairpersons of the commission
State-of-the-art report	Commission
Manual / Guide to good practice / Recommendation	Technical Council
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Any publication not having met the above requirements will be clearly identified as a preliminary draft.

fib Bulletin 91 was approved as a State-of-the-art report by Commission 1 *Concrete Structure* in 2017.

Cover image: "Floating Wind Turbine"
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ISSN 1562-3610
ISBN 978-2-88394-134-2
doi.org/10.35789/fib.BULL.0091
Proofreading by Gordon Clark. Layout by Corinne Bottollier (*fib*).
Printed by DCC Document Competence Center Siegmar Kästl e.K., Germany

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Acknowledgements

This report was drafted by Working Party 1.2.1 "Floating concrete structure", in Task Group 1.2 "Concrete structures in marine environments" within the *fib* Commission 1.

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Foreword

This bulletin is the first document prepared by TG 1.2 "Concrete structures in marine environments". This theme is considered important for Commission 1, since in the future several applications are forecast in marine environments.

Floating concrete structures allow the use of marine spaces with important developments for urban areas, industrial plants, infrastructures and energy production. In this bulletin a series of applications, projects and conceptual ideas are presented. This should be considered as a document representing the potentialities and the innovations on the use of structural concrete in marine environmental.

The floating structure presented in the bulletin should be of inspiration for new application that will be developed in the nearly future, representing a challenge not only for structural designers, but also for administrations, construction companies and industrial entities.

The use of structural concrete is becoming essential in these kinds of applications, in terms of cost, durability and sustainability.

As Commission 1 chair, I'm very grateful to Tor Ole Olsen and to all members of WP 1.2.1 "Floating concrete structures" for having produced this document that I consider very interesting not only for the *fib* members, but also for concrete community.

Alberto Meda

Chair of Commission 1: Concrete Structures

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Preface

The future will see an increased use of the sea, by man. Important reasons are, among others, demography, the need for food, and rising sea level.

Fortunately, well-designed and well-built concrete structures handles the sea very well. Structures may be designed for a life of several hundred years.

Floating concrete structures is not novel. As early as 1848 Lambot built a small boat of concrete. Since then a number of marine structures have been built.

In the last 50 years a lot of experience has been gained from the design and construction of offshore platforms for the oil and gas industry. Many of these are not permanently floating, but was afloat during construction.

fib has recognized the great potential of floating concrete structures, and taken the initiative to write a state-of-the-art report on the subject. It is hoped that the report will stimulate to creative use of a well suited material, to the best for the users and the industry. Hopefully also the report will give examples that may be helpful, with a lot of references.

Tor Ole Olsen

Convener of Task Group 1.2 and Working Party 1.2.1: Concrete structures in marine environments

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1. Introduction

The future will see an increased activity in the sea, for various purposes. This state-of-the-art report will, by examples, illustrate applications that may form a basis for many alternative uses and possibilities. Extensive references are provided.

Concrete in marine structures is not a new thing. Lambot built his concrete canoe in 1848, and it is still conserved. What is new are the improved material properties, gained experience, and an increased need for space and proximity to the sea. This also applies to rivers, for which development work is being carried out in Paris and London.

As weight is particularly important for structures that are designed to float, some advanced methods of design will be demonstrated. LWA (Light Weight Aggregate) concrete will also be described.

Immersed tunnels are not treated, although they are important, since the ITA (International Tunnelling Association) has carried out extensive work on the subject.

Some marine structures resist high loads and are quite sturdy, such as offshore concrete platforms for the oil and gas industry. Other applications call for smaller and lighter structures. Fibre reinforced concrete has been shown to be effective for such cases, and a chapter on this subject is included.

Many of the offshore concrete structures for the oil and gas industry are gravity based, meaning that they sit on the sea floor. However, while being constructed and installed, they were afloat, and much of the experience from working with them may be beneficial for permanently floating structures. For this reason bottom fixed concrete marine structures are also included in this report on floating concrete structures, where there is relevance.

It is anticipated that, in the future, cost reductions will be possible based on the experience gained.

Offshore platforms are dealt with in previous *fib* bulletins 50[1] and 18[2] and within general references[3,4,5].

It is emphasized that the examples shown in this report are not a complete sample of the possibilities. Rather the opposite, *fib* hopes this report will serve as an inspiration to find new and novel applications

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doi.org/10.35789/fib.BULL.0091.Ch01

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2. The beauty of the sea

The sea represents a wealth of opportunities, as it always has, and mankind has been clever in exploiting the opportunities. There are challenges ahead, and the sea will help to solve them.

Many people enjoy the sea, for good reasons. Recreation is one of the good reasons.



Figure 2-1 One of the beauties of the sea[6]

There are also other reasons, linked to usefulness; some key words are listed in Chapter 4.

Three very useful abilities of the sea are obvious.

The first is buoyancy. The value of buoyancy cannot be overstated; the largest object ever moved by man weighed 1.5 million tonnes.

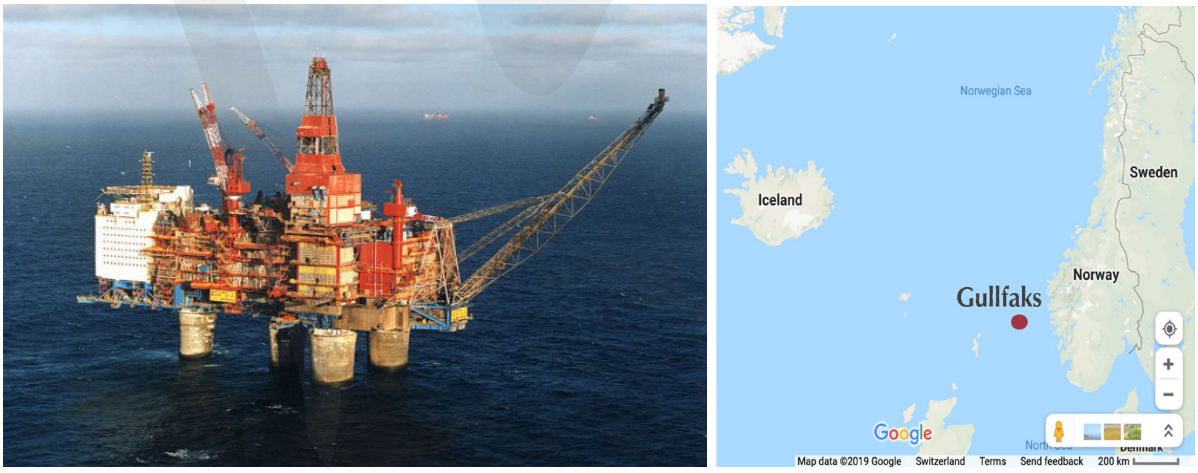


Figure 2-2 Gullfaks C[7] platform which was towed over a significant distance of some 250 km.

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doi.org/10.35789/fib.BULL.0091.Ch02

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The method of construction of these oil platforms is well developed and effective: the bottom part of the structure is built in a graving dock, floated out to be completed in a sheltered fjord. In parallel the topside factory is built, and placed on barges. When completed the concrete substructure is lowered by ballasting, the topside is towed over it, and lifted off the barges by de-ballasting. Then the integral platform is towed to site and installed.

Smaller structures may be built differently, on barges, in an existing dry dock for ship building, or on land and lifted into the sea.

Doris of France was the pioneer, with their first platform the Ekofisk Centre, built in Norway.



Figure 2-3: The Ekofisk centre platform[8]

The Troll A below carries a 22'000 tonnes deck 150 m above the water:



Figure 2-4 The Troll A platform on its way to installation[9]

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3. Some challenges of the sea

There are also challenges with the sea. The harshness of the sea has caused a lot of damage to people and man-made objects, as well as to nature itself. People depending on the sea have experienced this for thousands of years. Fortunately there is sound knowledge about many aspects of the environment, but predicting the environment everywhere 100 years ahead is not an exact science.

And the marine environment is feared to worsen in the future, with sea level rising and stronger storms.



Figure 3-1 The Oseberg A platform in the North Sea[10]

From a design and cost point of view there are many loads acting on a floating structure, including waves from all directions, ballasting changing over time, and live loads changing over time.

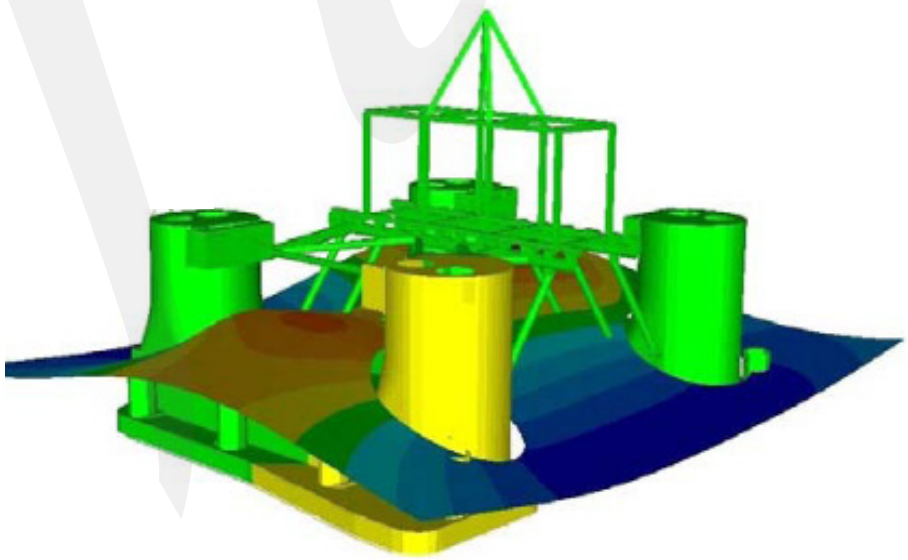


Figure 3-2 Wave loads on a floating structure[11]

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doi.org/10.35789/fib.BULL.0091.Ch03

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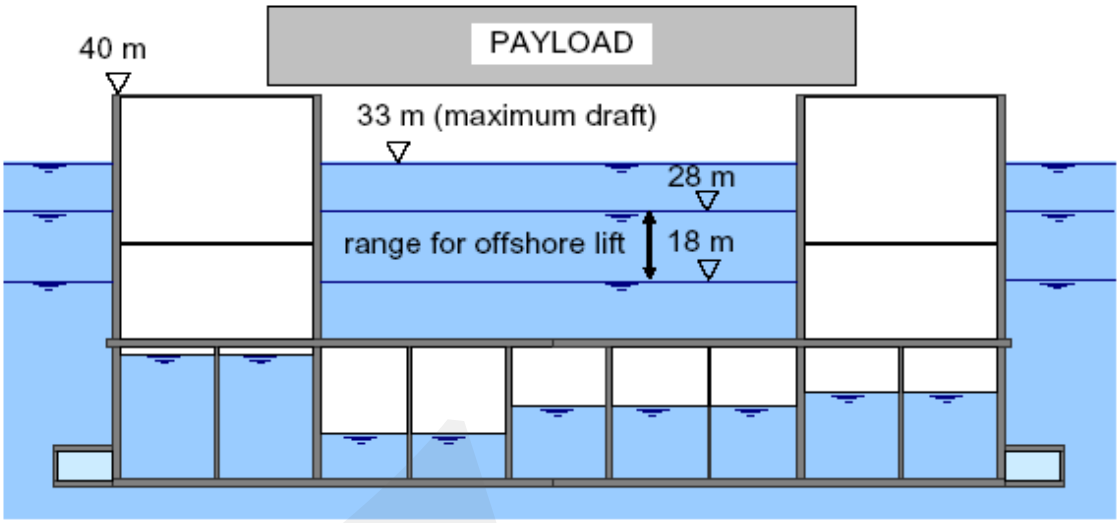


Figure 3-3 Static loads on a floating structure[11]

Another important element of marine structures is the method of construction, illustrated below in Figure 3-4 by a sea crane lifting a small structure from land to sea. Larger structures need other methods, so it is important to integrate construction and design closely.

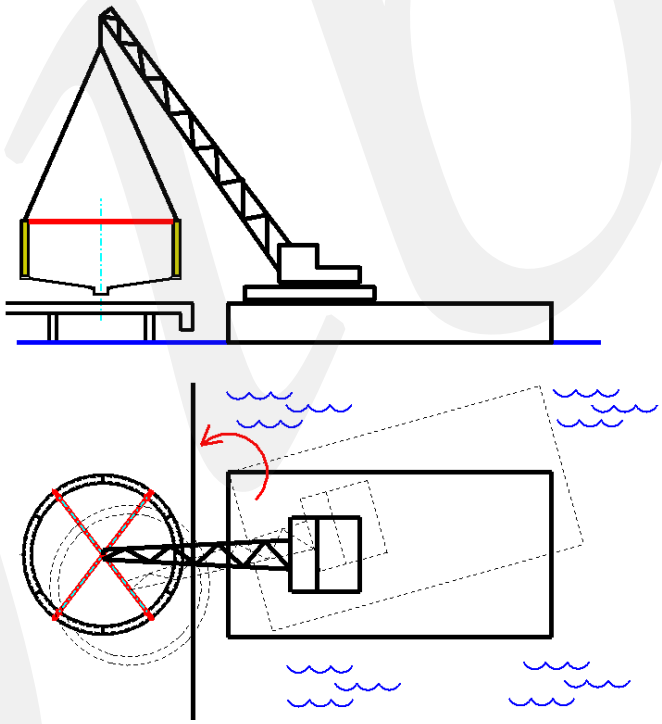


Figure 3-4 Lifting a small floating structure from construction on a dock into the sea. Larger structures need other ways of construction[11]

Because they need to float, marine structures are made as slender as possible. This means that the reinforcement content is high, calling for careful design and construction planning. Needless to say weight control is an important part of marine structure projects.

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4. General applications

As already stated, the future will see an increased use of the sea. Examples of application of floating concrete structures may include, but not be limited to, the following:

- Food production
- Infrastructure
- Energy
- Environment
- Dwellings and urban development
- Nearshore industrial development
- Offshore industrial development
- Storage
- Vessels
- Recreation
- Aid in catastrophes
- Military actions.

Mastering the technology to make efficient floating structures includes, but is not limited to, the following:

- Conceptual design
- Detail design
- Structural analysis
- Geotechnical analysis
- Hydrodynamic analysis
- Naval Architecture
- Construction
- Materials
- Sustainability
- Cost.

All these applications and all these technologies cannot be described in detail in this report, but are listed here to indicate potentials.

Another key indicator to support a belief that the sea will see an increased attention for manmade activities and use is the fact that the world's cities are growing. General growth in world population combined with increasing urbanization puts pressure on cities to exploit new areas. Reclaimed land is an option often used, but when not appropriate the floating city may be the best option.

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doi.org/10.35789/fib.BULL.0091.Ch04

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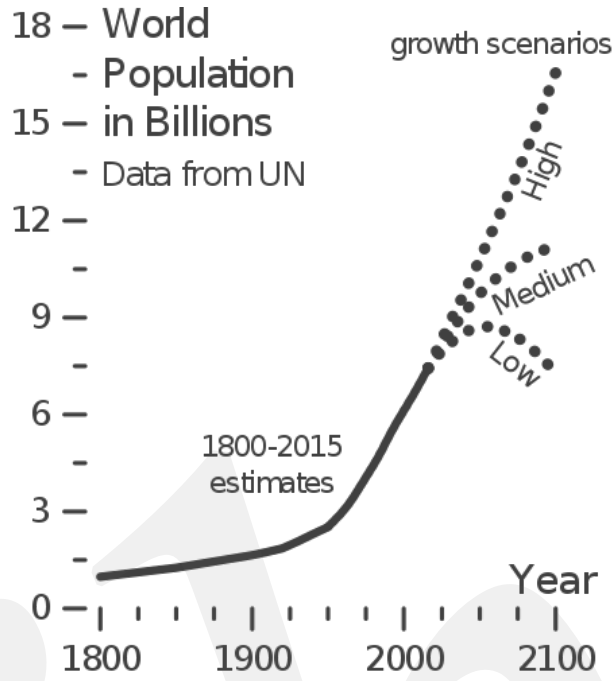


Figure 4-1 World Population[12]

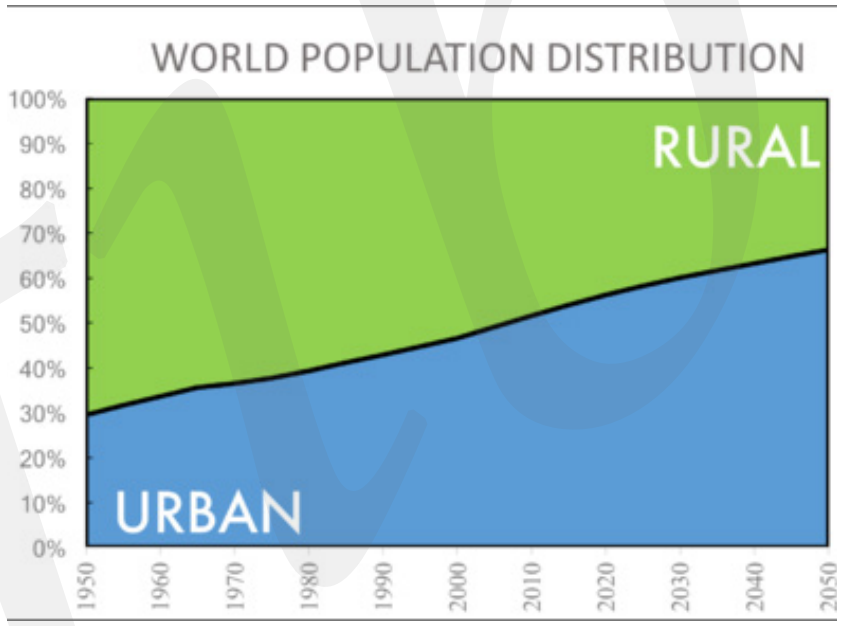


Figure 4-2 World population distribution[13]

Additionally a large percentage of the cities and the population in general are situated close to the sea; it is believed that 40% of the population lives closer than 100 km from the sea. Utilizing the sea as real estate will enable cities to grow further.

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5. Projects and ideas

One of the objectives of this state-of-the-art report is to inspire proper application of concrete for marine structures. Both completed projects and those imagined or in conceptual/planning stage are included.

Looking into the web and literature, there are many examples of floating marine structures as some are illustrated below. Many of these are made of steel, but increasingly more are of concrete, due to the good properties and recent development. Some of them are completed and in use and others are projects planned for the future.

It is really imagination that is the limit. Everything can be designed, and everything can be built. And creative skills can be learned, by experience from own activities and by inspiration from what has been achieved.

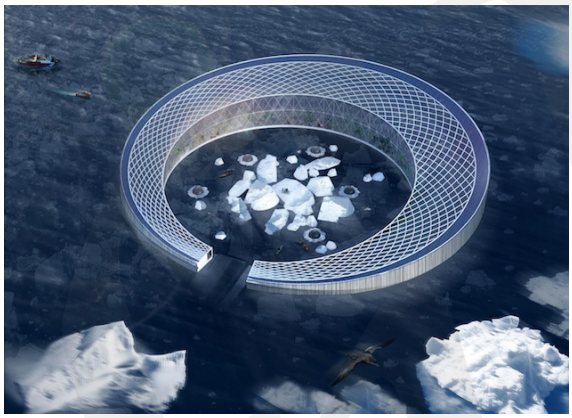


Figure 5-1 Floating Arctic City[14]



Figure 5-2 Floating Runway[15]



Figure 5-3 The Ark - Floating City[16]

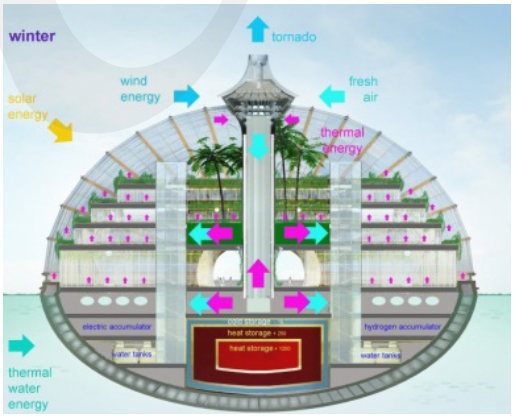


Figure 5-4 Chapter through The Ark - Floating City[16]

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doi.org/10.35789/fib.BULL.0091.Ch05

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Figure 5-5 Lilypad Floating City Concept[17]

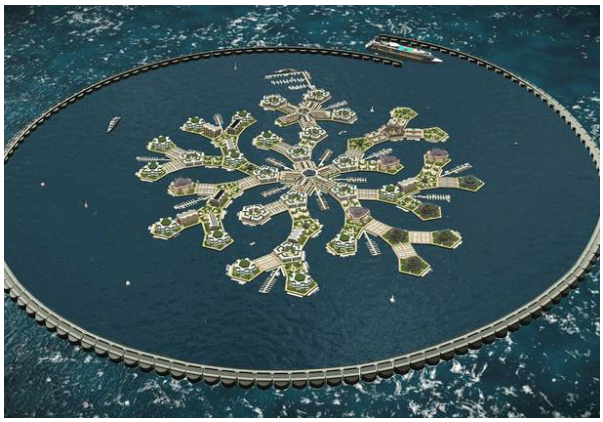


Figure 5-6 Floating City[18]



Figure 5-7 The Rion-Antirion bridge[19]



Figure 5-8 Mulberry Harbour, Arromanches, Normandy, France[20]

This astonishing structure was constructed during the Second World War, to facilitate the landing of allied troops in Normandy. Description of the project may be found on the Internet, where the picture above was found.

See for example "The Mulberry Harbour"[20]

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Figure 5-9 Adriatic LNG Terminal - During tow from Spain to Italy[21]



Figure 5-10 Floating Solar Power Plant[22]



Figure 5-11 Floating House[23]

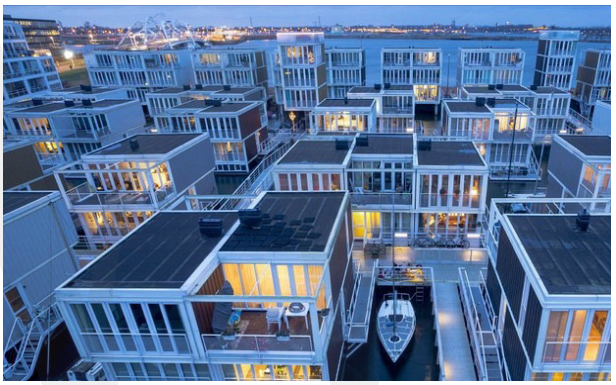


Figure 5-12 Floating homes in Steigereiland[24]



Figure 5-13 Floating Performance stage Singapore[25]

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Figure 5-14 Kami-goto floating oil storage facility[26]

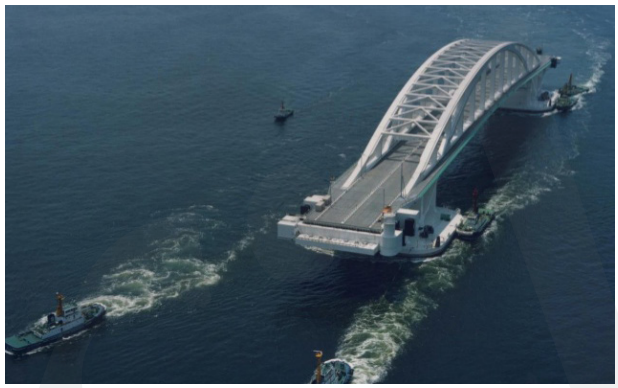




Figure 5-15 Yume Mai Bridge[27]



Figure 5-16 Floating homes in Maasbommel[28]



Figure 5-17 Floating Island Seoul[29]



Figure 5-18 Floating Concrete Breakwater[30]

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6. Design

6.1 General

Design is about creating a structure that fulfils the requirements set out. For floating structures physical requirements are very important, as is the construction method. For anchoring of the structure, geotechnical aspects also need to be considered.

Weight is important. Poorly designed structures may not float as expected. Subsequent phases of design may lead to increasing weight, which may result in unfortunate problems.

Concrete as a material is very well suited for the marine environment, when properly designed and properly built. Concrete is a less costly material as compared to steel, but it may be heavier. Therefore it is important to use efficient shapes for the structure, and as discussed later, lightweight aggregate concretes may be used.

There are hundreds of floating structures that are functioning well, and the design of them is not a novel discipline. Some key elements are mentioned here.

Design work will typically take place in three consecutive phases:

- The Concept phase, to define the global geometry.
- The Project Definition, or FEED (Front End Engineering Development) phase, to verify concept soundness and to provide the technical basis for Detail Design.
- Detail Design phase to produce the drawings and specifications for Procurement and Work Preparations.

The main objectives of the Concept phase are to screen, select and develop the best concept, and to define the global configuration deliverables needed as basis for the FEED phase. The concept work consists of three distinct tasks:

- Design Basis Validation, e.g. functional requirements, maintenance issues, relevant regulatory requirements, environmental considerations, etc.
- Concept Development loop, finalisation of concept geometry is an iterative process involving geometry proposals, load and weight estimates, construction constraints, floating stability, code reviews and other relevant factors affecting temporary and in-place performance.
- Global configuration definition, which is the main overall deliverable of the concept phase.

The Project Definition phase design objective is to confirm that the selected concept is mature enough to enter detail design (i.e. no major geometry adjustments foreseen) and consequently cannot commence until the global configuration task has been completed. This phase includes basic engineering activities necessary to define the selected concept and provides input documentation to enable detail design, procurement and tendering processes. It includes structural and outfitting design, preparation of specifications, material selections, concrete mix designs, preliminary construction method statements, reports, design dossiers, registers and data sheets.

This chapter was mainly authored by Drag Nikolai Jensse.

doi.org/10.35789/fib.BULL.0091.Ch06

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The Detail Design phase objective is to produce timely and complete engineering documents required for procurement activities and for the construction of temporary and permanent works. At this stage the structural geometry and main dimensions, e.g. wall thicknesses, are fixed. The purpose of finite element analyses during detail design is to determine reinforcement and post-tensioning steel dimensions and quantities. If the designer has been too optimistic at the concept stage the rebar density will exceed the space available and wall thicknesses must be revised which will affect load calculations, weight estimates and floating stability and the net effect is to risk needing to start all over at the concept phase.

6.2 Concrete design

The initial sizing of the concrete structure will be based upon data from experience and hand calculations in the Concept Development loop above. Several companies have developed IT tools to progress a project through the three design phases, such as Arup in the UK and Multiconsult and Dr.techn.Olav Olsen in Norway. Whilst design automation to calculate loads and perform code checks is part of everyday life in a design office the eyes and reviews of an experienced engineering team is still a prerequisite for successful design deliverables.

Both linear and non-linear finite element programs are commercially available. Practical experience when modelling and selecting element types is a critical aspect of the linear and non-linear FE analyses employed for concrete design. Reinforced concrete will demonstrate non-linear behaviour and methods such as the Modified Compression Field Method and the Modified Compression Field Theory, developed by Professor Collins and his team at University of Toronto, provide a rational approach to estimating shear capacity beyond the standard design codes.

The references[31,32,33,34] give specific guidelines and description of a modern tool for performing efficient and safe design.

6.3 Foundation design

Foundation design is a key issue in concept development and detail engineering of marine concrete structures. Functional requirements, as well as water depth, soil conditions, ice loads and pay load are parameters that will govern the selection of a gravity based (fixed) structure or a floater.

Gravity based structures will typically be designed with skirts penetrating soft upper seabed soil layers to achieve foundation stability. The skirt compartments may then be grouted after installation to improve load distribution and transfer. Water current velocities and base geometry may call for scour protection measures.

Floaters can be anchored in several ways such as pile sleeves, catenary mooring lines and tension legs. Anchors for mooring lines and tension legs are often concrete suction piles. Pontoon bridges have employed both catenary mooring solutions, providing intermediate side supports, and pontoons bolted together and anchored with land bearings at either end.

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6.4 Some Design Rules and Guides

Several institutions have contributed to design rules and guidelines:

- DNV-GL:
  - · DNV-OS-C502 Standard for Offshore Concrete structures
- ACI Committee 357:
  - · 357.2R-10 Report on Floating and Float-In Concrete Structures
  - · 357R-84 Guide for the Design and Construction of Fixed Offshore Concrete Structures
  - · 357.2R-88 Report on Barge-Like Concrete Structures
  - · 357.1R-91 State-of-the-Art Report on Offshore Concrete Structures for the Arctic).
- *fib:*
  - · Bulletin 18 Recycling of offshore concrete structures
  - · Bulletin 50 Concrete structures for oil and gas fields in hostile marine environments.
- FIP:
  - · Prestressed concrete ships
  - · Lightweight aggregate concrete for marine structures
  - · Cover to steel reinforcement for floating concrete structures
  - · Permanent anchorages for offshore structures, Design and construction of concrete ships
  - · Durability of concrete structures in the North Sea.
- Standard Norway:
  - · NS3473. Used worldwide for offshore concrete
- International Organization of Standardization - ISO
  - · ISO 19903 Petroleum and Gas industries – Fixed concrete offshore structures NOTE: Under revision to include also floating structures.

ISO 19903 states, in Section 8.2.1, that the reference standard to be used shall comply with the basic principles of ISO 19900. And further notes: NS 3473.E has been widely used and is recognized as meeting the requirements of this subclause (8.2.1). However, any standard can be used as a reference standard provided it is supplemented by additional requirements, where necessary, to ensure that all relevant aspects for the design are properly covered. The further subclauses of 8.2 can be used as a check list when selecting the reference standard to be used for a specific project.

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7. Loads

Real understanding of the loads is very important for the safety of structures. This applies to all structures, including floating structures.

Most of the loads on a floating structure come from "nature", as environmental loads. These need thorough investigations and measurements.

The main loads to be considered include:

- Wave loads. Wave loads are made up of an inertia component, predominant for large volume structures, and a drag force component; the latter dominates the loads on frame type structures. Both deterministic and stochastic models are used to calculate wave loads. Available calculation tools provide answers that compare well with model tests. Model tests may be needed to assess particular effects, e.g. wave run-up or shallow water effects; these should be performed at the FEED stage.

- Current loads. Drag forces, and wind loads, can cause vortex shedding which under given circumstances may excite natural frequencies and cause fatigue and instability. This potential effect must be addressed and mitigated during the FEED stage.

- Wind loads. Any superstructure will be subject to wind loads. Reliable wind data are available for most areas and the actual load analysis must account for location and orientation effects.

- Ice loads. Analytical models have a tendency to cause discussion, and it is important to make sure to address these up front, and resolve during FEED if at all possible. Several ice laboratories can run realistic model tests.

- Hydrostatic pressure may govern part of the design, pressure differentials may arise as a result of installation methods and operational features.

- Seismic loads. In general marine concrete structures are sufficiently robust to not be overly impacted by seismic loads, but outfitting design and foundation behaviour may need to consider accelerations caused by seismic loads.

- Accidental loads. Unplanned events during operation can introduce accidental loads resulting from aspects such as dropped objects, ship impact and loss of differential pressure.

- Temperature gradients. Hydrocarbon storage systems will impose thermal loads where tightness and durability are major concerns recognized by the codes.

- Swell and tide. The anchor loading on marine structures may be affected by swell and tide effects; in particular these may cause severe loading when combined with ice conditions.

Some of the Codes and Rules listed in section 6.4 include guidelines for calculating environmental loads. Other effects, like Accidental loads and Temperature gradients, are function specific and must be established and defined when developing the Design Basis.

This chapter was mainly authored by Drag Nikolai Jensse.

doi.org/10.35789/fib.BULL.0091.Ch07

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8. LWA (Light Weight Aggregate), and a LWA concrete structure

Concrete is preferable as compared to steel when robustness, cost and durability are considered. It may not be so if weight is important.

For obvious reasons, as already stated, weight is important for floating structures; weight is one of the most governing parameters for design.

Reducing weight may be achieved by designing efficient shapes of the structure, and/or reducing the weight of the material.

Using LWA concrete can help design considerably, in some cases even make the impossible possible.

The use of structural lightweight concrete in a permanently floating sea structure is not a novel concept. Manufactured LWA has been used in a large number of concrete ships and barges during the World War periods. More recently LWA concrete found application in two major floating platforms built for the North Sea; the Heidrun TLP and the Troll West, both installed in 1995. The Heidrun TLP was made of high performance LWA concrete with an in situ density of 1'950 kg/m^3 whereas a modified normal density concrete (MND) with a density of 2'250 kg/m^3 was utilized for Troll West. The use of LWA concrete in these platforms was the result of a Norwegian research campaign aimed to develop LWA concretes for floaters with typical densities in the range of 1'850-1'950 kg/ m^3. Significant research was also performed on lighter, high performance artificial light weight aggregate (ALWA) concretes with densities in the range of 1'450-1'550 kg/m^3. These studies made a valuable contribution to the implementation of production and design provisions for LWA concrete into Norwegian concrete standards, applicable for high-performance concretes with LWA in both coarse and fine fractions.

Some other offshore concrete platforms have also taken the benefit of LWA concrete, particularly when in high position, beneficially for hydrostatic stability reasons, and when draft needs to be limited.

The ability to choose a suitable concrete with suitable properties, including weight, is a powerful tool when designing floating concrete structures.

A good example of an efficient use of LWA concrete is a floater developed for a floating heavy lift vessel in the mid to late 2000's. The vessel was partly constructed, but never completed, due to the 2008 financial crisis.

The MPU Heavy Lifter was a multi-purpose buoyancy lifter for use in offshore platform installations and removals. The concrete vessel was designed to lift entire topsides or jackets in one piece by the Archimedes' principle. For optimum lightship weight versus displacement, the hull was made of high performance LWA concrete with an (unreinforced) in situ density less than 1'600 kg/m^3 based on lightweight aggregates in all fractions.

This chapter was mainly authored by Tor Ole Olsen.

doi.org/10.35789/fib.BULL.0091.Ch08

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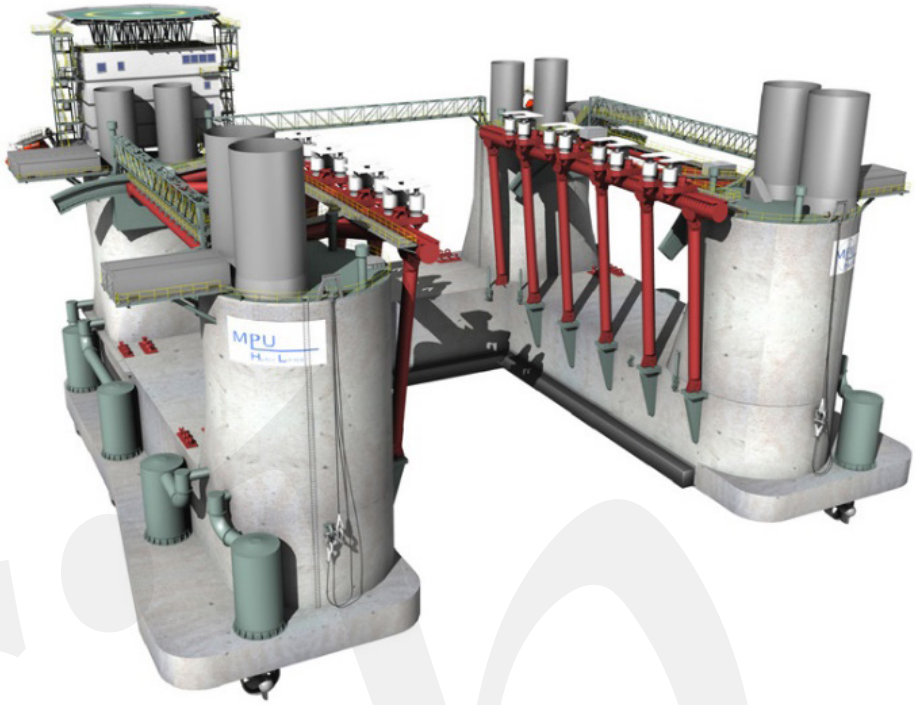


Figure 8-1 MPU Heavy Lifter (3D Rendering)[35]

Due to limited previous real-life experience, a comprehensive laboratory test program to investigate and document the concrete properties and structural performance was conducted. The concrete mix design was developed through a series of trial mixes, both in the laboratory and at full scale, in order to meet the workability and performance criteria relevant for marine structures. The structural design was carried out in accordance with the Norwegian design code NS 3473[36]. In the absence of relevant test evidence, particular design details were also investigated experimentally.

The extensive work on this concrete is described in reference[37].

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9. Fibre reinforced concrete

9.1 General

Reinforcement is a significant cost element in concrete structures, and rebars influence the time it takes to build a structure. Replacing rebars by fibres mixed into the concrete therefore makes sense.

Fibre reinforced concrete is a well-known material that has been used for a large number of structural applications since 1930. However, a lack of generally accepted design rules and standards limits the application of the material today. Since the year 2000 there has, however, been good progress in the development of standards and guidelines for design and execution of fibre reinforced concrete. *fib* Model Code 2010[38] now includes particular sections addressing fibre reinforced concrete, the standardization organization in Germany has adapted the DafStb standard "Steel fibre reinforced concrete" to the Eurocode system, and in the Scandinavian countries Denmark, Sweden and Norway, and also in Italy, either national guidelines or standards have recently been published.

The State-of-the-art is well established through international conferences as FRC: The modern Landscape[39] and FRC 2014 Joint ACI-*fib* International Workshop[40], and the *fib*-bulletin, being written as background document for the fibre concrete chapters in *fib* Model Code 2010 (MC2010), currently under preparation by *fib* – TG4.1 Fibre reinforced concrete.

Because the safety-issues are a national responsibility, and the concrete industry is still mainly used to national guidelines and standards, the practice and opportunity for solutions with fibres as the only reinforcement varies between countries. Concerning the work with the revision of Eurocode 2[41], originally planned for launching in 2020, there will be an Annex related to steel fibre reinforcement that will help to harmonize the European use of steel fibres in load carrying structures.

Fibres for structural applications are most commonly steel with lengths in the range 10 to 60 mm and aspect ratio (length/diameter) between 50 and 80. Polymer fibres (frequently denoted synthetic macrofibres) with similar dimensions are also commonly used, but have a more narrow range of application mainly due to their low strength and stiffness, and uncertain time dependent behaviour. Other special types of fibres also exist in the market, but the application of them is still limited.

Fibres may be used in all concrete strength classes ranging from ordinary low strength concrete for structures on grade to UHPC for special applications, and can improve the performance in several ways. Fibres may be used in ordinary vibrated concrete, but also in self-compacting concrete, and in extruded concrete. The fibres' contribution to strength, stiffness and crack performance is, however, largely dependent on the concrete composition, the consistency and the execution (casting process), and it is therefore important that the testing and dimensioning approaches are performance based as for instance in *fib* Model Code 2010[38]. The fibres' contribution to enhanced performance, can never be expressed by the fibres alone, it is always the combination of fibres and concrete that must be characterized.

This chapter was mainly authored by Terje Kanstad.

doi.org/10.35789/fib.BULL.0091.Ch09

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9.2 Characteristics of materials and structural behaviour

The main parameter used to characterize the contribution from fibres in structural design is the the residual tensile strength, i.e. the post-cracking tensile strength. Most guidelines and standards recommend determining this parameter indirectly by three or four point bending tests for practical reasons. The standards usually refer to design values determined at crack-widths in the range 0.5 to 3.5 mm, which are assumed to represent the structural properties in deformation stages corresponding to SLS and ULS design of ordinary reinforced concrete.

In Service Limit State design, it is well known that fibre reinforcement contributes to closer crack spacing and smaller crack widths, and therefore also considerably increased stiffness and tightness. Durability of fibre reinforced concrete is a complex topic due to the various materials used the fibres, the concrete qualities and the different degradation mechanisms that may occur. It is therefore difficult to express these properties in short and simple statements, but it should be noted that it is quite common to use fibre reinforcement as a tool to improve durability and thereby increase the service life of concrete structures.

Concerning structures exposed to bending in the Ultimate Limit States (ULS), it is important to note that sections with fibre reinforcement alone do not have the same large ductility as ordinary under-reinforced cross sections. However, there are many types of structures that have large redundancy (statically indeterminate), and structural systems where this is not decisive, so that fibres may fully replace the ordinary reinforcement also in cases exposed to bending. Combined with ordinary reinforcement (hybrid solutions) to secure robustness and ductility, fibres work very well to increase the moment capacity.

The shear capacity is enhanced by steel fibre reinforcement. This is well-documented both in research and practice, and it is generally accepted in concrete technology that ordinary shear reinforcement may be fully replaced by steel fibres. This statement holds for beams, slabs, punching shear, and also for special cases with web-openings, and for prestressed concrete. However it is important to note that this is not the case for other fibre materials which do not have the same large E-modulus as steel.

9.3 Structural applications

As stated above, fibres may fully, or partially, replace ordinary reinforcement bars and prestresing steels in concrete structures.

The most common structures where the ordinary reinforcing bars are fully replaced by fibres are rock tunnels with shotcrete, prefabricated tunnel linings, slabs on grade (industrial floors), wall and column foundations, cellar walls, and internal non-load bearing walls. Presently, these structures represent the major part of the market for the fibre concrete industry.

Fibres combined with ordinary reinforcing bars are already being used, and have a large potential for increased use in most types of concrete structures walls, pile-supported slabs, pile caps, elevated slabs and beams, abutment walls and other secondary structural members in bridges.

Implementing the effect of fibre-reinforcement in conventional design approaches is not straightforward, as tensile stresses in concrete are not included. The MCFT (Modified

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Compression Field Theory) on the other hand, is perfectly suitable for analysis of fibre-reinforced structures. The effect can be directly modelled by replacing the tensile stress-strain curve of the concrete with the corresponding curve for fibre-reinforced concrete. Such curves can be easily obtained from bending tests on smaller specimens. As fibre-reinforced concrete is capable of transferring tensile stresses across cracks, no crack check is necessary even if conventional reinforcement is present.

This design approach was utilized recently in the design of a fibre-reinforced closed floating concrete fish cage (see chapter 13). The outer diameter of the tank is approximately 16.5 m, the total height is 7.7 m, and it contains 1'092 m^3 of water during production, see also the section on fish farming in this report. Note the wall thickness of 100 mm. The tank was modelled with solid elements to properly capture the stiffness in structural intersections. Obtaining an efficient structure that satisfied the specified floating capabilities required minimization of the concrete weight. As a consequence of concrete cover requirements, the use of conventional reinforcement would lead to an increase of the concrete thickness. Limitation of the structural weight was therefore obtained by employing steel fibres, replacing conventional reinforcement in the main parts of the structure, with ordinary and prestressed reinforcement only used where required. The properties of the fibre-reinforced concrete were documented by tests of smaller specimens. Construction of the tank and deployment on site happened in June 2016. Figure 9-1 shows the finished tank and the analysis model.

Replacing conventional bar reinforcement with fibre-reinforcement can yield efficient construction of traditional structures, and enables the construction of new types of structures with very efficient concrete volumes.

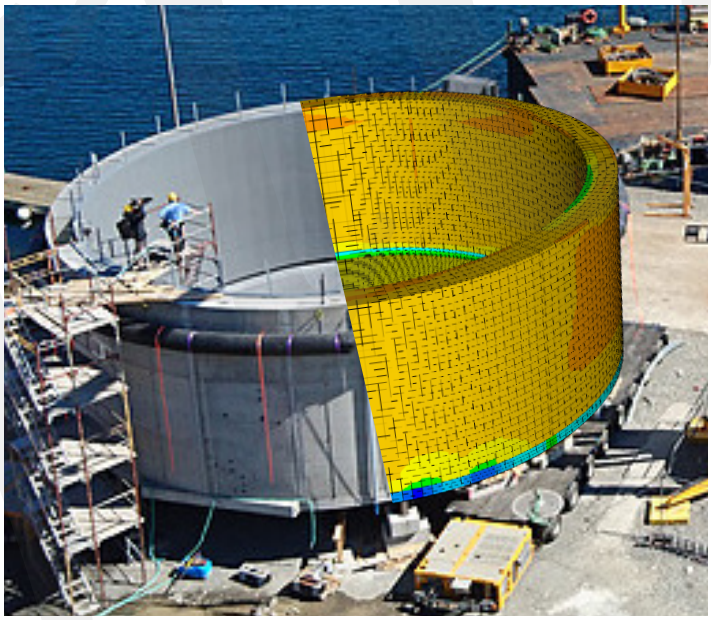


Figure 9-1 Finalized fish tank and analysis model (Fishfarming Innovation)[42]

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10. Floating wind generators

10.1 Introduction

Clean energy is in strong demand worldwide and the market for bottom fixed wind turbines is growing quickly in areas with suitable water depths and seabed conditions. However, the majority of offshore areas do not have the suitable shallow waters close to shore. For these areas floating wind turbines will be a preferred option.

The first floating wind generator to be installed was Statoil's Hywind, outside Karmøy, Norway. The second was WindFloat outside Agucadoura, Portugal. Both of these are steel structures.

The VolturnUS is another concept, by the University of Maine, similarly Nautulus is under development. DCNSNaval is developing a concept.

Information about these concepts is available on the Internet. All in all, floating wind generators have a future.

Larger turbines call for larger and more robust structures, and the concrete option is obvious.

The Ideol floating concrete box structure, to support a wind generator, is under construction in France.

A robust semi-submersible floating substructure, which can support large wind turbines in harsh environments, is shown in Figure 10-1. Both a concrete floater and a hybrid concrete/steel floater are shown.

The focus has been to develop a floater with a robust and clean design and which simplifies construction and installation as well, based on existing experience with floating concrete structures. The latter includes inshore completion and tow of the complete floating Wind Turbine Generator (WTG). The floater may be built in steel or concrete, or a combination of the two materials (hybrid). Steel is considered to be cost effective for small and medium size wind turbines (2-5 MW) due to light weight and easy material handling. Concrete is considered to be cost efficient for large wind turbines, typically 5 MW and larger (depending on fabrication facilities etc.). This is due to the favorable robustness, fatigue characteristics and design life for concrete. Another interesting alternative is a hybrid with concrete in the pontoons and central tower, and steel bucket cells, Figure 10-1.

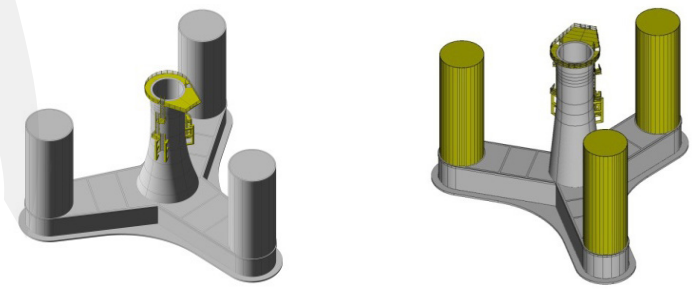


Figure 10-1 Floating hulls for wind generators, concrete and hybrid concrete/steel[43]

This chapter was mainly authored by Coen van der Vliet.

doi.org/10.35789/fib.BULL.0091.Ch10

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Development of large wind turbines is seen as one of the most effective ways to make offshore wind economic in the future, and hence using concrete as a building material seems to have the largest potential.

Compared to the oil & gas industry with reserves of limited size it makes sense to utilize concrete material with a long design life for development of offshore wind. Very long design life - 100 years or more – may be achieved for a concrete substructure and no regular maintenance is required. This opens up the possibility of reuse of the structures through repowering and adds significant value to the project since wind power will last indefinitely.

10.2 Concept description

The concept shown in Figure 10-1 is described in some detail. The floater is a column stabilized floating substructure with a central shaft, three corner columns and a star shaped 3-armed pontoon. As mentioned above, similar designs are under development.

The wind floater has been designed for large offshore wind turbines. The shape and structure has been developed with the characteristics of concrete in mind. Such characteristics are typically:

- Concrete is very stiff and robust with respect to environmental loads and impact loads
- Concrete is very suitable for upscaling in size
- Concrete is not very sensitive to fatigue
- Concrete can be designed for a long service life, 100 years typically
- Concrete is heavy and the design must allow for sufficient buoyancy in the lower structure to achieve a small minimum draft which is key for inshore completion of the WTG unit.

The philosophy behind the design has been to provide a floater that can be completed in a dry dock, on a barge or on land, and be set afloat inshore at a quayside with limited water depth. It has further been designed to allow for complete pre-assembly of the floating wind turbine, including tower and hub, while moored to the quayside, and allow for towing in a stable position during transit. The focus has been on simplicity and robustness in operation. One feature is that the pontoons will be flooded during installation and connected to the sea. This means that the pontoons during operation will have zero hydrostatic pressure and there will be no ballast system for maintaining the level of ballast in the pontoons. During temporary phases the pontoons will be empty and will contribute to optimal stability and small draft during assembly and transport. During operation they will be water filled and contribute to optimal motion behaviour by supporting the corner columns and provide added mass and a low center of gravity. The WTG is supported on the central shaft which is dry during all phases. The corner columns are used for trimming the unit. The base case will be a passive ballast system with temporary pumps, i.e. a possibility to add or reduce water from the corner columns during installation or removal, but not on a regular basis. There is also a

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possibility to have an active system if found beneficial for operation purposes.

Unless self-propelled, floating structures need to be moored. The base case for the mooring system for this structure will be a three line catenary chain (passive) system with equal spacing (120 deg.) and mooring attachments placed on the corner columns. The elevation will be at the top of the columns for simple installation and operation.

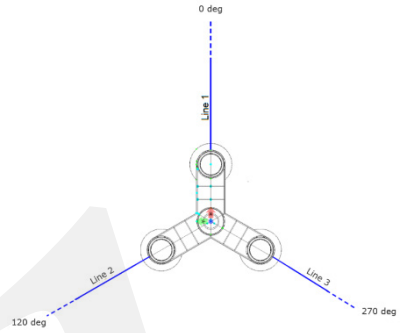


Figure 10-2 Wind Floater Mooring - Top View[44]

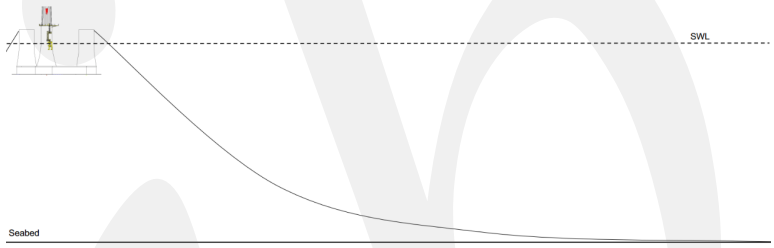


Figure 10-3 Wind Floater Mooring - Side View

The structure needs to be accessed. Access to the floater will typically provide for one boat landing placed on the centre shaft.

10.3 Fabrication

The floater can be constructed locally worldwide, with a limited infrastructure in place. Proper integration of design and construction is strongly recommended. Also recommended are strict weight control and good construction planning.

Fabrication of the hull structure can be carried out on floating barges, in a dry dock or on a quay. For low cost mass production, an efficient fabrication yard on land will be established. Mass production is expected to reduce the cost per unit significantly compared to one-off projects. This will raise challenges for the construction industry that does not routinely operate in this way, being more attuned to one off construction.

The floater allows for inshore completion, including tower, generator and rotor. The complete unit can float with a small draft and no deep water is required for fabrication or transit. The WTG tower, nacelle and rotor will be lift installed at the quayside. In this way the use of expensive offshore cranes for installation of WTG is eliminated, reducing time, cost and risk although the scale of the onshore assembly challenge should not be underestimated. The complete floater is towed to site and connected to pre-set mooring lines. The floater provides a robust and stable base for the wind turbine, preventing unfavourable motion and fatigue problems. The concept is robust with regard to turbine

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size, and the elevation of the interface between a concrete substructure and steel turbine tower is flexible in order to improve fatigue life for the steel tower.

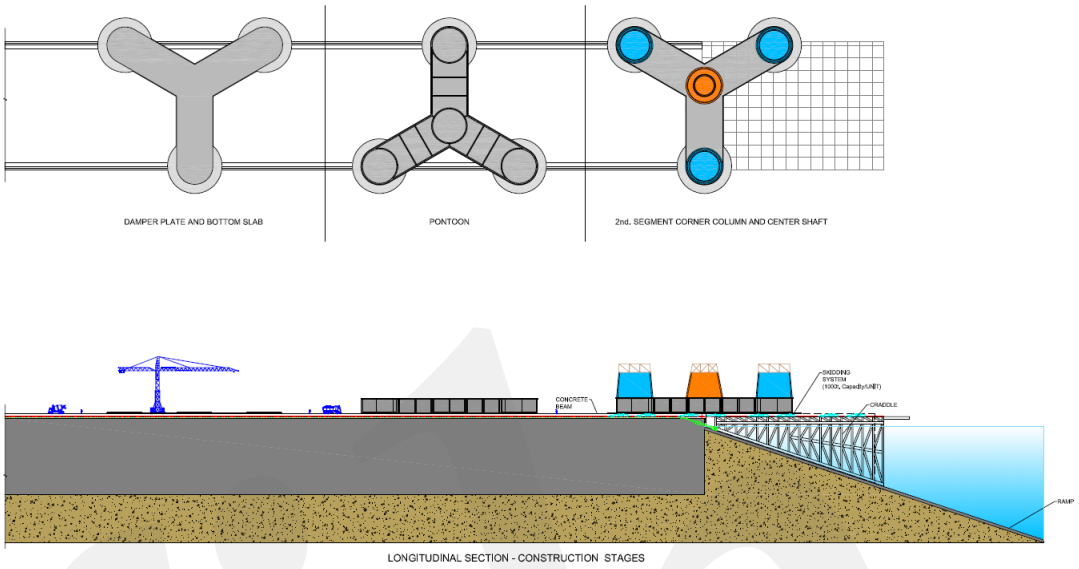


Figure 10-2 Wind Floater Mooring - Top View [44]

10.4 Transformer station

A transformer station is one of the objects that makes a wind farm, together with the generators (see previous sections of this chapter). It may be an economic solution to build these transformer stations on a concrete subbase as a self-installing platform. This makes them easy to transport and enables repositioning or re-use when necessary.

The general concept is shown in Figure 10-5. The subbase serves both as floater and ballast tank. During transportation to the installation site, the platform is in its lowest position (1). At the installation site, the tanks are gradually filled, using the buoyancy of the platform to guarantee stability (2). Once fully ballasted with water, the platform lifts itself from the sea level (3). Finally, the water ballast is

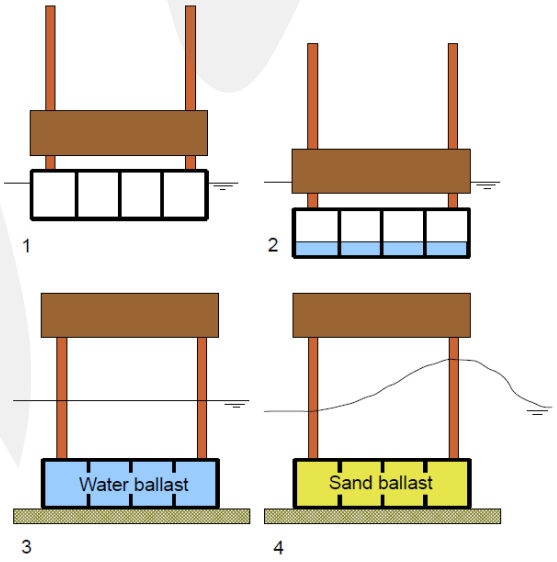


Figure 10-5: General concept [45]

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A feasibility design has been performed by the company Arcadis, that focused on the stability (both during transport and with respect to ultimate limit state wave actions in the final situation) and on the constructability of the concrete structure. The feasibility study has been performed for a situation characterized by:

- Maximum wave height H_{max} = 18.6 m
- Wave period T_{min} = 11 sec
- Seabed 30 m below Lowest Astronomical Tide (LAT)
- Current speed $v_{current}$ = 1.3 m/s
- Subbase plan dimensions: 40×40 m^2.

Due to the shallow depth and the relatively large maximum wave height, stream function wave theory had to be used. For the feasibility study, the wave load was simulated by an equivalent hydrostatic load. For further design steps, CFD, diffraction analysis or model tests should be performed for a more accurate prediction of hydrodynamic loads.

The wave and current loads have been translated into drag forces and wave pressures. The wave pressure on the subbase is less than the hydrostatic wave height due to attenuation of the pressure over depth. A hydrostatic pressure of 70% of the wave height has been used for this feasibility study.

The feasibility mainly depends on the stability of the structure in waves. For this situation, the mechanisms of sliding and overturning and the bearing capacity of the subsoil have been considered. The calculations have been performed with a 2D geotechnical FEA model. Neglecting the 3rd direction leads to an underestimation of the overturning stability and bearing capacity of the subsoil, as the load distribution 'out of plane' is omitted from the equilibrium.

Wave loads act both on the structure and the adjacent seabed. The duration of the wave load is such that it can be considered a quasi-static load. In the Plaxis model this is incorporated by introducing pressure loads on the structure and the subsoil according to the shape of the wave. Drag forces on the subbase are introduced as separate loads acting on the side of the structure. The superstructure is modelled with a fictitious beam in the centre of the structure. Loads from the superstructure are acting on this fictitious beam. The loads are short term and will cause excess water pressure in the soil. The soil model that is used allows for undrained behaviour. The subbase is modelled as a 40 meter wide rigid block with the volume weight of the concrete structure filled with sand ballast. On the bottom of the subbase an interface was modelled with maximum friction angle of $2/3\phi$. The effect of the passing wave is modelled by putting the wave in several positions along the structure. Different positions will lead to different failure modes. For instance a wave peak next to the platform will cause sliding but no overturning, whereas a position of the wave peak more outside the platform will cause overturning but less sliding. The design calculations showed that the assumed dimensions resulted in a stable design.

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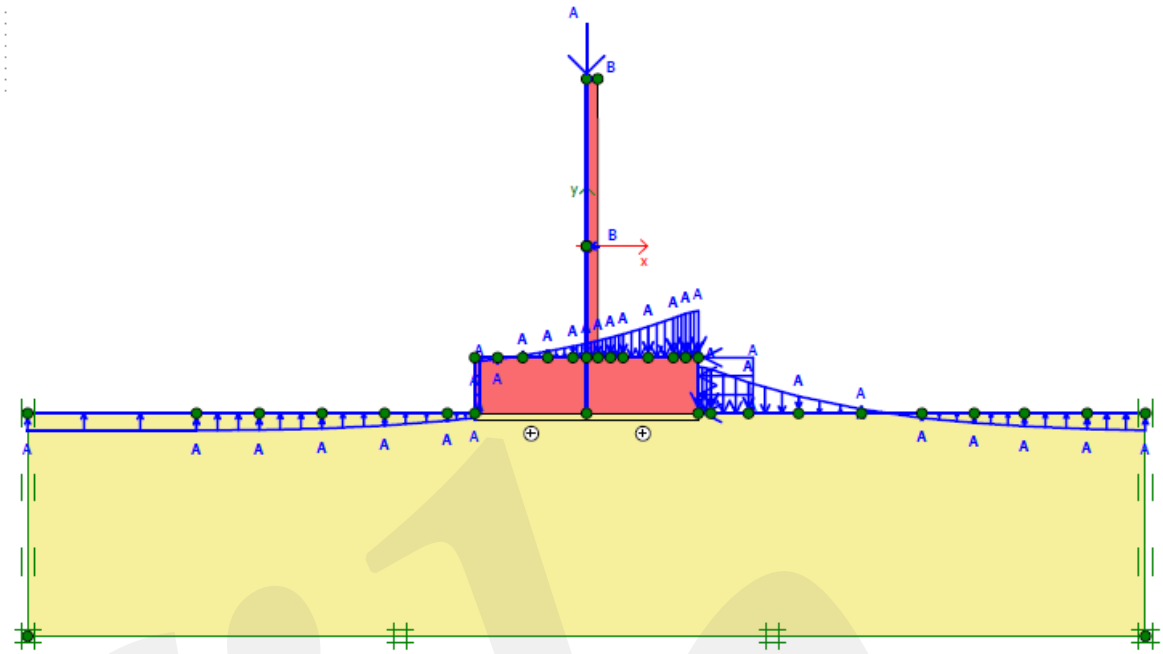


Figure 10-6: FEA model of subbase and wave load (travelling from the right)[45]

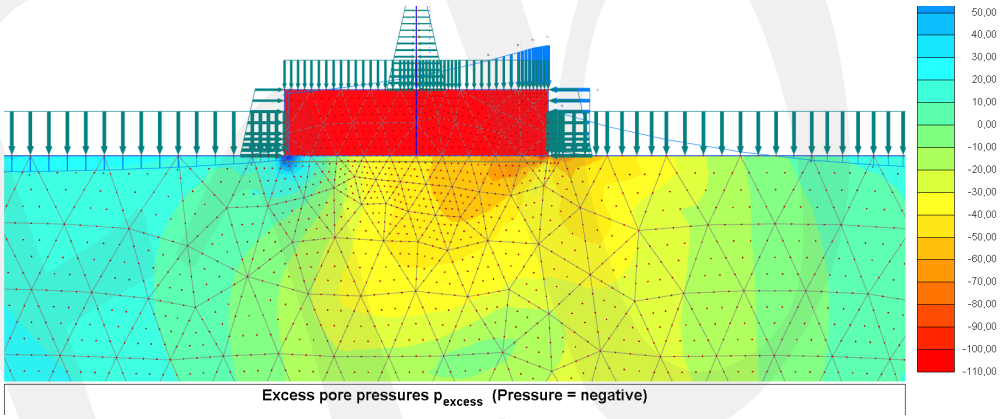


Figure 10-7: Excess pore pressures related to overturning[45]

The hydrodynamic stability during transport and installation has been verified mainly based on metacentre height. Stability in this sense has been regarded as the degree to which an object wants to return to its initial state when perturbed. The stability of an object is determined by the distance between the centre of gravity and the metacentre of the object: the metacentric height. This parameter is mainly dependent on the width of the floating body (rectangular cross-section). From this it follows that the stability of the object is determined by its centre of gravity (objects with a lower centre of gravity have more weight stability) and its width (wider objects possess more shape stability).

Typical requirements, as formulated for transport of concrete tunnel elements overseas by Det Norske Veritas are:

- The Metacentre height N must be at least one meter above the centre of buoyancy, B.
- Up to an angle of 30° there must be a stabilizing moment.

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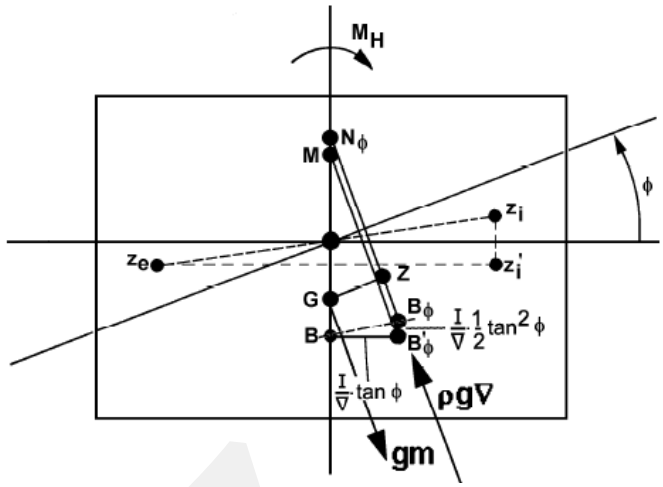


Figure 10-8: Metacentre M shifts to N due to a roll angle ϕ [45]

For the concrete subbase alone, the metacentric height was 19.5 m. After installing the complete hull and topside the draft increases and the metacentric height reduces to 9 m, still sufficient by a large margin.

A FEA model of the concrete structure was made, using shell elements for the walls, floor and roof slabs and beams for the legs of the platform. Typical dimensions were:

- Floor slab thickness: 1 m
- Roof slab thickness: 0.6 m
- Outer wall thickness: 0.6 m
- Inner wall thickness: 0.4 m.

Both installation phase and final position have been considered. For the section forces in the roof and walls of the subbase, the installation phase is governing. The final position is governing for the stresses around the connection of the legs.

The calculations show that the subbase can be constructed with realistic amounts of reinforcement. In the outside corner areas, prestressing might be necessary to reduce the risk of cracking.

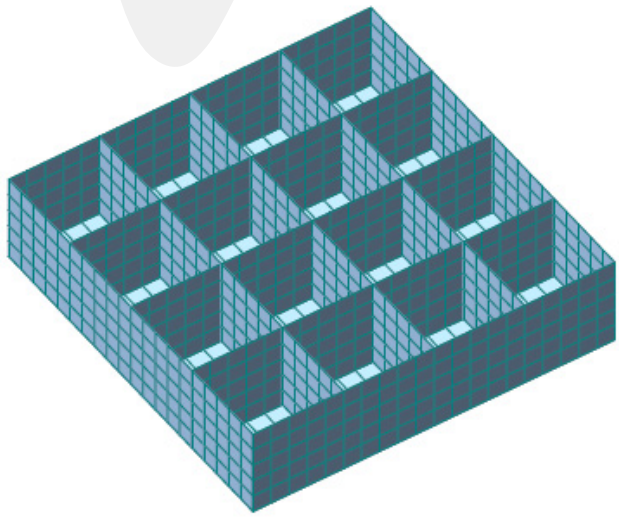


Figure 10-9: Subbase part or the FEA model[45]

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11. Sea beach

As initially stated the sea represents an increasingly attractive place to build for human activities. The Sea beach, Sea bath, in Oslo is a very good example of this. Even in low population Norway, with an enormous length of coast, the Sea bath is installed solely for recreational purposes.

How can this pay off? There are non-quantifiable aspects, of course, but three elements should be mentioned: the Sea bath was a gift from the Oslo Harbour Authorities to the developer of the nearby apartments, and the developer pays no rent. Additionally the Sea bath increased the number of apartments permitted to be developed. Additionally, the Sea bath turned out so popular that the price of the apartments increased.

The Sea bath is developed to be an innovative and efficient floater concept. As a basis, technology from offshore concrete structures is used, with an active slim down approach. The concrete hull has Styrofoam inside to replace water and ensure buoyancy, and this Styrofoam is the formwork for the internal walls.

Required design life is 100 years.

Use of the non-linear design program was instrumental in achieving slim walls with only one layer of reinforcement.

Strict weight and tolerance control has required careful construction procedures, including handling of the Styrofoam.

The design of the mooring and access was an important part of the design. Few and simple parts were required, that do not need maintenance.

Although Oslo is relatively protected, hydrodynamic analyses and design were performed, including waves, current, wind and impacts.

Some details from the design are included, and also some pictures. Most of the innovations are no longer visible.

The park consists of three concrete floater elements with a top deck thickness of 300 mm and underlying, centrally reinforced walls with a general thickness of 150 mm, constituting a total surface area of about 5'000 m^2. The three elements are linked and anchored by two land-connected steel bridges combined with a single steel pile driven into the seabed. Figure 11-1 shows the three elements after construction in the dry dock, and in their final state at site. The structural composition of the floating park is illustrated in Figure 11-2.

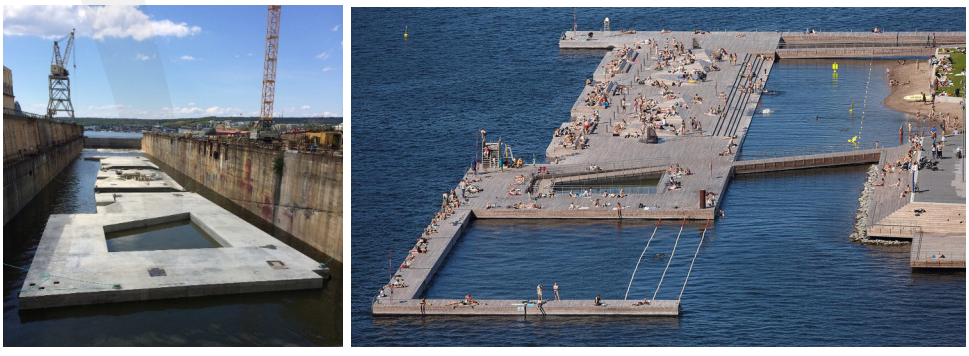


Figure 11-1 The floating park in dock and installed[46]

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doi.org/10.35789/fib.BULL.0091.Ch11

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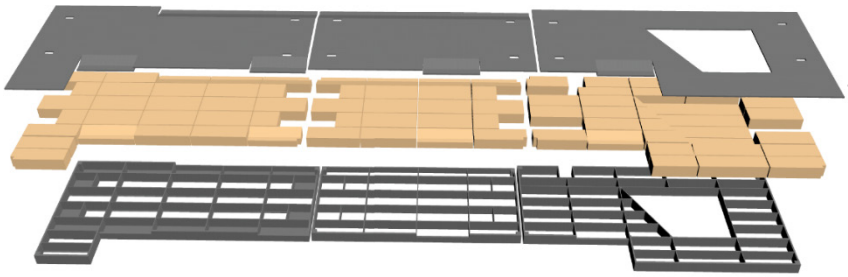


Figure 11-2 Structural configuration of the floating park (exploded view)

Self-weight, combined with buoyancy from centric Styrofoam (XPS) elements and peripheral ballasting, was one of the dominating loads in the design of the floating park. Figure 11-3 shows a part of the central concrete floater, and its cross section.

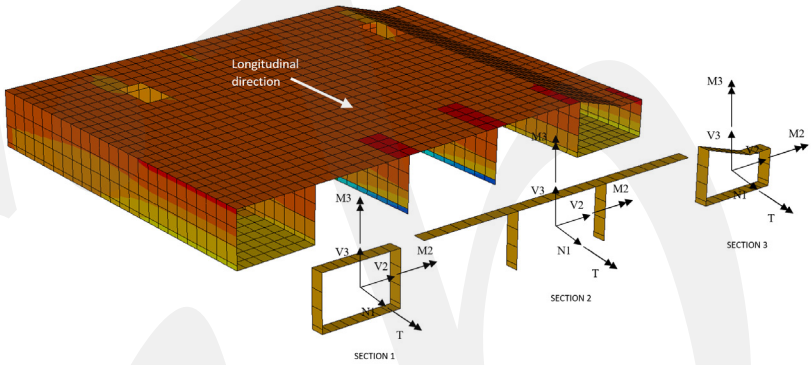


Figure 11-3 Beam section forces in the floating park[47]

The optimization of the floating park design, by using nonlinear analysis, helped create a significantly efficient design, limiting the weight of the structure and the construction cost. Without the use of nonlinear design the weight increase from additional reinforcement would lead to comprehensive conceptual changes in the design and most likely also significant additional costs for the client.

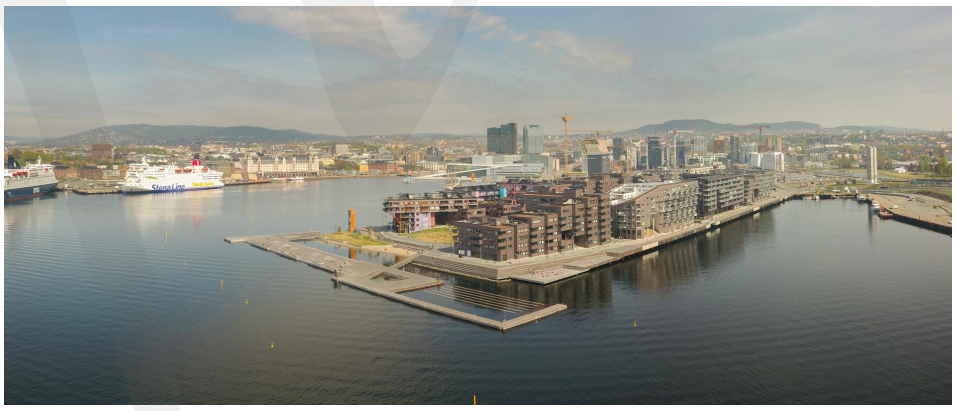


Figure 11-5 The Sea beach[48]

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12. Reinforced concrete floating caissons for ports and harbours

12.1 Function

Reinforced concrete floating caissons are prefabricated elements used for port and harbour construction that are afloat only during the construction stage, then being placed on the sea bottom for service. Buoyancy is essential for the construction process, this being key for the advantages of the concept.

Caissons are used for gravity maritime structures such as vertical breakwaters, quay walls, dolphins and other special structures. The use of floating concrete caissons for vertical breakwaters is a cost-effective solution that reduces the vulnerability of the structure to wave actions during construction when compared with other alternatives for vertical breakwaters such as concrete blocks, sheet pile cofferdams or underwater concrete, and also compared with rubble mound breakwaters. When placement tolerances are tight enough the breakwater can provide a berthing line in the harbour side that helps in obtaining economic return from the investment.

Caisson quay walls form monolithic retaining walls that are cost competitive for depths of around 10 m or more. As operational loads are transferred indirectly by earth pressure, the structure can easily be adapted to changes in the operational requirements (storage loads, cargo handling equipment, etc.).

Other uses of reinforced concrete floating caissons are dolphins and other gravity elements such as piles for bascule bridges, sometimes containing the mechanical elements and other operational plant.

Caissons for port structures have a long track record of successful applications, as is demonstrated in literature. The following sections describe what is thought essential for one way of construction.

12.2 Concrete structure

12.2.1 Description

The usual shape of Reinforced Concrete Caissons is four sided with parallel walls, although other layout variations are sometimes constructed. Usual dimensions are:

- Length: usually more than 20 m and up to more than 60 m. The length is limited by the manufacturing plant or the uniformity of the foundation.
- Beam or width, defined by stability conditions both in flotation (naval stability) and in service (geotechnical stability).
- Height, determined by water draft (defined either by natural water draft or operational constraints), plus tidal range, plus the freeboard needed either for operational or

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doi.org/10.35789/fib.BULL.0091.Ch12

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construction reasons. The height is limited by allowable foundation stress; a practical limit is reached at 29-32 m.

The caisson is composed of a base slab that transfers loads to the foundation and a shaft with hollow cells that provide buoyancy. The plan shape of the cells is usually square or rectangular (side 3-5 m), being sometimes circular (2-4 m diameter). The width of the bottom slab (usually 0.50 to 1.20 m thick) can be the same as the shaft or it can be increased by cantilevers in order to reduce the peak pressure on the foundation mound.

The caissons are placed by sinking to the foundation by filling the cells with sea water. The foundation can be: levelled sea bottom (sandy) or rubble mound with a graded gravel layer on top. The rubble mound is placed on a soil layer with sufficient carrying capacity. This can sometimes require dredging a trench and substituting natural soil by quarry stone (rubble mound). Direct foundation on rocky bottoms is not a standard solution due to the fact that the high stiffness of the caissons requires a foundation with a relatively uniform subgrade modulus. Concentrated reactions (due for instance to rock peaks) can cause punching of the base slab.

12.2.2 Construction

12.2.2.1 Caisson manufacturing

Main alternatives for standard reinforced concrete floating caisson casting are:

- Casting in slipways or dry docks. These are fixed facilities that allow for dry construction. Once finished the caissons are launched or put afloat and towed away to the site. Distance from the casting yard to the construction site can be relevant.

- Casting on submersible guided pontoon or synchro lift: these are semi-fixed facilities that require on site works but can be easily dismantled after finishing the caissons. Usually sliding moulds are used for casting.

12.2.2.2 Casting on floating dry-docks.

Floating dry docks are provided with a set of ballast tanks placed on the base pontoon and corner towers. Controlled flooding of the tanks through seawater intake brings naval stability and safety for the dock, the caisson under construction, moulds and scaffolding for all stages of the manoeuvre. The floating dry-dock can be transported from one site to another by towing and does not require on site civil works. The only requirements are: a gangway for personnel access, connection to a set of concrete pumps (one or two), being under the reach of the boom of a tower crane for reinforcement supply, and electric power connection.

Caisson casting on a floating dry-dock is carried out in the following stages:

- With the base pontoon fully out of the water, the deck is cleaned and the preparation for the caisson structure construction is carried out.

- Assembly of the caisson base moulds and placement of reinforcement of the base slab and wall fixing

- Pouring of the base slab

- Placement of the sliding mounds on top of the base.

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Sliding of the caisson shaft on a continuous and uninterrupted basis, 24 hours per day.

Simultaneous staged ballasting of the floating dock is achieved by balanced flooding of the ballast tanks placed in the pontoon and towers. Ballast allows control of naval stability of the dock-caisson assembly and keeping the support force between the caisson and the pontoon within acceptable limits. This simultaneous process allows the maximum capacity of the floating dock to be obtained.

Once shaft sliding is finished, the sliding mould is removed and the caisson is ballasted with sea water in order to ensure its naval stability after launching.

The dock is slowly ballasted until the caisson rises off the deck and floats alone. Then the caisson can be towed to where it is needed.

12.2.2.3 Caisson transportation

Once the caisson is afloat it will be towed to a berth or a sheltered area where it will be prepared either for site placement or for long-distance transportation or until concrete reaches the required maturity if towing forces would be relevant.

If the caisson is going to be placed near the casting site, the caisson is directly prepared for controlled placement by installing the filling and emptying system (pipe works, valves, pumps, etc.) and the geometric control system (capstans, GPS transceivers, control units, etc.).

When the caisson needs to be towed relatively far from the casting site, it needs to be equipped for offshore navigation with a top slab in order to avoid sinking or capsizing due to uncontrolled cell flooding by overtopping, and with towing equipment (bollards…) and day and night beaconing. Another option for very long distance (for instance, intercontinental transportation) and/or unfavourable sea climate conditions, is transportation on semi-submersible vessels.

12.2.2.4 Caisson placement and filling

Before placement, foundation levelling needs to be checked. The caisson, equipped for placement, is moored to previously placed caissons and/or anchors or concrete dead weights placed on the sea bottom and filling with sea water starts. When the caisson is near the sea bottom, position is tightly controlled by the capstans that operate the mooring lines and filling is slowly continued until the caisson is placed on the foundation mound.

After checking positional tolerances, additional stability needs to be provided by filling the caisson cells. This is usually done with granular materials (sand, gravel, run-off quarry…) that allows relatively accurate weight control with moderate pressure on the caisson walls. This needs to be done as soon as possible in order to reduce the chance of displacements or damage due to wave action.

12.2.2.5 Finishing

Once adjacent caissons are placed, the joint between them is filled and made tight for the backfill material and wave energy when required. Finally, remaining elements such as the superstructure are built.

For breakwaters, the main superstructure element is the crown wall or parapet that directly faces wave forces and protects the sheltered area from overtopping. When the structure

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provides a berthing face or in the case of quay walls, a capping beam that hosts the mooring devices, fenders, crane rails, etc. are also built. These finishing elements are usually cast on site.

12.2.2.6 Specific features

The construction method has a large impact on the structural design due to the many stages of the process and the fact that the caissons are floating structures during construction and gravity structures when in service.

Structural design needs to take into consideration the following construction stages:

- Flotation
- Placement on the sea bottom
- Cell filling with granular material following the construction method
- Service loads.

For breakwaters, the main service loads are due to wave action (crest and trough) while for berthing structures (sometimes breakwaters also provide berthing for ships) service loads to be considered are: operations and storage, cargo handling equipment, berthing and mooring and other climatic loads.

Due to the aggressiveness of the marine environment, durability needs to be taken into consideration for concrete mix design, checking of crack widths in service and prevention of reinforcement corrosion. For the construction stage, closure of cracks generated by construction loads needs to be checked, keeping stresses in the reinforcement steel below the proportionality limit (approximately 70% of the yield stress).

Alternative reinforcement solutions have been analysed, and used sometimes for reinforced concrete floating caissons, such as epoxy coating, stainless steel rebars, fibre reinforced polymers, etc. The use of fibres (steel or polymer) allows a reduction of conventional reinforcement but does not eliminate it.

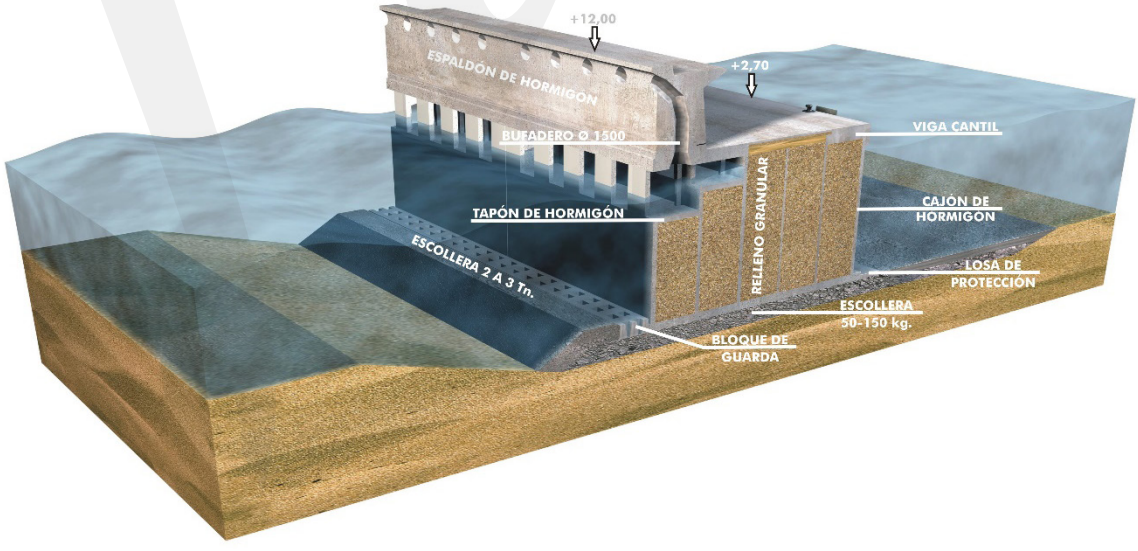


Figure 12-1 Sketch of the caisson breakwater of the Port of Castellón (Spain) with a low reflection wall integrated in the Caisson. The breakwater in service[49]

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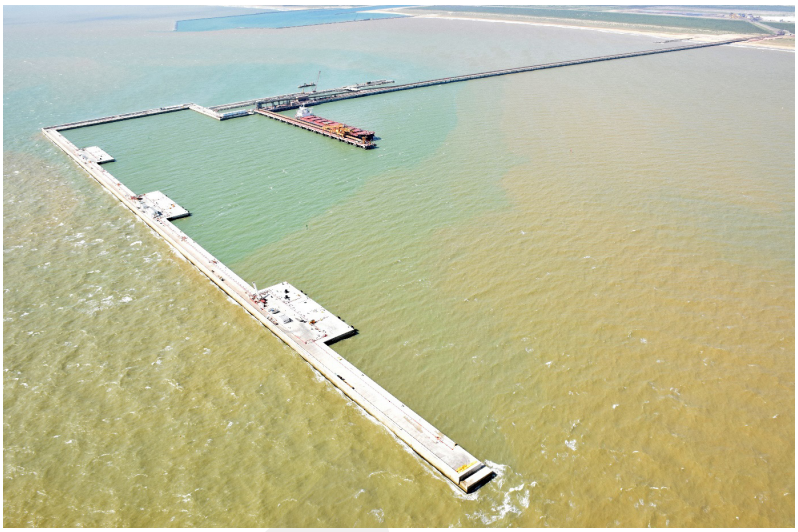


Figure 12-2 Caisson breakwater 2'020 m long in Port Açu (Brazil)[50]



Figure 12-3 Floating dock "Mar del Enol" manufacturing caissons for the Port Açu breakwater (Brazil) in the Port of Algeciras (Spain)[49]

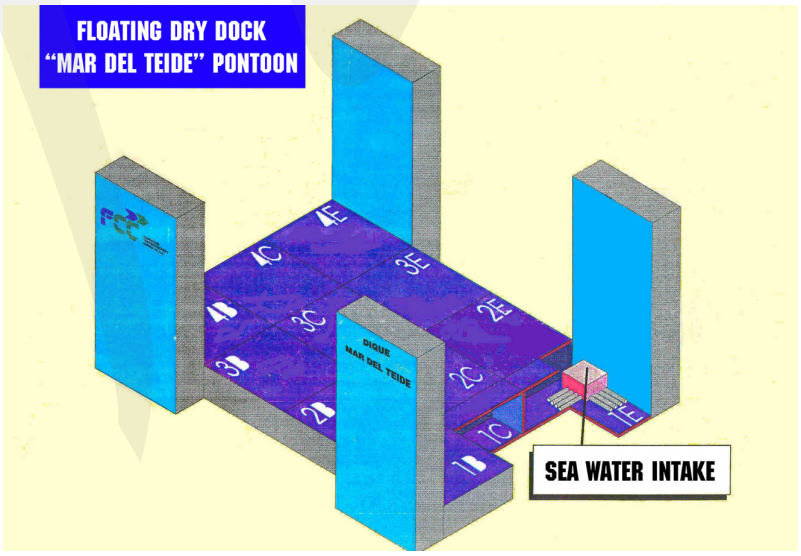


Figure 12-4 Scheme of the ballast tank distribution of the "Mar del Teide" floating dock[49]

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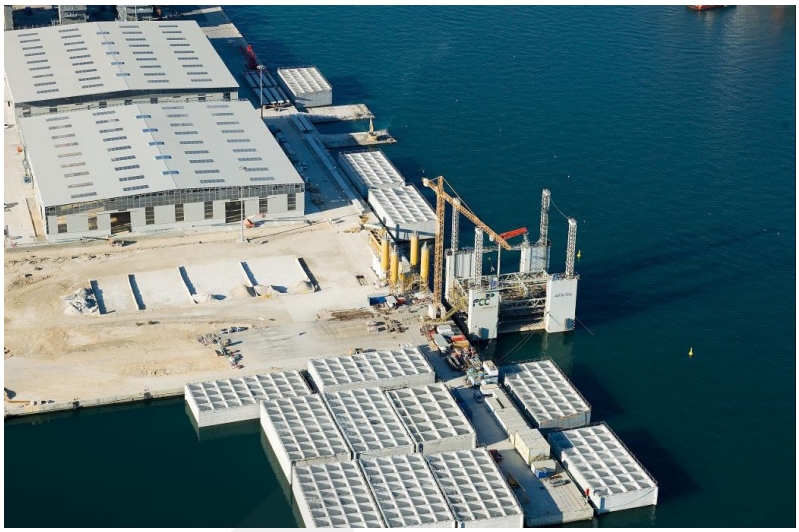


Figure 12-5 Caissons for a quay wall stockpiled afloat in the Port of Tarragona (Spain)[49]



Figure 12-6 Caisson breakwater in the Port of Granadilla (Tenerife, Spain) after caisson placement[49]



Figure 12-7 Caissons loaded on a semi-submersible boat for transportation from the Port of Algeciras (Spain) to Port Açu (Brazil)[49]

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13 Fish farming

We will here consider farming of salmon.

Traditionally fish farming is done in nets. The largest nets today are 160 m in circumference, that is, more than 2'000 m^2 in area. In Norway there are 1'000 salmon farming locations, altogether 4-5'000 nets, producing 1.3 million tonnes of salmon a year. The weight of all Norwegians is 0.3 million tonnes.

While relatively inexpensive, the nets have some serious drawbacks:

- They sometimes tear open, letting the fish escape, and mingle with wild fish
- They let lice enter the nets and reduce the value of the fish
- They let all the waste through
- Overfeeding is not healthy for the surrounding fish
- Medicine goes through
- Contagious bacteria and other particles come in.

Fish farming is the second largest export article in Norway, after Oil & Gas. Norway is blessed with pure water of nice temperature, and with protective fjords. However, the challenges listed above become serious, in particular when many farms are located in one location. The severity of the situation is such that an increase in fish farming is not possible, even though everybody wants it.

An obvious remedy to this is to make closed buckets and pump the water from greater depths, were the water is clean. Added to which the garbage can be collected for something useful.

It is difficult to beat the capital investment of a net, at least in the short perspective. But in the overall perspective it will pay with a robust and durable "bucket" where the salmon flourish in a well-controlled environment, including water current for fitness, and where the workers are comfortable.

The first concrete prototype bucket was launched on the west coast of Norway in June 2016. It is a very slender structure, 100 m^3 of concrete, housing 1'000 m^3 of water. It is anchored in a protected area, so there are few loads on the structure.

Experience with the bucket is very positive. The salmon grow faster than those in open nets, the salmon is more fit and the selling price is higher.

The following pictures show the bucket.

This chapter was mainly authored by Tor Ole Olsen.

doi.org/10.35789/fib.BULL.0091.Ch13

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Figure 13-1 The first concrete prototype bucket for fish farming, Salmon Home #1[51]



Figure 13-2 Lifting Salmon Home #1 into the water[51]

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Figure 13-3 Salmon Home #1 almost afloat[51]



Figure 13-4 Salmon Home #1 in operation, with salmon inside[51]

Fish farming may also be performed in larger structures, which may be located at more exposed sites.

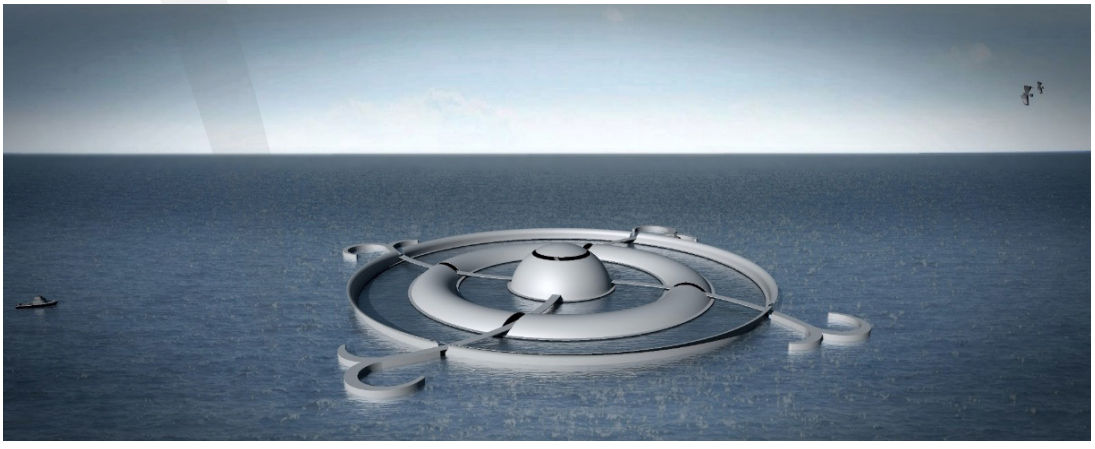


Figure 13-5 The jelly fish tank[51]

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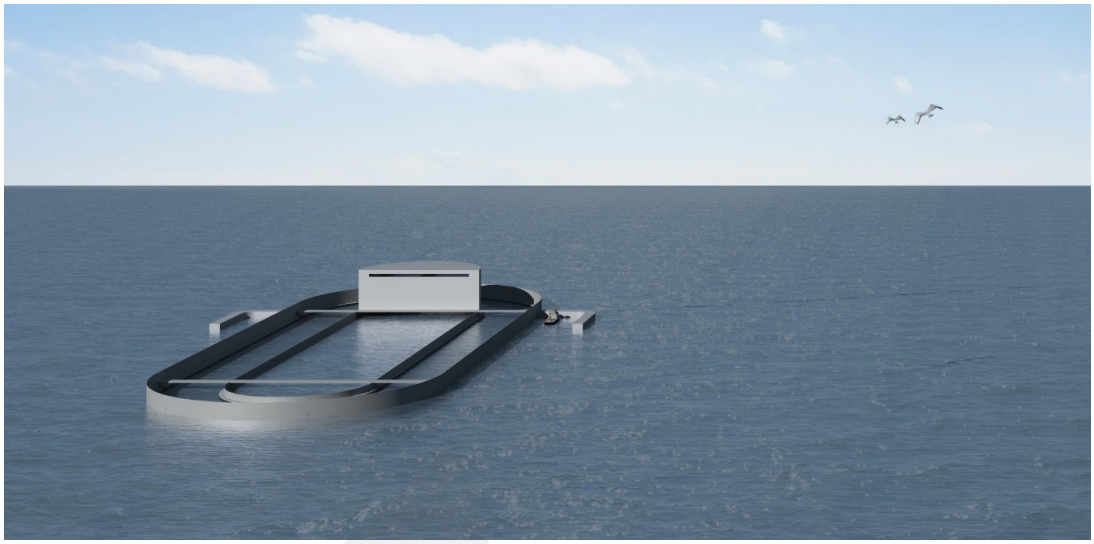


Figure 13-6 The oval compartments tank[51]

The farming industry is much more than just the containers for the fish. The fish need a lot of food, and this is typically provided through hoses from a purpose built barge. These barges are not always beautiful, reducing the impression of a nice fjord frequently visited by tourists.

To remedy this Tom Wike has sketched proposals of new thoughts for the fishing industry. The main function is a feeding platform, where the base chambers contain the food. Areas on the deck are for the technical services, working stations and living units for the employers. On the upper level is the inspection/control platform as well as a visitor centre with education facilities and a seafood restaurant right in the middle of the Norwegian "food basket".

This must be the most exotic and direct way to learn about and sell the story of an important industry.

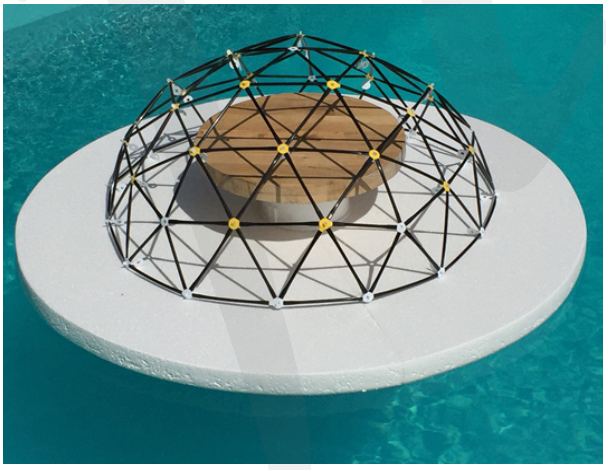


Figure 13-7 Proposed feeding barge[52]

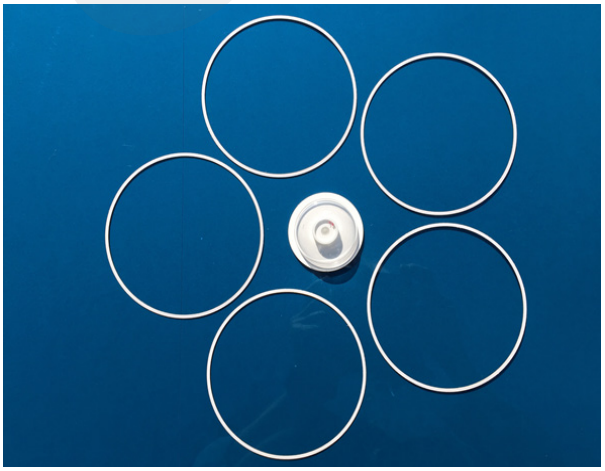


Figure 13-8 Proposed feeding barge among fish farming nets[52]

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14. Floating cruise harbour

14.1 Summary

In building a new set of jetties in relatively deep water, with objectives of better protecting the existing Condamine harbour entrance, and providing berthing and mooring facilities for large cruise boats, the Principality of Monaco completed the harbour extension process which started in the early nineties.

With regards to the 55 m water depth at the extremity of the new jetty, it had been decided to resort to unusual solutions in harbour civil works technology, by adapting well proven techniques developed for the oil and gas offshore industry.

The Principality of Monaco selected DORIS Engineering, the pioneer of concrete gravity and floating platforms in North Sea, to carry out all the studies and analyses for sizing the different works; to prepare the bid packages, and to act as the project manager on behalf of the owner during the period of the construction and installation works.

14.2 The Monaco site and harbour works

The Monaco sea shore configuration is a natural bay open to swells from the east and south east (Figure 14-1).

For more than a century, the harbour has been protected by two jetties made of dumped rock and stone blocks.

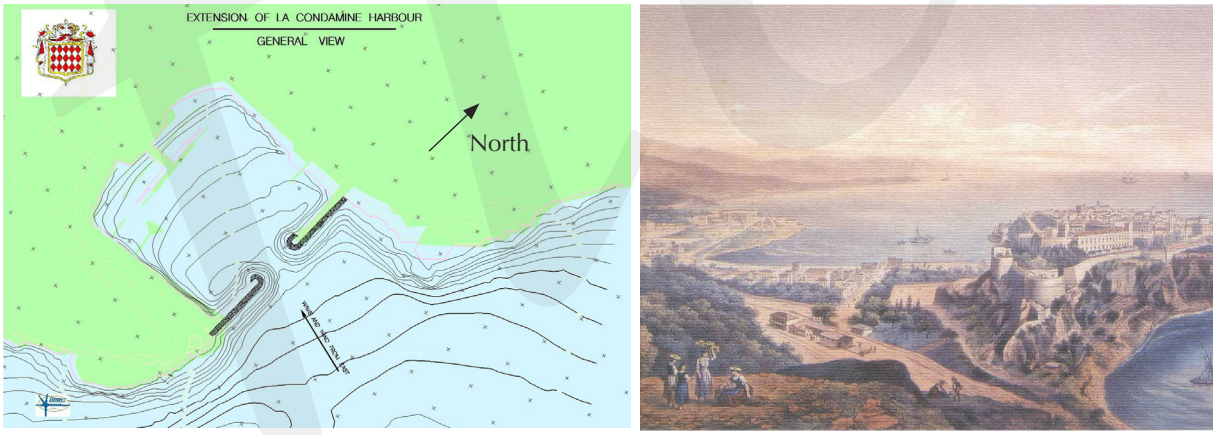


Figure 14-1 Former Monaco Harbour facilities[53]

Objectives of the Monaco harbour extension:

- Protect existing harbour
- Create extension to the existing harbour and doubling of moorings
- Allow for large cruise boat berthing, up to 200 m long & 30'000 tonnes displacement
- Provide warehouse and services area

This chapter was mainly authored by Michel Vaché.

doi.org/10.35789/fib.BULL.0091.Ch14

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- Minimise environmental impact during construction (water pollution, noise, view)
- Minimise trucks crossing and works in the Principality during construction.

The complete extension project includes three main pieces of works (Figure 14-2) which, in most cases, have been prefabricated in remote areas, floated and towed to site:

- The floating jetty: a 352 m long concrete caisson, towed in a single piece from the dry dock construction site near Gibraltar to the Monaco site.
- The embankment: made of several earth retaining concrete caissons, which provides transition between the floating jetty and the original shore. The abutment caisson (sitting in 30 m water) provides the attachment of the floating jetty to the shore, to which it is connected through a steel ball-joint which allows rotation in three axes, while preventing any displacement between the jetty and the shore.
- The counter-jetty: installed on the opposite shore to the floating jetty. It is made of a concrete caisson resting on steel and concrete supports.

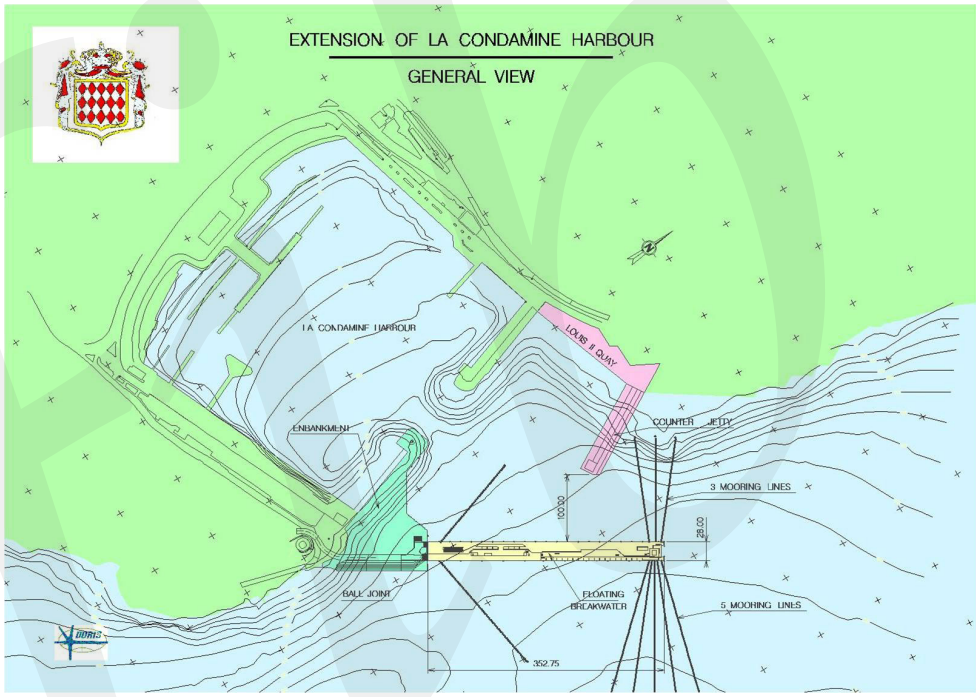


Figure 14-2 New configuration of the Monaco Harbour facilities[54]

Main design parameters and requirements:

- Water level variations (tide, surge, etc.): - 0.40 m to +1.00 m
- Waves : H_{max} = 7.90 m; Hs = 4.9 m; T_{peak}= 12.0 s prevailing in the sector 160° to 216° (the floating jetty axis is oriented 36° North)
- Wind: 100 year V_{3sec} = 44.7 m/s
- Design life: 100 years
- Durability: non-operated water ballast during service
- In service safety: adequate hull partitioning of the floating caisson.

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14.3 Description of the new harbour facilities

14.3.1 Embankment

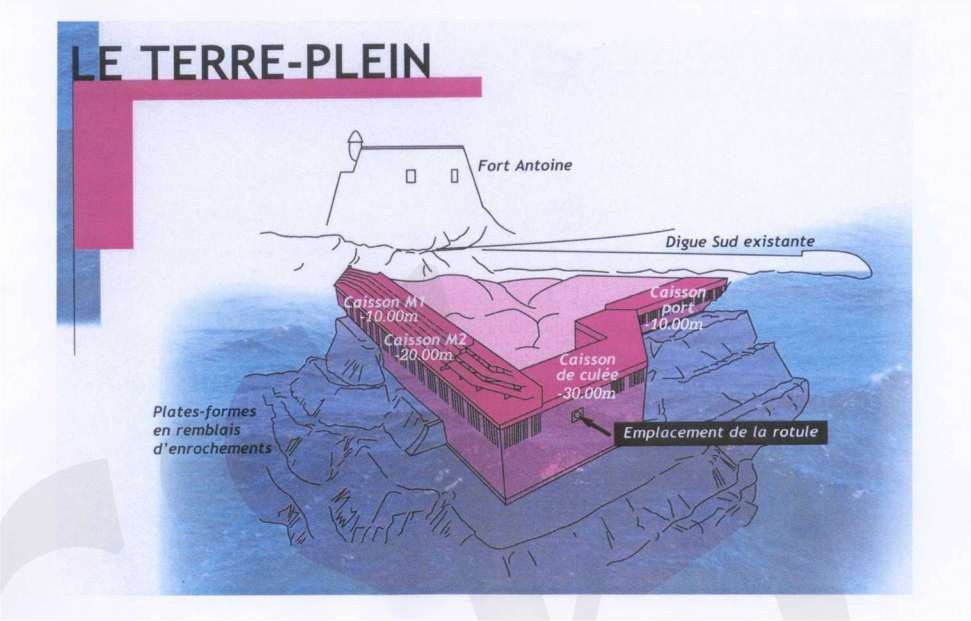


Figure 14-3 The embankment works[54]

The embankment works are composed of four retaining concrete caissons resting on the sea bed by gravity. Underwater platforms, in a water depth varying between 10 m and 30 m, have been prepared and levelled before the installation of the caissons (80'000 m^3 dredging, 400'000 m^3 underwater backfill, vibro-compaction, solid grouting, steel pile inserts).

The total concrete volume for the four caissons amounts to 22'000 m^3 (between 1'500 and 13'800 m^3 each) and the average reinforcing steel ratio varies between 240 to 350 kg/ m^3.

14.3.2 Counter-jetty

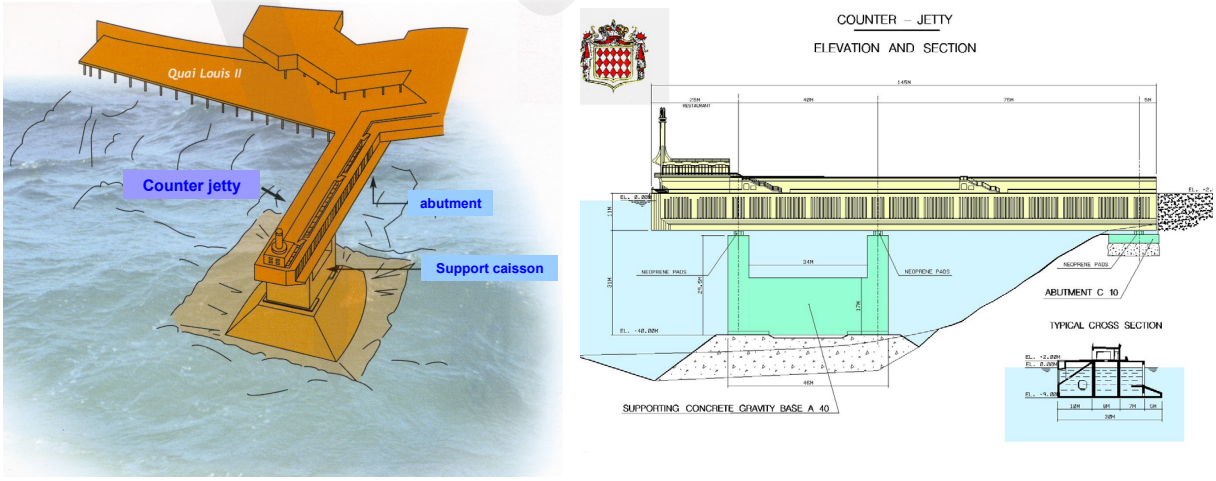


Figure 14-4 The counter jetty works[54]

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The counter jetty is made of a prefabricated reinforced and prestressed concrete caisson 145 m long by 30 m wide and 11 m high (11'700 m^3 concrete; 230 kg/m^3 and 25 kg/m^3 average reinforcing and prestressing steel ratios), which is supported on 3 lines of supports. It completes the closure of the outer harbour, in providing a protection from the eastern swells.

The prefabricated support caisson (8'700 m^3 of concrete) integrates two lines of supports. It is resting on a prepared underwater platform made of 50'000 m^3 dredging, 70'000 m^3 backfilling, steel pile inserts, micro-piles and jet grouting.

14.3.3 Floating jetty

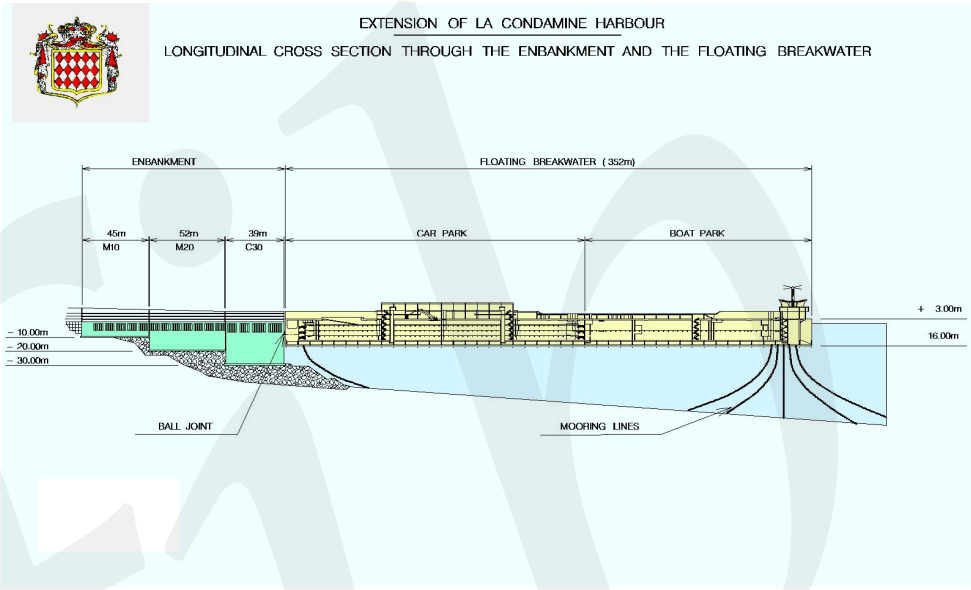


Figure 14-5 Floating jetty: elevation[54]

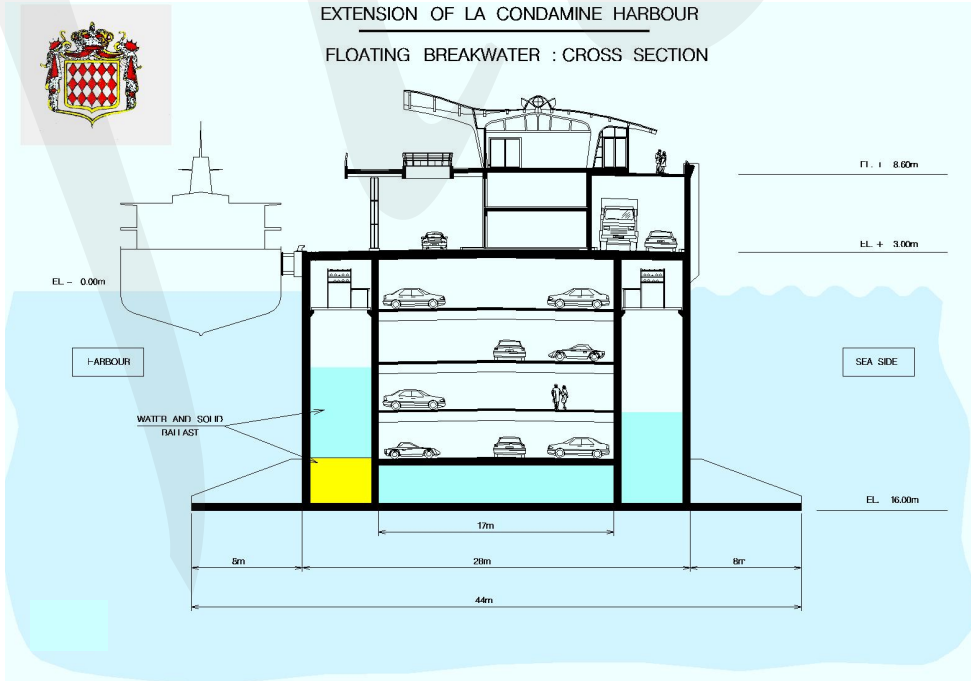


Figure 14-6 Floating jetty: transverse section[54]

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Primary objective of the 352 m long, 28 m wide, 19 m deep floating jetty is to protect the internal water plan from the open sea swell: this dictated the transversal shape and size of the work.

Secondary objective is to utilise the available space inside in an optimum fashion.

The caisson is a 28 m wide box shape, completed by two wings 8.0 m wide each at the level of the lower slab, which have been proven to have a very beneficial effect for the planned stillness of the new harbour water.

Nominal draught of the floating jetty: 16 m.

Upper main deck elevation: + 3.0 m (ground level of the Principality in the close vicinity).

The double hull, double bottom shape of the caisson provides the water and solid ballast compartments for the draught adjustment of the floating jetty and for minimising the longitudinal bending moments due to permanent loads, and to ensure the stability of the floating body in case of unexpected damage. Solid and water ballast are installed once for ever: they do not need to be modified during the normal operation of the facility.

The central core of the caisson inside the volume limited by the peripheral watertight water ballast compartments is utilised for public functions.

Space open to the public is divided into two main areas:

- Car park on four levels, over the first 200 m from the ball-joint
- Storage area, partly used as a "dry harbour": organised on two floors in the remaining volume on the sea side.

Above the main deck at elevation + 3.0 m: a series of secondary buildings are arranged for reception of passengers, luggage, storage, cars and trucks traffic, walks, lighthouse, etc.

Reinforced / prestressed concrete volume: 39'100 m^3 (C65).

Average prestressing and reinforcing steel ratios are respectively 80 kg/m^3 and 250 kg/ m^3.

In service displacement: 163'000 tonnes.

Auxiliary structures (intermediate floors, superstructures, ramps, stairs...): 5'500 m^3 of concrete.

Emphasis has been placed on the durability of the materials in order to guarantee the specified design life of 100 years. Cathodic protection has been provided for improving durability of the steel inserts.

A hinge system, at the near shore extremity, is set 8 m below sea level, corresponding to the depth of minimal motions of the floating jetty, in particular in roll. It includes the following elements (Figure 14-7):

- Spherical ball-joint: made of steel sliding on a fibre glass reinforced polyamide surface, internal diameter 1.60 m. It is fastened at the end of a 5.57 m diameter cylindrical tube.
- On the side of the floating jetty, a steel caisson 6.0 x 5.0 x 9.0 m, fastened to the concrete structure by a set of 60 prestressing rods, includes a conical housing on which the ball-joint axis is inserted.

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- On the side of the abutment caisson, a steel caisson 4.80 x 4.80 x 1.82 m, built in the concrete structure, supports the ball-joint.
- Ball-joint and supporting devices are made of 750 tonnes of cast steel.
- In spite of the fact that wear and fatigue have been designed for a 100-year life, inspection and replacement procedures have been developed as a precautionary measure.

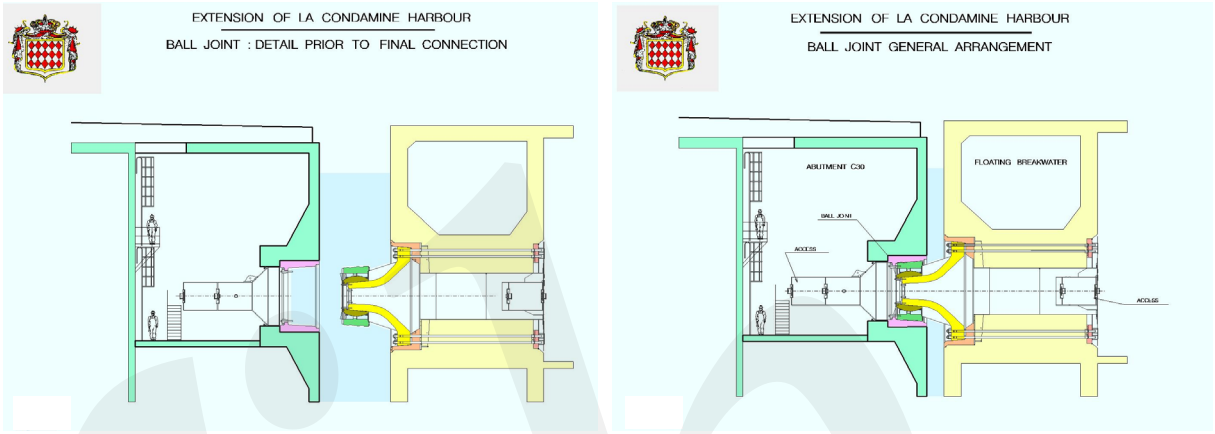


Figure 14-7 The ball-joint system[54]

At the sea side extremity, two sets of mooring lines are installed perpendicularly to the longitudinal axis of the floating jetty:

- Harbour side: 3 lines, 160 m long each
- Sea side: 5 lines, 540 m long each
- Lines are made of a combination of 95 and 152 mm diameter chain.
- Lines are individually moored by 5 x 72″ driven piles at sea side and 3 x 60″ piles at the harbour side for a total of 570 tonnes of steel.
- The ball-joint is attached to the embankment through fuse bolts breaking behind a certain calibrated load, releasing the floating jetty from its attachment to the shore. Consequently, two rescue lines are installed at the hinged extremity, in case of ball-joint disconnection under severe earthquake for instance.

Dynamic behaviour of the floating jetty and its mooring lines has been studied through a combination of numerical analyses and basin model tests (Figure 14-8).



Figure 14-8 Basin model tests[54]

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Table 14-1 Expected motions resulting from the computer simulations and basin model tests are:

	100 year wave	**annual wave**
Wave: Hs / Tpeak	4.9 m / 12.0 s	2.8 m / 7.2 s
Roll	± 1.12 °	± 0.20 °
Heave (sea side end)	± 1.81 m	± 0.11 m
Sway (sea side end)	± 2.73 m	± 0.45 m

14.4 Construction

Construction of the works has been split into two independent contracts:

- Construction of the floating jetty in a graving dock in Algeciras (Spain)
- Construction of all the other concrete works (abutment caissons and counter jetty) in La Ciotat and Marseilles (France), plus the underwater foundation preparatory works.

14.4.1 Works in La Ciotat and Marseilles (France)



Figure 14-9 Construction in La Ciotat and Marseilles[54]

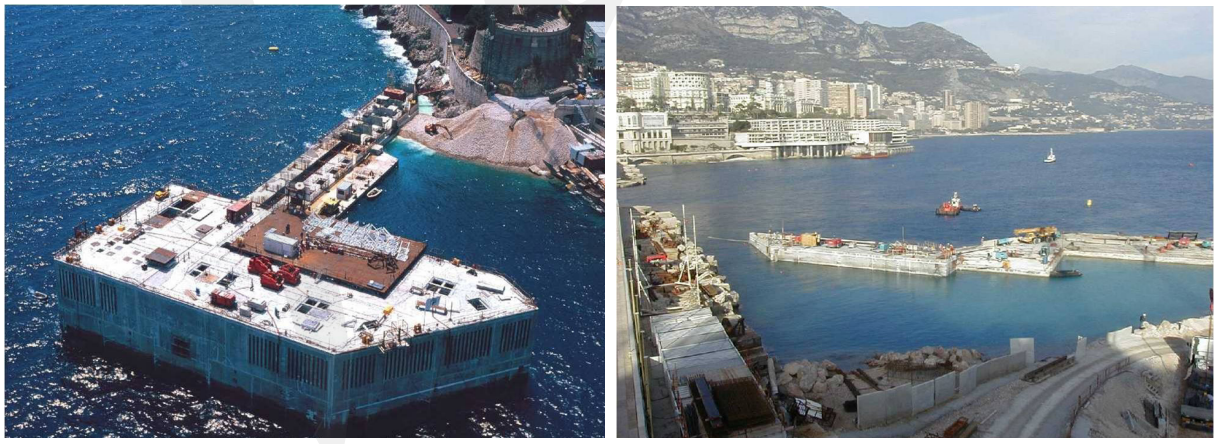


Figure 14-10 Installation of the caisson[54]

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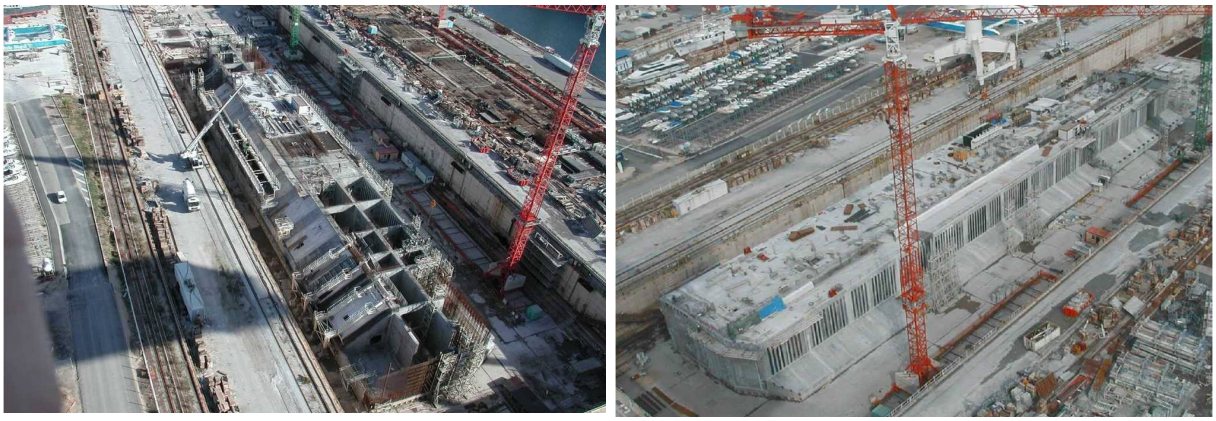


Figure 14-11 Construction of the embankment caissons in the shipyard dock in La Ciotat and Positioning and water ballasting in position of the abutment caisson[54]

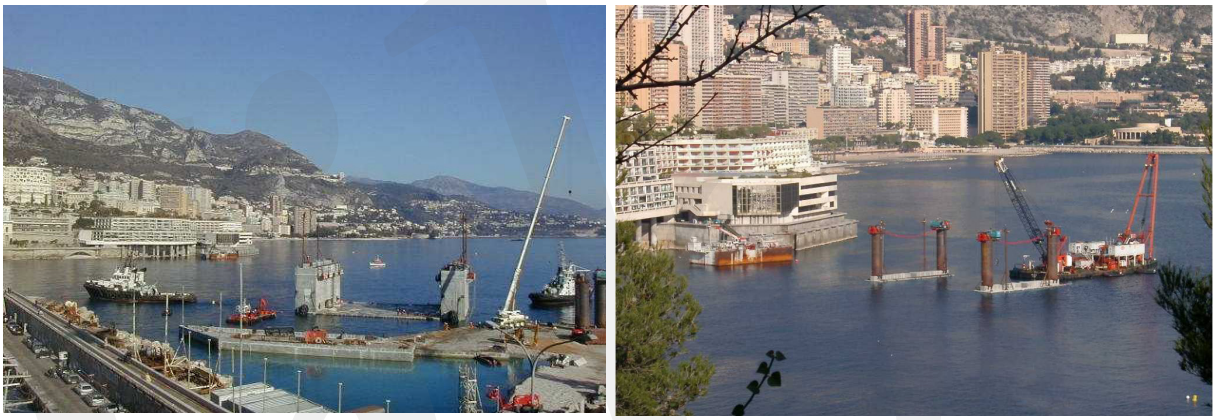


Figure 14-12 Installation of the part of the hinge system fitted to the abutment caisson taking place in the deep harbour of Marseilles and Embankment works viewed from the shore[54]

14.4.2 Works in Algeciras (Spain)

The construction of the floating jetty took place in a dedicated graving dock dug near the Algeciras harbour in the south of Spain in the vicinity of Gibraltar.



Figure 14-13 Construction site in Algeciras, as the construction in the dry proceeds[54]

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Figure 14-14 Installation of the ball-joint within the extremity of the jetty caisson[54]



Figure 14-15 General view of the jetty in the dry prior to flooding the dock and final tow to Monaco[54]

Note the steel cap for the protection of the hinge during the tow and the three temporary steel beams at the upper level for the final and accurate positioning of the floating jetty with respect to the abutment caisson.

14.5 Installation of the floating jetty



Figure 14-16 General view of the jetty floating in the dry dock[54]



Figure 14-17 Positioning of the floating jetty through the use of the temporary steel beams[54]



Figure 14-18 Floating jetty approaching the abutment caisson and engaging the two parts of the ball-joint device[54]

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14.6 Completed project



Figure 14-19 The new harbour at berth[54]



Figure 14-20 Large cruise boats[54]

14.7 Project players

The following organisations and companies have successfully contributed to a unique seven years project opening to public in 2003.

Table 14-2 Floating cruise harbour project players

Owner	**SERVICE DES TRAVAUX PUBLICS DE MONACO**
Project Manager	DORIS Engineering with the assistance of:
	SIMECSOL (Geotechnics)
	CETE-LCPC (Concrete Mix Advisor)
	CTIF (Cast Iron Advisor)
	COPLAN (Service Facilities)
Technical Control	BUREAU VERITAS
Embankment & counter jetty	SAIPEM S.A.
	BOUYGUES TP
	GTM CONSTRUCTION
	VINCI
	ENGECO
Floating jetty	BEC FRERES
	DRAGADOS
	FCC
	H. TRIVERIO
	SMMT
Sources, photos & illustrations credits	SERVICE DES TRAVAUX PUBLICS DE MONACO
	DORIS Engineering

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15. Floating Arctic Quay



Figure 15-1 Visualization showing floating quay at Longyearbyen port[55]

15.1 Introduction

Longyearbyen is the largest settlement and the administrative centre of Spitzbergen. Spitzbergen is located at the 78th parallel north, approximately 700 km off the Norwegian coastline, separating the Greenland- and Barents Sea. Longyearbyen has a population of approx. 2'000 and although coal mining is decreasing in Longyearbyen, there has been an increasing science and research activity and tourism. As a result, the port of Longyearbyen has a definite need for increased capacity for docking ships in the summer months. Longyearbyen does not have the materials nor infrastructure to build a floating quay, nor a fixed quay for that matter, and also a challenging climate, putting floating structures in an advantageous position.

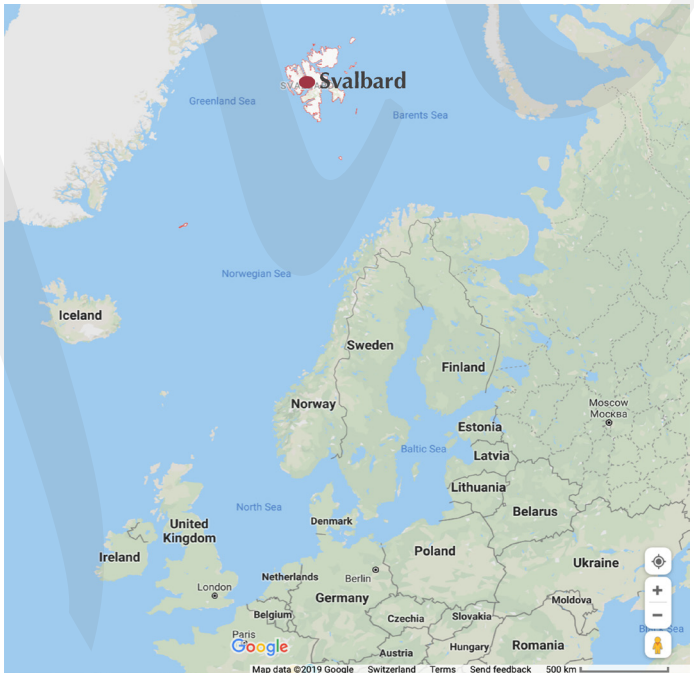


Figure 15-2 High arctic location of Longyearbyen, Spitzbergen[56]

This chapter was mainly authored by Rune Jerstad.

doi.org/10.35789/fib.BULL.0091.Ch15

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To solve the demand for increased quay capacity, engineering/design was requested to lead the development of a cost effective quay with several additional features needed in Longyearbyen. The concept is to fabricate a floating quay more cost effectively on the mainland, towing it to Longyearbyen, and mooring and installing it there. With the exception of the bridge landfall, a floating quay will eliminate expensive foundations on poor soil conditions. An added benefit to the concept is also utilization of the quay hull for other activities and the concept's flexibility through easy removal or relocation after it has served its purpose at the current position – giving the quay a possibly extended life cycle and value, and also leaving a smaller footprint in a vulnerable Arctic landscape.

15.2 Project description

The floating quay has outer dimensions 120 x 35 m. This gives over 300 m total length for docking of ships on all sides of the floating quay. The concrete floating structure is 9 m high with a variation in freeboard between 2.0 and 2.8 m, depending on payload. In addition, the floating quay supports two superstructures with three floors.



Figure 15-3 Architect's impression of section through the floating harbour[57]

The quay is required to handle cargo, passenger and expeditionary traffic. This means docking of vessels from small boats and up to a maximum size cruise ship of 250 m ship length. As the largest ships will dock at the outer main quay front, the smaller ships can also dock on the sides and on the inner quay front. The smaller boats will dock at a separate jetty for smaller boats sheltered behind the main floating quay.

The floating quay hull has room for 1'000 m^3 diesel fuel stored in separate tanks in the hull's watertight safety compartments around the hull. Inside the outer safety compartments, there are 4'000 m^2 arranged for daily use. The lower floor is utilized for "cold storage", with internal temperature corresponding to the temperature of the sea. The upper floor in the quay hull has facilities for education and research purposes and contains laboratories, refrigerated areas, and expedition briefing rooms.

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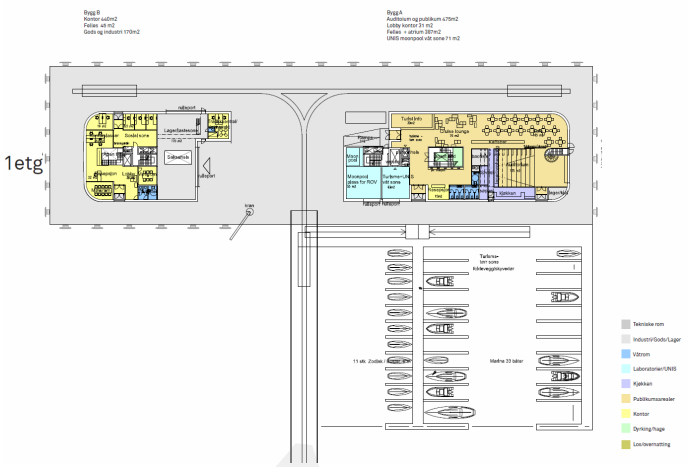


Figure 15-4 Proposed layout of the floating quay arrangement - Top of caisson (ground floor)[58]

Education, research and tourism often use ships and boats in their work in Longyearbyen, so the floating quay is a good location for housing facilities related to these activities. The superstructure houses offices for multiple user groups, a traffic centre for the harbour authority, a lounge for cruise passenger, with cafe, auditorium, and meeting rooms, a terminal for cargo handling, and a moon pool for indoor launching of divers and ROVs for education and research purposes.

15.3 Environmental protection

In line with Svalbard's unique location in the vulnerable Arctic environment, the project incorporates environmental measures. It is easy to remove the floating quay after it has served its mission at the current position – giving the quay a potential extended life somewhere else, and thereby leaving a smaller environmental footprint from the quay.

The quay and buildings are designed as low-energy buildings that set requirements for insulation and building physics. For heating the quay and buildings a heat pump will be used to collect the thermal energy from the sea. This is the most energy efficient and environmentally friendly solution as thermal energy from the sea is available in large quantities. Further, the project plan is to incorporate solar panels on the roof of the buildings to handle energy consumption peaks. These measures will, in addition to being environmentally friendly, relieve the need for investment in increasing the existing energy supply infrastructure in Longyearbyen.

15.4 Environmental conditions

High Arctic conditions set requirements for the design of the quay itself, the building exteriors and layout, energy conservation and, not least, for the mooring of the quay. The main considerations are wind, waves and ice.

15.4.1 Wind

The area topography is dominated by significant variations in terrain elevations. The fjord Longyearbyen is surrounded by steep mountainsides on each side, reaching up

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to 500 m height. The characteristic topography strongly influences the environmental actions in the location, determining the mooring system design. Especially when a large size cruise ship is moored to the quay, the wind force and direction will generate large mooring forces (direct wind pressure and wind driven waves that must be absorbed whilst achieving gentle, comfortable movement on the quay. For the largest cruise ships (up to 250 m length) separate mooring points on land are required.

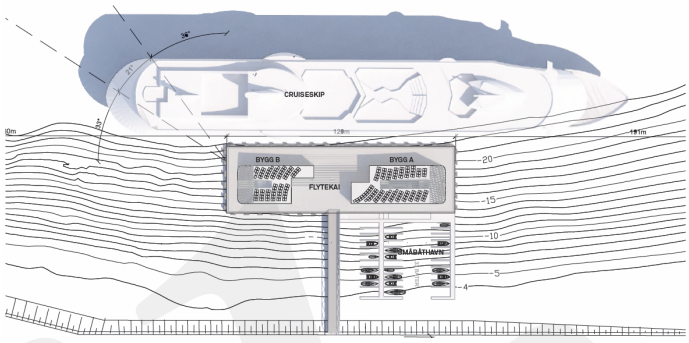


Figure 15-5 Showing 250 m cruise ship docked at the quay[58]

15.4.2 Waves

The significant wave height for the wind generated waves is determined by the fetch of 40 km in the north-northwest direction. Wave actions for the operational phase are extracted from a study performed by the Norwegian University of Science and Technology (NTNU). Determining the significant wave height for swell waves, the sheltered location of the quay compared to the measurement location is taken into account. The "Breakwater-effect" of a headland gives a considerable reduction to the design wave height.

15.4.3 Ice

The salinity in Adventfjorden is approximately 35 ppt, which gives a freezing temperature of -1.8°C. Combining different methods and measurements, maximum sea ice thickness is estimated to be 1.3 m at Longyearbyen.

Annual ice and ice-bergs may drift in to the Adventfjord. Annual ice may come drifting in from the Arctic shelf, and ice-bergs may be formed by the glaciers in fjords close by Advendtfjorden.

There are three main sources of ice loads on the quay when situated in a solid ice cover:

- Thermal expansion exerting a huge pressure along all sides of the quay
- Horizontal movements of ice cover as the quay is frozen to the ice
- Wind fetch on the ice floe. When the ice breaks or large floes drift close to the quay it is possible that a large floe may become attached to the quay. This floe will exert forces on the quay as a function of wind and current.

15.5 Design considerations

The outer walls are a robust 0.5 m thickness, but to ensure high degree of safety, the outermost part of the quay hull consists of watertight compartments that secure damage

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stability in the case of an unforeseen accident rupturing the side of the quay. These compartments are utilized for fuel tanks, ballast, mooring and equipment.

Towing over one of the harshest sea-stretches on the planet, even during the summer season, means that the hull can experience large global bending forces from unfavourable wave-conditions. The hull must be able to handle these forces, in a similar manner as ships do. Sufficient hull height and correct post-tensioning of the hull are the most important factors to cater for the structural stresses in this critical phase.

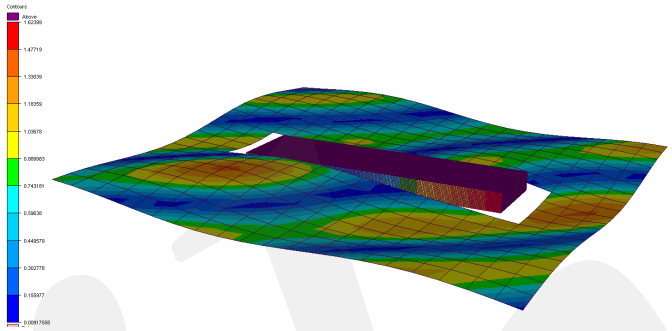


Figure 15-6 Dynamic wave analysis (Olav Olsen)[59]

Another complex dynamic analysis is that for the moorings to tune the system to allow for comfortable movements of the quay during operation in the port of Longyearbyen. Important factors are wind, wave, ice, current, bathymetry, docked ships, and the quay itself. Combined mooring, catenary mooring lines and the bridge to shore, as a part of the system to dampen and steady the movement of the floating quay, has been considered.

15.6 Summary

The results shown is from the concept development and Pre-FEED stage of a project conducted for Longyearbyen Lokalstyre. A separate evaluation of the different development opportunities of the Longyearbyen port, performed for the Norwegian Ministry of Transportation and Communication, has confirmed a floating quay to be the recommended solution. The work conducted has shown that a floating concrete quay is a robust and an economical solution for remote Arctic harbours. It gives flexibility in application and for removal or relocations.



Figure 15-7 Architect's impression of Longyearbyen floating harbour[55]

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16. Floating bridges

A floating bridge is useful for certain conditions when crossing straits. For example if the water depth is very large, or if it is difficult to construct at the bridge site, or if the bridge is temporary. There are also other governing factors to be considered by the designer and the owner. This chapter describes some floating bridges, and their merits.

The technique is very old. The Romans used it for example, when conquering land.

16.1 The concept

A floating bridge or a pontoon bridge is a bridge where a continuous deck for pedestrian or vehicle travel, the roadway, rests on floating structures, e.g. floats or shallow draft boats. The buoyancy of the supports limits the maximum load the bridge may carry.



Figure 16-1 The Dongjin Bridge (東津橋) in Ganzhou metropolitan area[60]

Historically, most floating bridges have been temporary structures, used in wartime and civil emergencies. More permanent floating bridges have been used for sheltered water crossings where it is not considered economically feasible to build a bridge supported by fixed piers. Typically, floating bridges have been used in river crossings where the loading is simple and monotonic; vertical loading from self-weight plus traffic and horizontal loading from the river flow. A challenge in using a floating bridge to span a waterway used by waterborne traffic is that such a bridge requires a section that is elevated, or can be raised or removed.

The floating structure could be of the classical type using several separated pontoons forming the support for the roadway structure (e.g. the "Nordhordland Bridge" situated outside Bergen in Norway) or a continuous floating pontoon using the top of the pontoon as roadway (e.g. the "Lacey V. Murrow Floating Bridge", situated in Lake Washington, Washington in USA) or supporting a roadway structure on a continuous pontoon (e.g. the new "William R. Bennett Bridge", situated in Lake Okanagan, British Columbia in Canada).

This chapter was mainly authored by Rolf Magne Larssen.

doi.org/10.35789/fib.BULL.0091.Ch16

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16.2. The development

Pontoon bridges have been in use since ancient times and have been used to great advantage in many battles throughout history, among them the Battle of Garigliano, the Battle of Oudenarde, the crossing of the Rhine during World War II, and during the Iran–Iraq War.

The most common use is for river crossings and several examples may be found in literature. One existing bridge of this kind with a long record is the Dongjin Bridge in Ganzhou in China. This pedestrian bridge exists today and was originally constructed during the Song Dynasty (960-1279). The bridge provides a simple way of crossing the river for humans and light transport but has obviously its limitations for more heavy transports as cars and lorries, see Figure 16-1.

Probably the most known European floating bridge is the crossing over the Golden Horn in Istanbul, Turkey. In ancient time several crossings have been established or been proposed for this crossing, e.g. Leonardo da Vinci's fixed crossing from 1502. The most recent, the Galata Bridge, has a history that dates back to 1845 when the first floating bridge was established here, built of wood. A total of four different floating bridges secured the connection over the Golden Horn from 1845 to 1994 when a fixed bascule bridge replaced the last one. The length of these bridges was moderate, between 460 and 500 m but they provided a secure connection for people and light transport.



Figure 16-2 The Interstate 90 floating bridges across Lake Washington, looking east from Seattle, Washington toward Mercer Island. Right: Lacey V. Murrow Memorial Bridge Left: w:Homer M. Hadley Memorial Bridge[61]

The next step in the development of floating bridges towards a bridge type for heavy transport over considerable length and diversified loading, is probably the Lake Washington Floating Bridge, opened in 1940, see Figure 16-2 (right). This bridge is a 2'020 m long floating concrete structure crossing Lake Washington outside Seattle on the west coast of the USA. The bridge concept was originally proposed by structural engineer Homer Hadley. His proposal of the concept at the American Society of Civil Engineers meeting in 1921 caused a lot of scepticism and discussion amongst Seattle's civic leaders and in the media. Hadley hoped to build it with private money and make it a toll bridge, but bankers nicknamed the idea "Hadley's Folly," and the project stopped. The idea was relaunched in 1937, after federal money became available to the states during the Great Depression, and the Director of Highways for the Washington Transportation Department, Lacey V. Murrow, became convinced that the building of the floating bridge was feasible. Construction started in November 1938 and the bridge was opened on July 2, 1940.

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Figure 16-3 The Hood Canal Bridge in western USA, Washington State[62]

Moving from fresh water in an inland lake to salt water with tide variations, current and larger waves bring further challenges to such a bridge concept. The Hood Canal Bridge across Hood Canal of Puget Sound in Washington State on the west cost of USA, see Figure 16-3, brought the floating bridge concept this step further in 1961. This 1987 m long floating continuous concrete pontoon bridge spans this salt water stretch having a tidal difference of 5 m. A floating bridge is preferred due to the depth of the Hood Canal channel – over 100 m in several places – making fixed piers for a conventional suspension bridge complicated and expensive. At the centre of the bridge a retractable pontoon draw span opens to a 180 m clearance in order to give access to open sea for ships e.g. from the naval base at Bangor, homeport of Trident submarines. The floating portion of the bridge is held in place by braided steel cable tied to 42 concrete-block anchors, each with a weight of more than 500 tonnes. The design is very similar to the Lake Washington Floating Bridge except that this bridge has an elevated roadway supported on the pontoon.

However, some engineers had at an early stage expressed doubt about the design and location of the Hood Canal Bridge, since the floating pontoon structure was the first of its kind to be built in salt water and that the canal would act as a natural funnel to intensify both wind and waves. In some way this proved to be partly correct as the western half of the bridge sank during a storm after 18 years in service, in 1979. The bridge was rebuilt and opened for traffic again in 1982.

Three other bridges of similar design have been built in the USA and one in western Canada, all using concrete as the main building material. The last one, the Kelowna or Okanagan Floating Bridge, was built in 1958. Due to substantial increase in traffic this bridge was rebuilt in 2008 and is now named William R. Bennett Bridge.

All the bridges mentioned have been held in position for sideways loading by anchors. The next step in floating bridge technology was realized through the Norwegian bridges built in the 1990's, the Bergøysund Floating Bridge opened in 1992 and the Nordhordland Floating Bridge opened in 1994, see Figure 16-4. These bridges were respectively 933 m and 1'246 m (floating part) long and held in position only by anchoring at both ends, with no side supports in the span. This solution provides feasibility to cross the Norwegian

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fjords which were more than 300 m and 500 m deep at the two bridge locations. The design of these two bridges was quite different from the continuous floating concrete structure used in the American and Canadian bridges. For these bridges, discrete concrete pontoons supporting a continuous elevated bridge beam were used. This design facilitates an elevated roadway that reduces traffic hazards in storms, reduces loading on the bridge from waves and current, reduces corrosion risk of the bridge deck and improves water passage beneath the bridge to minimize the bridge impact on the local water environment. The design of the bridge beam was however different for the two bridges; one steel tube space truss structure for the Bergøysund Floating Bridge and one box girder with internal stiffeners for the Nordhordland Floating Bridge.



Figure 16-4 The Nordhordland Bridge outside Bergen in western Norway[63]

The challenge of how to provide continuous access for waterborne traffic was also solved differently for the two bridges. For the Bergsøysund floating bridge the ship traffic was moved from the Bergsøysund to the Gjemnessund, on the other side of the island Bergsøya. This strait was then crossed by a suspension bridge having a navigation channel of 43 m height. Thus, the Bergsøysund was closed for ship traffic other than small boats with less than 6 m height. For the Nordhordland Floating Bridge a cable-stayed high bridge is built in the end of the floating bridge giving a sailing channel of 32 m height. To bring the roadway from the floating bridge beam up to the high bridge a light viaduct structure is used.

In total around 50 floating bridge structures have been built around the world. Most of these are rather short, simple structures used for pedestrian crossing (around 15 structures) or river crossing (around 10 structures) and some are temporary structures used for some years while waiting for a better permanent structure to come. In total, 11 floating bridge structures for road traffic of some size exist, five in the west coast of USA, one in Canada, two in Norway, two in Guyana and one in United Arab Emirates.

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16.3 Some examples of design and construction of floating bridges

In order to give a closer description of floating bridge technology some examples are described in more detail. For these examples the Nordhordland Floating Bridge from 1994 and the rebuilding of the Okanagan Floating Bridge, the William R. Bennett Floating Bridge in 2008 have been selected.

16.3.1 The Nordhordland Bridge

The Nordhordland Bridge is a floating bridge situated on the west coast of Norway, close to the city of Bergen. Plans for a bridge at this location had existed since the 1960s, and after the decision to construct the bridge was passed by the Parliament of Norway in 1989, construction started in 1991. The design of the bridge was performed by a team lead by Dr. ing. A. Aas-Jakobsen AS and the construction was performed by a joint venture AF Salhus Bru consisting of the contractors Norwegian Contractor, Aker Entreprnør, Veidekke and Kværner Eureka.

The "Nordhordland Bridge" is a floating bridge which spans a strait of 1'470 m width. A requirement for this crossing was to provide a navigation channel having a width of 50 m and a free height of 32 m. The solution is shown in Figure 16-5.

The "Nordhordland Bridge" thus consists of a floating part with 1'246 m length. This floating part is connected to the shoreline in one end and to a pier constructed at the other end. This pier is placed on the sea bottom at 30 m depth 163 m from the other shoreline. The remaining gap is spanned by a one tower cable stayed bridge over the established navigation channel.

As the floating part of the bridge is held in position only by supports in both ends no anchor system is needed. The effect of water level variation (tide) is taken care of by allowing rotation about the horizontal axis in both ends and having an appropriate length of each end span (distance to first pontoon).

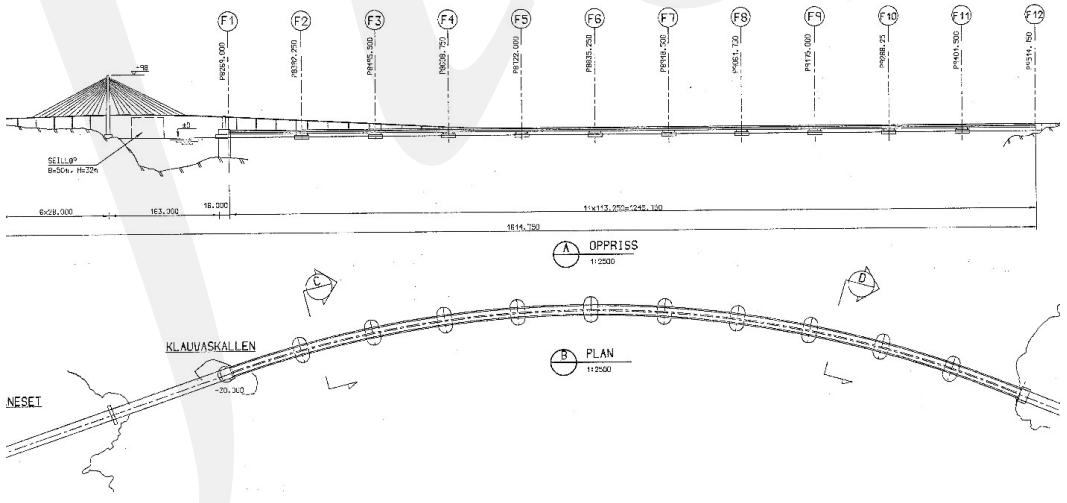


Figure 16-5 Overview of Nordhordland Bridge[64]

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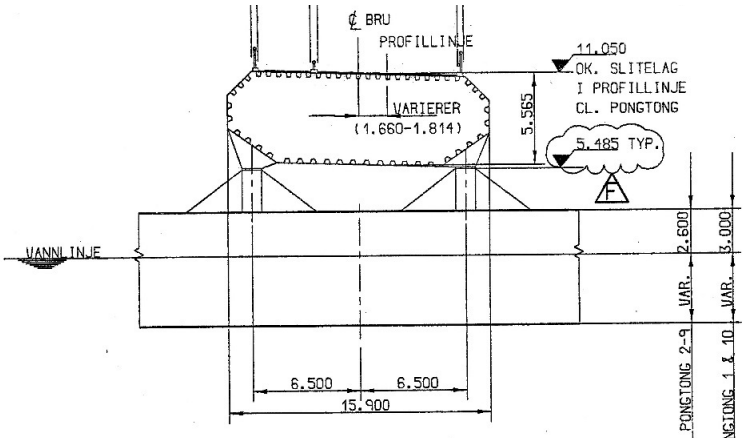


Figure 16-6 Cross-section of floating part of Nordhordland Bridge at pontoon[64]

A detailed description of this bridge is given by Meaas, P., Landet, E., & Vindøy, V.[65]. The following information is partly based on this source and direct communications with persons involved in the design.

Floating bridges are sensitive to environmental loads i.e. wave height, direction and period, wind speed and direction and tidal variation. It is vital in the design of such structures to have accurate predictions of these effects. Therefore, a measurement program at the bridge site was carried out. Wind measurements were performed in the period 1989 to 1991 and wave data was collected from December 1990 to March 1991. Based on these data design parameters for the ultimate limit state and long term statistical wave data, in the form of scatter diagrams, were derived.

Environmental loads with return period 100 years are as follows: The design 10 minutes average wind speed 10 m above sea level is 27.1 m/s. The significant wave height is Hs = 1.67 m. The corresponding wave periods are in the range of 3.6 to 5.1 s. The sea current is 1.75 m/s. The tidal variations are +1.6 m above and -1.4 below mean water level. For fatigue calculations a constant tidal variation of ± 0.61 m is applied.

The ice load for global analysis of the bridge is 11.4 kN/m. The load is assumed to be evenly distributed over the whole width of the strait. The air temperatures are + 25°C and - 15°C. In addition variation of the density of sea water and marine growth is specified and considered in the design.

The floating part of the bridge consists of a continuous main girder designed as a closed steel box, 15.9 m wide and 5.5 m high, see Figure 16-6. The girder is vertically supported at the separated pontoons and shaped as an arch in the horizontal plane (radius of 1'700 m) in order to minimize loading effects due to mean wind, current and temperature. The roadway is placed on top of this girder for most of the span, 11 m above the waterline. In the end towards the navigation channel bridge, a 350 m long ramp is required to connect the bridge deck on the cable stayed bridge to the bridge deck on the floating bridge. The free height below the girder down to the waterline is 5.5 m allowing for passage of small boats. The plate thicknesses vary from 14 mm to 20 mm. The plate stiffeners are traditional trapezoidal shaped and they are spanning in the longitudinal direction of the girder. The stiffeners are supported by cross frames with centre distance of maximum 4.5 m. At the supports on the pontoons, bulkheads are used instead of cross frames as the loads in these section are significant larger than in the cross frames. The plate thicknesses in these bulkheads vary from 8 to 50 mm.

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The total steel weight of the box girder is 12'500 tonnes of which approximately 3'000 tonnes are high strength steel.

Each of the 10 pontoons is 42.0 m long, 20.5 m wide and with a half circle of diameter 20.5 m at each end, see Figure 16-7. The freeboard for the pontoons is generally 2.6 m. This freeboard is designed such that there will be no flow of water over the top of the pontoons under severe storm conditions. As the tidal variation has an effect on the freeboard of the pontoon near the abutment at each end of the bridge, the freeboard is increased to 3.0 m for these pontoons. The draft varies from 4.3 m to 5.6 m. The variations are mainly due to the elevated ramp at one part of the bridge and that the end span is equal in length with those in the middle. Each pontoon will be provided with some solid ballast. This is not necessary for stability reasons, but is only used to account for uncertainties in actual weight after construction.

Each pontoon is divided into 9 compartments which are separated by watertight bulkheads. Each compartment has access through a watertight hatch from the top deck. The sizes of compartments are determined such that the floating bridge is still intact if two adjacent compartments are flooded due to some accident. The thickness of the bottom slab is 300 mm; the outer walls are 310 mm thick while the top slab and the internal bulkheads are 200 mm thick.

Light weight aggregate concrete, LC55, has been used in the entire pontoons in order to minimize the weight. A reduction of weight is favourable because the draft is reduced which again reduces wave and current loading on the entire structure. The savings are therefore mainly achieved by reduction in the structural steel weight.

- The main properties of the LC55 concrete are:
- E-modules: 20'000-23'000 MPa
- Density (saturated) is maximum: 1'950 kg/m^3
- Type of aggregate: Liapor 8.

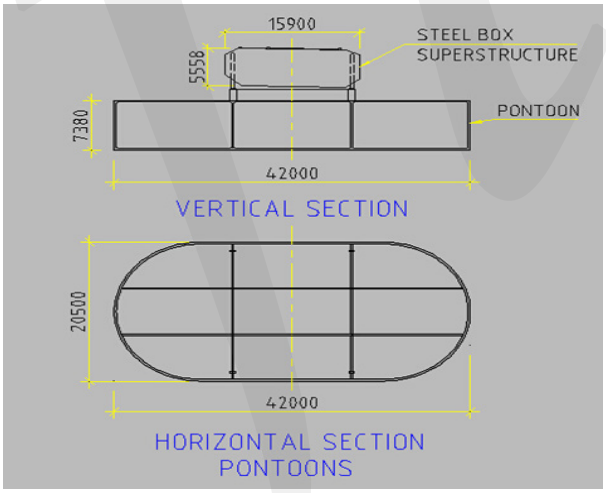


Figure 16-7 Pontoons plan and section[65]

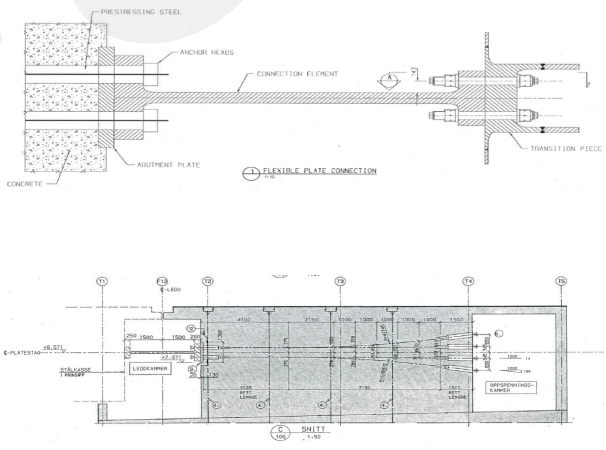


Figure 16-8 Arrangement for connection between floating bridge and abutment[65]

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Figure 16-9 Construction of pontoons in light weight aggregate concrete for the Nordhordland Bridge[64]



Figure 16-10 Installation of the bridge[64]

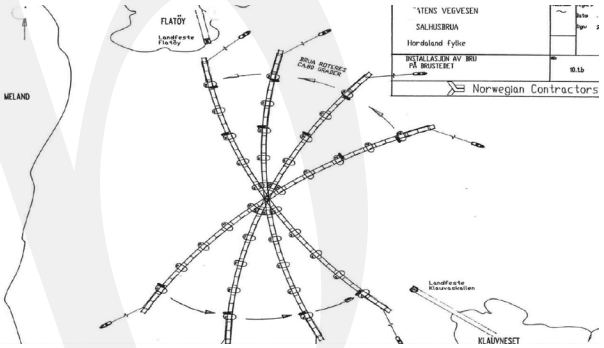


Figure 16-11 Installation of the bridge[66]



Figure 16-12 Assembly of the bridge girder at Lonevågen, close to final location for the Nordhordland Bridge[64]

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The steel box girder is fixed to the abutments by means of special purpose made "flexible" plate connections made in high strength steel, see Figure 16-8. The plate connections transfer bending moments about a vertical axis, axial forces and horizontal shear in the girder to the abutments. The plate connections, however, are flexible about a horizontal axis perpendicular to the bridge in order to allow deformations due to tidal variations. Vertical shear and torsion is taken by separate neoprene bearings. On each side of the steel girder there is one set of neoprene bearings supporting downward vertical forces and another set supporting upward vertical forces. At each abutment 4 "flexible" steel plate connection elements are used.

The flexible plate connections are forged in order to obtain excellent fatigue properties and a high yield stress. The thickness of the flanges and transition curvature between the plate and flange is designed in order to minimize stress concentrations. The steel plate connection elements are fixed to the steel girder by means of bolts specially made for this project, see Figure 16-8. The flexible plates are bolted to transition elements, which are welded into the box girder. The transitions elements are forged and the final heat treatment was by normalizing.

The challenging abutment is the transition between the high level bridge and the floating bridge. This abutment is founded on bedrock at deep water, at a water depth of 30 m. It consists of a caisson with a cross section of 20 m x 21 m with 16 internal cells. The caisson is filled with gravel and it is closed with a slab 5 m above the waterline. Due to the large height of the caisson it was designed such that the lower part, 12.5 m high, could be constructed in a dry dock and the upper part could be constructed by slip forming afloat. This construction process is the same as used for concrete offshore platforms. All marine phases were checked with respect to hydrostatic stability. As part of this design all walls were designed such that any compartment could be accidently filled during construction without loss of the structure. The largest loads to be carried by the abutments are axial forces in the box girder in combination with a moment about the vertical axis of the abutment. Vertical prestressed rock anchors are required.

All elements of the bridge were produced in Norway. The ten pontoons were built in a dry dock in Fredrikstad, see Figure 16-9, and towed to Lonevågen, see Figure 16-10, a small sheltered fjord just east of the bridge site. The steel box girder was built in sections (92 to 155 m) at Kværner Eureka in Moss/Fredrikstad and shipped to Lonevågen on barges. In Lonevågen assembly of the bridge girder and the pontoons was performed, see Figure 16-12, before the complete floating bridge were towed 12 km to the final location in the Salhusfjord and installed in one piece, see Figure 16-11.

The bridge was opened to traffic in 1994 and has functioned well without any major problems. The concrete pontoons have been monitored closely, especially related to chloride corrosion of reinforcement. Measurements show an excellent behaviour. Similar results are also found for the second Norwegian floating bridge which has similar pontoons. The steel girder has also behaved well during the 23 years this bridge has been in service.

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16.3.2 The William R. Bennett Bridge



Figure 16-13 William R. Bennet Bridge, Okanagan Valley of British Columbia, Canada[68]

The Kelowna or Okanagan Floating Bridge, was built in 1958 and due to substantial traffic increase, rebuilt in 2008 and now named William R. Bennett Bridge, see Figure 16-13. The Ministry of Transportation of British Columbia found out in the early 1990's that the existing structure could not handle the increasing traffic volumes and that the structure was near the end of its available service life. In 1995, Westmar Engineering Consultants of North Vancouver was retained to design a replacement structure. Eventually it was decided to build the bridge as a Design/Build/Finance/ Operate project and a team lead by SNC Lavalin was awarded the contract. The design of the new bridge was performed by a team lead by Buckland & Taylor Ltd and using the Norwegian companies Dr. ing. Aas-Jakobsen AS and Johs. Holt AS to do the floating structure design, based on their experience from the design of the Bergøysund Floating Bridge and the Nordhordland Floating Bridge. Arup acted as checking engineer for the owners.

The new bridge is located along the north side of the previous bridge, a location that permits re-use of the existing road infrastructure. Also, to minimize the cost of the project, the new bridge re-uses the anchoring system from the previous bridge. This use of common anchors required that both bridges were connected during construction, which added significant complexity to the design and erection process.

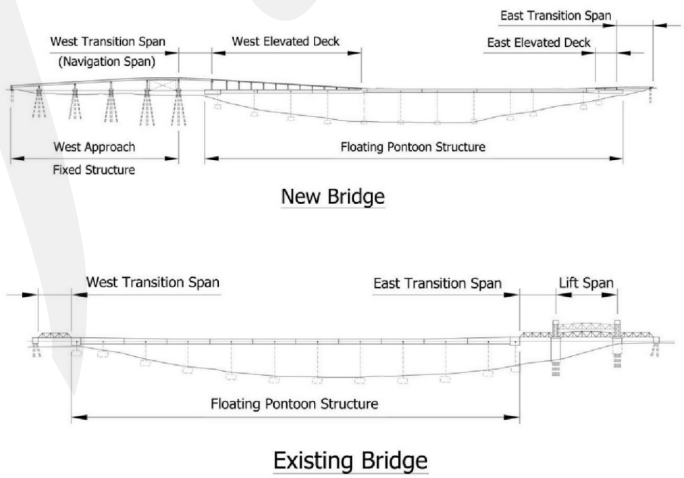


Figure 16-14 Elevation of new and previous William R. Bennet Bridge[67]

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Figure 16-15 Erection of final pontoon of the William R. Bennett Bridge[67]

A detailed description of this bridge is given by Matson et.al.[68] and Jakobsen, S.E., Larsen, P.N. & Matson, D.[67]. The following information is partly based on these sources and direct communications with persons involved in the design.

The new design eliminates the lift span by providing a fixed, elevated approach structure with a navigation span on the west side of the bridge, see Figure 16-14. The middle portion of the bridge comprises a floating pontoon string, and transition spans are provided from the west approach structure and the East shore to the pontoons.

The floating section of the bridge consists of a continuous concrete pontoon-string approximately 700 m long and 25 m wide. The depth of the pontoon varies from 5.14 m to 7.8 m to account for different weights of the superstructure. Each pontoon contains longitudinal post-tensioning in the top slab, bottom slab and walls.

The roadway is situated directly on the pontoon top slab over the middle length of the pontoon-string. Roadway access from the high level West Approach Fixed Structure is via an elevated deck supported on piers that are integral with the pontoon.

The pontoon was designed and constructed as nine separate parts in a graving dock (dry dock) and subsequently floated out to the bridge site where they were connected together by means of post-tensioned tendons, see Figure 16-15. The pontoon is supported sideways by a number of anchor lines fixed to the lake bed. The pontoon is designed with a number of internal walls forming watertight bulkheads. These watertight bulkheads are arranged such that flooding of two adjacent compartments will not affect the integrity of the bridge. In addition the internal walls provide supports for the external walls and slabs which consequently can be made relatively thin in order to reduce weight. Reduced weight gives reduced draft which results in lower wave load.

The Canadian Bridge Code (CAN/CSA-S6-00)[69] did not address all the design requirements for this type of structure. Thus, other codes, particularly the Norwegian code, were used to supplement the Canadian code. The pontoon is designed for a 100 year wind speed and significant wave loading (the peak wave height is approximately 2.3 m). The pontoon is post-tensioned in the longitudinal direction in order to give membrane compression in the submerged part in Serviceability Limit State. In Ultimate Limit State through crack widths are limited to prevent ingress of water into the pontoon.

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Dynamic wave analyses were performed, for transportation, free floating and erection phases, as well as the completed bridge. Transfer functions of global moments showed peak values in actual excitation frequencies, implying quite severe response. Slow drift effects were also implemented. The final results were combined with the results from the extensive global static analysis model. Here all effects from permanent loads, deformation loads, wind loading, ice loads, temperature, traffic loading and accidental flooding were combined into relevant load combinations in all limit states. A plot of the model is given in Figure 16-16.

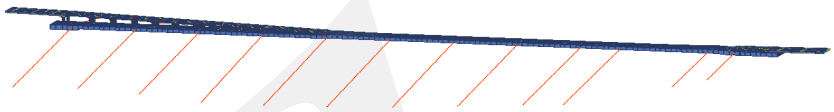


Figure 16-16 Plot of global analysis model[67]

For each of the 9 pontoon parts, a model was developed, see Figure 16-18, taking into account global sectional forces from the global analysis and local loading acting on the considered structural part. The result from the analysis was processed by means of the concrete design program in order to find utilizations of concrete and reinforcement. In addition hand calculations were performed to verify the results, based on the modified compression field theory and is a tailor made program for check of concrete cross-sections of various shapes.

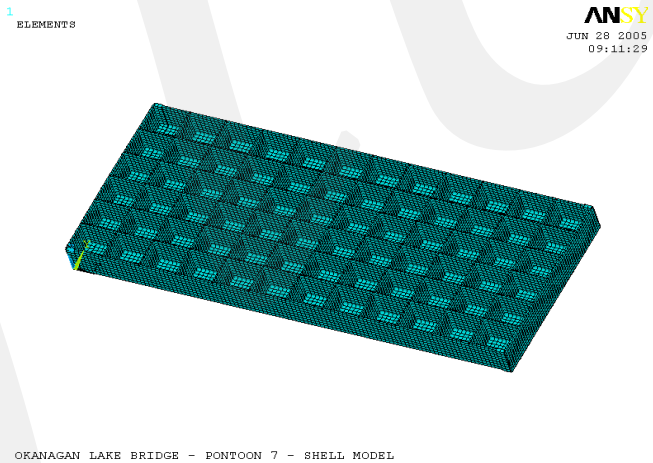


Figure 16-17 Plot of typical finite element model (top slab not shown)[67]

The dominant action on the pontoon is the wave loading. This action is combined with other significant load actions such as traffic, temperature and accidental flooding in Serviceability Limit State and Ultimate Limit State according to design criteria. One of the governing load combinations in Ultimate Limit State is wave action without traffic loads. In Serviceability Limit State a combination of accidental flooding with wave action and traffic is governing for parts of the pontoon. The dynamic wave analysis showed that the moments about the two axes are essentially uncorrelated; therefore the orthogonal responses were combined with the 100%/40% rule.

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To minimize the built-in forces in the pontoon during construction, a construction scheme for the construction phases was developed utilizing temporary ballast placed in positions of the pontoon where future permanent weight shall be included. The preliminary ballasting scheme differs from the actual ballasting scheme used by the contractor on the site. The difference in theoretical weight and actual weight is compensated by solid ballast in order to reach the final draft. The response in the pontoon during the construction phases was controlled by giving the contractor design limits for global bending moments and shear forces along the pontoon to operate within. The limits were given both for temporary operations as well as for the final condition.

The elevated decks are designed as a light weight steel plate composite girder structure, with relatively short spans, to distribute the weight of the superstructure evenly onto the pontoon and to improve stability (the elevated structure is supported as much as 18 m above the top of the pontoon and any extra weight reduces the stability of the pontoon).

The 9 separate pontoons were connected together to form a 700 m long pontoon string. The Concession Agreement for the project required that the pontoon connections must be stronger than the pontoon structures, and that the joints must be under compression for all service load combinations. As such, each pontoon connection includes a total of 146 post tension tendons (7-15ø strands), and 28 high strength DYWIDAG bars (44 mm diameter), around the perimeter of the pontoon (see Figure 16-18).

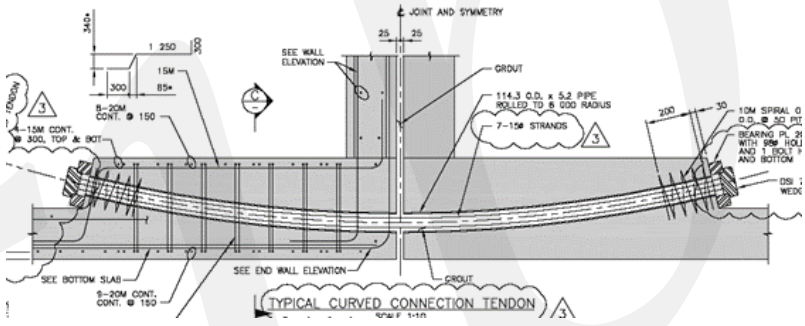


Figure 16-18 Pontoon Connection[67]

To allow for construction tolerances, and to be able to connect the pontoons and install all the PT tendons and the high strength bars across the joint, it was necessary to design the pontoons with a relatively large tolerance at the location of the tendons and the DYWIDAG bars. As such, it was decided to place a steel pipe with an inner diameter of 104 mm in the concrete bulkhead of the pontoons, and then feed the PT tendons and DYWIDAG bars inside these pipes. This allowed the steel pipes on the two connecting surfaces to be slightly misaligned, but to still be able to get the steel across the joint.

The floating pontoons are anchored with anchor cables that from anchor end up consist of a heavy sea link anchor chain (acting as a damper), a 69 mm diameter lower cable, and a replaceable 69 mm diameter upper pennant. The total length of the anchor cables varies from 90 m to 240 m. Due to the water level fluctuation on the lake, the floating pontoons are designed with a cable gallery that allow for the anchor cable tension to be adjusted. To obtain a minimum tension in the cables (even during a 100-year return period wind storm), but also to avoid exceeding the capacity of the anchor cables and the anchors, the cables require adjustment on a bi-annual basis.

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The construction process started in August 2005 with the commissioning of a graving dock on the North West shore of the lake. Pontoon construction started on January 2006 and the new bridge was opened to traffic in June 2008. The demolition of the old bridge continued until spring 2009.

The schedule was so tight that required steel to be ordered within six weeks of contract signature and pontoon construction to start before the final design was completed. The contractual model, which integrated the design and construction teams, allowed a complete coordination and reduced the time between the issue of "For Construction" drawings and placement of concrete on site to as little as two weeks.



Figure 16-19 Typical pontoon under construction[67]

The construction of a floating structure presents a rather unique set of challenges which require innovative thinking. The pontoon design is driven by buoyancy and water tight compartments. These requirements result in tall and slender walls. Typical internal walls range from 150 mm to 200 mm thickness and heights in excess of 5.4 m. Placing concrete in such thin, tall and heavily reinforced walls was achieved by using self-compacting concrete (SCC), and developing placing techniques and tools which reduced the risk of segregation and other defects, see Figure 16-19. The top and bottom slabs (25 m by 90 m) were cast without construction joints, and the exterior walls were poured on two sections, each one 115 m long by 5 m high.

After a quite steep learning curve, the typical cycle for a pontoon (90 m long, 14'000 tonnes of concrete, 1'100 linear m of walls) was established at 70 days, including post-tensioning. This was the time from the start of construction until tow out from the dock, see Figure 16-20 .

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Figure 16-20 Float-out of pontoon[67]

The pontoons, once finished and floated, had to be installed in their final position, next to the existing structure. Since the old and new bridges share 20 out of 24 existing anchors during the construction period, it was imperative to ensure the integrity of the old structure and the safety of the public. The installation procedure requires either one or two sets of anchors to be de-tensioned during the installation of a pontoon, so the operation had to be quick and safe.

Pontoons were installed without the superstructure. Therefore, the elevated deck was installed on a floating structure to which additional pontoons continued to be added at regular intervals. Water was used inside the pontoons as temporary ballast, placed in such a way as to mimic the final pontoon draft and to maintain the shear stresses and bending moments within acceptable limits. This was achieved by the complex ballasting procedure developed by the WRB Bridge Group. This involved pumping water in and out of specific locations every time the construction process required temporary or permanent weights to be added, removed or moved. The temporary construction ballast was replaced by permanent gravel ballast after the construction of the bridge was finalized.

16.4 The future

Section 16.2 identified a total of 11 floating bridge structures for road traffic of some size around the world. The structures located on the west coast of USA and Canada are all sideways anchored and have a length between 700 and 2'400 m while the Norwegian structures are only sideways fixed in the end and have a length between 900 and 1'250 m. Could these structures be made even longer and sustain even more severe loading?

In Norway, the Norwegian Public Roads Administration (NPRA) is currently planning a coastal route E39 whose objective is to connect Kristiansand to Trondheim without ferry crossings. This project has many great challenges due to the topography of the west coast of Norway, and especially crossing the fjords requires innovative thinking. Many of the Norwegian fjords are both deep and wide, which makes the crossing of them a great

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challenge. Traditional bridge design is thus not applicable. The widest fjords cannot be crossed with a single span bridge, and with depths varying from 450 m up to 1'300 m the bridge foundations is a major challenge. Due to these conditions, the NPRA in cooperation with the consultant industry in Norway have developed new fixed link concepts.

The most challenging crossing would be the Sognefjord connection, since at the chosen crossing site the fjord is 3'700 m wide and 1'250 m deep. Consequently a feasibility study was launched with the objective to demonstrate that it is feasible to cross Sognefjord with a bridge structure. If that were to be the case, then all remaining deep fjords could also be crossed by bridges. Following an initial phase of conceptual development in 2010, the conclusions were that the following concepts should be further addressed; a suspension bridge, a floating bridge, a submerged floating tunnel and a combination of the last two concepts. Late in 2011, an invitation was launched for companies to develop solutions for floating bridges, submerged floating tunnels and a combination of these. The aim was that through a "dialogue tender phase" this should result in teams being selected to do feasibility studies of the various concepts. At that stage, in spring 2012, a team consisting of the companies Aas-Jakobsen, Johs. Holt, Cowi, NGI and Skanska were chosen to perform a feasibility study of the floating bridge concept. Several concepts were investigated as a part of this study, both end- and side-anchored floating bridges and concepts with ship passage near the shore area, concepts with global load transfer below sea level and one floating cable-stayed bridge. But the final preferred concept for crossing the Sognefjord is a three-span suspension bridge supported partly on floating pontoons, as shown in Figure 16-21 Proposal for crossing the Sognefjord on the west coast of Norway[70]. A detailed description of this concept is given by Jakobsen et al.[71] and Larssen et al.[72]. The following information is partly based on these sources and direct communications with persons involved in the design.

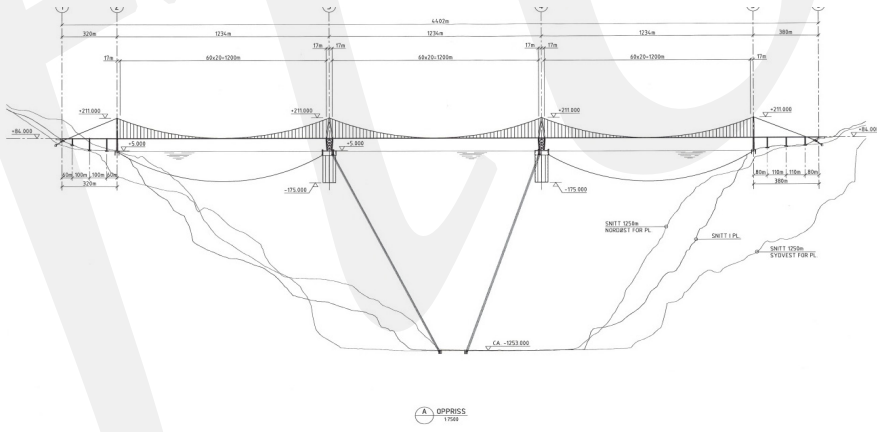


Figure 16-21 Proposal for crossing the Sognefjord on the west coast of Norway[70]

The length of each span is 1'234 m and the total bridge length is 4'402 m including viaducts on each side of the fjord. The bridge deck carries two lanes of traffic and one pedestrian/bicycle lane. The ship clearance height is 70 m. The main pontoons are anchored to the sea bed and to the shore through an anchoring system similar to that used in floating marine structures for the oil and gas industry.

The anchorage system consists of stay cables attached to the fjord sea-bed and to the pontoons. The main part of the cables are locked coil steel spiral strands for subsea use

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with a breaking load of approximately 22 MN, see Figure 16-22. Composite materials could also be used for this part. In the top and bottom part of the cable marine chains are used in accordance with common practice. Suction anchors are located at the sea-bed where the soil surface is relatively even and where there are thick layers of clay. The dimensions of the suction anchors are calculated to be approximately 18 m deep into the soil and with an outer diameter of 6 m. At the top, the anchor chains are fixed to the pontoon by using specially designed fairleads attached to the pontoon. The anchor force is controlled by adjustable jacks located on the top slab of the pontoon.

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

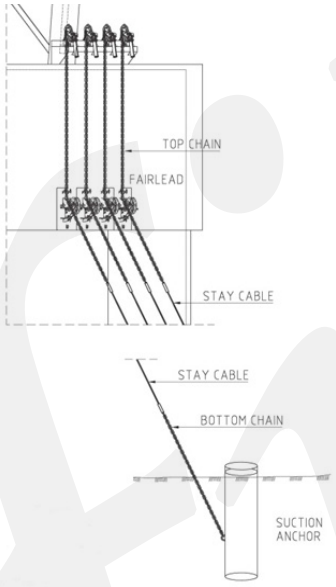


Figure 16-22 Anchorage system for the Sognefjord crossing[70]

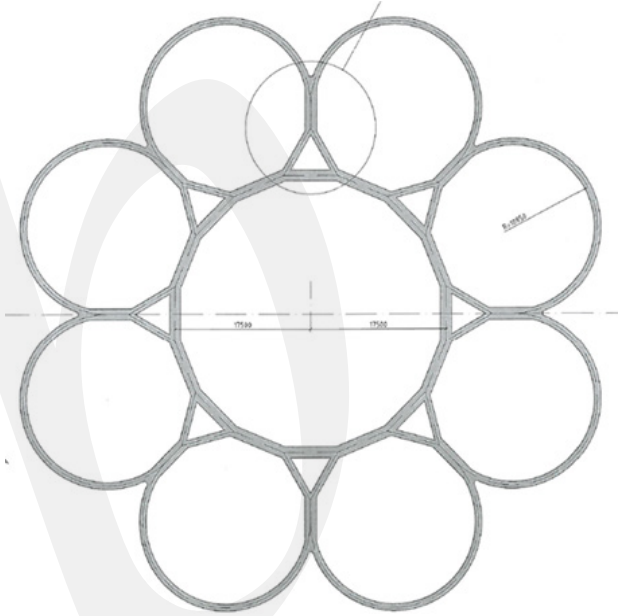


Figure 16-23 Pontoon section for the Sognefjord crossing[70]

The floating pontoons are concrete structures with a 9-cell cylindrical configuration with diameters from approximately 35 m for the centre cell to 11 m for the outer ones, see Figure 16-23. The draft is proposed to be 175 m with a freeboard of 7 m. In the top 25 m, the pontoon configuration is specially designed to resist and absorb ship collisions through several secondary walls and light weight aggregate concrete in-fills. The total concrete volume is 105'000 m^3. To ensure sufficient global stability, ballast comprising 155'000 m3 of olivine aggregate is used.

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

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

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The towers on pontoons will be steel towers, chosen to obtain minimum weight. The towers are four legged in order to be able to resist large bending moments in two directions, and with a diamond shape configuration. This shape is found to be optimal since there is a need for wide leg spacing at bridge deck level to limit deformations in the top of the towers and smaller leg spacing at the base in order to limit the size of the pontoons. The steel towers are 206 m high and the overall widths vary between 32 m at the base, up to 42 m below the deck, and down to 6.7 m at the top, see Figure 16-24. The steel towers are erected section by section using floating offshore cranes for the first sections and cranes mounted on the pontoons for the remaining sections. The towers on shore are concrete towers with the same outer geometry as the steel towers except being two instead of four legged. They may be constructed by means of a climbing scaffolding system or by slip forming.

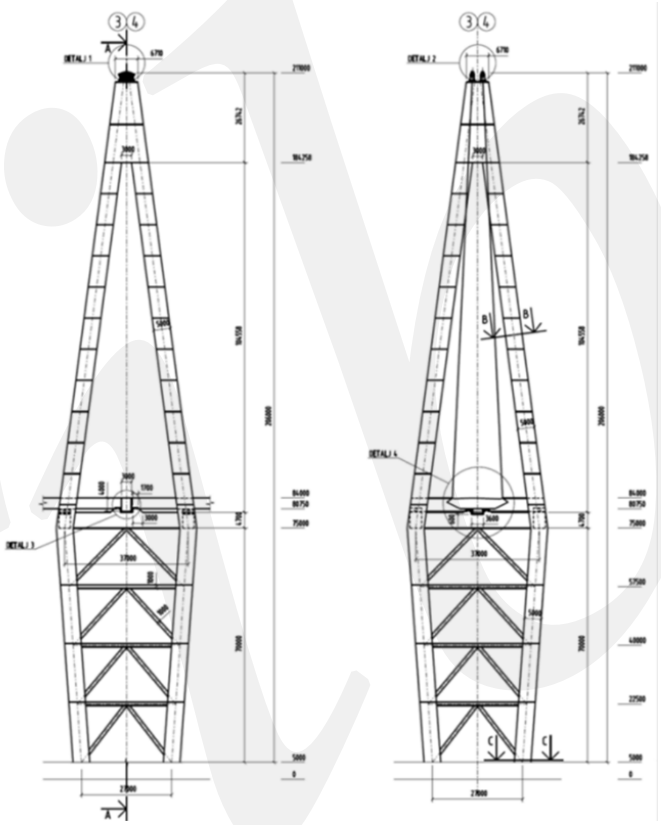


Figure 16-24 Tower layout for the Sognefjord crossing[71]

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

The main cables consist of 19 bundles with 384 wires each and with a wire diameter of 5.3 mm. The wire yield strength is chosen to be 1'770 MPa. Aerial spinning is proposed, but prefabricated cables are also considered to be a feasible solution. The main cables are fixed by traditional rock anchorages in the abutment areas. A visualization of the bridge proposal is shown in Figure 16-25.

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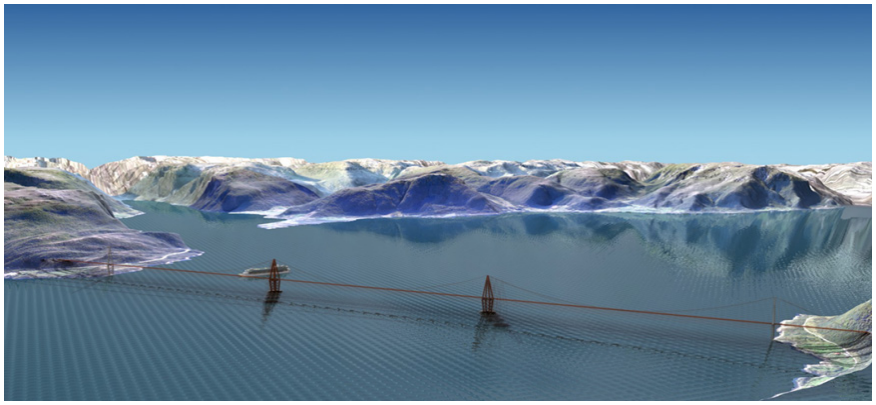


Figure 16-25 Visualization of the proposed floating suspension bridge for Sognefjord, Norway[70]

The most recent (2015-2017) proposal for a record breaking floating bridge in Norway is for the Bjørnafjord, just south of Bergen. This crossing is also a part of coastal route E39 and the crossing is scheduled to be the first crossing to be built with such a fixed link. The fjord is some 5 km wide, and has a depth of up to 550 m. These conditions make the crossing not feasible with traditional bridge design. With this challenge in mind three concepts have been developed: One is a floating pontoon bridge, either as a straight side-anchored bridge, see Figure 16-26, or as a curved end-anchored floating bridge, see Figure 16-27 and Figure 16-28. The third bridge concept is a multi-span suspension bridge on floating foundations, see Figure 16-29.

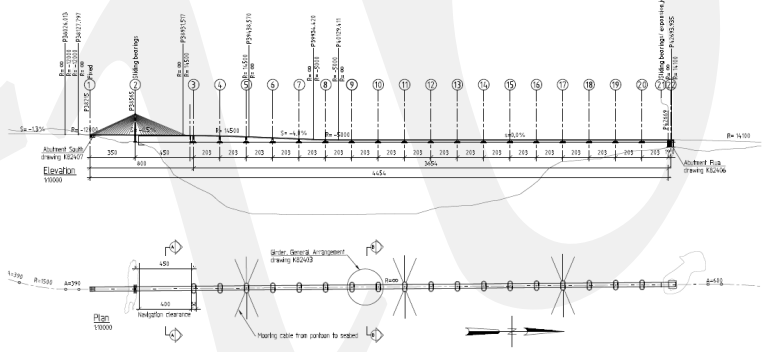


Figure 16-26 Overview of the proposed straight side-anchored low floating bridge for Bjørnafjord, western Norway[73]

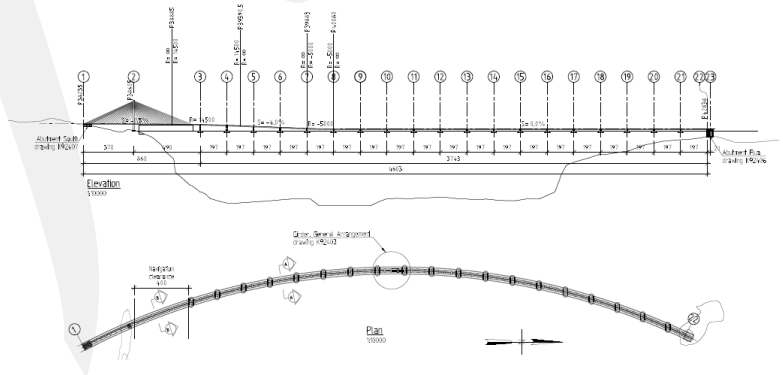


Figure 16-27 Overview of the proposed curved end-anchored low floating bridge for Bjørnafjord, western Norway[73]

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Figure 16-28 Visualization of the proposed curved end-anchored low floating bridge for Bjørnafjord, western Norway[73]

The design of floating bridges is moving to longer span and deeper water and to more severe loading environments. The development on the Norwegian west coast is motivated from combining the knowledge from bridge engineering with the knowledge from the offshore oil industry into a boost of the infrastructure development. The shift in depth and span length means taking new materials into consideration and to utilize the knowledge gained from other sectors into spectacular structures for the future. One consequence might be that the dominating building material for floating bridges no longer will be concrete.

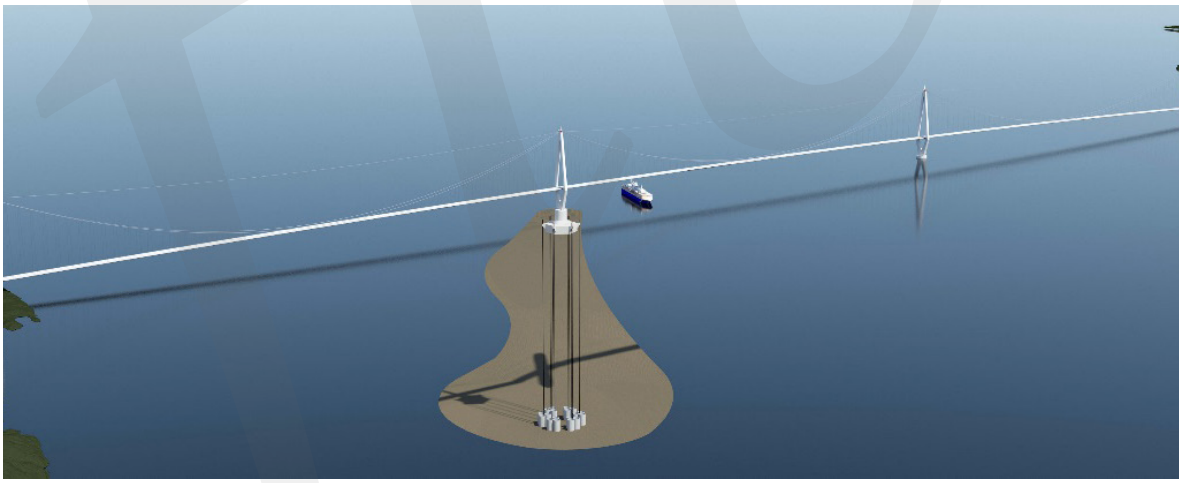


Figure 16-29 Visualization of the proposed multi-span suspension bridge on floating foundations for Bjørnafjord, western Norway[74]

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17. Floating platforms for the O&G industry

Most structures for the oil and gas offshore industry are made of steel. Many of them are located in shallow waters.

The harsh environment of the North Sea triggered the benefits of concrete platforms. Doris was the real offshore concrete pioneer with the Ekofisk platform in the early 1970s, built in Norway and utilizing the Norwegian fjords, followed by the Norwegian contractors and designers, as well as others.

The design specifications vary widely.

All of the platforms are floating, at least during construction. However, there are three which are permanently floating; the Tension leg platform Heidrun, the catenary moored Troll B, and the N´Kossa barge.

The floating platforms need to be anchored, either by tension legs or catenary. Anchoring is described in this report in some detail in the section for wind energy generators.

This chapter was mainly authored by Tor Ole Olsen.

doi.org/10.35789/fib.BULL.0091.Ch17

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18. Floating apartments

All projects with structures need a clear design specification, which is very important for floating structures and hopefully is understood from this report.

Offshore platforms for the oil and gas industry, hardly seen by anyone and very dependent on physical constraints, have very different design specifications as compared to floating apartments, where physical constraints are important in addition to aesthetic constraints.

The idea of floating communities in different scales are inspired by water lilies as balanced circular platforms,; from a single hut as well as circular organized village around a common square.

Perhaps this could be considered as a modern improvement to living near congested and expensive areas, or to save the communities living on the low lying islands that are washed away by the rising sea level or brutal storms.

A first sketch of a floating society might look like this:

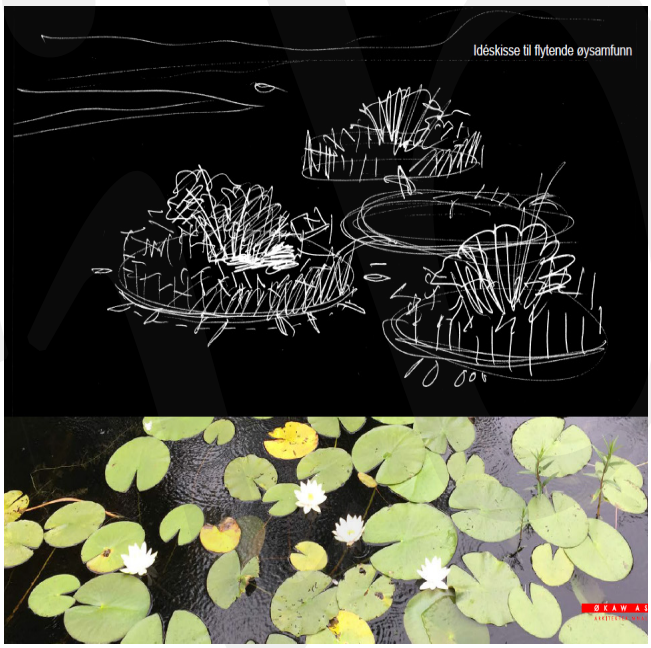


Figure 18-1 Inspiration, floating community[75]

The next examples are floating platforms for revitalization of the harbour in a small city, Tvedestrand, in the south of Norway.

Here the circle is used for a floating restaurant "The Lifebuoy", a floating scene "The Hood" and a floating bathing platform "The Swimming Ring" (Figures 18-2, 18-3 and 18-4).

This chapter was mainly authored by Tom Wike.

doi.org/10.35789/fib.BULL.0091.Ch18

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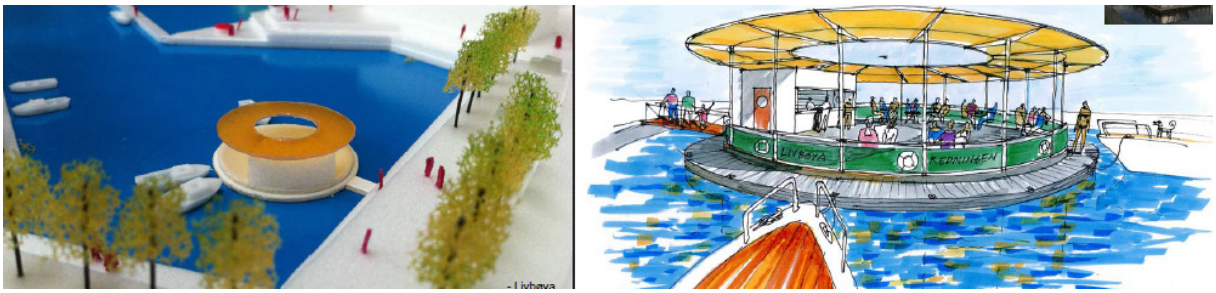


Figure 18-2 Sketches, floating facilities[75]

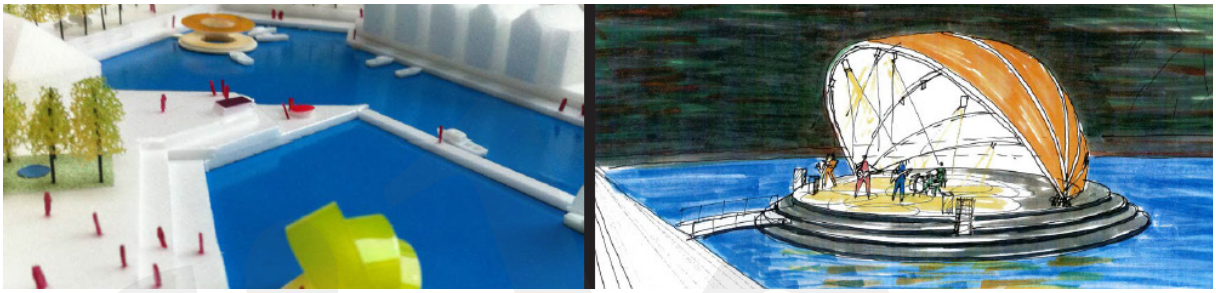


Figure 18-3 Floating facilities[76]

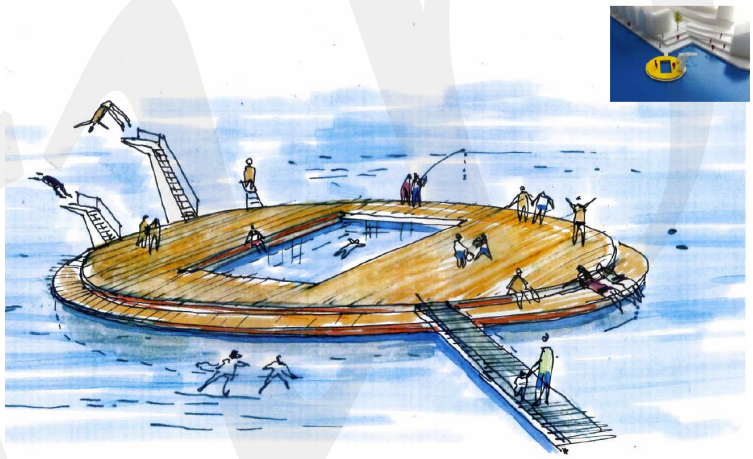


Figure 18-4 Floating leisure[75]

The next project illustrates in Figure 18-5 a floating community on a platform in the shape of a ship.

Two apartment blocks varying from two to eight stories high are placed alonside each other and leave interesting spaces in between them. The terraced roof areas are used for growing food as well as leisure for the residents.

The model in Figures 18-6 and 18-7 illustrates three "boat units" connected with bridges to a larger community, and at the photomontage of the island Langøya (Figure 18-8) the boats are organized in different ways around the Island.

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This island, which is a former chalk mine, has been used for disposal of polluted waste material for years, and is now being sealed. The idea is to use it as a "Leisure Island" with all kinds of entertainment, golf course and sport facilities.

What could be a more clean, efficient and beautiful "modern city", than to let the floating communities lie like boats around the re-established island in the Oslo fjord.



Figure 18-5 Floating apartments[75]

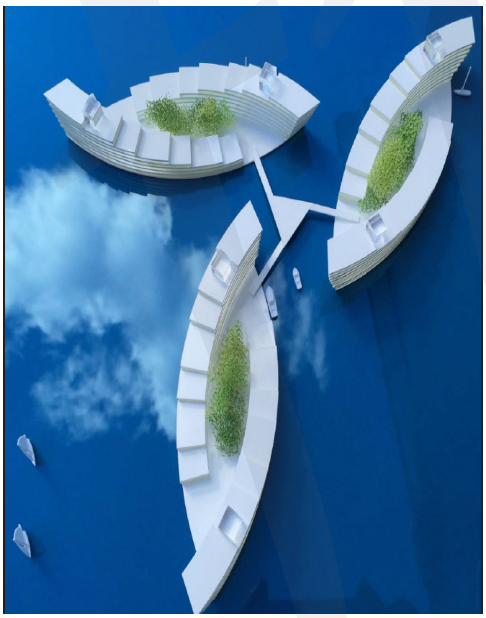


Figure 18-6 City development[77]



Figure 18-6 City development[75]

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Figure 18-8 System of floating apartments[75]

From creative ideas for the future to a completed project; Brockholes Visitor Centre, located in Lancashire, UK. This project was completed in 2011. It is not very large, but the challenges met and solved are very descriptive for a floating structure.

Adam Khan Architects won the design competition, with structural engineers Price & Myers on the team; the owner is Lancashire Wildlife Trust.

The Brockholes site comprises a 67 hectare triangle of land between a bend in the river Ribble, the M6 motorway and a steep escarpment covered with ancient woodlands (a Site of Specific Scientific Interest). Geographically a floodplain, the site was formerly a gravel quarry until the owner began its remediation. The quarry operator had excavated large pits, which were left to become shallow lakes, ideal for the wading birds the area attracts, and had piled up mounds of topsoil forming an artificial bund cutting off the site from the river. However, the site is still at risk of flooding and the Environment Agency has given it a 1 in 100 year flood risk of 3 metres inundation.

The Architect's vision of a "Floating World" was selected, as it provided unlimited flood protection and located the new Visitor Centre at water level where visitors would feel truly immersed in the wildlife.

At the Competition stage, the initial ground investigations showed that the site was covered by a thick layer of poor bearing capacity topsoil. To build on the land would have required a deep raft foundation and the thought was: "if we're going to build a raft foundation anyway, why not build a proper raft?" This would provide the required flood protection, with the cost and carbon comparable to the traditional foundations.

An important part of the design for Brockholes was that the owner wanted to aim for the highest environmental performance from the outset, which could be maintained through construction to completion and into operation.

Adam Khan's scheme was based on the Iraqi Marsh Arab floating villages, sadly destroyed in the 1990's by the draining of their Wetlands, where clusters of tall barn-type buildings were built on floating reed rafts.

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The Architect's "floating world" left the challenge of designing a 65 m by 42 m rigid buoyant base for the buildings, on a scale without precedent in the UK, with Adam Khan's stipulation that it had to float as close to the water level as possible.

Several initial ideas were schemed, including using either welded steel plate or thin-shelled Ferro-cement boxes, where deeper units would be used to counteract the areas of greatest loading. This developed into a more robust scheme of one large pontoon of uniform depth, standardising construction and better able to resist unbalanced forces.

Three options using different construction techniques were developed: the original steel box idea (1.6 m deep), steel trusses with plastic floats (3 m deep) and thick concrete boxes (2.25 m deep). These were sized for the required stiffness and appraised based upon their buildability, durability and quantity of material used. For equal stiffness, the steel options were up to four times more expensive than the concrete boxes and would require ongoing maintenance to prevent corrosion. The fabrication of the concrete boxes would also produce significantly lower C02 emissions than the steel options and could be produced by a wider range of Contractors, improving the chance of a more competitive tender.

The pontoon was designed to comply with BS8007- "Design of concrete structures for retaining aqueous liquids". However, the structure was not designed to prevent water ingress, but to ensure that any water penetrating the concrete was at a manageable rate. To eliminate the risk of the concrete cells ever filling up with water, the base was cast around large polystyrene void formers, which also simplified construction by acting as permanent formwork.

From information from suppliers of polystyrene it was discovered that standard-sized blocks cost almost half that of bespoke sizes, so the internal cell dimensions were based on multiples of the standard block. This saved time on site as the blocks required little site cutting or preparation and helped keep site wastage to a minimum, contributing towards the BREEAM credits.

The pontoon was modelled on a finite-element software package, "Oasys GSA", to predict how it would behave under the anticipated loadings. Given the irregular grid pattern, a complex Excel spreadsheet was written to output the positions of the 3'597 nodes and the 4'912 panel elements that defined the pontoon. The simulation modelled the scheme on over 1'100 springs, where the stiffness of each spring was directly proportional to the volume of water it displaced. This principle only worked where the springs were in compression as water cannot apply a tensile force. Fortunately, this never became an issue as the pontoon dead-load was sufficient to keep all parts of the pontoon "submerged" in all load cases.

The most onerous part of modelling the pontoon was meticulously calculating the weight of each building, without load factors, to determine the exact floatation level and stresses in the base. It was a frustratingly iterative process that had to be updated constantly throughout the design and construction stages. An error was discovered in the software where the concrete in any perpendicular wall and slab intersection would be counted twice, and although it seemed a very small amount of repeated material, without correction it would have raised the floating height of the pontoon by 40 mm.

The live loading was simpler to calculate, but we challenged the typical loads from British Standard BS6399-1. Designing for a blanket imposed load of 4.0 k N/m^2 would

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have required the pontoon to displace 400 mm of water, equating to over 10'000 people on the pontoon. Instead, we opted for a worse-case load of 2.0 k N/m^2, which was still very conservative given the car park would effectively limit visitor numbers to 400. The worst-case out-of-balance live loading was checked assuming the 400 people were crowded into any one corner of the pontoon. This always gave lower deflections than the 200 mm from the 2.0 k N/m^2 uniform load, satisfying Preston Building Control.

Having finished the model, the computer deflections and tilts were checked against rigid-body hand calculations, which matched surprisingly well. To combat the tilt created by the uneven building loads some of the base cells were designed to be without polystyrene, which could be filled with water to act as ballast tanks, trimming the pontoon. From the model, the exact volume of water required in each tank was calculated to ensure that the Centre would float as close to an even level as possible.

Pontoon drainage and services within the pontoon are connected to land via looped cables allowing for vertical movement while floating. A 250 mm high permanent service trench below the pontoon top slab was provided, through which the ductwork could be fed from hatches post-construction. The rain-water run-off from the buildings and open spaces discharges directly through the pontoon into the lake. At first, this principle seems counterintuitive and that penetrations in the raft would cause the pontoon to sink. However, by keeping the top level of the drainage pipes above the level of the lake the buoyancy is retained.

To prevent the pontoon drifting around the lake, two methods of securing it in place horizontally were schemed. The first was using large steel piles embedded in the lake bedrock which the pontoon would slide up freely, and the second was tethering it in place with large chains. The pile method was eventually chosen as it was felt that there would be too much lateral movement and maintenance with the chains. This meant that the sequencing had to be carefully considered, as the large mass concrete bases fixing the piles into the bedrock extend under the footprint of the pontoon, meaning that they had to be installed before works started on the base.

The piles were designed to withstand the wind and lake-current forces, with the pile section size reducing above the water level where the moments are lowest. The top of the piles are 3.2 m above the normal lake level, ensuring that the pontoon does not become free even in the highest flood condition. Steel H-frames with bearing pads were cast into the side walls of the pontoon, fixing around the piles and locking the Centre in place while allowing it to rise vertically.

To ensure year-round access to the Visitor Centre for all members of the public two bridges were designed to span onto the floating pontoon from mass concrete islands founded on the lake-bed. The bridges articulate allowing for seasonal adjustment of the lake-water level while remaining within a 1:20 slope in all conditions. Pinned at the pontoon and with rollers onto the islands the bridges could potentially "roll-off" in a significant flood event, so "catcher frames" have been designed to support the bridges before that would happen, ensuring that the Visitor Centre can settle back down unimpeded as the water recedes.

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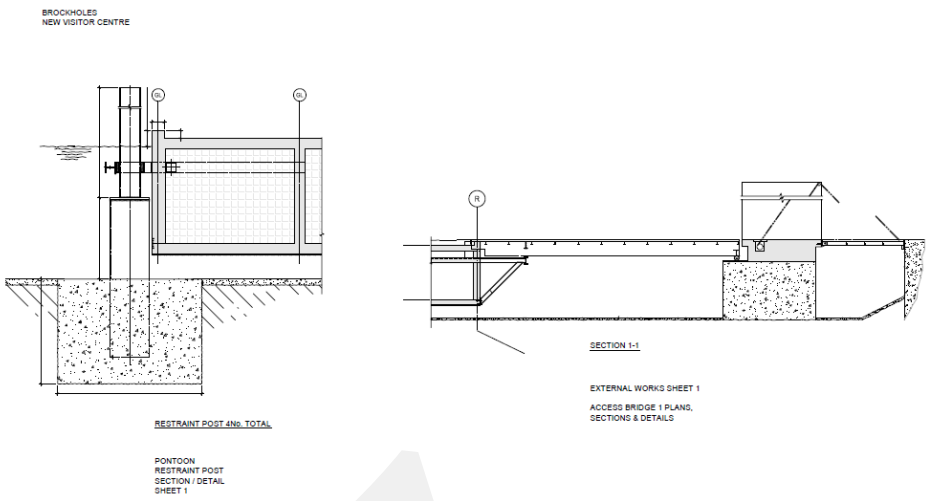


Figure 18-9 Bridge to floating centre

The car parking was designed to provide a range of different surfaces, from tarmac through to reinforced grass mesh, allowing for different user needs whilst reducing the visual and landscape impact of car parking. The concept for the parking design means that it could be expanded or reduced at will, leaving a natural infrastructure of trees, swales and vistas.

Design workshops, presentations and discussions were held with specific user groups throughout the design process, and the owner carried out Community consultation at early stages, assessing what impacts the proposed Centre would have in the area over time.

The owner has an extensive programme of community engagement projects and a large network of volunteers who continue to be involved in the everyday business of the centre. The aim is that the Centre will attract over 250'000 people annually from across the UK, and will provide a valuable resource for local communities to use, encouraging healthy, active lifestyles and increasing interaction with the natural environment. To date, it is on target to exceed projected visitor numbers in the first year of opening.

A large concern during the Design stage was that the potential for vandalism had to be designed-out. However, there have been no instances of vandalism to the visitor village, largely helped by the owner's commitment to community engagement: for example by working closely with the neighbouring housing estates over many years and running community angling schemes amongst other initiatives.

Investment was made into Brockholes because of the project's potential to become a high profile tourism gateway to the region, stimulating the economies of Preston and Lancashire. It has already created several dozen new local jobs, encouraged many new visitors to the area and dramatically improved an area of land previously classified as brownfield.

The project aimed for zero-carbon both in use and production, using materials of low embodied energy, high levels of thermal insulation and building airtightness, natural ventilation, and off-site prefabrication as well as on-site energy generation and waste treatment. The design was strongly influenced by the sourcing and durability of the materials used, potential for recycling, and the distance to site.

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While the BREEAM "Outstanding" target was an important driver, further environmental analysis was undertaken, including whole building embodied energy analysis, on-site sewage treatment analysis, low energy catering analysis and attendance of owner's own site specific ecologist throughout design and construction. Landscaping works were designed to create suitable habitats for planting, mud flats and reed beds to enhance site ecology and observable species.

Construction site impacts were minimised by monitoring and reporting energy, carbon and water use, implementing best practice policies for air, water and ground pollution, using an environmental materials policy and ensuring at least 80% of site timber was responsibly sourced and 100% was legally sourced.

Construction waste was minimized through a Site Waste Management Plan, achieving less than 5 tonnes of waste per 100 m2 of floor area and some 80% (by volume) of non-hazardous construction was diverted from landfill, recycled or reused.

The most efficient method of constructing the pontoon was to build it in-situ by partially draining the lake to create a dry-dock. This required significant earthworks yet improved buildability and removed the hazards of working in and around water. The calculated pontoon depth, 2.4 m deep in the final case, was greater than the shallow lake water depth, so the dry-dock would serve as a future permanent deeper basin in which the Centre would float.

To speed up construction of the pontoon the temporary formwork was simplified by designing pull-out starter bars into the base slab and cell walls. The Contractor (Mansell) was provided with a construction methodology, outlining the casting sequence of the concrete elements, where the least number of construction joints possible were expected in the external walls to limit the chance of water ingress. Due to the predominant edge loading on the pontoon, the top surface is in tension, so casting large parts of the top slab monolithically was crucial in providing a rigid base.

Towards the end of the construction phase a floating methodology was produced for the Contractor, advising on the lake refill rate as the pontoon approached buoyancy and specifying how much tilt was acceptable. It was also used to raise awareness of potential complications during flotation and possible solutions should they occur.

There was concerned that when the water was released back into the dry-dock the completed pontoon may stick to the bottom of the lake. The solution to achieve water pressure under the whole base was to cast a thin concrete screed over the whole dry-dock and then cast the base slab on top of an egg-crate cavity wall membrane. This provided the base with thousands of small "fee" which allowed the water to flow underneath unimpeded.

Filling the dry-dock took several days, and there was nervousness on site as the water was approaching the flotation level. Five laser targets were set up on the pontoon, one at the centre and one at each corner, and two EDM stations were used to monitor their positions every 15 minutes. As the water level rose and the corners started to lift off (to the great relief of the designers) on-site direction was provided, based on the feedback from the EDM readings, advising on additional water in the ballast tanks to level the pontoon and the speed of the water refill rate.

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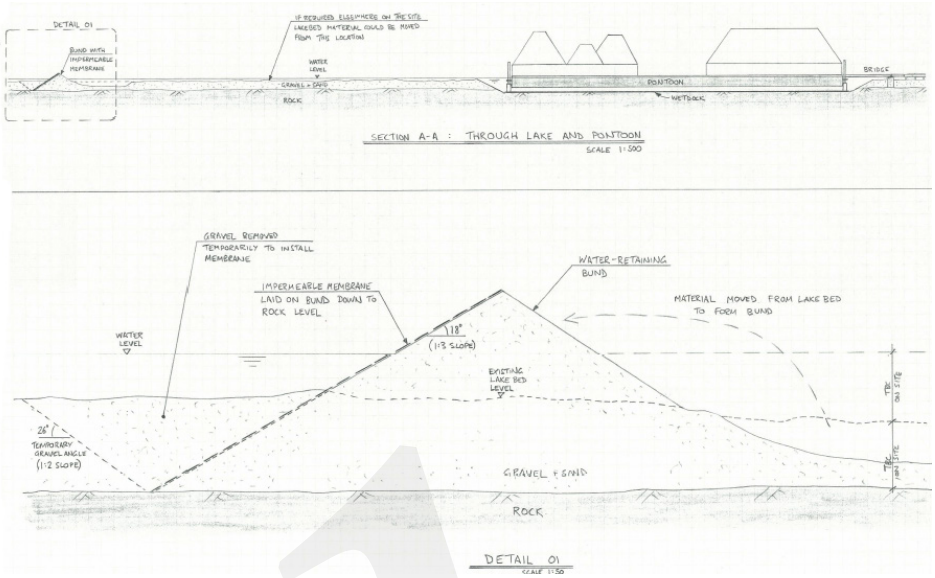


Figure 18-10 Establishing dry dock

Once completely floating, there was a slight difference in level across the pontoon, despite the ballast tanks being completely full. This was perhaps due to small inaccuracies in the loading calculations, or in the Centre's construction, so the required volume of heavier gravel ballast to be pumped into the tanks was calculated in order to trim the pontoon further until level.

The pontoon base is a beautifully simple concept but one that was very tricky to realize. The pontoon now floats on average just 150 mm above the water level, a huge engineering success, proving that the meticulous design work and constant re-checking paid off and that the Contractor built the structure very accurately. The low freeboard provides Adam Khan and the owner exactly what they wanted - a visitor experience where you get down to nature's level and feel part of the wildlife.

After two years of design and eighteen months of construction, the Centre opened in Easter 2011 to much acclaim. It was the first project in the country to be awarded interim BREEAM "Outstanding" and is currently being assessed at Post Construction Stage. The project achieved innovation credits for Considerate Constructors, daylight, low or zero carbon technologies, water metering and construction site waste management, and has already won numerous awards, including the Wood Awards 2011 and the Civic Trust Sustainable Building Special Award 2012. It has been presented at the Solid Wood Solutions Seminar and has featured in articles, such as the Architect's Journal, Building Magazine and the TRADA website. Since the project’s completion there has been a significant amount of interest from Architecture students, which has helped with their case studies based on Brockholes.

The design for Brockholes is simple but effective and most importantly, the solution is thought provoking to both professionals and members of the public alike. Particularly in light of its sympathetic approach to dealing with the effects of climate change this kind of project defines what being a Structural Engineer in the 21st Century is all about. Requiring creative and collaborative thinking, working with nature rather than against it, Brockholes provides an attractive, robust and cost effective public amenity with educational opportunities for people of all ages and backgrounds.

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Figure 18-11 Construction of wooden houses on barge



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Figure 18-12 Afloat

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19. Submerged floating bridges

A submerged floating bridge is a structure that spans through the water. It is similar to an immersed tunnel, except the immersed tunnel rests on the sea floor. A submerged floating tunnel is more favourable at larger water depth and/or where there is a rugged sea floor. Norway is one of few places with this type of topography.

While there are many immersed tunnels in the world, there are no submerged floating tunnels. But then there are not so many strait crossings over deep waters.

In Norway it has been decided for political reasons to build a ferry free road along the western coast. This means that 8 major strait crossings must be built from the following options: subsea tunnel, immersed tunnel, submerged floating bridge, floating bridge or bridge. These options are illustrated in Figure 19-1.

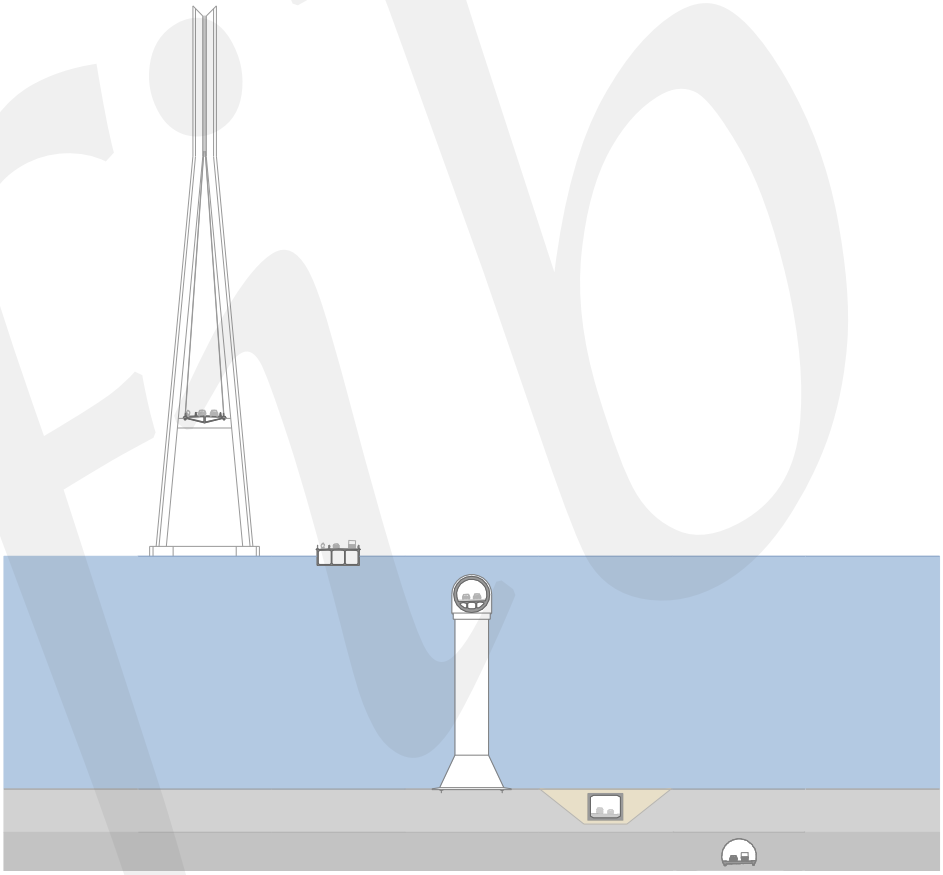


Figure 19-1 Immersed tunnel[11]

The west coast of Norway is rather rugged with deep fjords. The Sognefjorden, for example, is 1.3 km deep and 3.7 km wide at the place of crossing. Obviously many of the traditional means of crossing are not easily adaptable. In this case a submerged floating tunnel is proposed, held in position beneath the surface in the vertical direction, and arching in the horizontal direction.

This chapter was mainly authored by Tor Ole Olsen.

doi.org/10.35789/fib.BULL.0091.Ch19

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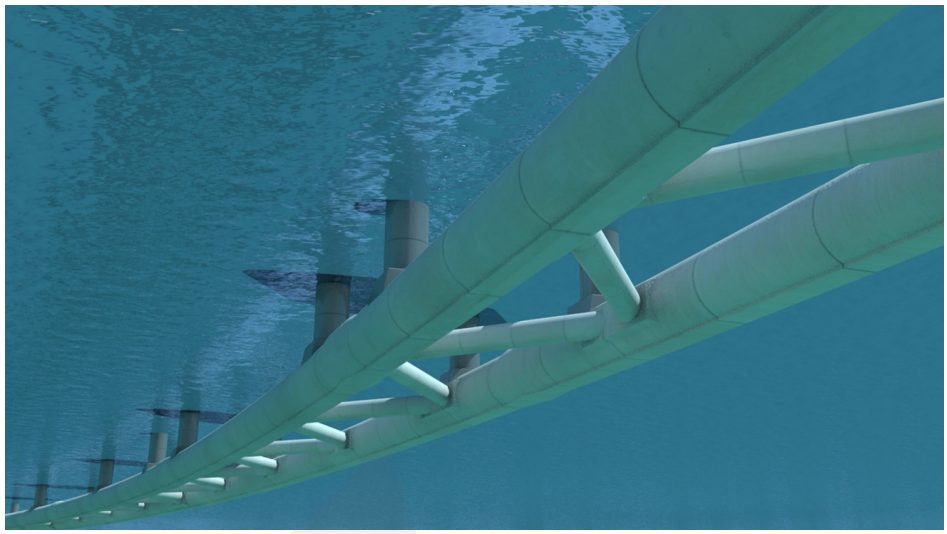


Figure 19-1 Immersed tunnel[11]

At more shallow crossings it is easier to anchor in vertical direction via tension legs or catenary cables to anchors placed at the sea floor. Below is the proposed submerged floating bridge at Bjørnafjorden:

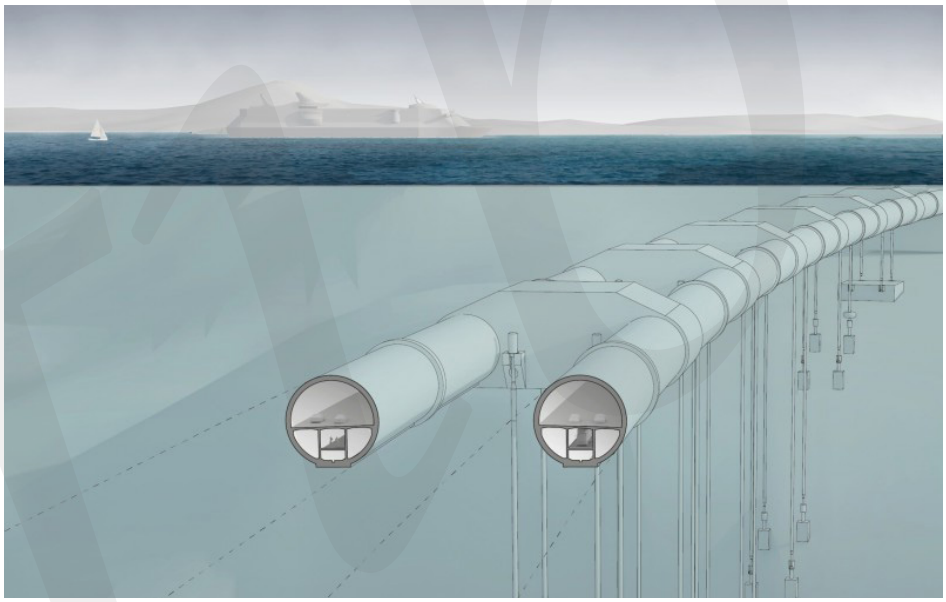


Figure 19-3 The proposed submerged floating bridge at Bjørnafjorden[78]

A subsequent *fib* report will deal with submerged floating tunnels in more detail.

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Floating concrete structures

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ISBN 978-2-88394-134-2

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ISSN: 1562-3610