Statens vegvesen

Ferry free E39 –Fjord crossings Bjørnafjorden | 304624

0	18.06.17	Final issue	JoS	SAH	JoS	
Rev.	Publish date	Description	Made by	Checked by	Project appro.	Client appro.

Client Statens vegvesen	
Contractor Norconsult DR. TECHN. OLAV OLSEN AkerSolutions	Contract no.: 16/40194

Document name: K7 Bjørnafjorden End-anchored floating bridge Summary report		
Document no.: SBJ-30-C3-NOR-90-RE-100	Rev.: 0	Pages: 63

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Summary

Statens vegvesen (SVV) pursues the development of a permanent link over the Bjørnafjord through parallel studies comprising both floating and submerged floating bridge concepts. In this third phase of the concept development, three alternatives remain under consideration; end-anchored floating bridge, side-anchored floating bridge and multi-span suspension bridge on TLP foundations.

This report briefly summarizes the work performed for end-anchored floating bridge (designated K7). For detailed documentation, reference is made to the Detailed Appendices A – Q.

The project has been executed by Norconsult AS and sub-consultants Dr.techn Olav Olsen and Aker Solutions. Other contributors are Institute for Energy Technology (IFE) and Aas-Jakobsen.

The scope of the project was to assist SVV in developing a basis for selecting the alternative(s) for further planning and engineering. To make the correct choices, it is important to investigate any feasibility issues and perform analyses and design to give confidence in the performance, quantities and construction methods proposed in this study. Key tasks were:

- Global analyses including sensitivity studies
- Prediction of aerodynamic loads
- Ship impact analyses, investigation of local and global effects
- Fatigue analyses
- Design of structural elements
- Corrosion protection of steel structures
- Engineering geology evaluations

Reducing cost and uncertainties has been governing for the Phase 3 Concept Development Process. Simplifications of structural components and reduction of loads and load effects have therefor been essential. This has led to a significant reduction of material quantities and a simplified structure compared to Phase 2, without compromising the robustness of the structure.

The proposed solution consists of the following structural parts:

- Abutments in both ends on dry land
- Tower for cable stayed part of the bridge on dry land
- Bridge girder, continuously from abutment to abutment supported vertically by cable stays in ship channel in south, and by pontoons in the floating part.
- Steel pontoons and steel columns

There is no anchoring to seabed, no expansion joints and no bearings. All foundations sit on solid rock above sea level.

The concept meets all requirements in the Design Basis.

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1 Introduction

1.1 Project context

Statens vegvesen (SVV) has been commissioned by the Norwegian Ministry of Transport and Communications to develop plans for a ferry free coastal highway E39 between Kristiansand and Trondheim. The 1 100 km long coastal corridor comprise today 8 ferry connections, most of them are wide and deep fjord crossings that will require massive investments and longer spanning structures than previously installed in Norway. Based on the choice of concept evaluation (KVU) E39 Aksdal Bergen, the Ministry of Transport and Communications has decided that E39 shall cross Bjørnafjorden between Reksteren and Os.

SVV is finalizing the work on a governmental regional plan with consequence assessment for E39 Stord-Os. This plan recommends a route from Stord to Os, including crossing solution for Bjørnafjorden, and shall be approved by the ministry of Local Government and Modernisation. SVV pursues the development of a permanent link over the Bjørnafjord through parallel studies comprising both floating and submerged floating bridge concepts. In this third phase of the concept development, three alternatives remain under consideration; end-anchored floating bridge (K7), side-anchored floating bridge (K8) and multi-span suspension bridge on TLP foundations (K1).

1.2 Project team

The project has been executed under a frame agreement for bridge engineering between Statens vegvesen Region vest and Norconsult. Norconsult is the largest multidiscipline consultant in Norway, and is a leading player within engineering for transportation and communication.

The team has been strengthened with selected subcontractors who are all highly qualified within their respective areas of expertise. Here we find Dr.techn Olav Olsen, who have been involved on many of the fjord crossing studies in Norway and Aker Solutions, a global supplier of products and services within offshore oil and gas.

Besides the companies mentioned above, the following have contributed to the team:

- Tor Anders Nygaard with IFE, PhD in aerodynamics and the developer of the aero-servo-hydro-elastic FEM code 3DFloat which is tailored for nonlinear, coupled time-domain simulations.
- Aas-Jakobsen, who has been a link between phase 2 and phase 3 development projects and a resource on analysis tasks.

1.3 Project scope

The scope of the project is to assist SVV in developing a basis for selecting the alternative(s) for further planning and engineering. In order to make the correct choices it is important to investigate any feasibility issues and perform analyses and design to give confidence in the quantities and construction methods proposed in this study. Key tasks are:

- Global analyses including sensitivity studies
- Prediction of aerodynamic loads
- Ship impact analyses, investigation of local and global effects
- Fatigue analyses

- Design of structural elements
- Corrosion protection of steel structures
- Engineering geology evaluations

1.4 Project development

Reducing cost and uncertainties has been governing for the Phase 3 Concept Development Process. Simplifications of structural components and reduction of loads have therefor been essential. This has led to a significant reduction of material quantities and a simplified structure compared to Phase 2, without compromising the robustness of the structure.

The Development Phase 3 (compared to Phase 2) can be described by the following:

- Load Reduction
  - Pontoon spacing reduced from 197 m to 100 m
  - Static loads from self-weight reduced by about 80%
  - Consequently, bridge girder height could be reduced from 6,5 to 3,5 meters
  - Reduced height and more aerodynamic shaped box girder
  - Horizontal forces from wind reduced by about 70%
  - Consequently, the width of the bridge girder was reduced from 55 to 27,6 meters
  - The twin box girder was changed to a single box girder
  - Torsion from traffic was significantly reduced
  - Required roll stiffness from pontoons was reduced
  - Steel pontoons selected leading to reduction in displacement
  - Reduced displacement leads to reduction in wave loads
  - Turning the arch with apex towards east gives axial tension from dominant western wind
  - Consequently, arch compression is reduced

- Simplification – Structural Components
  - Simple single box structure well known from cable-stayed and suspension bridges
  - Circular steel columns known from offshore practise
  - Steel pontoons known from ship and offshore practise
  - Lateral connetion/bearings between bridge girder and stay cable tower removed

- Simplification – Change of Road Alignment
  - Complex and costly underwater foundation on Flua avoided
  - Approach bridge from Flua to shore avoided
  - Shifting of North landing avoided conflict with recreational area
  - No interface with seabed
  - No geotechnical issues
  - All foundations on solid rock above water line

During design, extensive parametric studies have been executed, both by simplified calculations and by advanced time domain analyses. The parametric studies covered variation of e.g:

- Bridge overall length
- Arch radius and direction
- Stiffness

- Pontoon material, volume and mass
- Pontoon spacing

These studies have given us valuable knowledge about the structural performance of the bridge and how each parameter influences the structural behaviour. It has also given us valuable information and understanding of the influence of the different loads acting on the structure.

2 Design approach

2.1 Concept development and chosen solutions

A broad range of parameter studies has been performed and choices has been made based on these studies. The effects of the parameter on a specified response are qualitatively summarized in chapter 2.2. The main parameter variations that led to concept solutions are:

- Road alignment and curvature
- Bridge girder cross-section
- Pontoon spacing
- Pontoon material

These parameters and chosen solutions are described more in detail below.

2.1.1 Road alignment and curvature

There were several objections to the proposed Phase 2 road alignment:

- Expensive and complicated underwater foundation on Flua. Water depth 40 – 50 meters and large forces. Requires large caisson.
- Expensive and complicated approach bridge from Flua to shore. Water depth 70 – 90 meters combined with tight curve, makes it difficult with large spans. Expensive foundations. High risk.
- Bridge approaches a valuable recreation area. High impact on environment.
- Bridge arch pointing at west, gives compression from dominant westly wind.

Parameter studies concerning the road alignment were performed:

- Effect of longer bridge, avoiding Flua, with abutment on shore in north. Increased natural periods, and some increase in wind response due to higher energy in the wind spectrum at higher periods. Effect is compensated by using a lower aerodynamic box. See below.
- Different arch radius investigated. Higher radius gives higher axial load. This affects the stiffness to some extent, but not significant effect on response.
- Arch pointing at east. Effect, as described above, is reduced. Positive effect on static buckling from mean wind. Positive effect on the road alignment further north.

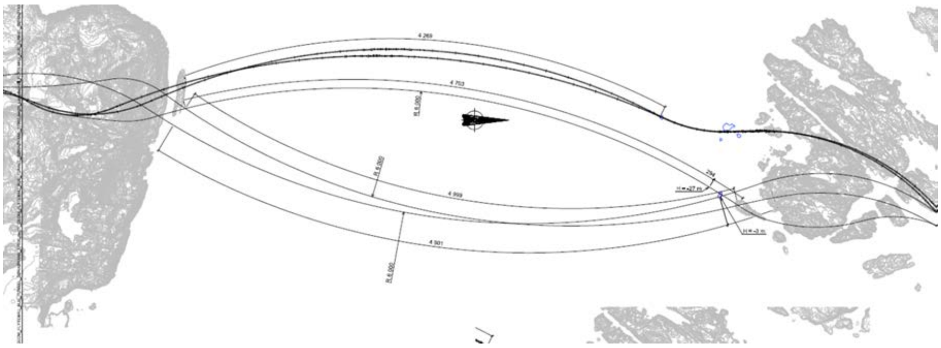


Figure 2-1 Different road alignments investigated

Conclusions:

- Bridge spanning from shore to shore without deep water foundation on Flua is chosen due to assumed cost saving and reduced risk. Abutment North placed on islet Gulholmen.
- Arch pointing at east is chosen due to increased buckling safety and favorable road alignment further north.

2.1.2 Bridge girder cross section

Influence of box height and drag factors on the wind response has been investigated, running analyses with 3 different cross sections:

- The twin box from Phase 3.
- A suspension bridge single box with height 4,0 meters and drag factor ≈ 0,8
- A cable stayed bridge single box with height 3,5 meters and drag factor 0,53

The effect is very important and reducing the box height and drag factor is considered as a precondition for an end anchored bridge at this size.

The low drag factor of 0,53 has been questioned. The factor was determined in Phase 3 based on CFD analyses. We have run more detailed 2D and 3D CFD analyses which confirm the drag factor.

Consequently, the 3,5-meter-high single box is chosen due to remarkable reduced wind response.

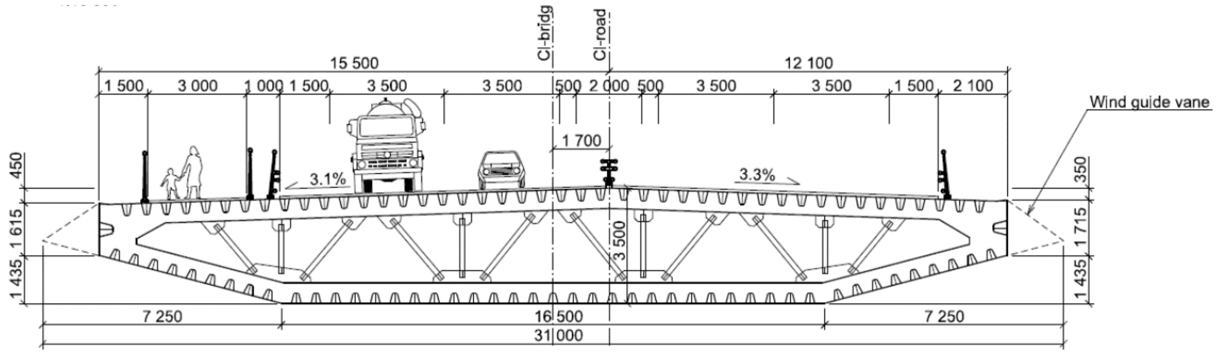


Figure 2-2 Chosen general cross-section

Plate dimensions must be chosen based on the load actions. As minimum plate thicknesses, recommendations in NS-EN 1993-2 is adopted. The minimum thickness of the deck plate is chosen to 14 mm. In bottom plates 12 mm are chosen as minimum. The stiffener arrangement is also chosen from recommendations in NS-EN 1993-2. Trapezoidal stiffeners divide the deck plates into 300 mm wide parts. This arrangement is also chosen in the bottom plates. Normally, trapezoidal stiffeners are made of cold formed plates with minimum 6 mm thickness. This means that there is a minimum quantity of steel in this type of structure. The minimum quantity for this structure is approx. 11,7 ton/m including transverse bulkheads. The cross-section chosen for this concept has an average weight of 13,7 ton/m.

2.1.3 Pontoon spacing

The pontoon spacing is the most important parameter regarding the load actions from permanent loads. As the bridge acts like a continuous beam on fixed supports for permanent loads, the bending moment is proportional with square of the span length. If box height is constant, the steel amount will vary accordingly.

Analyses has been run with two different span lengths (pontoon spacings):

- Span length 150 m
- Span length 100 m

If the cross-section is considered unchanged, the pontoon displacement is increased with a factor approx. 1,5 when span length is increased similar. Also, the hydrodynamic loads will increase the same. The effect of this is complex and cannot be concluded unconditional. Figure below shows the effect on the bending stresses in the 4-meter-high box with 100 and 150 meters span. The pontoons are semi type pontoons with two columns.

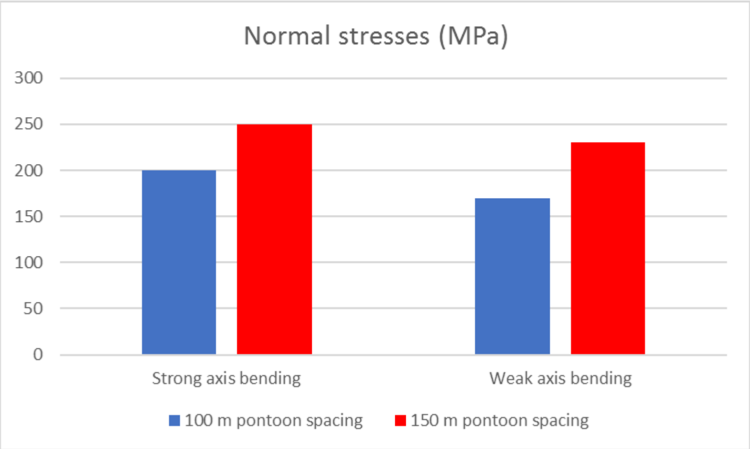


Figure 2-3 Stresses from wind sea for two different pontoon spacings

The figure indicates that stresses will increase when pontoon spacing increases. For the pontoons chosen for this parameter variation, an increase of ≈ 25% on strong axis bending and ≈ 35% on weak axis bending can be expected. The increase may be reduced due to pontoon/girder optimization. Tentatively the increase is assumed as the root of the span increase.

The figure below indicates the correlation between steel girder steel weight and span length based on the following assumptions:

- Box girder height is constant 3,5 meters
- Increase of permanent bending moment equal to square of increase of span
- Increase of bending moment from wind sea equal to the root of increase of the span
- Characteristic values from permanent bending and wind sea are typically equal and about 80 MPa for an optimized concept.
- All other load actions unchanged.
- Load factor 1,2 used for permanent load and 1,6 for wave load.

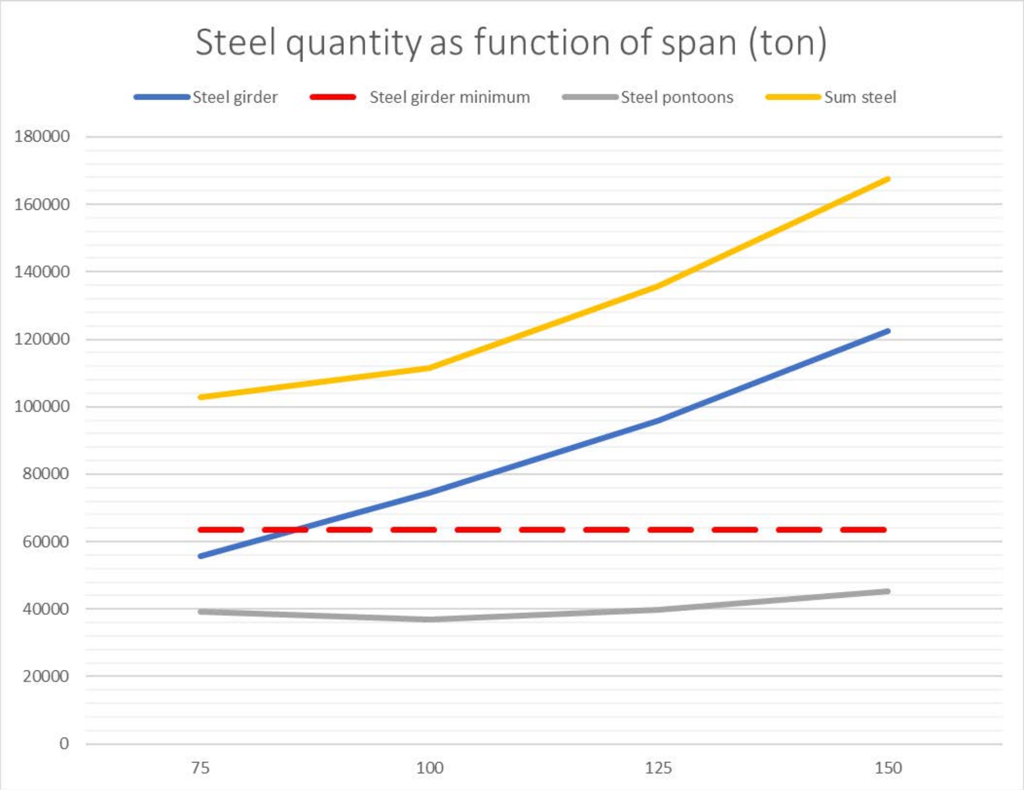


Figure 2-4 Estimated steel weight as function of span length

The above figure indicates that 100 m is close to optimum for the chosen cross-section. A reduction of the span length gives some reduction in steel quantity for the girder. However, at about 85 m span, the cross-section cannot be reduced any further due to minimum thicknesses of plates and stiffeners. This is indicated by the red-dot line. A reduction in steel quantity will also reduce bending capacity about strong axis and reduce buckling safety. 100-meter span is therefor chosen.

2.1.4 Pontoon material

Both floating bridges build in Norway have pontoons made of concrete. In Phase 2 of the concept study, the proposal was also concrete pontoons. However, there is extensive experience with steel pontoons from the offshore industry. Pontoons in steel have clearly some advantages compared to concrete. The most obvious is the weight. Using concrete for the pontoons means that most of the buoyancy is used to carry the weight of the pontoon itself. This means that a concrete pontoon has much larger displacement than a steel pontoon. Since wave excitation is proportional to displacement, the use of steel pontoons will reduce the hydrodynamic loading.

The figure below shows a parameter study where both steel and concrete pontoons are analysed. Both pontoon types are semisubmersible pontoons with two columns. Pontoon spacing is 100 meters.

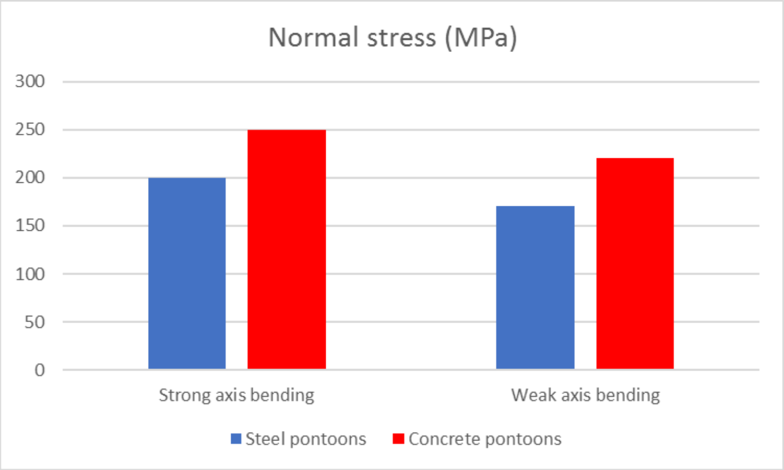


Figure 2-5 Stresses from wind sea for steel and concrete pontoons

The figure shows that the girder response will increase when concrete pontoons are used. For the pontoons chosen for this parameter study, an increase of 25% till 30% can be expected.

Based on the simple fact that large concrete pontoons give remarkably higher response in the bridge girder, steel pontoons are chosen.

2.2 Parameter influence matrix

In the concept development phase, a lot of parameters were investigated. The influence of the various parameters on a specified response are qualitatively summarized in tables below. The colors indicate sensitivity; Yellow and red means that the parameter has large effect on the specified response. Green and dark green means that the parameter has little or insignificant effect on the specified response.

	GENERAL CONFIGURATION		
	Length	Radius	Pontoon spacing
WIND RESPONSE IN GIRDER	Some increase with increased length due to higher periods	Some increase with reduced radius due to higher periods	No significant effect
WIND DRIVEN WAVE RESPONSE IN GIRDER Tp=3-6 seconds	No significant effect	No significant effect	Increase with larger spacing
SWELL RESPONSE Tp=12-18 seconds	Some increase with increased length due to higher periods	Some increase with reduced radius due to higher periods	Some increase with larger spacing
TRAFFIC RESPONSE IN GIRDER	No significant effect	No significant effect	Some increase with larger spacing
TRAFFIC RESPONSE IN PONTOONS	No significant effect	No significant effect	Some increase with larger spacing
DEAD WEIGHT RESPONSE IN GIRDER	No significant effect	No significant effect	Increase with larger spacing
TIDE EFFECT ON GIRDER TOWARDS SHORE	No significant effect	No significant effect	Some decrease with larger spacing

(red)	Important parameter that strongly affects the specified response
(yellow)	Important parameter that significantly affects the specified response
(light green)	Parameter that less significantly affects the specified response
(green)	Parameter that has no significant effect on the specified response

Table 2-1 General configurations impact on response

	BRIDGE GIRDER				
	Horizontal properties (sway)		Vertical properties (heave)		Torsional properties (roll)
	Width	Stiffness	Height	Stiffness	Stiffness
WIND RESPONSE IN GIRDER	Increase with reduced width due to reduced stiffness	Increase with reduced stiffness due to higher periods	Increase with increased height due to higher wind load	No significant effect	Effect depends on waterplane stiffness
WIND DRIVEN WAVE RESPONSE IN GIRDER Tp=3-6 seconds	No significant effect	No significant effect	Some increase with increased height due to increased stiffness	Some increase with increased stiffness	Effect depends on waterplane stiffness
SWELL RESPONSE Tp=12-18 seconds	Some increase with reduced width	Some increase with reduced stifness due to higher periods	Some increase with increased height due to increased stiffness	Some increase with increased stiffness	Effect depends on waterplane stiffness
TRAFFIC RESPONSE IN GIRDER	No significant effect with single box	No significant effect	Effect depends on waterplane stiffness	Effect depends on waterplane stiffness	Effect depends on waterplane stiffness
TRAFFIC RESPONSE IN PONTOONS	No significant effect with single box	No significant effect	Effect depends on waterplane stiffness	Effect depends on waterplane stiffness	Effect depends on waterplane stiffness
DEAD WEIGHT RESPONSE IN GIRDER	No effect if weight is not increased	No effect if weight is not increased	No effect if weight is not increased	No effect if weight is not increased	No significant effect
TIDE EFFECT ON GIRDER TOWARDS SHORE	No significant effect	No significant effect	Increase with increased height due to higher stiffness	Increase with increased stiffness	No significant effect

Important parameter that strongly affects the specified response
Important parameter that significantly affects the specified response
Parameter that less significantly affects the specified response
Parameter that has no significant effect on the specified response

Table 2-2 Bridge girder properties impact on response

	PONTOONS			
		Heave	Pitch i.e. torsion in girder	Roll i.e. weak axis bending in girder
	Mass	Stiffness	Stiffness	Stiffness
WIND RESPONSE IN GIRDER	Increase with increased mass due to higher periods	Some heave reduction with increased stiffness	Some torsion reduction with increased stiffness	No significant effect
WIND DRIVEN WAVE RESPONSE IN GIRDER Tp=3-6 seconds	Increase with increased mass due to higher excitation and periods	Increase with increased stiffness	Some torsion reduction with increased stiffness	Some increase with increased stiffness
SWELL RESPONSE Tp=12-18 seconds	Increase with increased mass due to higher excitation and periods	Some heave reduction with increased stiffness	Some torsion reduction with increased stiffness	No significant effect
TRAFFIC RESPONSE IN GIRDER	No significant effect	Increased hogging moment with increased stiffness	No significant effect	No significant effect
TRAFFIC RESPONSE IN PONTOONS	No significant effect	Reduced displacement with increased stiffness	Reduced torsion rotation with increased stiffness	No significant effect
DEAD WEIGHT RESPONSE IN GIRDER	No significant effect	No significant effect	No significant effect	No significant effect
TIDE EFFECT ON GIRDER TOWARDS SHORE	No significant effect	Increase with increased stiffness	No significant effect	No significant effect

Important parameter that strongly affects the specified response
Important parameter that significantly affects the specified response
Parameter that less significantly affects the specified response
Parameter that has no significant effect on the specified response

Table 2-3 Pontoon properties impact on response

3 Concept description

3.1 General|

The End-anchored floating bridge across the Bjørnafjorden is split into the following major structural elements (sub-systems):

- Bridge girder (carriage way)
- Pontoons
- Columns (between pontoons and bridge girder)
- Abutment South (south support of bridge)
- Cable-stayed bridge (tower, cables)
- Abutment North (north support of bridge at Gulholmen)
- Filling and approach bridge North of Gulholmen

An overview of the end-anchored bridge is shown in Figure 3-1. The structural parts are more thoroughly described in the following chapters and in the respective Appendices.

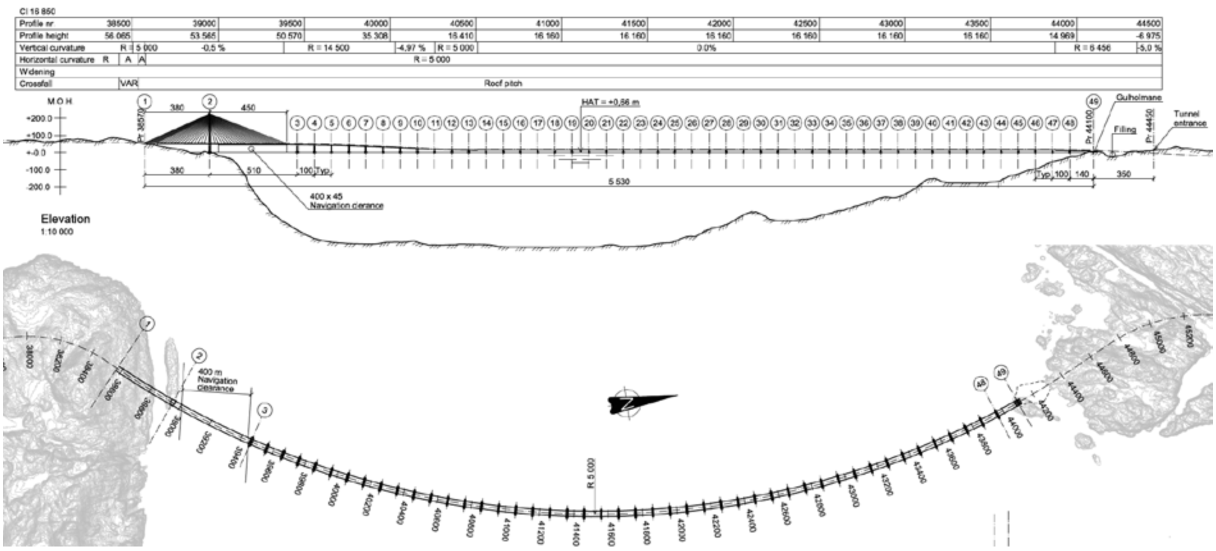


Figure 3-1 Overview of end-anchored floating bridge

3.2 Key Figures

Bridge length between abutments	5439	m	
Length cable stayed part	769	m	
Length floating bridge part	4670	m	
Pontoon spacing	100	m	
Number of pontoons	46	m	
Width of box-girder structure	27,6	m	
Width including wind guide vanes	31,0	m	

Steel quantity in girder	74 369	ton	(13,7 ton/m)
Steel quantity in columns	5 610	ton	
Steel quantity in pontoons	31 400	ton	
Total amount of structural steel	111 379	ton	
Concrete in pylon	12 634	m^3	
Concrete in pylon foundation	4 725	m^3	
Concrete in abutments	13 500	m^3	
Stay cables	1 552	ton	
Rock blasting	93 000	m^3	
Rock fill	260 000	m^3	

3.3 Bridge girder

The bridge girder is defined as the steel box from abutment south to abutment north.

3.3.1 General description

The structural part of the bridge girder is a single steel box, 27,6 meters wide and 3,5 meters high. The cross section is build up from panels stiffened by trapezoidal stiffeners. The deck plates are 14 mm thick, the bottom plates 12 mm thick. The spacing between stiffeners are 600 mm. The vertical web plates are 35 mm thick, stiffened by a single trapezoidal stiffener. The panels are supported on transverse beams with a general spacing of 4 meters.

The cross section is constant without variation in the low bridge (axis 13 to axis 48), and in the cable stayed part of the bridge. In the high bridge from the nose of the cable stayed bridge to axis 13, the cross section is strengthened using heavier trapezoidal stiffeners. The bridge girder is also strengthened locally at the end supports and at the column connections.

In compliance with the Design basis, the steel grade in all structural plate elements are S420 N according to NS-EN 10025-3.

The general cross section is showed below:

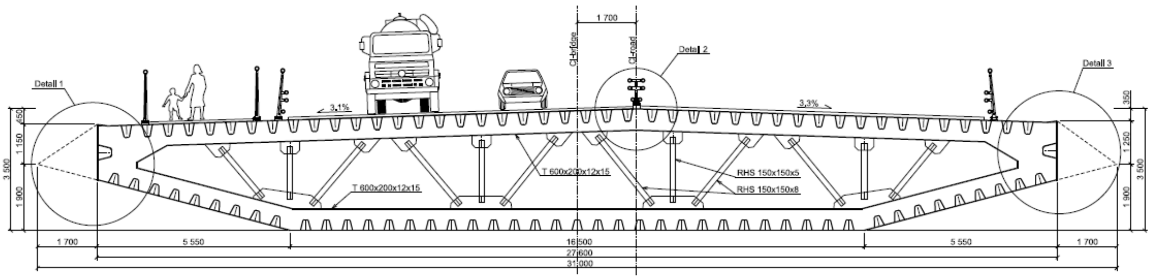


Figure 3-2 Bridge cross section

To ensure access for welding and inspection in the outercorners of the girder, a vertical web-plate with one single stiffener is used. The aerodynamic shape of the box girder will be ensured using guide vanes with no influence on structural behavior.

Table 3-1 shows the variation of plates and stiffeners along the bridge. This is also shown on drawings -012, -021, -031, -072 and -073.

Table 3-1 Variation of steel plates and stiffeners along the bridge. S1-S5 denotes stiffener types.

	X - coordinate	Cross section type	Deck plates	Vertical web plates	Bottom plates
Cable stayed bridge	0 – 679 m	Type 1	14 mm S1	35 mm S5	12 mm S3
High bridge	679 – 1757 m	Type 2	14 mm S2	35 mm S5	12 mm S4
Low bridge	1757 – 5269 m	Type 1	14 mm S1	35 mm S5	12 mm S3
Low bridge	5269 – 5439 m	Type 2	14 mm S2	35 mm S5	12 mm S4

As seen from the table above, the bridge is divided into 3 typical parts. These are shown below.

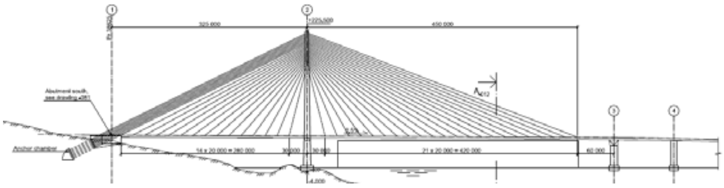


Figure 3-3 Cable stayed bridge

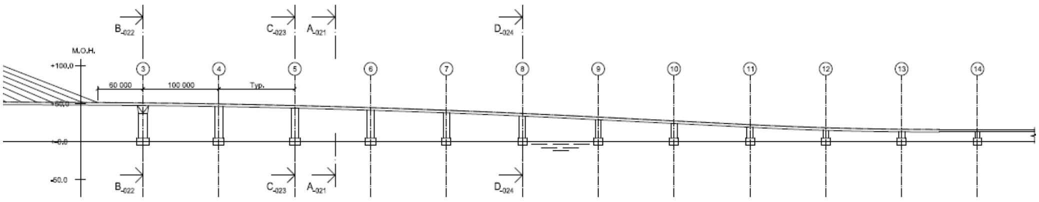


Figure 3-4 High bridge (axis 3-13)

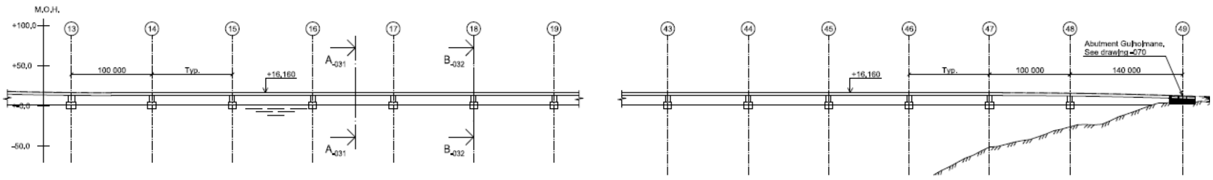


Figure 3-5 Low bridge (axis 13-49)

Resistance of the cross-section is increased in the high bridge using heavier stiffeners as shown in Figure 3-6 below. Outer dimensions are unchanged, but plate thicknesses are increased. All plate stiffeners are within limits for cold forming.

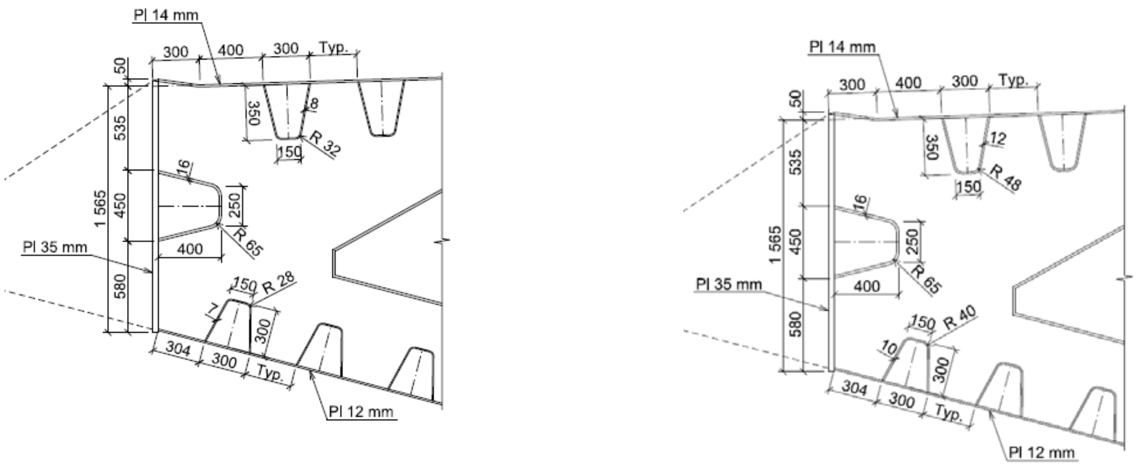


Figure 3-6 Details at corner of steel girder shows variation in stiffeners in Cross-section Type 1 Left, and Type 2 Right

3.3.2 Steel weight summary

Total steel weights are summarized in table below

Table 3-2 Steel weights in bridge girder

	Weight	Total length (m)	Steel weight (ton)
Cross section type 1	12.55 ton/m	4 151.5	52 101
Cross section type 1	14.55 ton/m	1 189	17 300
Reinforcements abutments*	22.7 ton/m	98.5	2 236 *
	Additional weight	No	
Reinforcement axis 3	101 ton	1	101
Reinforcement axis 4 - 6	71 ton	3	214
Reinforcement axis 7 - 12	59 ton	6	354
Reinforcement axis 13 - 47	55 ton	35	1925
Reinforcement axis 48	71 ton	1	71
	Total steel weight		74 371 ton

*) For reinforcement at abutments, see Appendix G

3.3.3 Cross section properties

Cross section properties used in the analysis are calculated based on the plate thicknesses and the stiffeners. The stiffener areas are assumed to be equally distributed over the plate width placed in the COG of the stiffener, forming an equivalent plate. This equivalent plate thickness is used to calculate all cross-section parameters except for the torsion parameters where the actual plate thickness is used.

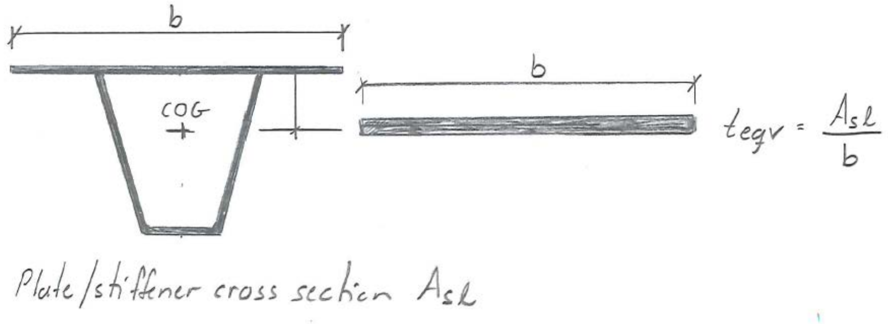


Figure 3-7 Equivalent plate for cross section properties

The table below gives the cross-section properties used in analyses:

Table 3-3 Cross section properties. For location of points, refer to figure below.

	Cross section Type 1		Cross section Type 2	
A	1,43 m^2		1,68 m^2	
I_{strong}	115,62 m^4		132,47 m^4	
I_{weak}	2,68 m^4		3,20 m^4	
I_t	6,10 m^4		8,25 m^4	
$W_{t.}$	1,98 m^3		2,85	
Mass per meter *	17836 kg/m		19798 kg/m	
Section modulus **	Weak axis	Strong axis	Weak axis	Strong axis
Point 1	1,715 m^3	68,010 m^3	2,039 m^3	77,925 m^3
Point 2	2,520 m^3	9,555 m^3	2,995 m^3	10,948 m^3
Point 4	-1,388 m^3	14,014 m^3	-1,653 m^3	16,057 m^3
Point 5	-1,388 m^3	0,000 m^3	-1,653 m^3	0,000 m^3
Point 6	-1,388 m^3	14,014 m^3	-1,653 m^3	16,057 m^3
Point 8	2,304 m^3	9,555 m^3	2,738 m^3	10,948 m^3
COG vertical	1,93 m		1,93 m	

*) Mass includes transverse beams, railings, asphalt

) Section modulus refers to points in the cross-section in **Error! Reference source not found.

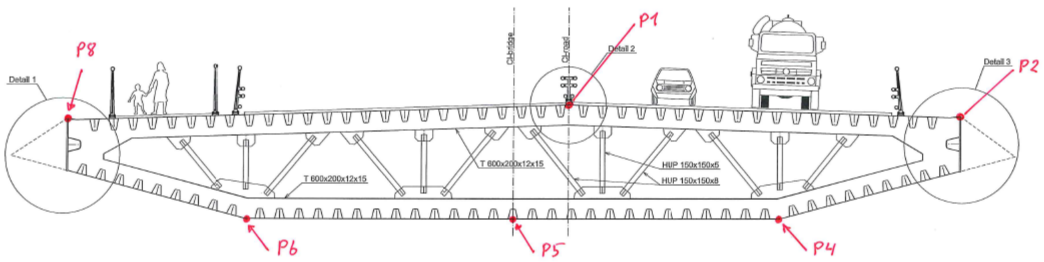


Figure 3-8 Location of points in Table 3-3

3.3.4 Cross-section resistance in ULS

For details regarding bridge girder design, reference is made to Appendix D.

Design stresses are calculated and reported in the Global Analyses Report, Appendix A. Stresses are reported in the stress point shown on figure above. The stresses are envelope of maximum ULS stresses along the bridge. Stresses are checked against buckling and yield stress.

The cross-section resistance is calculated according to NS-EN 1993-1-5 with the following procedure:

1. Calculation of plate buckling effects due to direct stresses according to section 4
    - Column type buckling behavior will be dominant for all stiffened plates
    - The buckling load resistance gives the maximum allowable compression stress
2. Calculation of shear resistance according to section 5
3. Interaction between shear and bending are neglected when utilization in shear is less than 50% according to EN 1993-1-1, section 6.2.8
4. As a robustness check, the cross-section resistance for axial and bending loading are determined using a Finite Element Method according to NS-EN 1993-1-5 Annex C.

Buckling capacities are calculated for the stiffened plates and summarized below. The stresses are in the centroid of the effective column:

Buckling resistance in plates

Cross section type	Deck 14 mm plates	Bottom 12 mm plates	Side panel
Type 1	320 MPa	312 MPa	344 MPa
Type 2	342 MPa	334 MPa	344 MPa

The cross section is reinforced locally at all axis with longitudinal bulkheads and in axis 3 – 6 and 48 thicker bottom plates. At axis 3 the transverse girder spacing is reduced to 3,75 meters. Thicker bottom plates and transverse girder spacing affects the resistance:

Location	Deck 14 mm plates	Bottom 18 mm plates	Side panel
Axis 3	347 MPa	336 MPa	349 MPa
Axis 4 – 6	342 MPa	330 MPa	344 MPa

Shear buckling resistance is calculated for all stiffened plates. All plate panels can be utilized to full yield.

I may be noted that overall buckling resistance of the bridge girder (global buckling of the arch), is checked in the global analyses for each time step in the analyses. The analyses program accounts for the effect of large deformations and axial force level in the girder for each time/load step. The section forces and moments from the analyses thus contains the effect of second order moments due to compression, i.e. a full non-linear buckling analysis is executed for each load-time step.

Verification of cross section resistance against the design forces and moments from the analyses, therefor is a complete capacity check including possible buckling of the arch, and strictly according to NS-EN 1993-1

Capacity of the bridge girder is also analysed according to NS-EN 1993-1-5 Annex C using a Finite Element Model. These analyses show 22% increase of capacity for weak axis bending, and 42% increase of capacity for strong axis bending. In combinations of bending about weak and strong axis, the increase is even higher. See Appendix J.

This redundancy is not utilized in the design, but gives valuable information about the robustness of the bridge girder.

3.3.1 Cross-section resistance in FLS

The resistance in Fatigue Limit State is verified according to NS-EN 1993-1-9. For details, see Appendix I.

The fatigue assessment shows fatigue lives for the bridge girder that generally are well above the target design life of 100 years. One critical section has been identified with insufficient fatigue life. However, two efforts have been identified which is believed to bring the fatigue life above 100 years also at this location. These efforts will have a small impact on overall weight and cost. This is more extensively described in chapter 4.4 and in Appendix I.

Fatigue life for details in column connections is not analysed. However, the detailing is according to recommended practice for Ship and Offshore Design, and fatigue life is considered as a matter of plate thicknesses.

3.3.2 Cross-section resistance in ALS

The resistance against ship impact is proved to be sufficient for the scenarios:

- Bow collisions with bridge pontoons, different angles of impact (30 MJ – 350 MJ)
- Deckhouse collision with bridge girder (188 MJ)
- Bow collisions with bridge girder
- Sideway collisions
- Submarine impact (10 MJ)

An impact energy of 500 MJ is also studied for a load case with impact on the pontoon in axis 3. The study shows limited increase in stresses for the column and bridge girder. Some plastic deformation occurs in the bridge girder. The lateral displacement of the bridge girder at the tower are somewhat larger than the spacing between the tower legs and will result in local damage. Mitigating measures must be considered if 500 MJ is set as a design case.

Further details in chapter 4.3 and Appendix C.

3.3.3 Global buckling safety

The resistance against global buckling safety is included in the global analyses. The analyses are non-linear time domain analyses which in every time step accounts for:

- Effect of deformation (Second order effects)
- Change in stiffness matrix due to axial load level
- Change in drag, lift and moment wind coefficients due to change in deformation
- Change in hydrodynamic forces (Mass, damping, excitation)
- All environmental loads such as turbulent and mean wind, swell and wind sea in fully coupled load time-series

The reported load actions, section forces, moments and stresses are final load actions including second order effects. Consequently, capacity checks as described in 3.3.4 is a complete verification of resistance according to NS-EN 1991-1-1 and NS-EN 1991-2.

3.4 Pontoons and columns

This appendix summarizes the phase 3 work related to sizing and structural design of the pontoons and columns supporting the bridge girder of the end anchored floating bridge across the Bjørnafjord. Key focus has been to minimize the overall investment cost of the bridge with due considerations of minimizing the impact related to maintenance and operation.

The main rationale for selecting steel floaters for the bridge was to reduce the overall investment cost of the floating bridge considering both the bridge girder and the floaters combined.

Pontoons

The pontoons are designed as conventional plated steel ship type structures with due considerations of the long design life of 100 years. A special study considering the corrosion protection requirements has been performed in this project phase and the current recommendation is to substitute the normal carbon steel plates in the splash zone (about 3 m) by a corrosion resistant alloy (Super duplex stainless steel with 25%Cr), see Figure 3-10 and Appendix Q for details.

The sizing of the “floaters” has been based on an iterative process considering a range of geometries, spacing and associated hydrodynamic properties.

The proposed “floaters” in this study phase are monohull pontoons sized according to the following main criteria:

- Satisfactory dynamic behavior for the floating bridge considering both extreme and fatigue loading
- Sufficient water plane area and rotational stiffness to support the permanent and variable loads
- Relatively low structural weight, conventional ship type structure with standard dimensions, plate thicknesses and stiffeners and thus suitable for efficient world-wide fabrication, transport and efficient integration with bridge girder
- No requirement for permanent active components such as ballast or bilge pumps and limited requirements for structural inspections
- Subdivided into compartments so that an accidental flooding of 2 compartments will not jeopardize the floating bridge integrity and post-accident behaviour

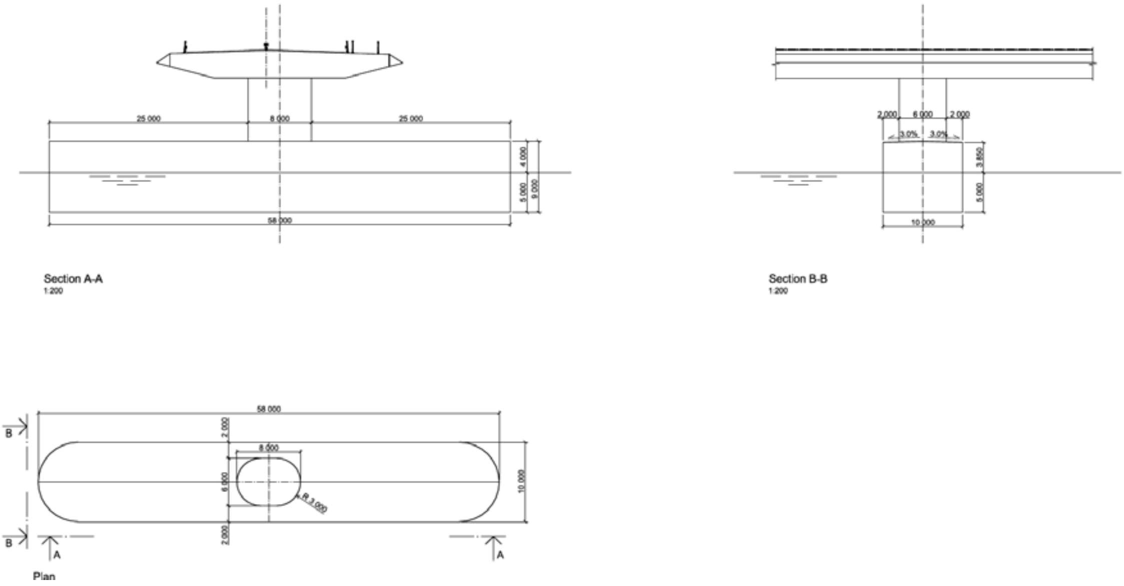


Figure 3-9 Pontoon global geometry - low bridge. Type 1, -DR-050, ref. Appendix A

The pontoon widths are different for the low bridge and the bridge in the ramp and high bridge area, see Table 3-4. The structural arrangement is similar, see Figure 3-10, but the type 2, 3 and 4 have longitudinal watertight bulkhead and thus more compartments than the low bridge (type 1).

Table 3-4 Pontoon global dimensions and weights – pontoon ends half circles – see Figure 3-9

Pontoon types	**L (m)**	**B (m)**	**H (m)**	**Weight (tonnes) Incl. outfitting and margins**	No off
1 Low bridge - Axis 13-48	58	10	9	750	36
2 Ramp - Axis 7-12	58	12	9	900	6
3 High bridge - Axis 4-6	58	14	9	1040	3
4 Ship navigation channel - Axis 3	58	16	9	1180	1
Total				**36 700**	**46**

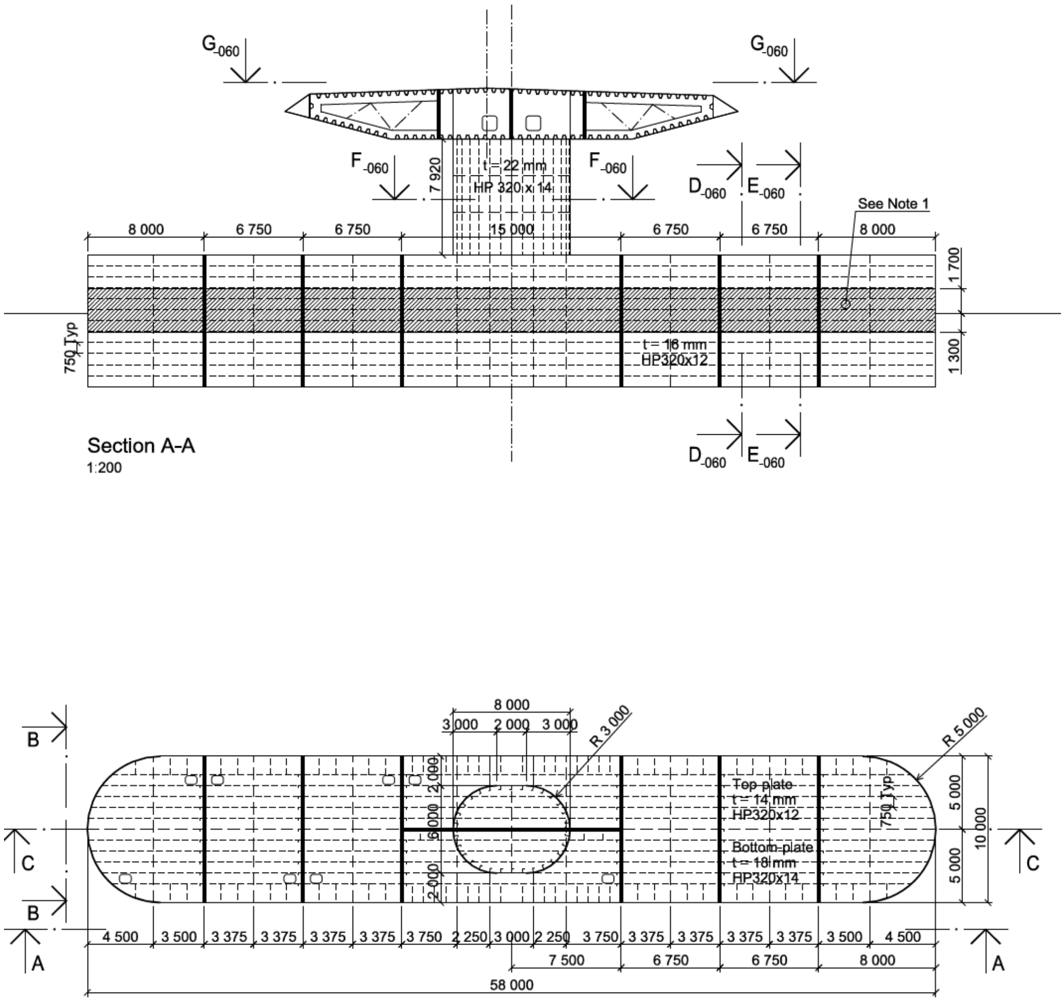


Figure 3-10 Structural arrangement and main scantlings pontoon type 1 (low bridge,-DR-054, ref. Appendix A

Since the pontoons are expected to be fabricated at ship yards, it should be considered to apply DNVGL material and fabrication standards. Typically, the plate material quality has σ_y = 420 MPa, whereas the profiles and stiffeners has σ_y = 355 MPa to ensure availability of for example HP-stiffener profiles which are not produced with σ_y = 420 MPa.

There are 2 manholes/hatches to each pontoon compartment located at the pontoon deck which is approximately 4 m above the sea. The general access to the pontoon deck is from a vessel.

The manholes/hatches will be used for ballasting and to deploy de-ballast pumps. The intension is to remove all the water ballast in operation and dry and seal the compartments to prevent corrosion.

The pontoons are designed for the complete service life of 100 years. Present experience from offshore show that proper coating systems at the cathodically protected part has lower coating breakdown rate compared to what is in the current basis for dimension of anodes. Consequently, the current design has been based on dimensioning the cathodic protection system for 50 years, with provision for replacement of anodes, if required. Following an unforeseen event, it is possible to replace a pontoon while the floating bridge is in service, by supporting the portion of the bridge using a barge during the replacement period.

Columns

There is one central column at each pontoon supporting the bridge girder. The column shape is round/rounded to minimize the wind load, visual impact and steel weight. In general, the ship impact onto the pontoon is the governing design criterion for the columns. This implies that the column dimension varies for the low bridge, ramp and high bridge pontoons due to the increase in moment arm and increasing design ship impact energy towards the ship navigation channel. The column plate thickness and stiffeners are governed by the ship impact and final dimensioning will require a more refined ship impact calculation model to evaluate post-impact capacity.

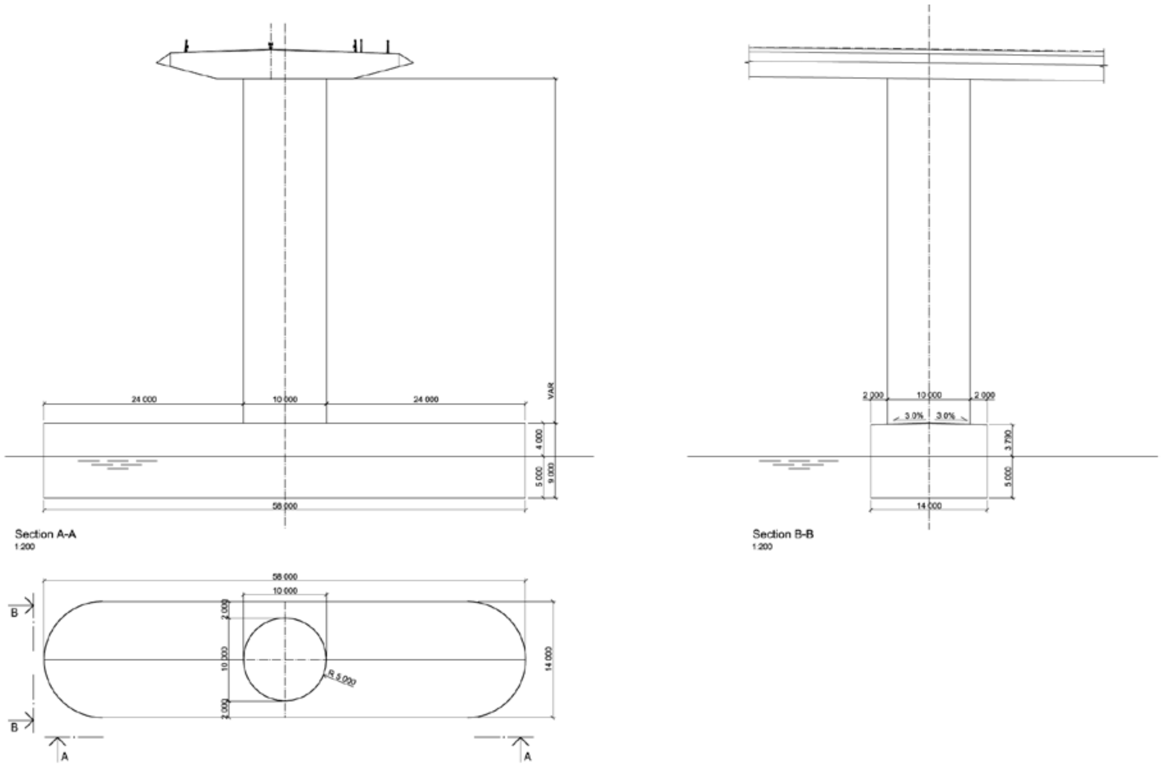


Figure 3-11 Pontoon and column – dimensions for High bridge (type 3), -DR-052, ref. Appendix A

Since all the low bridge and ramp pontoons are oriented normal to the bridge axis, the structural interface for column against the bridge girder and the pontoon will be same for all pontoons along the curved bridge, thus greatly simplifying the design, fabrication and installation logistics. For the high bridge area, the columns are circular which allows rotation of the pontoon to be parallel to the ship navigation channel without affecting the internal reinforcements inside the bridge girder or pontoon structures.

Table 3-5 Column dimensions and weights

Column types	L (m)	B (m)	R (m)	H (m)	Weight (tonnes)	No off
1 Low bridge - Axis 13-48	8	6	3	7,5	65	36
2 Ramp - Axis 7-12	10	8	4	23 (average)	230 (average)	6
3 High bridge - Axis 4-6	10	10	5	38,5 (average)	380 (average)	3
4 Ship navig. ch - Axis 3	12	12	6	41	750	1
Total					~ 5600	**46**

The column structural arrangement is based on a column shell supported by vertical stiffeners and radial frames similar to a semi-submersible column, but without a requirement for watertight subdivision by decks or vertical bulkheads. Figure 3-12 shows the structural arrangement of the column closest to the ship navigation channel – which is the tallest and has the highest ship impact energy. As shown in Figure 3-12, the top part of this column is rectangular to provide structural continuity between the column and the local internal reinforced structures inside the bridge girder. Similarly, the bridge girder frame spacing is reduced and additional longitudinal stiffeners are included to take the large bending moments from the design ship impact energies in the relevant directions. A local finite element analysis has been conducted for the governing ship impact case against the pontoon next to the ship navigation channel to confirm that the expected load path from the pontoon into the bridge girder for the governing ship impact scenario.

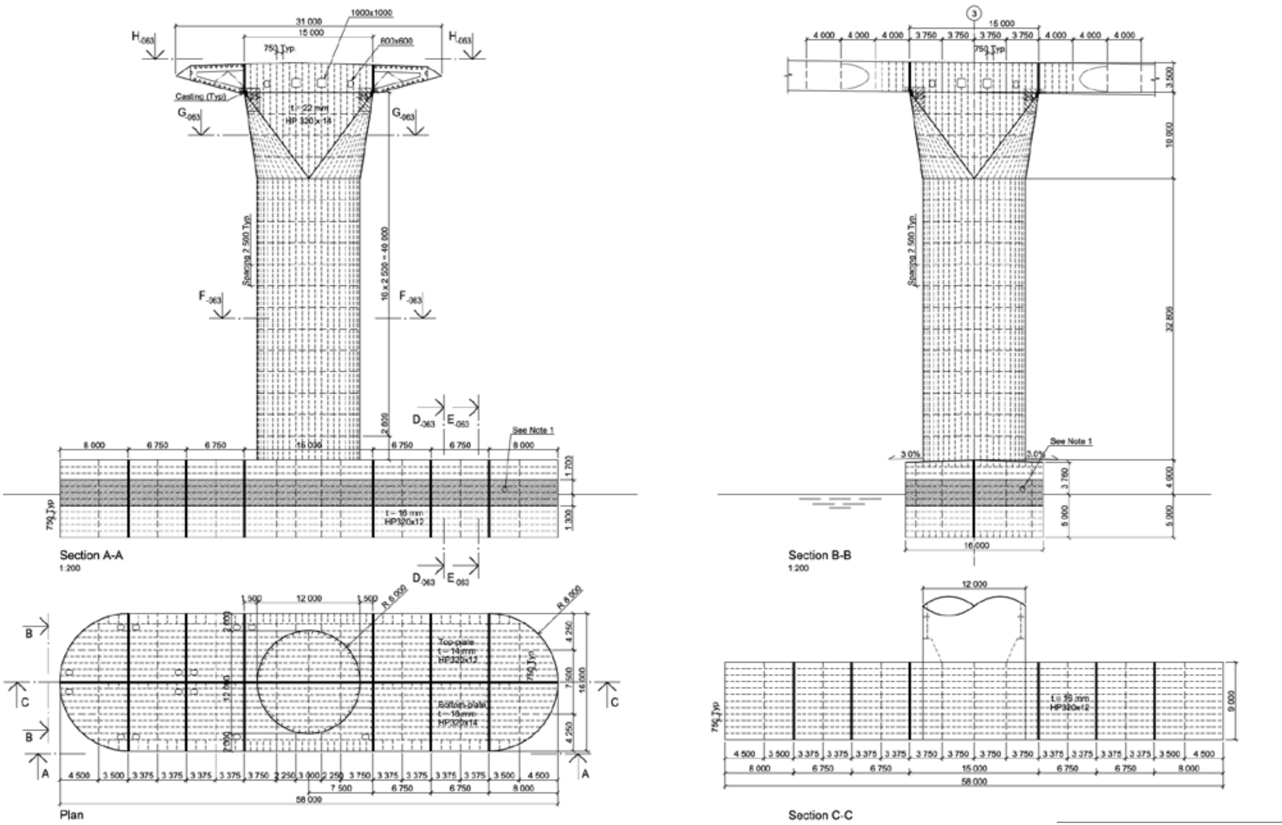


Figure 3-12 Structural arrangement – Pontoon and Column – Type 4 (Ship Navigation Channel), -DR-057, ref. Appendix A

3.5 Abutments and end-anchoring

Following the optimization of the crossing alignment both abutments are now located on dry land. Both abutments are founded on prepared bedrock base. Vis á vis the previous project phase the crossing alignment and thus also the location of the North abutment has been altered to site on the Gullholmane islet. The location of the South abutment is unchanged, however the anchorage of the backstays has been separated from the abutment and the fixed end support of the superstructure.

The bridge box girder is monolithically connected to the abutments in both ends. The restraint of the superstructure is resolved by concrete gravity base structures with a box-shaped, cellular configuration. Solid ballast (Olivine) and post-tensioned rock anchors are used to enhance the overturning and sliding resistance.

This report focus on the design of the structural features deemed crucial for the feasibility and performance of the integral abutment concept:

— The direct, integral connection between bridge girder and abutment
— Abutment global stability
— Transfer of bridge end reactions through the abutment to the base

The flexural response in the bridge girder increases substantially towards the abutments and is significantly higher than what can be resisted by the standard box girder cross section (Type 1) generally adopted for the Low Bridge. To strengthen the steel box girder at the ends (~50 m), the trapezoidal section is transformed into a rectangular section by removing the chamfer and introducing longitudinal diaphragms as well as T-stiffeners for the arrangement of post-tensioning anchors at the joint. The rectangular box has a tapered width from 27.6 m to 40 m towards abutment north, and constant width of 27.6 m towards abutment south, with an equivalent deck and bottom plate thickness of 40 mm. The maximum utilization of the reinforced section towards abutment north is 0.82, while the maximum utilization for the reinforced section towards abutment south is 0.56.

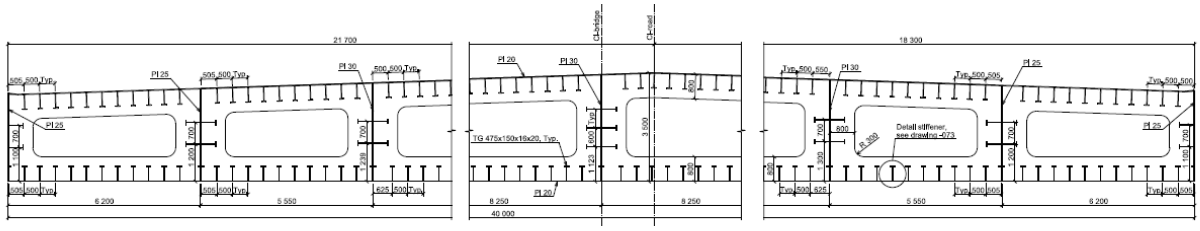


Figure 3-13 Strengthened bridge girder section at bridge ends at abutment North (truss bracing of cross stiffener not shown)

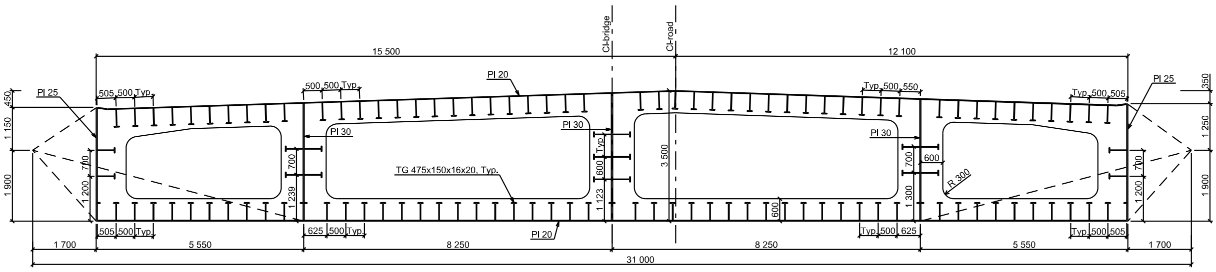


Figure 3-14 Strengthened bridge girder section at bridge ends at abutment South (truss bracing of cross stiffener not shown).

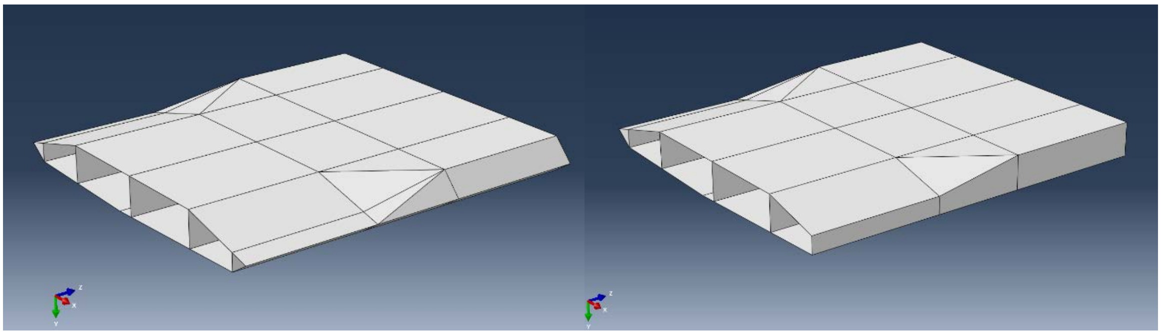


Figure 3-15 Transition from trapezoidal bridge girder section to rectangular section viewed from below, with and without airfoils included respecetively

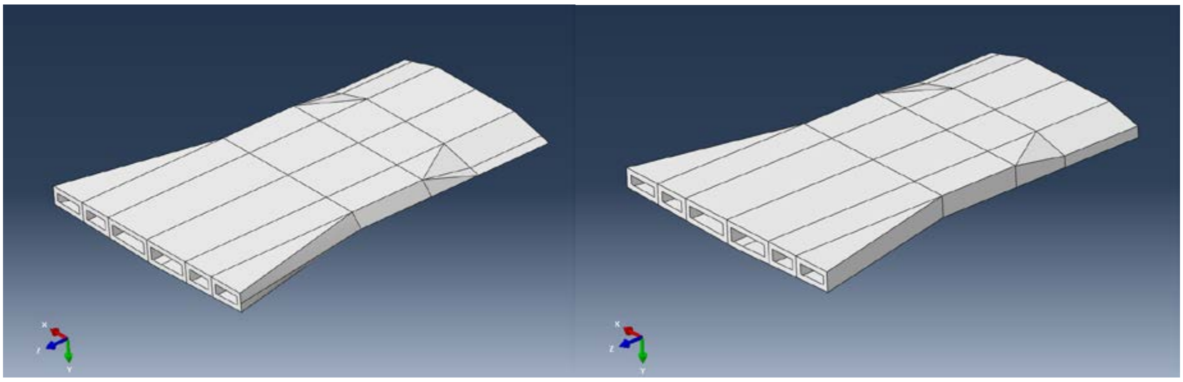


Figure 3-16 Transition from trapezoidal bridge girder section to rectangular section to 40 m width, viewed from below, with and without airfoils included respecetively

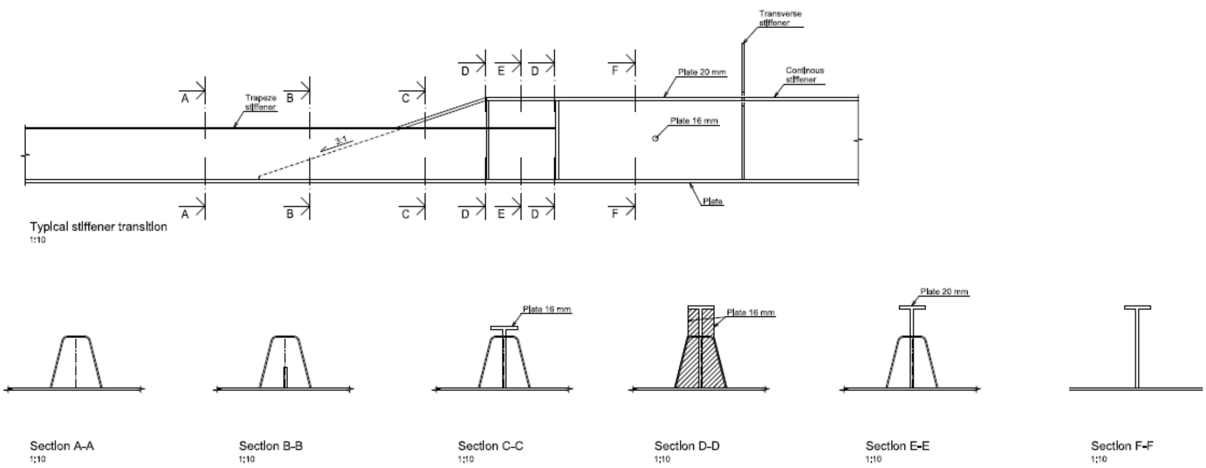
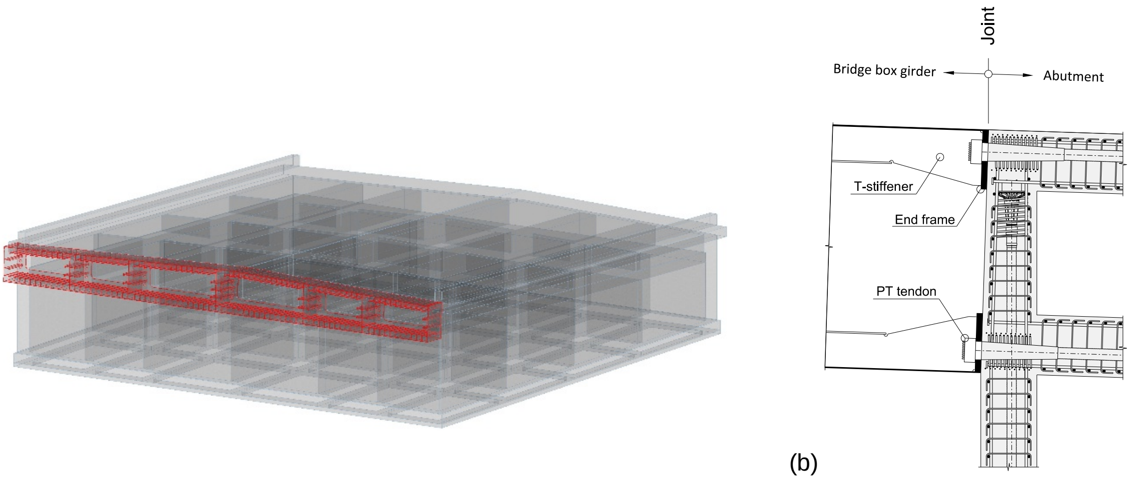


Figure 3-17 Stiffener transition from trapeze stiffener to T-stiffener

Bridge end connections

The fixed end restrain of the bridge is obtained by means of post-tensioned tendons closely arranged along the periphery of box girder and anchored directly into the girder end frame. In order not to interfere with the assumptions for the dynamic behaviour, the joint is designed to remain in compression in the ultimate limit state. The required PT level has been determined from the simplified assumption of plain strain distribution over the interface (i.e. ideal rigid end frame) but accounting for the non-linear behaviour of concrete in compression.

Due to the large forces at abutment North the width of the bridge girder has been extended to 40 m to increase the capacity locally in the area where the monolithically connection result in high forces. Due to large differences in the forces at abutment North and abutment South the width of the South abutment is kept equal to the width of the general width of the bridge girder of 27.6 m.



(a) (b)

Figure 3-18 Isometric view of bridge end connection in North (a) and detail of post-tensioned joint (b).

As can be seen from Figure 3-19 and Figure 3-20 a high level of prestress is required to maintain compression in the joint under full loading at ultimate. The assumption of a rigid end frame yields a total prestressing force of 1300 MN ($e_{y,p} \approx 0$; $e_{z,p} = 0.0$ m) which implies a PT intensity of 6-37 tendons @ 0.5 m for the all parts at abutment North. For abutment South the total prestressing force of 907 MN distributed to 134 6-36 tendons @ 0.5 m. For comparison, utilizing the full yielding capacity of the steel box with an equivalent plate thickness of 40 mm corresponds to a PT intensity of 6-45 @ 0.5 m.

The shear is transferred by means of multiple steel keys welded to the back of the end frame and arranged in the same pattern as the stiffeners. The shear capacity of the joint is a function of the net force normal to the joint and development of friction on the joint face. The shear resistance at the interface is predicted according to the construction joint provisions in EC2 6.2.5 with the bevelled shear keys configured in compliance with the indented surface specifications.

The end frame plate has a general width of 800 mm matching the thickness of the adjoining concrete slabs and walls. The net contact area is 73.4 m^2 when accounting for the holes for the PT trumpets (net-to-gross ratio ~0.91) at abutment North. A high strength concrete with a concrete grade of B85 (f_{cd} = 48 MPa) is required to resist the bearing stresses in the joint in ULS. The average concrete compressive stress resulting from prestressing is 17.7 MPa (0.2 f_{ck}) at abutment North. For both joints the compressive stresses at service load level is well within the limits to avoid longitudinal cracks, micro-cracks and excessive creep.

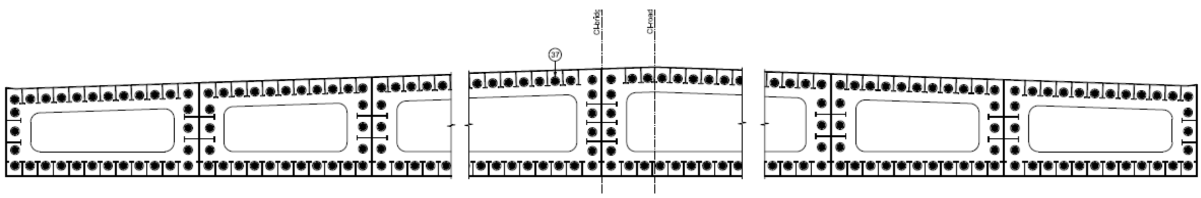


Figure 3-19 Post-tensioning demand at the joint in abutment North(number of 0.6" strands per tendon)

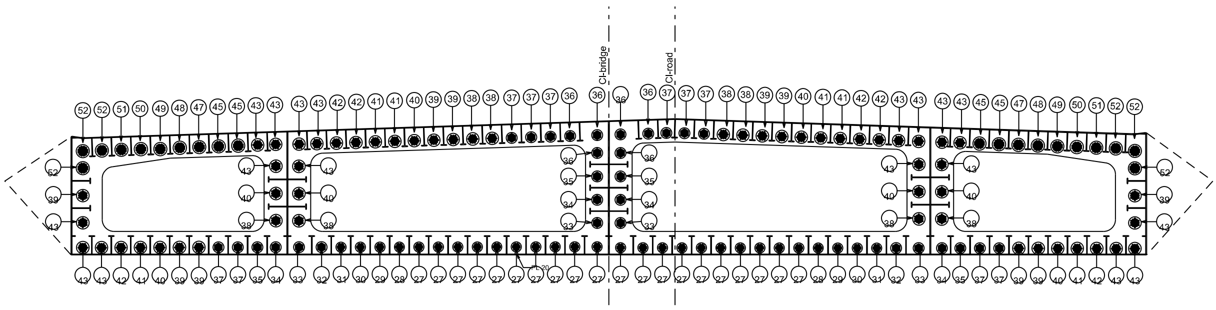


Figure 3-20 Post-tensioning demand at the joint in abutment South (number of 0.6" strands per tendon)

To enhance overload performance of the interface, unbonded tendons are used due to their ability to redistribute strains over their full length. Compared to bonded post-tensioning cables, unbonded tendons yields at a larger overall joint rotation. The result is a connection that can undergo large non-linear bridge end deformation without yielding the post-tensioning steel and without a significant loss in self-centering capability. The unbonded tendons will be installed in rigid steel sheating/ducts, stressed from the rear end of the abutment and filled with grease for corrosion protection.

The behaviour of the joint beyond decompression is assessed by a cross-sectional analysis based on the same assumptions described above. The strain in the unbonded prestressing steel is incompatible with the strain profile of the joint. Instead of accurately determined the total elongation between the anchorages (which would require an extensive global analysis model) the strain increase is simulated by reducing the stiffness of post-tensioning steel and apply the prestress as external loads. The predicted moment – curvature relationships in Figure 3-21 indicates that the joint has a capacity reserve of ~50 % the ULS loading to compensate for potential uncertainties in dynamic response, material strength, geometric deviations, inaccuracies in the design model or quality of workmanship. The ultimate resistance is governed by concrete compression failure for both axes.

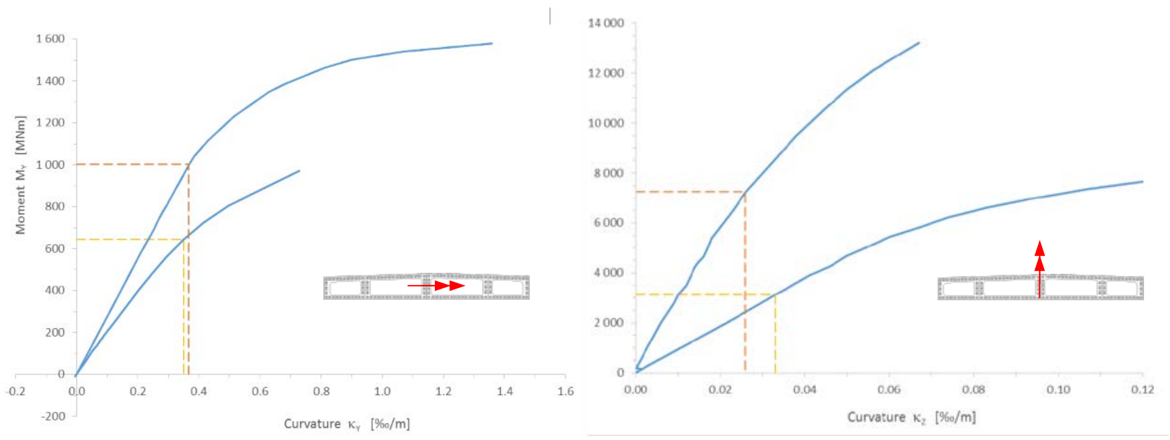


Figure 3-21 Moment-curvature diagrams (ALS) for bridge weak axis (left) and strong axis (right) .The upper graph represent the abutment North while the lower graph represent abutment South. Dotted line indicates decompression limit.

The caisson is designed as a box composed of slabs and walls which are predominantly subjected to membrane action. Slab and wall thicknesses are mainly controlled by geometric requirements for fitting orthogonal post-tensioning tendons. The unbonded coupling tendons are anchored at the rear end of the abutment (without staggered anchorage) in order to obtain a favourable transfer of the bridge end reactions through the longitudinal walls to the base. No detailed design has been made, but the appropriate layout has been verified by means of qualitative strut-and-tie models that illustrate the force flow for main loading situations.

Foundation

The abutment is founded direct on the bed rock. A level base is established whereby weathered and fissured rock is removed/excavated by blasting. To assure a predictable transfer of base shear and normal pressure, only the outer and long wall stems are cast directly onto rock whereas the base slab is cast onto a sand/gravel layer. For the foundation design, it is conservatively assumed that the overturning resistance is provided by dead weight (i.e. gravity) alone. As for the joint, to conform with the boundary conditions assumed for the global dynamic analysis, no uplift at any point within the foundation footprint is accepted for the ultimate and accidental limit state. The sliding capacity is determined from base friction only neglecting any contribution from backfilling or lateral hard rock support. The contribution from post-tensioned rock anchors to the base friction capacity is within the limits prescribed by N400 11.6.2.2

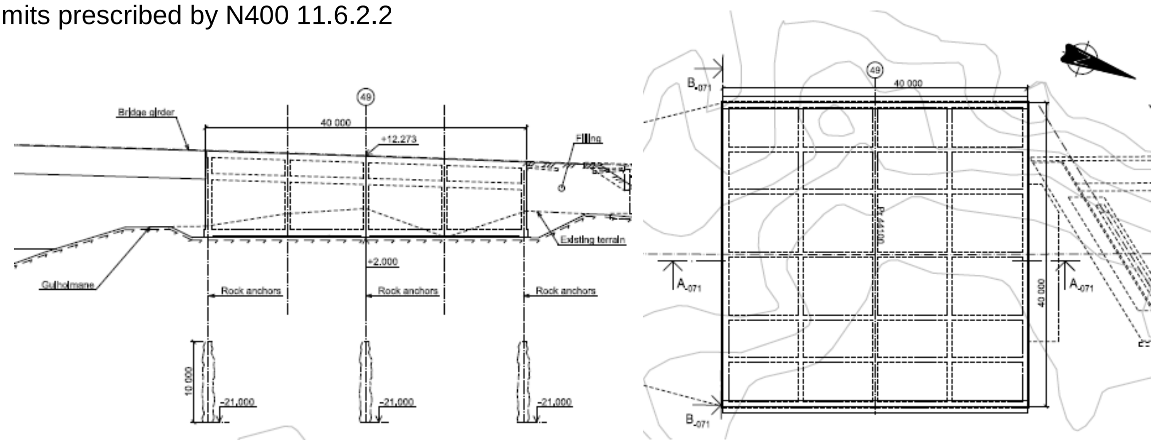


Figure S-10: Elevation and plan of North abutment

To get sufficient capacity for both sliding and overturning at foundation North the size of the gravity base structure is 40 m x 40m with an average height of 10 m. Generally, the forces at abutment South is smaller than for abutment North, except for the axial force which is almost double. Therefore, the foundation at abutment South has been enlarged to 60 m in the longitudinal direction, while the width is 27.6 m and the average height is 10 m.

3.6 Cable stayed bridge tower

Sizing of the tower and verification of the response due to global loads are investigated by use of a standalone, local FE-model. Twelve load combinations (combined static and dynamic forces) are evaluated in ULS. After the initial sizing of the tower, the geometry of the tower is included in the global FE-model. From the global FE-model, design forces in 13 representative locations are extracted and used for verification of the cross sections.

By use of the design tool Beam Design, cross sectional dimensions and tentative reinforcement amounts are verified at twelve representative locations of the tower. The design sections are verified for both ULS and SLS. In addition, the principal stress level in the shell sections in Beam Design are compared to the tensile strength of the concrete to evaluate to probability of cracking in the section. Major portion of the tower is found to be un-cracked for the characteristic load level.

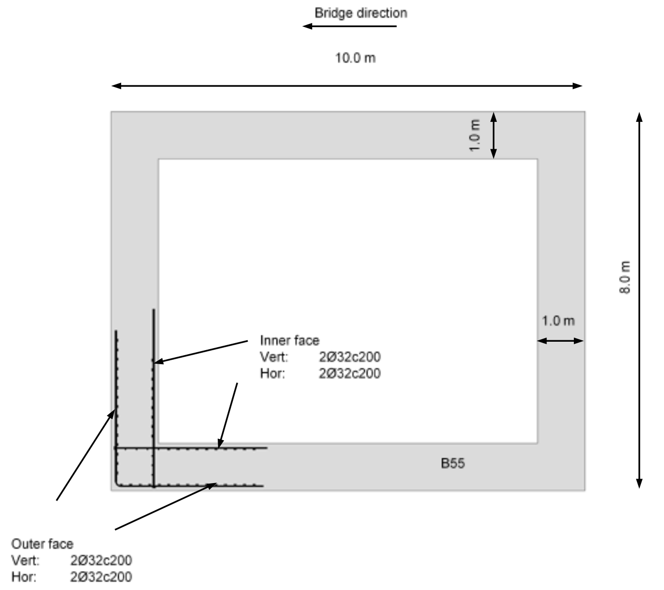
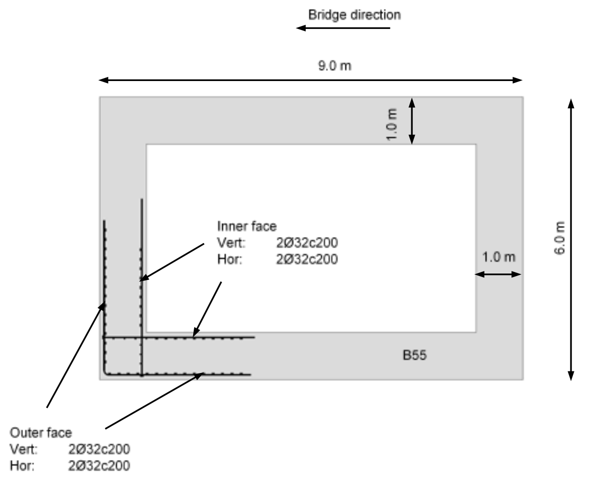
The utilization ratio in ULS for major part of the tower are moderate. The required reinforcement is in general governed by the crack width limitations.

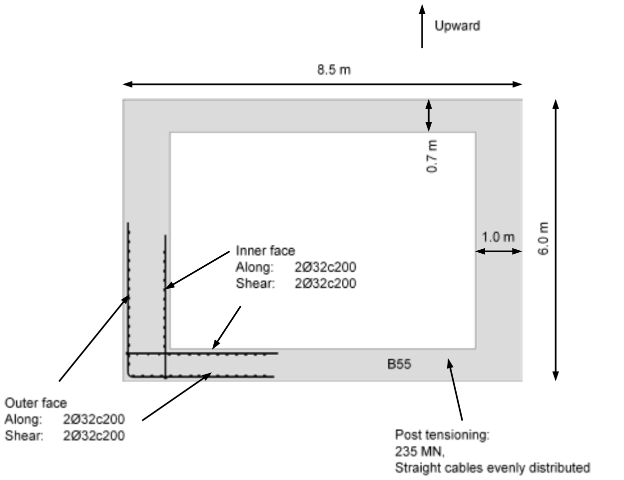
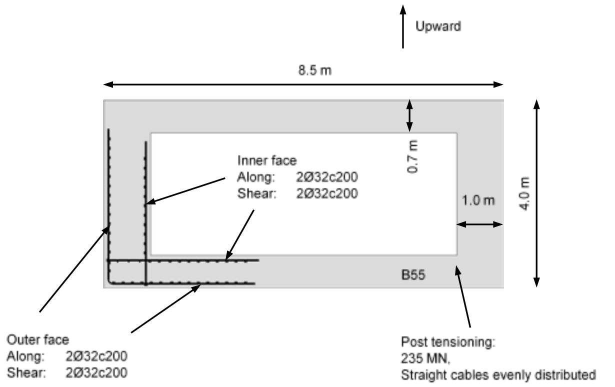
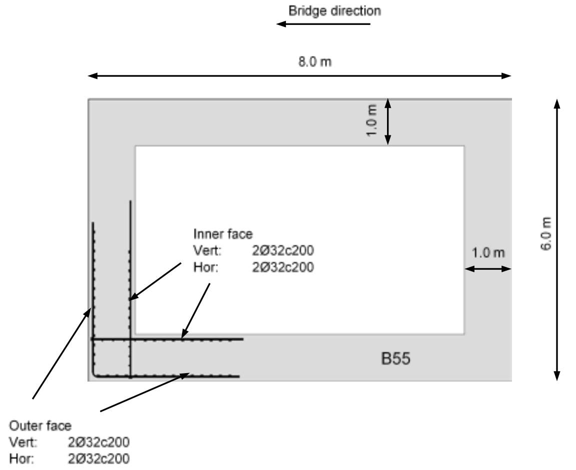
The critical section of the tower is found to be the junction between the inclined tower legs and the mono tower at elevation 169 m. The response in this section is governed by self weight of the structure and unsymmetrical bending due to stay cable forces. In following phases of the project, adjustment of the tower position should be considered to avoid bending moment in the mono tower due to self-weight. In addition, the building stages of the tower could be utilized to reduce the hogging moment in this critical section.

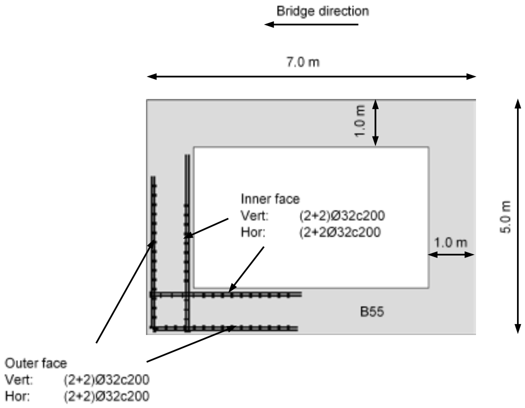
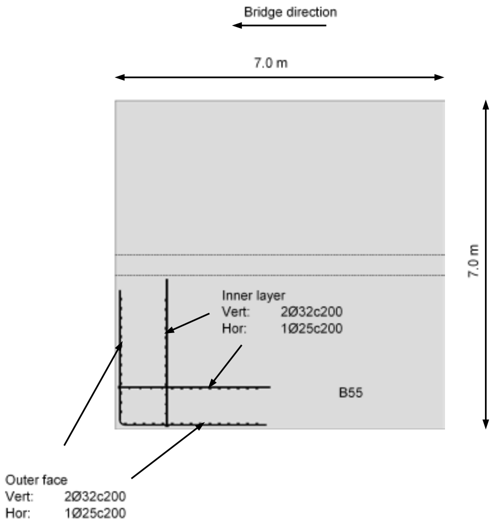
The stability of the tower legs are verified by use of the eigen modes found in the standalone FE-model. Taking both creep and cracking of the tower legs into account result in a 27% increase of bending moment in the tower legs. The critical section is verified for the increased bending moment in ULS.

The general utilization of the tower is low, with moderate reinforcement density. In following phases, special attention should be payed to the upper part of the inclined tower legs.

Table 3-6 Summary of verified cross sections and reinforcement amounts

Location of section	**Geometry and reinforcement layout**
Base of towers at foundation level	
Tower legs just below the cross beam	

Cross beam at supports	
Cross beam in span	
Tower legs just above the cross beam	

Top of the tower legs at the intersection with mono tower	
Base of the mono tower	

3.7 Approach bridge

The approach bridge from Gulholmen to shore is a conventional concrete slab bridge with 3 spans. The bridge is founded on piles to rock. A possible alternative to a bridge, would be a rock fill. The bridge is suggested from an aesthetic point of view. Here it is possible to use surplus rock from the adjacent tunnel to create new land to reduce the visual impact and traffic noise on Røtinga.

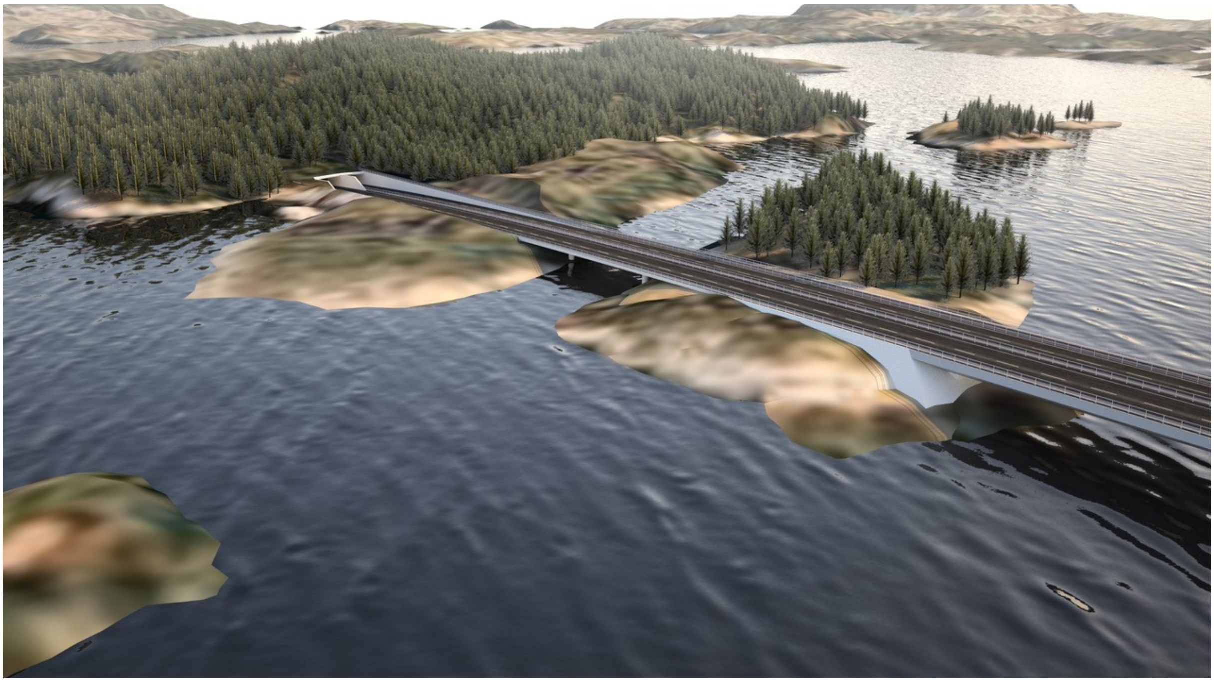


Figure 3-22 Aerial view of shore approach north with possible new terrain

Figure 3-23 Channel between Gulholmanene and extended shoreline on Røtinga

4 Global analyses

4.1 General

The different analyses covering global response of the bridge are described in the following chapters. They can be summarized as follows:

- Dynamic analyses with 3DFloat. This is time domain non-linear analyses including coupled loads from wind, wind-sea and swell. Includes slow drift forces from waves. May also be run with uncoupled loads.
- Static analyses with Sofistik. Includes permanent loads, traffic loads, temperature loads, tidal loads etc.
- Dynamic ship impact analyses with Abaqus. Includes analyses of ship impact on different pontoons with different attack angles and impact energies.
- Fatigue analyses with Orca-Flex/NovaFrame. Includes load effects from wind, wind-sea, swell, tide and traffic.
- Bench-mark analyses with Orca-Flex/NovaFrame. Verification analyses of load action from wind, wind-sea and swell.

The analyses are described in detail in the following Appendices:

Appendix B	Global analyses with 3DFloat and Sofistik
Appendix C	Ship impact analyses
Appendix I	Fatigue analyses
Appendix O	Bench-mark analyses

4.2 Global analyses

This chapter summarize design response for the proposed design. Given the current design basis, the design is found to be feasible. Both the ultimate limit state and the accidental limit state shows feasible structural response. The deflection, displacement and accelerations of the bridge are within the motion criteria described in the design basis.

Design response

The proposed design has been analysed. The following findings are discussed and documented:

- The ultimate limit state load cases and sensitivities shows feasible structural response
- The accidental limit state load cases and sensitivities shows feasible structural response
- The deflection, displacement and accelerations of the bridge are within the criteria's described in the design basis for all relevant load cases.

Based on these findings, the proposed design is concluded to be feasible and the response of the concept is inherently robust.

Modal response

Results from the Eigen value problem shows that added mass has a significant effect on the Eigen periods of the structure. Especially for Eigen periods in between 5 and 10s. The slope of the added mass curve is very steep in this area. The first horizontal Eigen modes are found at 120s and 62s, and thusly not very sensitive to wave structure interaction.

Dynamic response

Dynamic response from wind sea, swell waves and wind has been analysed with coupled time domain analyses. The following findings are discussed:

- Wind is the dominating load contribution for response about strong axis of the bridge. The strong axis response occurs mainly in the two first horizontal Eigen periods.
- Swell triggers horizontal modes in between 12 and 25s, but the contribution is minor compared to wind loading.
- Wind generated sea dominates the response about weak axis. A multitude of Eigen modes are triggered and the resonant behaviour is thusly very complex.

Horizontal wave loading is a significant contributor to the dynamic response, inducing rotational deformation of the bridge girder. The structural response therefore depends on the height of the pontoon towers due to larger moment induced by the horizontal excitation force on the pontoons.

Sensitivities

The most important and relevant information from the environmental documentation is evaluated with regards to finding the governing environmental load cases for this construction. Through thorough sensitivity studies, evaluating wind directions and speed, wave period and directions including spread for all parameters, nine governing load cases have been selected. The sensitivities are evaluated for each part of the bridge, as no single case is governing for the whole bridge. Major findings include, but are not limited to:

- Wind generated waves induces the largest stresses when the wave period is high and the dominating wave direction is normal to the pontoons.
- The swell waves induce the largest response for high wave periods at an angle normal to the bridge. The wind direction producing the largest response is with an angle of attack of about 60 degrees.

Additionally, a series of sensitivity studies have been performed to evaluate assumptions and limitations in the global analyses. The results of these analyses substantiate that the assumptions and limitations made in the global analyses are appropriate.

Methods and theory

Several methods have been applied to extract the extreme response from the global analyses. Methods/theory include, but are not limited to:

- Estimation of dynamic stress
- Statistical extreme value approximation
- Statistical combination of response contributions.
- Modelling method concerning potential theory elements.
- Mean and slow drift

- Implementation of turbulence
- Air loads in general

Model and software

To estimate the global response of the bridge, two modeling approaches and software's have been used.

1. A dynamic model is created to be used of the software 3DFloat
2. A static model has been created to be used by the software Sofistik.

Both software's are commercially available and extensively verified through both research programs and real-world application. They read the geometry-, material input, boundary conditions and mesh from the same database in order to minimize modeling errors.

For supportive calculations and input generation to the dynamic analyses, the following tools have been used:

- Pontoon hydrodynamic behavior included by potential theory elements in the dynamic model. Relevant input parameters have been generated using the DNV software WADAM (DNV, 2010).
- The aerodynamic coefficients of the bridge girder have been estimated by use of CFD analyses.

A visualization of the global analysis model in 3DFloat is shown in *Figure 4-1*. The orientation of the pontoons is illustrated in *Figure 4-2*. As seen in the figure the 10 first pontoons are oriented parallel to the ship-lane, the remaining pontoons are oriented normal to the bridge axis.

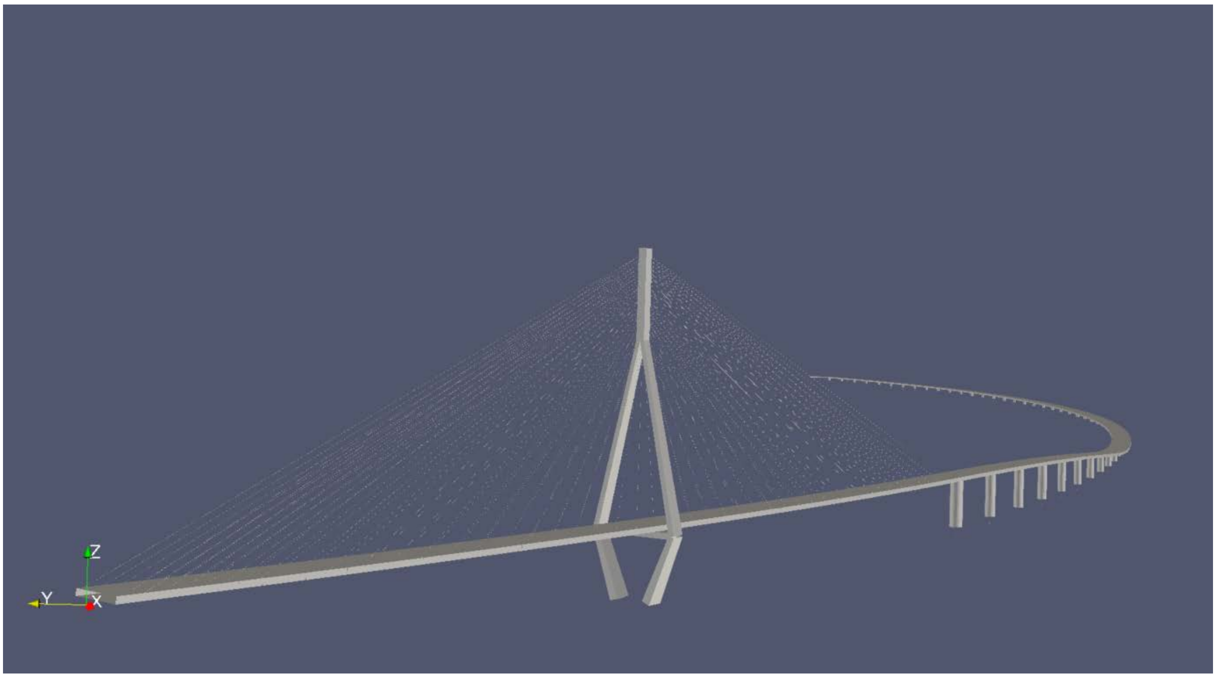


Figure 4-1: Global analysis model of the bridge in 3D.

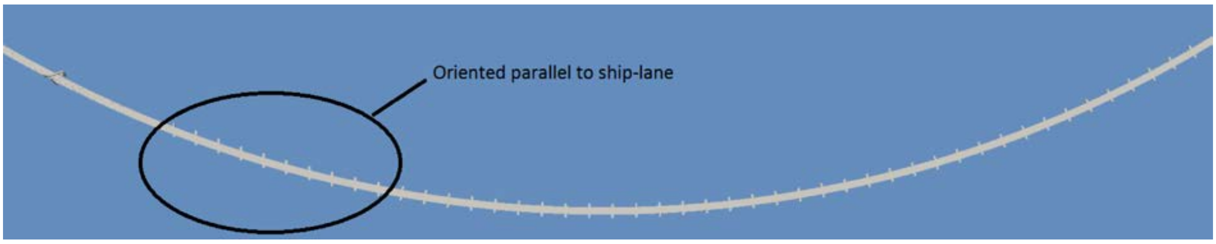


Figure 4-2: Orientation of pontoons

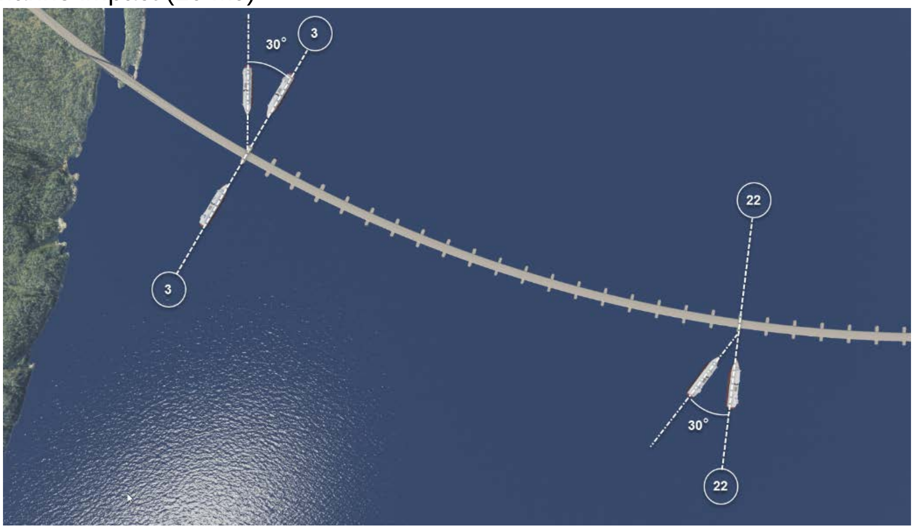
Coupled analyses

By use of the software 3DFloat (Nygaard, T. A., De Vaal, J., Pierella, F., Oggiano, L. and Stenbro, 2016) coupled analyses have been performed on the set of governing sea states. The results of these analyses show that different sea states produces the maximum response for different sections along the bridge. A contour of the extreme response for every section has been taken further and are applied in a total combination of the response.

4.3 Ship impact analyses

Impact scenarios are based on the cases listed in the Design basis:

- Bow collisions with bridge pontoons, different angles of impact (30 MJ – 350 MJ)
- Deckhouse collision with bridge girder (188 MJ)
- Bow collisions with bridge girder
- Sideway collisions
- Submarine impact (10 MJ)



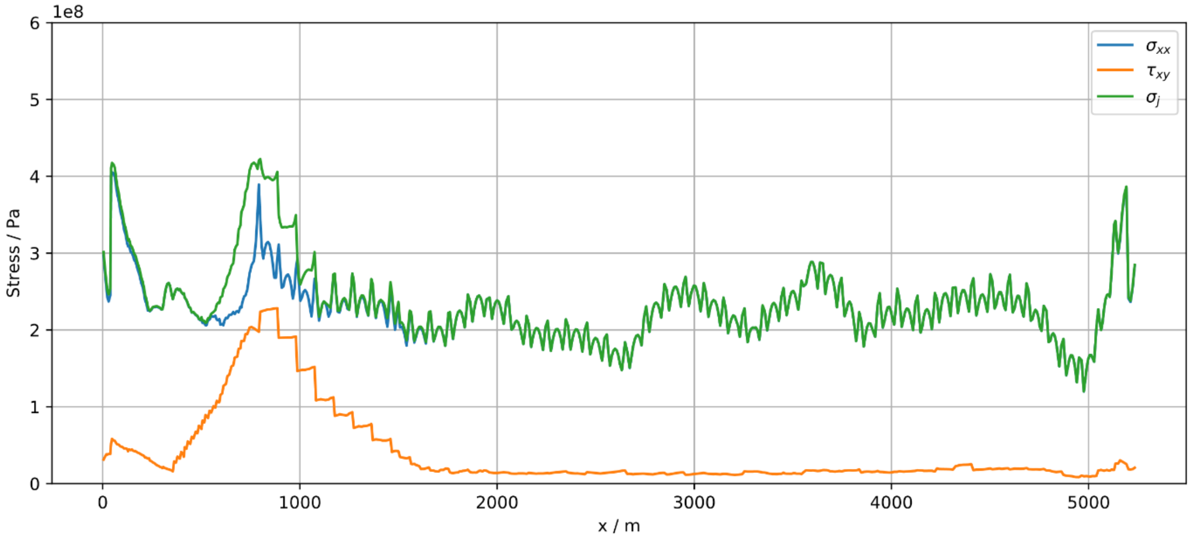
Impact scenarios for bow collision with bridge pontoons. Selected scenarios are shown.

The design philosophy is to utilize the deformation capacity both locally, and more important globally. By including the stiffness, mass and plastic deformation capacity of all main components in the same analysis, we achieve a distribution of impact energy throughout the system. The large deformation capacity of the bridge girder results in small differences in column response for the different impact energies.

The dynamic analyses are built up by including response characteristics derived from local analyses of ship and pontoon respectively. Response from the local analyses are represented by force-indentation curves in the global model. The bridge is analyzed with the software Abaqus 2017. For the ship impact analyses the implicit solver is primarily used, backed up with a comparison test using the explicit solver.

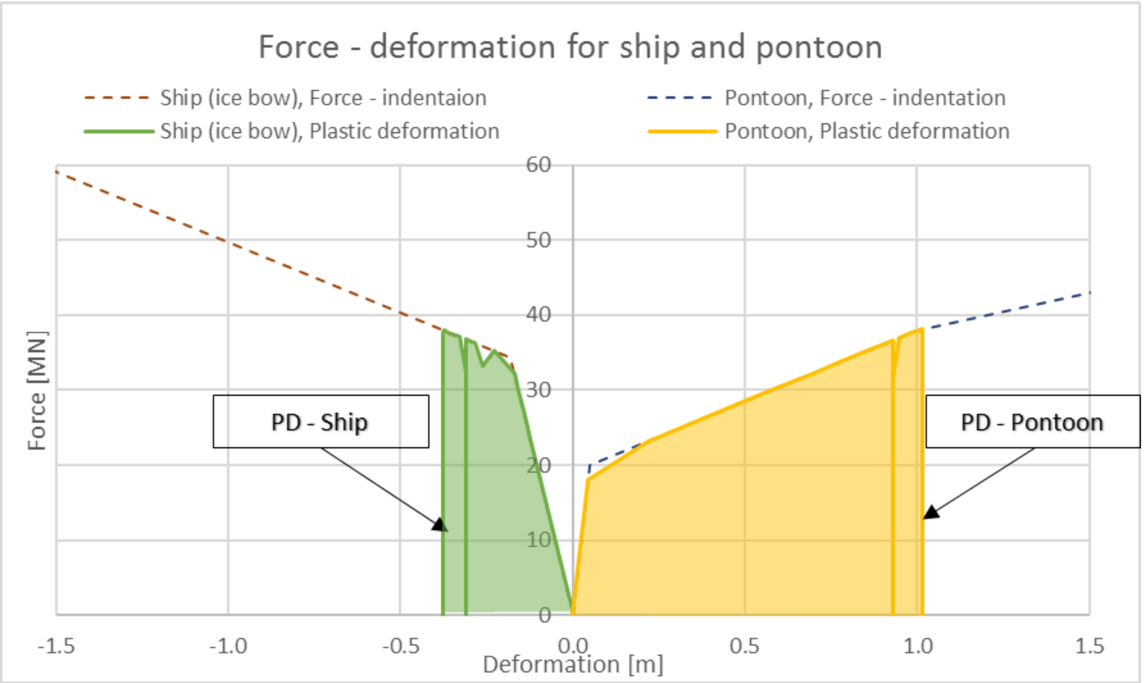
Response in bridge structure:

- Bridge girder: Areas of plastic deformation may occur in vicinity the impact area for energies of 350 MJ. For smaller impact energies, the bridge girder remains elastic.
- Columns: Elastic calculations show that it is necessary to increase amount of steel in the columns in the low bridge compared with the capacity needed for ULS. This is considered in the quantities for the cost estimate. Detailed calculations and utilization of plastic capacity may reduce the column size.
- Abutments: Forces are on the same level or smaller than from ULS. Forces from impact analysis are considered in the abutment design.
- Cable stayed bridge
    - Deformations at the tower give a limited impact between bridge girder and tower.
    - The cables close to the floating bridge are subjected to increased tension, but found to have sufficient capacity.



Stress plot for bridge girder. The plot shows envelope stress values for the scenario with impact on pontoon in axis 3, angle of attack 30˚, 350 MJ energy.

Plastic deformation and energy absorption in ship bulb and pontoon wall are illustrated in the following figure.



Force – deformation plot of ship bulb and pontoon wall. Area under the graphs represent the energy dissipated by plastic deformation.

Robustness

An impact energy of 500 MJ is studied for a load case with impact on the pontoon in axis 3. The study show limited increase in stresses for the column and bridge girder. Some plastic deformation occurs in the bridge girder. The lateral displacement of the bridge girder at the tower are somewhat larger than the spacing between the tower legs and will result in local damage. Mitigating measures must be considered if 500 MJ is set as a design case.

Input from the risk analysis

The impact energies used in the conducted analyses are based on a risk analysis for a different design. When an updated risk analysis is ready it should be compared with the impact scenarios used in this report, and the need for updated calculations must be considered.

It is of great value to have an open dialog between the risk analysis team and the design team. This will improve the consistency in how the scenarios are defined and how they are incorporated in design. The case of 30° impact angle at the most outer part of the pontoon is by far the most critical impact scenario. All other impact scenarios are less severe for the bridge columns. When defining the different impact scenarios, it should be considered to split the scenario for impact on pontoons in smaller impact zones, i.e. to calculate different impact energies for different areas of the pontoon.

4.4 Fatigue analyses

General

A design check for the Fatigue limit state (FLS) has been performed for the bridge girder. The design check takes into account loading from wind, wave, traffic and tide. The fatigue calculation follow the procedure proposed by DNVGL, ref **Error! Reference source not found.**, for conservatively combining fatigue contribution from all relevant types of loading.

Structural Details

The structural details that has been considered are transverse butt welds and transverse attachments (fillet welds). These are details that are subject to longitudinal girder stresses and are therefore considered the most critical with respect to fatigue. These details are common along the entire length of the bridge. For butt welds, stress concentration factors (SCFs) have been calculated based on actual plate thicknesses and a fabrication tolerance (misalignment) of 2 mm. SCFs for but welds are in the range 1.15 to 1.86. The attachment detail has been checked with an E-curve (with nominal stress), and the butt welds have been checked with a D -curve (with relevant SCF).

The fatigue check has been performed at four points on the girder cross-section, the two outermost points on bottom and top flange. In the floating part of the bridge three sections on each bridge segment have been checked, at support (above columns) and at midspan. In the high bridge, checks have been performed approximately every 100 m.

A Design Fatigue factor of 3 has been used in the fatigue life calculations as all the details that checked can be inspected.

A so-called system effect (or weld length effect) has been included in the calculations. A weld length of 100 meters have been used which gives a reduction factor of the calculated fatigue life of 0.66

Simulations and fatigue calculations

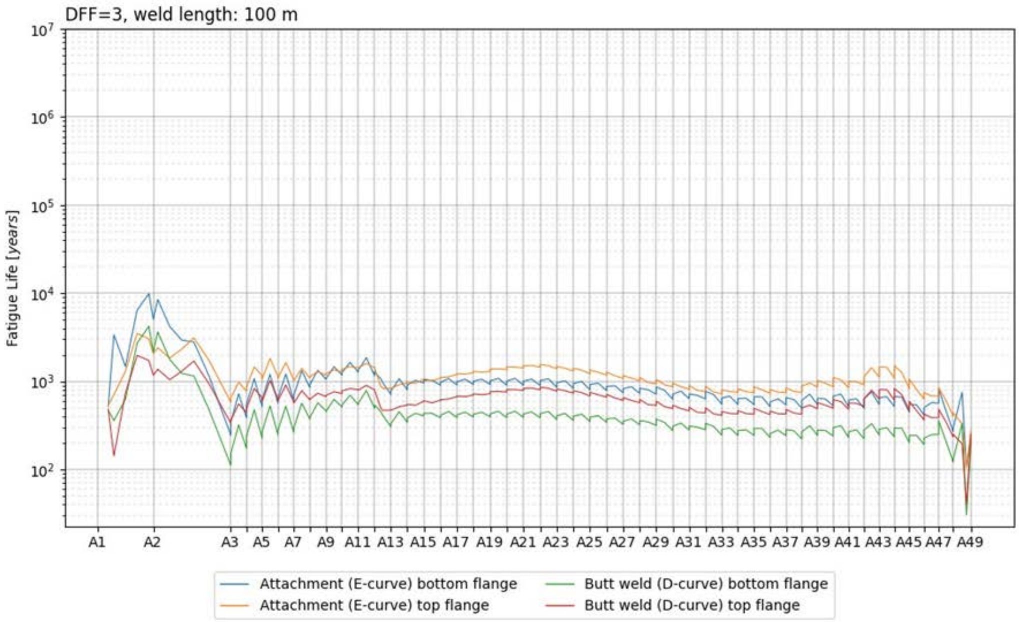
Fatigue contribution from environmental loads are calculated based on the rainflow cycle counting method. A set of stress time series has been established for each environmental load group (wind, wind seas and swells) representative for the long-term distribution of the loads. The stress time series from each load group has then been combined to time series representing the total environmental loading. Wave loads have been analysed in the frequency domain and the stress results have subsequently been transformed to time domain. Wind loads have been analysed directly in time-domain.

Fatigue contribution form traffic and tidal loads are calculated based on the DNVGL method with stress ranges calculated based on envelope values of the weak axis bending moment in the bridge girder.

Results

Generally, all calculated fatigue lives are above the target design life of 100 years with a DFF of 3. The exception is at the transition to the special cross-section towards the north abutment near mid span of the last bridge segment towards north. The calculated fatigue life at this location is 30 years.

Two main circumstances can be pointed out to explain the low fatigue life at this location. Firstly, the analysis model used for the fatigue calculations does not reflect the latest reinforcements of the bridge girder towards the north abutment. This means that the stresses used in the calculation is based on the regular girder cross-section and not the one with reinforced stiffeners. Updating the calculations with the reinforced cross section will improve the fatigue life. Secondly, the transition to the special cross-section is located near mid span where contribution from traffic is large. Contribution from traffic is significantly reduced towards the columns and by moving the section transition towards the column at Axis 48. Moving the transition towards Axis 48 will also reduce contributions from tide, which is highest towards the north abutment. These two efforts are believed to give acceptable fatigue lives for the entire bridge girder.



Conclusions

The fatigue assessment shows fatigue lives for the bridge girder that generally are well above the target design life of 100 years. One critical section has been identified with insufficient fatigue life. However, two efforts have been identified which is believed to bring the fatigue life above 100 years also at this location. These efforts will have a small impact on overall weight and cost.

Further Work

In addition to the two typical details addressed in the current assessment several other areas can be pointed out which will need attention with respect to fatigue when further developing the design. Both the bridge girder to column and column to pontoon connections are areas where fatigue hotspots typically will occur. A ‘fatigue-friendly’ design should be sought early on using FE-hotspot models to find appropriate structural details.

At the cross-section with lowest fatigue life, traffic is the main contribution to fatigue. Measures to reduce conservatism in the traffic fatigue calculations should be looked into, e.g. to refine the load model or the combination method for damage from environmental loads and traffic loads. Further to obtain a ‘fatigue-friendly’ design FE-hotspot models should be used for the end regions also to find appropriate structural details.

4.5 Bench-mark analyses

Extensive benc-hmark analyses have been carried out, and this is documented in appendix O. Two different models, Phase 2 model and Phase 3 model have been analysed with three different commercial structural analysis programs:

- OrcaFlex by Orcina
- 3DFloat by IFE
- NovaFrame by Aas-Jakobsen

In addition a fourth program TMcode is also used in the benchmark process.

The benchmark process started out with the Phase 2 and was originally limited to this model. As one had difficulties to update the models to behave equal and the Phase 3 model did differ fundamentally in stiffness and eigenperiods, it was decided to include both Phase 2 and Phase 3 models into the benchmark process.

Wind loading

The main objective of the wind modelling benchmark is to identify differences between the calculated response from 3DFloat and the calculated response from OrcaFlex. Results from Phase 2 models are presented in Vedlegg A and from Phase 3 models in Vedlegg C.

When the benchmark process started out the differences between the programs was rather distinct. During the process results have been much closer and now we feel that the wind actions are calculated reasonably well in all three programs. Some differences are present but the really large differences are due to different choices of input parameters that not are related to the calculation process. For instance, the choice of a reference point of turbulence to the highest location of the bridge beam instead of a weighted average point, means much more than the differences shown in the results in Vedlegg C.

A general finding is that the seed variability is quite large and that the differences found between the models as reported in Vedlegg A and C is largely due to seed variability. As may be seen from Vedlegg A.4 in order to separate differences between the models larger than 10-15 % from seed variability much more than 10 seeds must be used.

For the most important response components for wind loading, the investigations demonstrate that 3DFloat and OrcaFlex give results that are quite similar and that NovaFrame give equally precise response values taking the limitations inherent in the used procedure into consideration.

Wave loading

In the work presented in this report we have investigated the validity of the calculations performed in the FE software 3DFloat and OrcaFlex, with regards to response in a complete bridge structure exposed to environmental loading from wind and waves. In order to perform this comparison 4 wave load cases have been selected, denoted Case 2,3, 4 and 5 for Phase 2 models and Case 3-3, 3-4, 3-5 and 3-6 for Phase 3 models. The results presented in Vedlegg B and D show that there are differences, especially for one of the swell load cases.

We have tried a top down approach, where we started by comparing the response in the bridge girder from a complete model of the bridge structure with all effects included. The reason we went with this approach was that this is the ultimate goal. The programs should give equal results for the complete model with all effects included. However, this approach has shown not to give results that compare satisfactory. We have seen that a frequency domain approach with a lot more components and directions compare very well with time domain results in OrcaFlex. However, we cannot conclude definitely that the results from OrcaFlex are true and the results from 3DFloat are false. This is to a large part due to the top down approach. It is tedious work and maybe not possible to drill down through the differences to extract results that we can hand calculate to get an idea of which program is more correct. Therefore, we suggest that for future work related to benchmarking, resources must be available to follow the following procedure:

- First one should thoroughly benchmark the frequency domain model versus the time domain model by running one frequency at a time for important frequencies in the bridge. By important frequencies is meant frequencies that are important for understanding the behaviour of the bridge structure. Often this will be frequencies that induce large forces in the girder, but not necessarily. One needs to set up some criteria for determining which frequencies are important.

- Further one should compare 3DFloat and OrcaFlex for one frequency at a time and turning on one effect at a time. If the models give satisfactory comparison for all effects we can turn on all effects and compare the results from this. Still only with monochrome waves. When we get this to work, irregular waves can be compared, without spreading. Finally, if irregular waves give satisfactory results, spreading can be implemented. This will constitute a down up method. The benefit of this method is that we qualify 2 time domain models at the same time, calibrated against a frequency domain model.

Taking all of this into account the conclusion must be that for the most important wave responses do 3DFloat and OrcaFlex give similar results.

5 Construction and installation

This appendix summarizes the phase 3 work related to the fabrication, construction, assembly and marine operations related to the end anchored floating bridge across the Bjørnafjord. Key focus has been on establishing a base case for safe and efficient assembly and installation of the end anchored floating bridge. Alternatives to the base case assembly sites are also indicated.

The end anchored floating bridge includes the following main construction elements:

- Bridge girders
- Pontoons
- Columns (between pontoons and bridge girders)
- Abutment south (south support of cable stay bridge)
- Cable stay bridge tower
- Cable stay bridge (bridge girder and cables)
- Abutment north (at Gulholmane)
- Filling and road north of Gulholmane

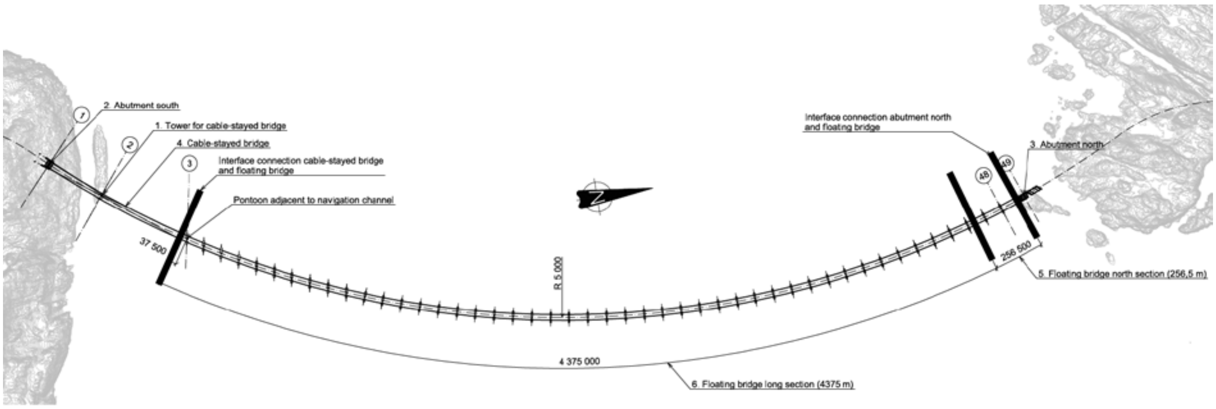


Figure 5-1 Overview end anchored floating bridge – installation sequences extract from –DR-100, appendix A

The end-anchored floating bridge installation sequence is proposed as follows:

1. *Tower for cable stay bridge and anchor (axis 2) and anchor chamber (axis 1)*
2. *Abutment south (axis 1)*
3. *Abutment north with 30 m steel transition structure (axis 49)*
4. *Cable stay bridge (axis 1 to ~ axis 3)*
5. *Floating bridge north section ~256 m (axis ~47 to foundation north steel interface)*
6. *Floating bridge long section ~4400 m (axis ~ 3 to ~47)*

- Fabrication and transport of bridge girder elements are considered “proven” and not subject for detailed evaluations. Focus has been on typical section split and logistics related to bridge assembly and to ensure that the bridge girders design provides means for efficient connection of column structure. The fabrication approach implies staggered delivery of bridge girder elements and does therefore not require large storage facilities
- Pontoon and columns are standard stiffened steel plated structures and thus well suited for fabrication at a large range of ship yards and fabrication methods. The pontoons can be transported on own keel (towed) or transported on a barge or transportation vessel. The columns may be pre-installed at the pontoons for the low bridge
- Construction of abutments on shore and cable stay bridge is only briefly addressed as these are known construction elements using conventional methods

The fabrication and installation schemes have been based on proven methods as well as vessels and equipment commonly available. The main assembly principles are similar to what was proposed in phase 2 with some simplifications.

The base case assembly method has been established based on a temporary assembly site located on 3 standard North Sea barges located in a sheltered fjord, Søreidsvika, which has suitable geometry for the entire 4400 m floating bridge part, infrastructure and close to the Bjørnafjord, see *Figure 5-2*.

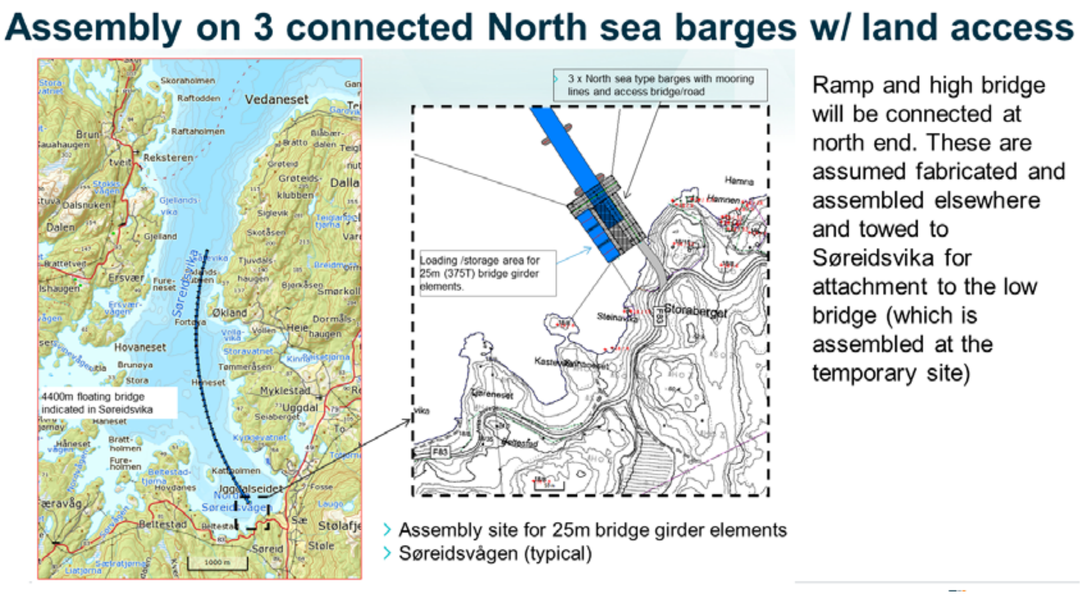


Figure 5-2 Proposed assembly site Søreidsvika

The concept includes minimum bridge girder connections in the fjord since the bridge girder welding for the low bridge will be performed at assembly barges and thus subject to minimal external forces. After completion of joining 4 x 25 m bridge girder elements, the bridge is shifted in the temporary mooring and a new pontoon is installed by first ballasting and then de-ballasting into a guiding system before welding, see *Figure 5-3* and *Figure 5-4*.

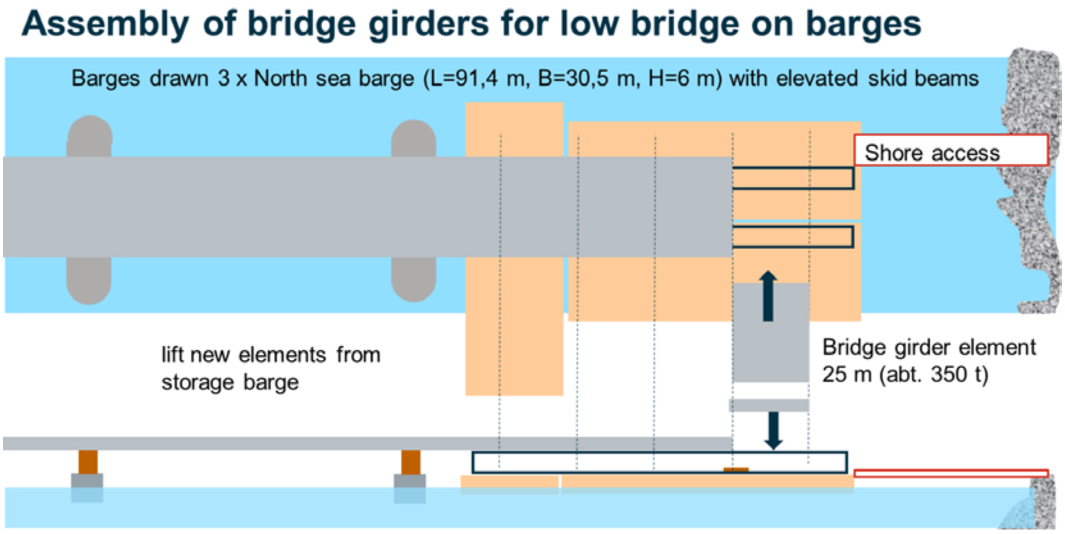


Figure 5-3 Assembly of bridge girder on floating assembly site

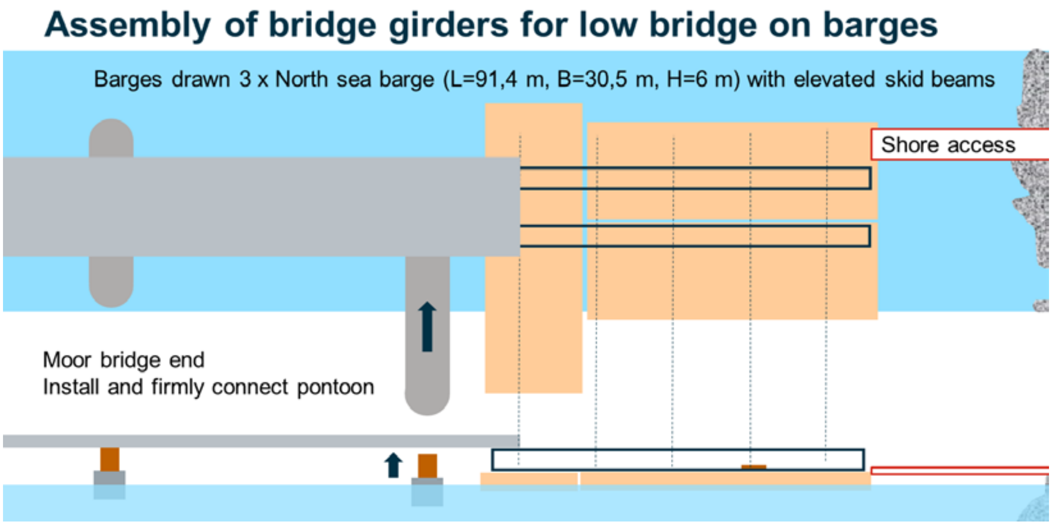


Figure 5-4 Completion of joining 100 m bridge girder, shifting in temporary mooring and installation of pontoon

Since the barge is afloat, the assembly site will experience the same tide as the completed floating bridge. To match the low bridge elevation at the final draft, the assembly will be performed on elevated skid beams. Welding can be performed at all girder sections simultaneously and the estimated turnaround for welding and shifting of the 100 m segment is approximately 3 weeks.

Pontoons for later installation can be stored afloat while bridge girder elements can be stored on barge/barges in the vicinity of the fabrication/assembly site(s).

The barge assembly site can also be located in another sheltered fjord.

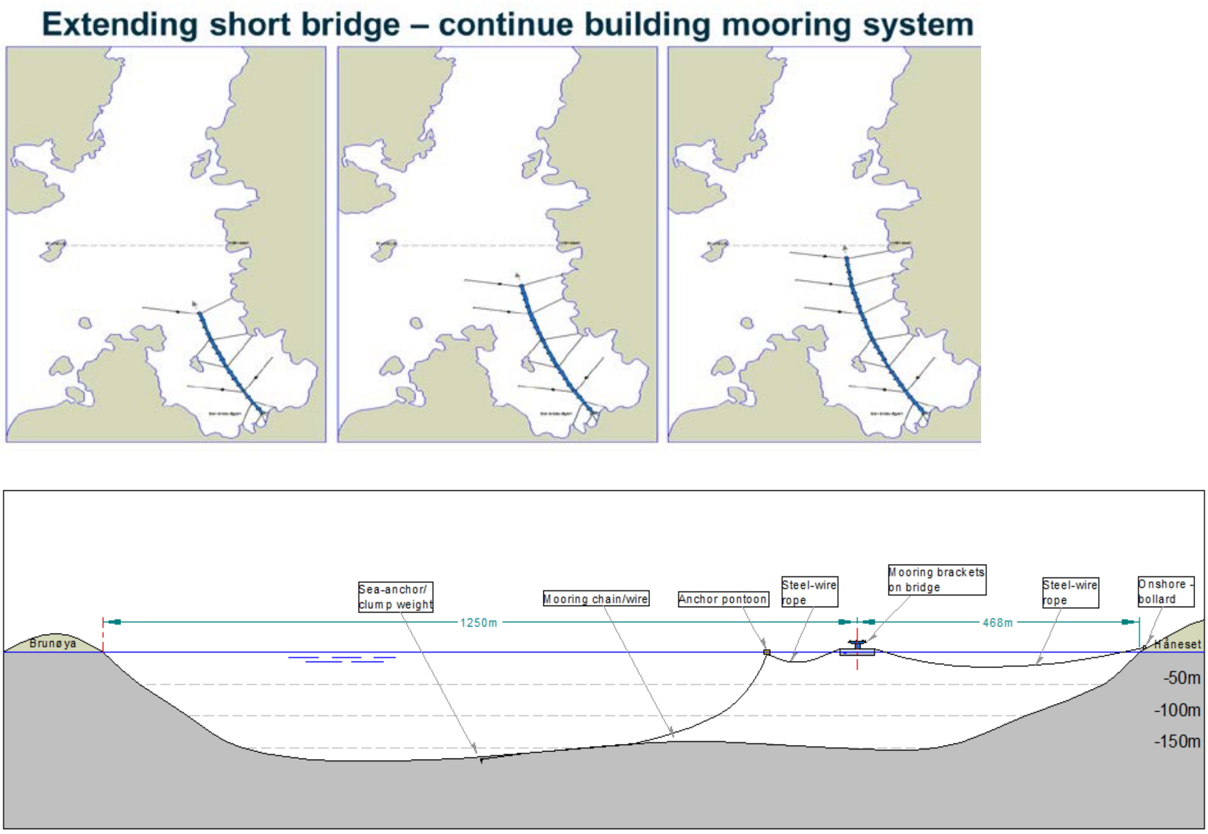


Figure 5-5 Typical temporary mooring system – typical plan view at various assembly stages and profile

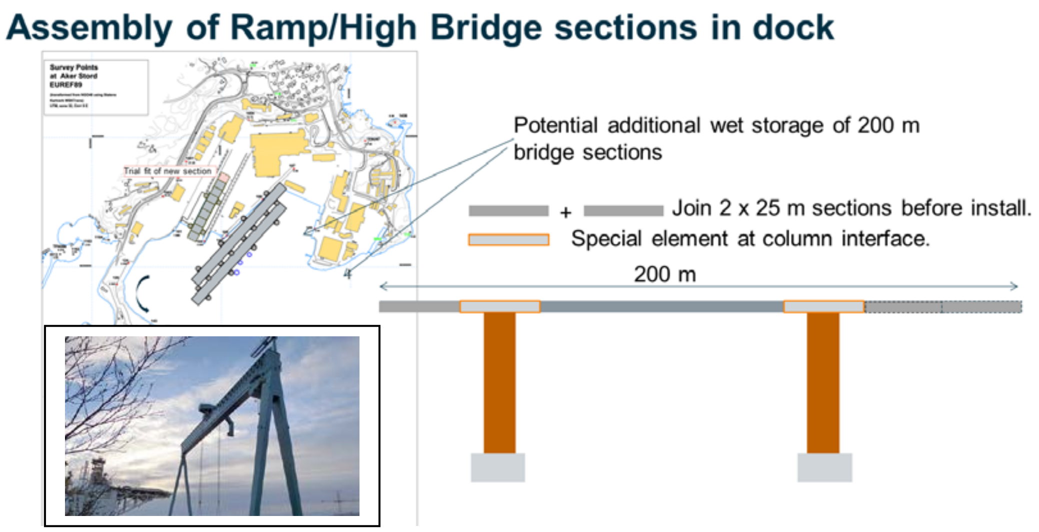


Figure 5-6 Assembly of ramp sections and high bridge section proposed in dock – example Kværner Stord

The base case considers assembly of ramp and high bridge sections at a yard – for example Kværner Stord. The pontoons are floated into the dock and the dock gate is closed. The columns are then installed onto the pontoons using the gantry crane with 800 tonnes capacity. The bridge girders elements are typically joined to 50 m pieces using temporary structure to support the span. Two 200 m segments are then joined to a 400 m segments quayside and stored ready for tow to the assembly site.

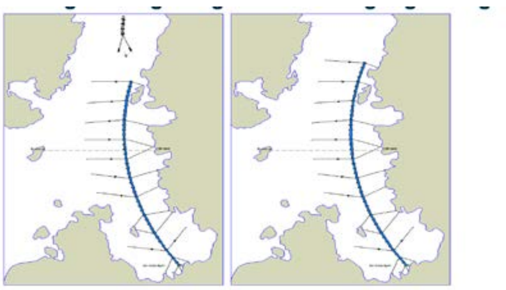
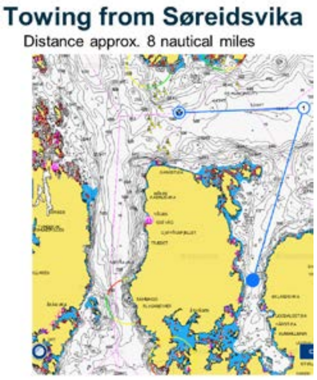


Figure 5-7 Towing and connection of ramp and high bridge segments at assembly site



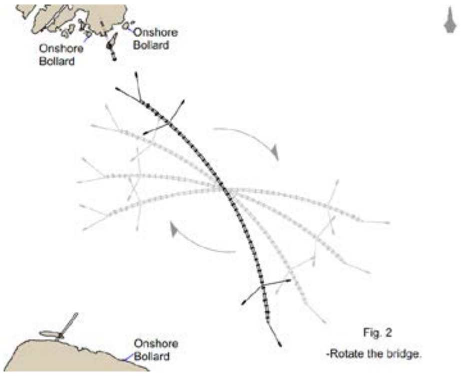
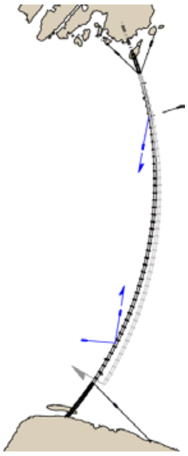


Figure 5-8 Towing from assembly site and turning of 4400 m end anchored bridge

The final installation will be performed by connecting the end tugs to the shore bollards and utilise their winches. The bridge will be fine positioned by a heavy bumper and guide system which will be locked upon reaching target position. Ballasting is required to match bridge girder interface at the cable stay bridge interface. The locking is designed for quick connection within the weather window and designed to withstand a 10 year return period summer storm so that welding is not on the critical path.



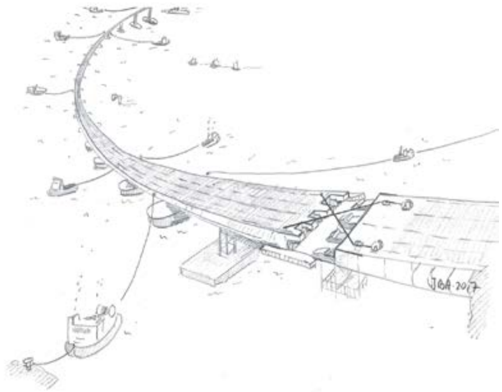


Figure 5-9 Final positioning of 4400 m bridge segment and bumper and guide system

A separate HAZID has been arranged considering the proposed methods. No major risks were noted and towing and connection of the 4400 m floating bridge segment is considered feasible. The total displacement mass is “only” at the order of 134 000 tonnes, so the wind will be governing factor during the marine operations. The main focus in later phases is to further develop the temporary connection system. Co-ordination of the many tugs requires planning and possibly simulator training, but is known from offshore projects and successfully performed for a 931 m floating bridge in 1982 (Bergsøysund).

6 Operation and maintenance

This chapter contain operation and maintenance issues which you do not have with ordinary bridges.

6.1 Corrosion protection

Adequate corrosion protection is required for the steel sections of the floating bridge to prevent excessive maintenance work during the design life of 100 years. For the section above the splash zone, metallising or an ordinary coating system, for example based upon epoxy or polyester is recommended, in combination with a focus on a coating friendly design. For the submerged sections, a combination of sacrificial anodes and a coating system is the best solutions.

In the splash zone, coating maintenance is very difficult, and it is recommended to prevent corrosion by use of corrosion resistant materials without a protective coating; either as a replacement of carbon steel in the full width of the splash zone, or by lining the steel with thin plates of corrosion resistant materials.

For the end anchored bridge, the splash zone has an extension of 3 m, and the base case includes replacement of the side shell plates in this region by plates with Duplex Stainless Steel with 25% Cr (25Cr DSS). This material can be welded directly to the carbon steel plates applying properly qualified welding materials and welding procedures for this type of connection. Corrosion resistant materials are commonly used in for example the offshore industry.

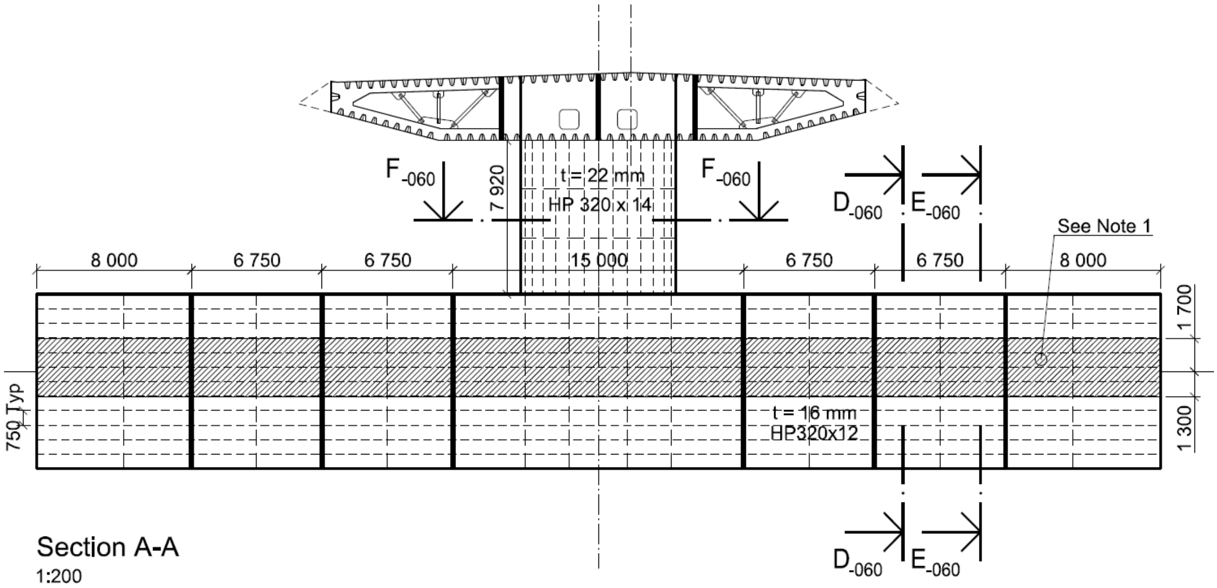


Figure 6-1 Typical pontoon - side view – shaded area indicate splash zone with 25CR DSS

6.2 Method for major maintenance for steel pontoons

Adequate corrosion protection is required for the steel sections of the floating bridge to prevent excessive maintenance work during the design life of 100 years. In the splash zone, coating maintenance is very difficult and the phase 3 pontoon design includes plates of corrosion resistant material (Duplex Stainless Steel with 25% Cr) in the splash zone.

At present, there has not been found any reason for planned major pontoon repairs during the 100 year service life. However, there may be unforeseen degradation or accidental events.

A method for "dry docking" any end anchored bridge pontoon based on commercially available submersible barge (such as BOA 29,30, 35 or 36) fitted with dock walls, has been developed.

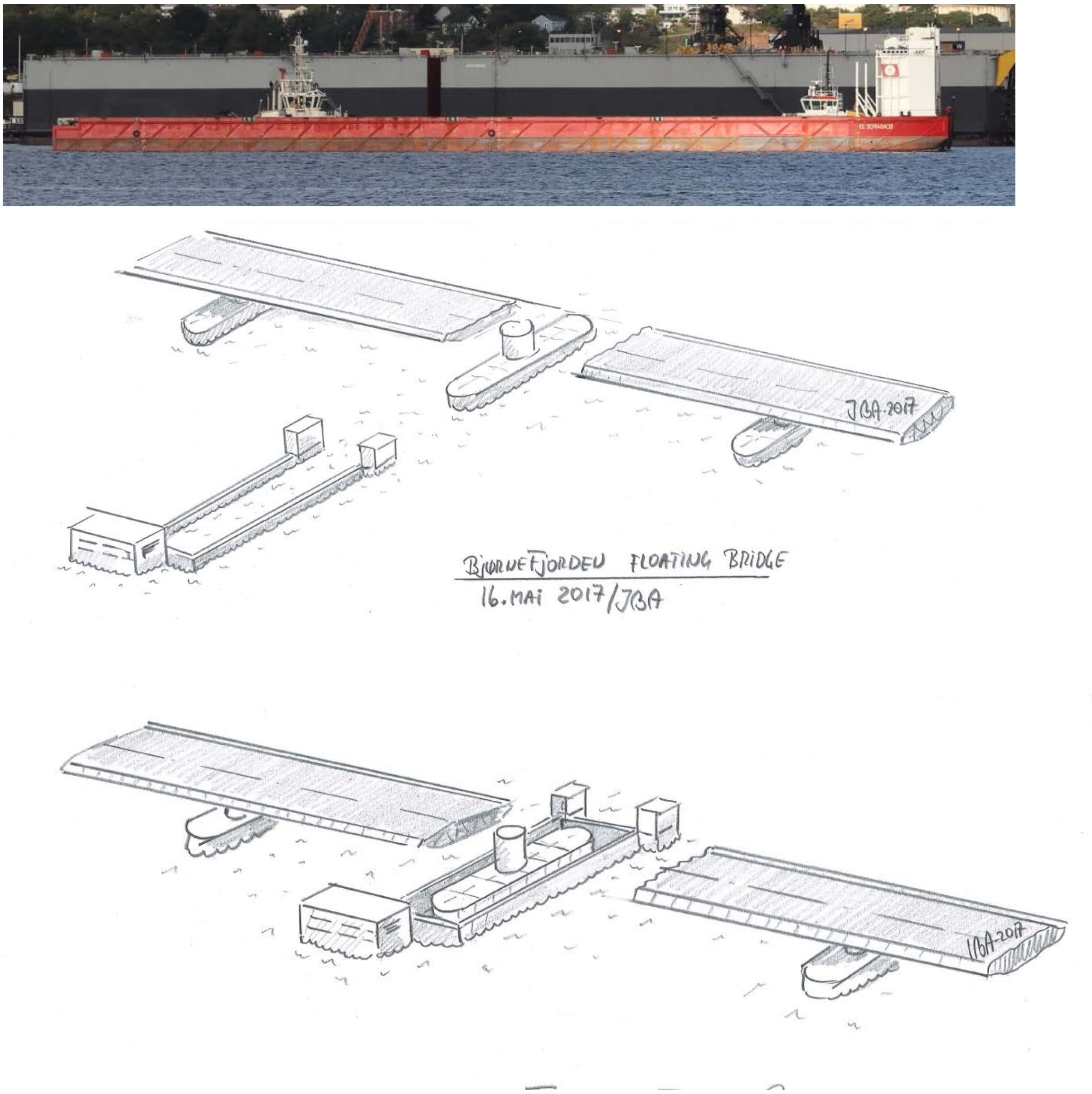


Figure 6-2 Principal method for "dry docking" of pontoon while in service

The emptying of the "dry dock" will introduce a lift onto the pontoon which is transferred to the bridge girder. The amount of water to be discharged from the "dry dock" is dependent on the dock dimension and height of keel blocks. Since there is residual ballast capacity in the submersible barge, a portion of the water in dock can be "moved" into the barge ballast compartment. This is more practical than ballasting the bridge pontoon – with subsequent requirement for cleaning. A net uplift from the "dry dock" onto the bridge pontoon corresponding to a minimum bending radius of 5000 m has been considered which implies a static bending stress of bending 74 MPa. This stress combined with relevant other loads and a summer storm is acceptable for the bridge girder structure. This implies that a speed limit of 80 km/hr can be achieved during the docking operation.

An alternative method also applying the same type of submersible barge, simply lifting the splash zone area above water (about 2 m) has also been developed. To avoid large stresses and significant traffic restrictions, also the neighbouring pontoons should be lifted (by about 1 m). The latter will require temporary buoyancy elements or similar barges.

Finally, a method for complete replacement of a pontoon has been outlined. This implies installation of lifting brackets near the bottom of the column, outfitting a barge with a lifting "fork" and temporarily transferring the bridge column load from the pontoon to the barge by ballasting the pontoon and de-ballasting the barge. The column will then be cut below the lifting fork and the pontoon will be replaced.

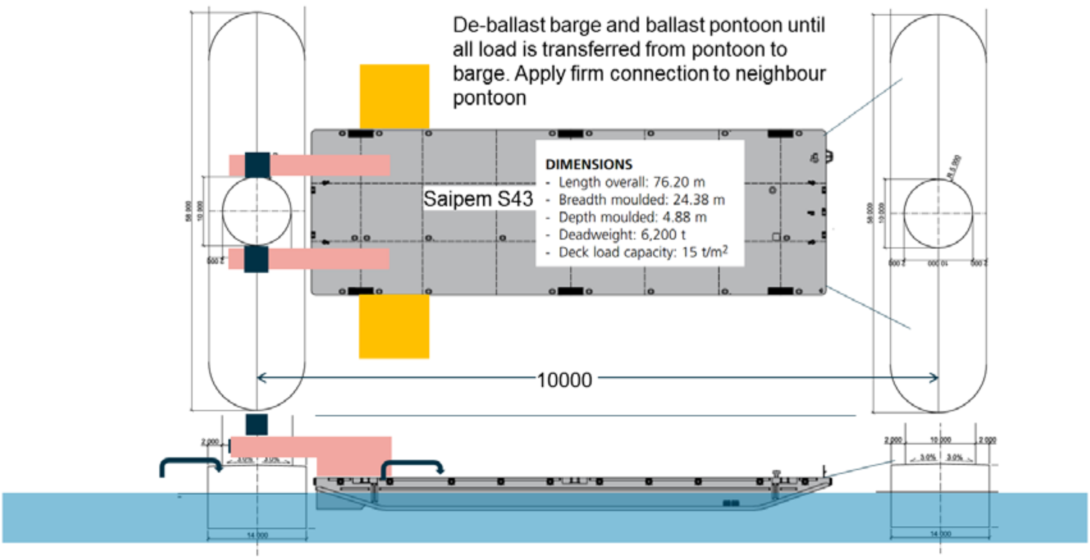


Figure 6-3 Transfer of load from bridge pontoon to barge prior to pontoon replacement

6.3 Availability

The availability of end-anchored floating bridge, defined as when it is open for traffic in at least one lane in each direction, is estimated to 99,8 % in the availability analysis. This is well above the minimum target of 99,5 %. High winds causes approximately 75 % of the downtime, estimated to 13,6 h per year by Aas-Jakobsen, Cowi & Johs.Holt in phase 2.

The current phase 3 concept for the end-anchored (curved) floating bridge is very similar to the side-anchored (straight) bridge from phase 2 with regards to both configuration and traffic management. The main difference in an inspection and maintenance perspective is the side-anchorage and expansion joint for the straight bridge, but that is conservatively neglected and the downtime for inspection and maintenance for the straight bridge from phase 2 is used in this availability analysis "as is". Partial downtime, when the capacity is reduced mainly due to inspection and maintenance, is estimated to 0,3 %.

Input data	
AADT	12 000
Bridge length [km]	5,825
Mill. veh.km/year	25,51

		Downtime		Partial downtime	
		[h/year]	[%]	[h/year]	[%]
A	PLANNED CLOSURE				
A1	Continuous maintenace	0,000	0,000 %	20,500	0,234 %
A2	Rehabilitation	0,000	0,000 %	6,400	0,073 %
B	UNFORESEEN CLOSURE				
B1	Traffic accidents	0,354	0,004 %	1,430	0,016 %
B2-1	Weather conditions	13,600	0,155 %	0,000	0,000 %
B2-2	Vessel impact pontoon	0,012	0,000 %	0,020	0,000 %
B2-3	Vessel impact bridge girder	0,005	0,000 %	0,034	0,000 %
B2-4	Vessel on collision course		0,000 %		0,000 %
B2-5	Ice on stay cables	2,800	0,032 %		0,000 %
B2-6	Fire affecting stay cables	0,004	0,000 %	0,022	0,000 %
B3	Structural failure	0,026	0,000 %		0,000 %
	SUM	**16,801**	**0,192 %**	**28,406**	**0,324 %**

7 Risks and uncertainties

7.1 Risk management process

A key driver from NPRA during phase 3 has been to reduce the cost considerably from phase 2 without jeopardizing the end-anchored floating bridge functionality, availability or robustness.

The starting point for phase 3 was the concept from phase 2 developed by another consultant group.

The phase 2 end-anchored floating bridge concept comprised a curved bridge 4600 m long with a double girder at 6,5 m height supported on concrete pontoons with 200 m spacing and anchored laterally at the ends at the cable stayed bridge tower (sliding support) and at an abutment built on north on “Flua” at ~40 m water depth.

The first step in the risk identification process was to evaluate the concept to understand the reasoning behind the design solutions, uncertainties and risks as well as major cost drivers. This included document reviews, lessons learnt session with chief designer from phase 2 as well as having a central resource from phase 2 integrated in the phase 3 project team. Important benchmarking analyses applying completely different analysis tools and teams applied to the phase 2 were performed as an important task to mitigate risk in analysis assumption, modelling and response evaluations for key parameters.

As a result of above evaluations, significant simplifications and cost and risk reduction potentials were identified and a range of parameter studies were carried out to test the hypotheses. The following main cost and risk reduction philosophies were the underlying goals during the concept design phase:

- Reduce loads on the bridge and thereby structural weights without affecting robustness
- Simplicity in design – simple and known design principles and interfaces, fewest possible construction elements and maintenance friendly design
- Further improvement of construction and assembly methods

Some of these required challenging the initial design constraints regarding positioning of the landing points and road alignment.

The “ultimate” part of risk management is to implement the risk reducing measures in design. A risk management matrix with risk identification, risk evaluation and risk treatment (risk mitigation) is attached to this report to document the key concept decisions made.

The following main results of risk mitigations have been implemented in phase 3 design:

- Bridge girder overall size and complexity considerably reduced
- Pontoons made of steel and substantially smaller than concrete pontoons. The change to steel material and reduced pontoon spacing (from 200 m to 100 m) was vital to achieve the significant reduction of the bridge girder. Risk reduction measures related to potential added maintenance requirements of steel pontoons incorporated in design.
- All foundations on land reducing complexity and construction. Improved design of structural attachments to foundations
- Elimination of all bearings or supports outside the end foundations, ie. no lateral support to the cable stayed bridge tower
- Costly bridge from “Flua” to shore replaced by extending floating bridge now going directly to shore
- Changing orientation of floating bridge arc towards east providing larger robustness and also expected to simplify shore access and continuation of road tunnel

- Safe and efficient bridge girder assembly site and method. Application of known construction and installation methods - tools and vessels commonly available. Large degree of parallel activities benefiting construction schedule.

7.2 Technology assessment

Two end-anchored bridges are already built and operating in Norway. This implies that the concept is proven, but the Bjørnafjorden bridge is considerably longer and there are some structural elements which are somewhat different. From a technology qualification perspective, all structural elements, interaction between them and interfaces are known and proven and thus not requiring technology qualification. Phase 3 has performed special studies concerning long term performance of steel pontoons. There is extensive operational experience in the offshore industry of steel floaters operating offshore. The “new” technology element and largest uncertainty can be attributed to the current design basis environmental parameters and also analysis modelling accuracy considering both the time and space dependent environmental parameters experienced by this very long structure.

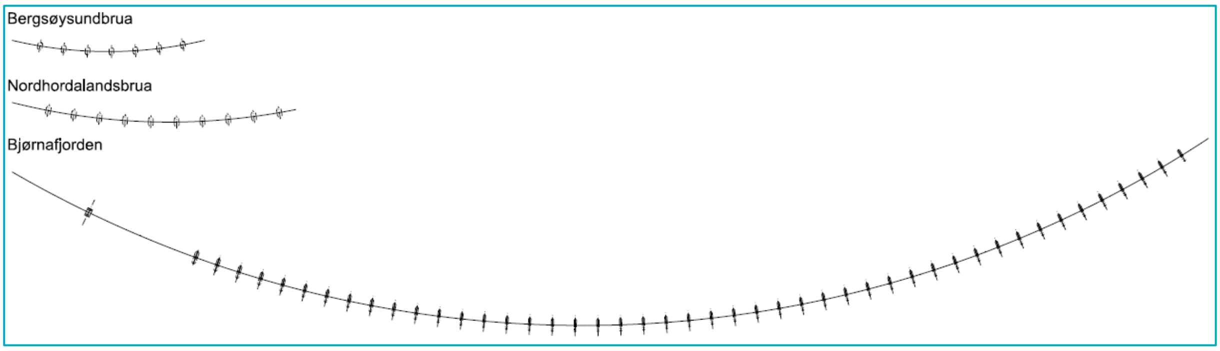


Figure 7-1 Comparison of Norwegian floating bridges

A key risk mitigation effort has been to use proven components in proven combinations to avoid/minimize requirements for extensive technology qualifications.

Table 7-1 Technology categorization as per DNV-RP-A203

Table 7-1 Technology Categorization

Application Area	**Degree of novelty of technology**		
	Proven	Limited Field History	New or Unproven
Known	1	2	3
Limited Knowledge	2	3	4
New	3	4	4

This categorization indicates the following:

1) No new technical uncertainties (proven technology).
2) New technical uncertainties.
3) New technical challenges.
4) Demanding new technical challenges.

Table 7-2 Technology categorization of end-anchored bridge main structure

Element	Technology	Technology references	Application Area	Similar application references	Cat.	Proposed actions next phase
Bridge girder	Proven	Commonly used steel structure in bridge designs.	Known	Structure almost identical to commonly used bridge girders applied in cable stayed bridges and suspension bridges	1	Standard detailing and focus on structural interface/reinforcement of column from pontoon.
Pontoons and columns	Proven	Pontoons and columns in steel are proven components.	Known	Pontoons are used in existing floating bridges. Long term operation of steel pontoons are known from barges, ship and semi submersibles. Design includes a corrosion resistant alloy in the splashzone to limit/eliminate maintenance requirement. Application of such alloys are widely known in offshore sea water systems and also applied to ship (eg. ice breakers).	1	Check whether application of corrosion resistant alloy will limit available pontoon fabrication yards. Note: The design is not dependent on the corrosion resistant alloy, but normal construction steel in splashzone will require periodic maintenance.
Cable stayed bridge	Proven	A range of cable stayed bridges.	Known	Several cable stayed bridge of similar size existing.	1	Consideration of adjustment of tower geometry and/or straighten cable stayed bridge.
Foundation south and north	Proven	Standard anchor solution – placed on land	Known	Existing bridge structures	1	The loads and moments from the end-anchored bridge are large and the foundations are relatively highly utilized so attention to potentially higher loads. Further detailing of structural interface with bridge girder.
Bridge connection to shore	Proven	Filling of rocks and interfacing plate bridge are “common” solutions.	Known	Filling and plate bridges are proven.	1	No structural interaction with end floating bridge

7.3 Risks and uncertainties

The risk management and technology assessment report includes a risk register which includes a systematic evaluation of rationale, activities and status related to key concept decisions. Since several risk reduction reductions measures are related to maintenance as well as construction and installation phases, these are also included with a status on implementation.

In addition, an uncertainty register has also been used an active tool during the design process. This is included in the report mentioned above mentioned report, and also includes an identification of which are considered adequately captured in phase 3 design and which need to re-visited at later project phases.

Key risks for further assessment are:

- Ship impact scenarios. Ship impact scenario with up to 500 MJ (design impact is 300 MJ) analysed in phase 3 and found acceptable. The bridge girder is relatively soft horizontally and shown to "absorb" significant energy by displacement (and has large capacity, ref. -90-RE-103.) Pontoon compartment damage effect analysed, ref. -90-RE-105. With potential "reasonable" increase in ship impact, this is not expected to be global concept issue. When

new ship collision data is available, clarify scenarios and update calculation based on potential new requirements. In phase 3, the cable stayed bridge is moved onto the Svarvhellaholmen. This implies that a ship on collision course will ground prior to hitting the tower, thus reducing collision energy (if the event is relevant). If the tower ends up to be exposed to ship impact, a protection structure should be considered.
- Geometry of cable-stayed bridge. The curved cable-stayed bridge adds complexity and costs in the construction phase. It is recommended to evaluate a straight cable-stayed bridge in the next phase. If a curved cable-stayed bridge is kept, the tower geometry could be further optimized to cater for the eccentric loading from the cables

In addition to the bulletpoints above, there are several risks identified which normally are handled with detailing and planning in coming project phases. With regards to this it is important to ensure continuity of key personnel from phase 3 to next phase and/or extensive lessons learnt.

There are some uncertainties where it would be beneficial to refine/investigate before or during the next project phase:

- HsTp-gamma combination in extreme sea states. Limited data to date. Preliminary evaluations indicate conservative metocean specification in current phase. Could potentially lead to suboptimization against current design basis and potential overdesign.
- Wind coefficients for bridge girder.
- Quantification of damping to be used in analysis model. Many resonant modes are excited and damping is important to limit responses

Another risk/uncertainty that should be evaluated when planning the future design phases is the challenges in quality assurance of complex load and response calculations. Here, one mitigating action could be to require independent verification analyses with a different analysis program as a part of the design process.

8 Recommendation for further work

In next phase of the study, focus should be on more detailing regarding design and verification of capacities. Examples are:

- Bulkheads – optimization and verifications
- Cable stayed anchors and other details in cable stayed bridge - optimization and verifications
- Pontoon structural optimization and verifications
- Stiffener arrangement in bridge girders, especially with respect to fabrication
- Bridge equipment – doors, ladders, walkways, drainage, monitoring, pumps etc.
- Detailed analyses with special focus on fatigue in pontoons and connection details
- Detailed analyses of abutment/girder connections
- Structural arrangement in box girder edge. Beneficial to include the aerodynamic sharp edge in the structure, but focus must be on fabrication and access for inspection and maintenance.

However, there are still important matters which should be further investigated, to improve the overall design of the bridge. Recommendations for further works are listed below:

- Straightening out the cable stayed part of the bridge – beneficial because it will simplify the construction of the cable stayed bridge

- Optimize pontoon spacing. Evaluate minimum spacing acceptable.

- Consider S460 steel. Pontoon spacing can be increased to some extent, without impact on steel quantities in bridge girder.

- Optimize pontoon shape at axis 48 closest to abutment north, and end span length. This will possibly reduce the load actions at the abutment.

- Detailing transition between ordinary cross section and reinforced cross section at abutments. The reinforcement should have minimum impact on response, and a smooth transition is deemed to be beneficial.

- Detailing of cable stayed bridge including shape and size of tower.

- Consider shorter and wider pontoons in low bridge. Beneficial for installation due to better stability. Beneficial for ship impact alongside bridge girder, because a shorter pontoon will reduce torsion in columns when ship hits outermost part of pontoon.

- Define global collapse modes and run push-over analyses

9 References

DNV. (2010). *WADAM User Manual.*

Nygaard, T. A., De Vaal, J., Pierella, F., Oggiano, L. and Stenbro. (2016). *Development, Verification and Validation of 3DFloat; Aero-Servo-Hydro-Elastic Computations of Offshore Structures.* Energy procedia 2016, Vol. 94, pg. 425-433.

SBJ-30-C3-NOR-22-DR-001 General arrangement. (n.d.).

SBJ-30-C3-NOR-90-RE-102-4 Appendix 4 - Methods and theory. (n.d.).

Sofistik. (2016). *Sofistik Basics.*

Statens vegvesen. (2017). *Design Basis Bjørnafjorden Side- and end anchored floating bridge.*