Statens vegvesen

Ferry free E39 –Fjord crossings Bjørnafjorden | 304624

0	28.11.19	Final issue	GSOL	SIGU	FKAM	
A	30.08.19	First issue	GSOL	SIGU	FKAM	
Rev.	Publish date	Description	Made by	Checked by	Project appro.	Client appro.

Client Statens vegvesen	
Contractor DNV·GL	Contract no.: 15/255967

Document name: Verification of OON Floating Bridge Concepts BJF 2019		
Document no.: SBJ-32-C5-DNV-62-RE-026	Rev.: 0	Pages: 58

FERJEFRI E39 - RAMMEAVTALE FJORDKRYSNINGSPROSJEKTET

Verification of OON Floating Bridge Concepts BJF 2019

Statens vegvesen region vest

Report No.: 2019-0300, Rev. 0
Document No.: 110UE04C-62
Date: 2019-11-28

Project name:	Ferjefri E39 - Rammeavtale fjordkrysningsprosjektet	DNV GL AS Oil & Gas
Report title:	Verification of OON Floating Bridge Concepts BJF 2019	Offshore Structures Veritasveien 1 1363 Høvik
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Date of issue:	2019-11-28	
Project No.:	10042624	
Organisation unit:	Offshore Structures	
Report No.:	2019-0300, Rev. 0	
Document No.:	110UE04C-62	

Applicable contract(s) governing the provision of this Report:

Objective:

Prepared by:	Verified by:	Approved by:
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Floating bridges, E39, ferryfree, verification

Rev. No.	Date	Reason for Issue	Prepared by	Verified by	Approved by
A	2019-08-30	First issue	G. Solland	G. Sigurdsson	F. Kamsvåg
0	2019-11-28	Final issue	G. Solland	G. Sigurdsson	F. Kamsvåg

Table of contents

1 EXECUTIVE SUMMARY

On behalf of SVV, DNV GL has performed 3rd party verifications (document reviews and independent analyses) of phase 5 conceptual studies performed by OON for the Bjørnafjorden crossing. Four (K11, K12, K13, K14) floating bridge alternatives have been evaluated. Main focus has been on OON chosen alternative, K12, as shown Figure 1-1 below.

Per agreement with SVV, revision 0 of this report has been issued without any adjustments compared to revision A.

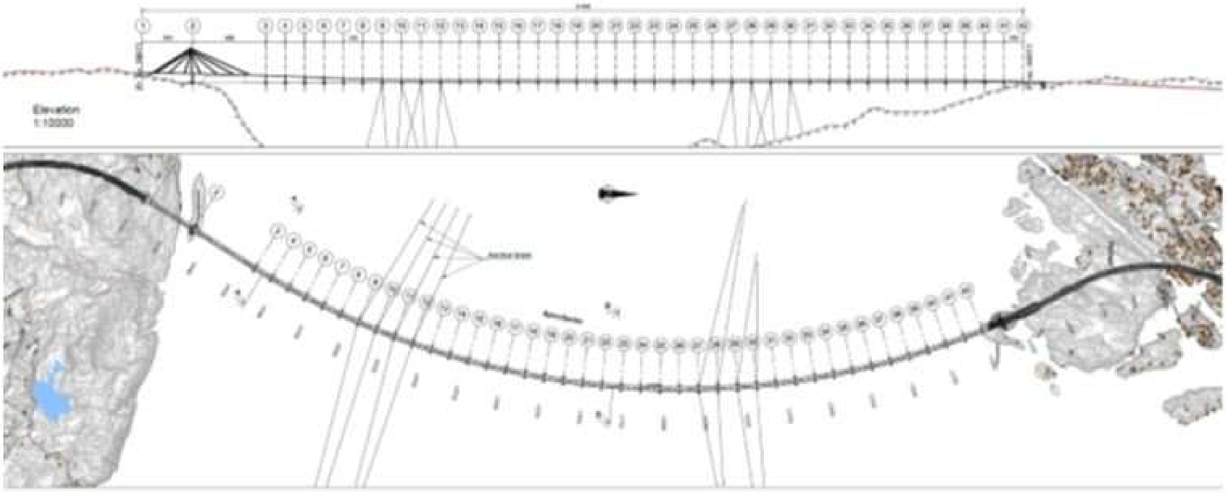

Figure 1-1 OON Chosen alternative K12

1.1 Conclusions

1.1.1 Feasibility of the selected concept

The design review for the K12 concept has not revealed any major deficiencies that may impact the feasibility of the project, but there are uncertainties for certain items that may affect the cost and schedule as listed below:

- There are considerable fatigue loading on the bridge both from environmental loads and traffic. Fatigue capacity checks are carried out for typical details along the bridge. The most fatigue loaded details are not designed to sufficient detail and therefore the calculated fatigue life is uncertain. Furthermore, stress concentrations due to shear lag and changes in cross sections are not accounted for. There are assumed favourable fabrication methods leading to little margins for deviations that may be experienced during construction. The need to design and fabricate fatigue loaded structures is more costly than predominantly static loaded structures. The current design needs to be improved to meet the fatigue requirements, but with future technology developments the cost of these improvements can be reduced.

- The effect of the local traffic has been considered together with the effect of the environmental response for fatigue assessment of details subjected to stress ranges in the longitudinal direction of the bridge. Based on this work a plate thickness equal 14 mm has been decided. However, so far, a fatigue assessment of the dynamic stresses transverse to the longitudinal stiffeners has not been reported. Based on experience from other projects and literature from other countries it is expected that this condition may be governing for the traffic on the deck with heavy traffic. It is expected that the requirement to traffic model will be revisited before a further design phase is started and based on this it is recommended that fatigue analysis for stress ranges transverse to the longitudinal stiffeners are performed before a recommended plate thickness in the bridge deck is decided.

- The selected design has not been proven for the specified energies to boat impacts for the bridge end at the North abutment. Furthermore, energy absorptions from ship impacts against the pontoon is assumed to be taken by plastic deformations in a concentrated zone in the columns but with extensive penetrations of the ships bow into the pontoon structure. It is judged that design development on the basis of the specified energies resulting in limited damages that can be repaired will be costly and that it should be considered to follow an ALARP philosophy for the development of the design. It is judged that the probability of total loss of the bridge is small even with considerable damages to pontoons and bridge girder and the robustness of the bridge is considered to be good against total loss.

- The mooring design analyses have been based on the linearized quasi-static stiffness. This means that the mooring loads will be under-predicted, both for ULS and FLS. However, loads the in the bridge girder will be over-predicted.

- The concept for fabrication and installation as presented in this phase is considered feasible. However, fabrication and installation of the bridge are at this stage described on a high level. Consequently, DNV GL consider the basis for cost and schedule estimates as very uncertain for the construction phase.

1.1.2 Ranking of concepts

The designer's arguments for the selection of the preferred concept is presented in their report SBJ-32-C5-OON-22-RE-002 Rev. C dated 24.05.2019. DNV GL concur with K12 as the preferred alternative but would rank K11 behind K13 and K14 due to the uncertainties in dynamic behavior of such a slender bridge and the risk of losing the entire bridge as there is no redundant load carrying system. The robustness is hence judged to be less for K11 than the other concepts.

1.2 Recommendations for further design development

- In the fatigue analyses very low stress concentration factors have been assumed for the butt welds in the bridge girder. This will require special attention to other hot spots such as where the longitudinal trapezoidal sections are welded to the transverse frames. This relates to stresses in the longitudinal direction of the bridge due to the global response and also to local stresses resulting from the traffic load on the bridge. Due to a significant number of welded connections between the longitudinal stiffeners and the transverse frames it is recommended to investigate this further in the next project phase to arrive at optimal connections that can be used for documentation of fatigue.

- Calibration with experience is recommended to avoid a conservative design. For this purpose, the stiffness of the asphalt layer may be included in the fatigue assessment for transverse stresses due to local traffic loads.

- It is noted that trapezoidal sections are used as stiffeners in general. It is assessed that open stiffeners such as bulb section may be easier to weld to the transverse frames without large stress concentrations. Therefore, it is questioned if other types of stiffeners than trapezoidal sections should be used in areas away from the traffic loaded deck plates. This consideration applies both to the bridge girder and also to the columns where it is expected to be a challenge to achieve good details by using trapezoidal sections as plate stiffeners. However, due to limited drawings of details it is not clear how acceptable details can be achieved at important connections by using trapezoidal sections. Thus, development of drawings showing significant details and welded connections should be given the highest priority in a further concept development. This is also in line with the recommendations for further work by the designer (SBJ-33-C5-OON-22-RE-016).

- It is noted that the wind response is quite sensitive to the statistical variations of the wind field characteristics. It is recommended to establish load cases which consider unfavourable combinations of wind field parameters.
- Towing and installation of the complete floating bridge are complicated marine operations, and further planning is required.
- The locking system for the construction joints in the main bridge girder must be engaged quickly (i.e. within the weather window) during the installation of the main floating bridge, but the documentation regarding this is immature. The locking system should be further developed.
- Local reinforcements and temporary steel are required to transfer loads during construction/assembly. Further detailing and to clarify possible consequences of remaining temporary steel on in-place (fatigue) stresses are recommended.
- It is recommended to further develop the metocean design basis for the next phase in the development of the Bjørnafjorden bridge. The analyses of the K12 concepts from AMC and OON have shown that the dynamic response in the bridge is sensitive to the current speed; a large current speed will reduce the response due to the increased damping. It is therefore necessary to define the current speeds and directions that shall be combined with extreme wind and wave conditions and also FLS conditions. For the FLS analyses it should also be specified how to combine wind sea and swell. Analyses so far by the designers and DNV GL have been performed without any wind load on the bridge girder in longitudinal direction. It should be investigated if this simplification is acceptable.

2 INTRODUCTION

2.1 General

During fall of 2018 SVV set out two conceptual studies to develop a floating bridge concept for crossing Bjørnafjorden (BJF). DNV GL has been chosen as independent verifier by SVV for this conceptual work. This is reflected in Frame agreement no 15/255967. DNV GL scope of work related to 'BJF 2019' is described in CTRs 610, 615, 620, 625 and 630. For this report reference is made to CTR 615 with focus on document review and CTR 620 with focus on independent analyses of OON chosen bridge concept. This DNV GL report is charged to Ctr 630, reporting to SVV.

This report deals with the concepts evaluated by design group OON. A total of four (4) concepts have been investigated by each of the design groups and one of these is recommended for the next phase (part B, Dec. 2019 – Dec. 2020). The activity plan (part A) set up by SVV were as follows:

Time	SVV activity plan	Responsible
19/11-18	SVV hand over design basis documentation to the two chosen design groups for Part A and project kick-off	SVV
18/01-19	Routing of roads for the 4 bridge alternatives accepted by SVV	OON
28/01-19	Status report no 1 with concept ranking issued by OON	OON
29/03-19	Status report no 2 with estimates of masses, costs and updated drawings/descriptions for all 4 alternatives issued by OON	OON
07/05-19	Verification of technical quality completed based on review of existing documentation for the 4 bridge alternatives. This verification also including interviews of OON. Interviews to be performed by DNV GL.	SVV
24/05-19	Report from OON on their chosen bridge concept including evaluations for the three other bridge concepts.	OON
30/06-19	Documentation basis (drawings and descriptions) for investment estimates of chosen bridge concept	OON
15/08-19	Final documentation delivery of recommended bridge concept	OON
31/08-19	Final documentation of the three (3) other bridge concepts	OON
31/08-19	Resource-diagram prognosis for the period Dec. 2019 – Dec- 2020 (part B)	OON
31/08-19	Part A completed	OON

For the Bjørnafjorden crossing several different bridge alternatives have been considered for the last 2 – 3 years. Currently the BJF crossing is into project Phase 5 and the following 4 floating bridge concepts have been evaluated:

K11 – Curved, end-anchored floating bridge in accordance with phase 4 of the project.

K12 – Curved, end-anchored floating bridge with supplementary side moorings

K13 – Straight, side anchored floating bridge

K14 – 'Straight' S-shaped, side anchored bridge

3 BASIS FOR WORK

3.1 Governing documents from SVV

SBJ-32-C4-SVV-90-BA-001 Design Basis Bjørnafjorden floating bridges Rev. 0 dated 19.11.2018

SBJ-01-C4-SVV-01-BA-001 MetOcean Design basis. Rev. 1 dated 30.11.2018

SBJ-32-C4-NTNU-22-RE-001 Dynamic stability of elastic nonlinear systems subjected to random excitation Rev. 1. Dated 17.12.2018

3.2 Definition of verification objects

The conceptual verification is based upon drawings and reports from the designer. The four different concepts investigated are:

K11 – Curved, end-anchored floating bridge in accordance with phase 4 of the project

K12 – Curved, end-anchored floating bridge with supplementary side moorings

K13 – Straight, side anchored floating bridge

K14 – 'Straight' S-shaped, side anchored bridge

K12 has been selected by the designer and has been further detailed compared with the other alternatives. The verification of the K12 concept is based on the drawings as listed in Appendix B and with review of the reports also listed in Appendix B.

The remaining concepts are reviewed assuming a structural design as listed in drawings in Appendix C and with review of the relevant reports also listed in Appendix C.

3.3 Scope

The scope for the review is related to check that the load carrying capacity and the sustainability of the four floating bridge concepts is according to the defined specifications. Comments to the plans for fabrication and installations are given in order to identify possible risks that may impact the conclusions about cost and schedule.

Bridge aesthetics and road alignment is not commented upon by DNV GL. The review of the design documentation has not included check the correctness of the summary tables for the quantities and the cost estimates.

3.1 Verification objective

The verification objective is to assist SVV to select the best of the four concepts chosen for design developments for the Bjørnafjorden crossing. Furthermore, it is to ascertain that design flaws will not lead to increase in cost and schedule estimates outside the intended limits as the project develops.

3.2 Verification methodology

The verification of the bridge concepts is made by a combination of review of design documents as drawings and reports and independent analyses. Results from the independent analyses are given in a separate report. The verification activity has been carried out in parallel with the design development. Verification comments are issued and discussed with SVV and the designers in meetings. Verification comments and answers are included in Appendix A.

4 VERIFICATION COMMENTS TO THE SELECTED CONCEPT K12

4.1 General

The following comments are noted by review of the drawings and reports as listed in Appendix B. The review benefits from the independent analyses for certain load cases that are carried out in parallel and reported in (SBJ-32-C5-DNV-62-RE-024-A).

4.2 Hydrodynamics

4.2.1 Introduction

The hydrodynamic loads on a floating bridge involve loads due to waves and current and possible combined loads due to wave-current interaction. The hydrodynamic loads act on pontoons and mooring system. Loads include both excitation and damping loads. While exciting loads are usually obtained from well-established software, the damping loads often requires special considerations and are estimated from tabulated values in codes. Standard software for analysis of hydrodynamic loads on general marine structures may be used, however there are some effects that are unique for a long floating bridge inside a fjord that requires special considerations.

Since a long floating bridge will have a wide range of significant natural frequencies, from high and moderate frequencies for vertical motions to low frequencies for horizontal motions, hydrodynamic loads in the same frequency range should be investigated. The dominant hydrodynamic load contributions are the first order wave loads on the pontoons. However, slowly varying wave loads caused by low-frequency second order difference frequency wave load components that may excite horizontal resonant motions, must be included. Also, an assessment of approximations usually applied for such low-frequency loads on marine structures should be carried out.

The proximity of the pontoons may affect the wave loading. Hence, hydrodynamic interactions between the pontoons needs to be evaluated. Another effect, different from marine structures in open sea, is the inhomogeneous wave and current conditions, both in magnitude and direction, along the bridge. The effect of this on the global response must be checked. A combined wave-current condition influences the excitation (encounter) frequency which again influences which eigenmode that may be excited. This effect also needs to be checked. Depending on the freeboard of the pontoons and the severity of the governing wave conditions, freeboard exceedance and possible green sea may affect the hydrodynamic loading on the pontoons.

The pontoon hydrodynamics are described in OON document SBJ-32-C5-OON-22-RE-008.

4.2.2 First order wave loads on pontoons

The hydrodynamic pontoon characteristics are analysed applying the frequency domain programs Wamit and Wadam.

The global effects of variations in the hydrodynamic properties are investigated both in time domain (3DFloat) and in frequency domain (DynNo). The following variations with respect to influence on hydrodynamic properties are studied:

- Differences in lower order and higher order panel representation in Wamit/Wadam
- Diamond-shaped pontoon compared to canoe-shaped pontoon
- Length, width and draft changes on the canoe-shaped pontoon

As expected, the difference between results from low order and higher order panel representation is found to be negligible for the first order wave loads.

The diamond-shaped pontoon has approximately the same cross-sectional area and displacement as the canoe-shaped pontoon. The results show that the wave excitation loads differ between the two shapes. There is however no clear conclusion with respect to which of the shapes that gives lowest loads. Which of the shapes that gives smallest loads depends on motion mode and wave direction.

Likewise, the investigations in length, width and draft changes give variable results with respect to possible reduction of excitation loads.

The actual bridge response is not dependent on the wave excitation alone as it is influenced by the resulting bridge modes. In order to evaluate the bridge and pontoons as a whole, frequency domain analyses of the global bridge model are performed applying computer program DynNo. The global bridge analyses show that a small change in pontoon geometry gives significant effect on global bridge response. However, none of the analysed geometry changes gives only favourable or unfavourable results.

OON has also performed a CFD analysis applying regular waves on a single pontoon. The derived excitation forces are higher than the Wamit analyses, except for surge force at 0 degrees wave heading. The difference in forces between the two methods may be as much as up to a factor of two. The reason for this is stated to be due to non-linear and viscous effects included in the CFD analyses.

DNV GL considers this deviation between CFD and Wamit to be too large to be explained by non-linear and viscous effects.

4.2.3 Pontoon added mass and damping

Pontoon added mass and damping are studied in the same way as the wave excitation loads. The difference between higher order and lower order panels are found to be negligible. Geometry changes give no clear answer with respect to optimal shape.

The computed hydrodynamic properties are reported by showing effects of geometry variations and panel modelling. Complete result overview of base case pontoons is not given. Hence, a full comparison between DNV GL results and OON results cannot be given. The given results seem however to compare fairly well with the DNV GL computed hydrodynamic properties.

4.2.4 Hydrodynamic interaction effects on pontoons

OON concludes that the wave induced loads and responses are affected by the hydrodynamic interaction between the pontoons. Hydrodynamic interaction has also been briefly checked by DNV GL where the same conclusion is found.

The interaction effect depends on the wave period. An example of this is given in Figure 4-1 where heave added mass and potential damping for a single pontoon computed by DNV GL is compared with results for three pontoons. Typically, multi body simulations show large interaction effects at shorter wave periods while the results tend to coincide with single body results at longer wave periods.

The long and slender pontoons give rise to a standing wave field between the pontoons, and a large amount of energy is propagating between the pontoons with limited dissipation of energy out from the standing wave field. Since linear potential theory analyses do not include viscous energy dissipation, the interaction effects are likely to be overestimated, especially at frequencies with standing waves.

DNV GL has suggested to include a damping lid on the water surface. A damping coefficient effectively reduces the wave amplitude. This approach which removes unrealistic amplification of the water surface elevation has in other projects improved the results compared with model test results. The values of the damping coefficient could preferably be experimentally determined, but it should be noted that even with a damping coefficient close to zero the damping lid model will improve the results.

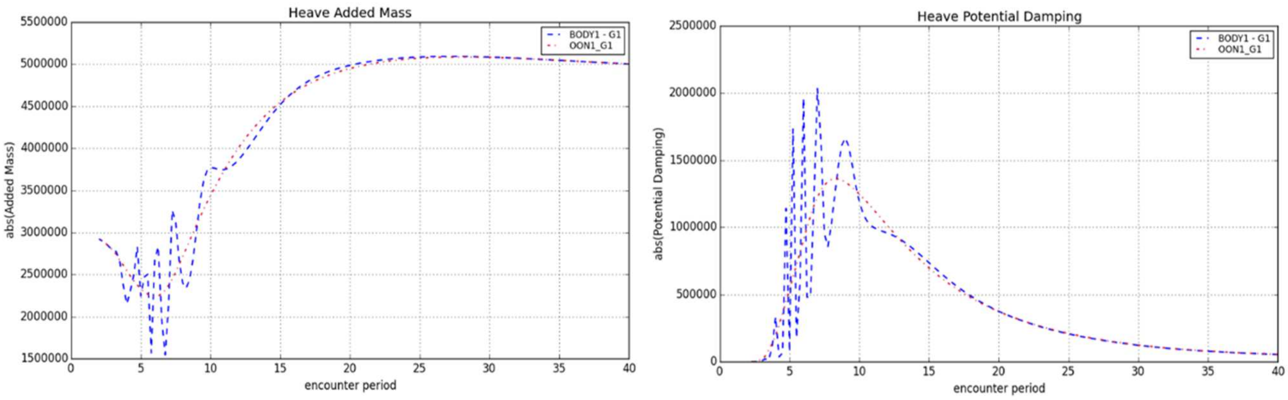

Figure 4-1 Typical differences between single body and multi body analyses – DNVGL computed heave added mass and potential damping for one body (red) and three bodies (blue).

OON also investigates the hydrodynamic interaction effect on a global bridge model both in frequency domain (DynNo) and in time domain (3DFloat). The conclusion is that interaction impact the global bridge behaviour. The effect is largest for weak axis bridge girder moments, while the impact is limited for strong axis moments and torsional moments.

4.2.5 Second order wave effects

Mean wave drift forces are investigated regarding panel models and geometry changes. As for the first order wave forces a clear optimal geometry is not found with respect to second order wave forces. It should be noted however that OON concludes that higher order panels give more accurate results than lower order panels for mean drift forces.

Simplified bridge response evaluations are performed where higher order hydrodynamic load effects are found to be significant for difference frequency loads while sum frequency loads are negligible. The difference between applying the full quadratic transfer function (QTF) and applying the Newman's approximation is investigated in Wadam on a fixed pontoon. OON concludes that the Newman approximation is found to be questionable. DNV GL questions this conclusion.

4.2.6 Viscous damping

OON has estimated pontoon viscous damping based on CFD analyses. Results are compared with values given in DNVGL-RP-C205. In addition, model tests results performed at Sintef Ocean are evaluated.

CFD analyses in steady current are reported in OON document SBJ-32-C5-OON-22-RE-008 both in chapter 7.2 and 7.9. The difference between these two analyses are not described in the document. In chapter 7.2 the CFD analyses gives a surge drag coefficient of Cd=0.3. In chapter 7.9 a surge drag coefficient Cd=0.569 is given. The deviation is not commented upon in the document.

The Cd=0.3 result is stated to be comparable to the values provided in RP-C205. It should however be noted that these values given in the RP are for Reynolds number Rn≈105 while the flow past the pontoons is in the post-critical flow regime Rn>106 where the drag coefficient should be somewhat higher.

The steady state drag coefficient obtained in the model test is Cd=0.55. The tested pontoon geometries differ however from the ones currently being used in the design. Hence, these results cannot be used directly.

Viscous damping from mooring lines are limited as OON uses fibre rope moorings in their design. In order to reduce the strong axis global bridge response, it may be relevant to apply means for increasing the drag damping. The possible use of appendices on the pontoons to increase the viscous drag damping is investigated. It is noted that in addition to increased damping, the applied drag plates will increase the added mass and the wave excitation forces.

4.2.7 Pontoon freeboard and wave run-up

Pontoon freeboard and wave run-up have not been discussed in the OON documentation.

4.2.8 Wave-current interactions

The wave-current interaction effect is investigated by comparing first order wave excitation forces computed in Wasim. OON concludes that the current has almost negligible effect on the wave excitation forces.

Global response analysis of the bridge is not carried out with respect to the wave-current interaction effect. In DNV GL opinion, a global response analysis should have been included. The encounter frequency for the waves on the pontoon will be affected by the wave-current interaction which again could excite different eigenmodes of the bridge.

4.2.9 Inhomogeneous wave conditions along bridge

Variation of wave conditions along the bridge is included and discussed in the verification and validation of global analysis, OON report SBJ-32-C5-OON-22-RE-004. Local scaling of the wave field is implemented. Location specific wave spectrums are given specifying, position, significant wave height, wave peak period and the Jonswap shape parameter. Wave direction is unchanged.

From the performed time domain simulations OON concludes that especially the weak axis moments are sensitive to the local wave conditions.

4.2.10 10.000 year conditions

The 10.000 year return period conditions have not been investigated by OON at this stage of the project.

4.3 Aerodynamics

4.3.1 Wind field characteristics

The MetOcean Design basis (SBJ-01-C4-SVV-01-BA-001) highlights that there are statistical variations in the wind field characteristics. In report SBJ-32-C5-OON-22-RE-005-0-AppC, OON has carried out a sensitivity analysis of the effect of these variations on the strong axis bending moments along the bridge. Both variations in the wind spectrum and in the coherence function were considered, but only for the along wind turbulence component. It was found that the sensitivity to these parameters are quite significant. Higher values for the spectral density coefficient Au were shown to give higher bending moments. Similarly, lower values for the coherence coefficient Cuy were found to give larger bending moments. As a consequence, these effects should be considered in the design. It is, however, not fully clear to DNV GL how the wind fields have been generated for the coupled analyses, and therefore also which parameters that have been applied in the design analyses.

4.3.2 Aerodynamic stability

The Bjørnafjorden Bridge may be exposed to potential threats arising from aerodynamic and aeroelastic instabilities. Several aerodynamic instability phenomena, both local and global instabilities, have been examined in report SBJ-32-C5-OON-22-RE-004. Instability phenomena investigated include,

- Coupled flutter
- Torsional flutter
- Galloping
- Vortex shedding
- Static divergence
- Wake-induced instability
- Rain-wind induced instability

An overview of the various wind-induced instabilities being assessed for different bridge components are shown in Figure 4-2.

Instability / Bridge element	*Bridge deck on a floating part*	*Cable-stayed bridge*	*Stay cables*	*Entire bridge*	*Bridge under construction*
Coupled flutter		✓		✓	✓
Torsional flutter	✓				
Galloping	✓		✓		
Vortex shedding	✓	✓	✓		✓
Wake-induced			✓		
Static divergence	✓				
Rain-wind-induced			✓		

Figure 4-2 Table 6 in report SBJ-32-C5-OON-22-RE-004

Coupled flutter instability is investigated using aerodynamic derivative data from three other bridges since such data are not available for the specific Bjørnafjorden bridge section. The check was carried out for the full bridge and for cable stayed bridge. It was found that neither is susceptible to flutter instability. *Torsional flutter* instability has been checked for the bridge deck using static load coefficients for the three other bridges (as above). It is concluded that bridge deck is not sensitivite to torsional flutter. Similar checks are required when aerodynamic derivative data become available for final bridge design.

The bridge deck is checked with respect to possible *galloping instability* and it is found that the bridge deck is not sensitive to this instability. Simplified calculations are performed based on the latest geometry and structural properties for the stay cables. It is found that stay cables may be sensitive to *dry galloping*. It is recommended to examine this instability using more sophisticated methods. The stay cables are also checked for possible *wake galloping instability* and *rain-wind-induced instability* and are found to be not sensitive to these possible aerodynamic instabilities. The bridge deck is checked for *static divergence* and found to be not sensitive.

Simplified calculations have been carried out to check whether the bridge deck and the stay cables are sensitive to excessive vortex-induced vibrations (VIV) during operation. The calculations show that the bridge deck is not sensitive to excessive VIV. However, wind tunnel tests are needed to exclude possibility of VIV. The analysis did not include the effect of traffic. This should be included in a more sophisticated analysis of VIV. Using calculation methods in Eurocode, the stay cables are found not to be sensitive to excessive VIV during operation, however VIV may potentially be an issue during construction.

With aerodynamic derivatives from wind tunnel tests and state-of-the-art calculation methods aero-elastic response can be assessed and possible aerodynamic instabilities mitigated. DNV GL judge that aerodynamic stability is satisfactory investigated at this stage of the project.

4.4 Parametric excitation

OON has carried out an extensive analysis of possible parametric resonance for the bridge. The physics of parametric resonance is explained by a dynamic axial force which effectively causes a time-varying geometric stiffness which can lead to a typical Mathieu instability if the frequency of excitation is twice a natural frequency of the system. Theoretically, instabilities can also occur for other frequency ratios as described in the NTNU report number SBJ-32-C4-NTNU-22-RE-001. NTNU has derived a simple stability criterion that has been used to show that all load cases are passed except for a load case with pure swell waves. OON has further investigated this load case and derived a second criterion where hydrodynamic damping is included and an upper bound for response due to parametric excitation is established. The analysis shows that the resulting stresses at resonance are not likely to exceed the design stresses for the bridge for the pure swell load case.

DNV GL has not reviewed nor verified the criterion developed by NTNU and has not previously been involved in similar bridge projects where Mathieu instability has been an issue. Since the criterion is based on analysis of a simple structure, we support OON's statement that if the criteria are not passed it does not mean that parametric resonance will be a problem, rather that such resonant response cannot be excluded and that further investigations are needed. OON has proposed recommendations for further work. DNV GL supports this.

4.5 Ship impacts

Global dynamic non-linear analyses in time domain are performed accounting for non-linear effects in both load and response. Local assessments have been performed for impact between ship bows vs pontoon and ship deckhouse vs bridge girder. Based on the calculated damage, the following damaged condition is considered: flooding of pontoons, loss of anchor mooring lines and damage to the deck girder.

DNV GL is aware that the criterions for boat impact might be made less strict in the next phase of the project. In the following, DNVGL will comment as if the requirements given in the design basis remains. The design basis is requiring that impact is investigated in all directions. It is also observed that bow impact parallel with the bridge are using same speed as orthogonal to the bridge which is according to the present design basis.

The performed non-linear analyses cannot be properly verified without performing similar kind of independent analyses, but as far as we have observed, the chosen input parameters appears to be adequate.

The design of the column and bridge girder is performed with the purpose of simple repair if damage occur. Since repair or substitution of the column with pontoon is easier than repairing the bridge girder, the design is aiming to make the column weaker than the bridge girder in such a way that any damage will occur in the column and not in the girder. Damage to the structure due to an ALS condition is normally acceptable due to the low probability, and the risk for expensive repair could be accepted in order to save construction cost.

The moment in the north end exceeds the present bridge girder capacity. It is stated that local reinforcements are needed. If so, the cost for these reinforcements should be accounted for. As mentioned above, repair after an ALS event is acceptable. Thus, if it can be proven without reinforcements that sufficient energy can be absorbed during impact and that integrity is maintained for post impact condition, these reinforcements could be voided by accepting the risk of repair in the north end due to impact.

It is said that overtopping of waves must be expected in the damaged condition due to increased draft of 3.1 meters. It is thus required that the local design is accounting for this.

DNVGL has not seen any drawing showing the watertight compartments. It is assumed that the compartments used in the calculations are according to what is accounted for in the cost estimate.

4.6 Global response analyses

The global response analyses have mainly been performed in the following programs:

- 3DFloat: Time domain
- DynNO/ABAQUS: Frequency domain
- Sofistik: Static analyses

For dynamic response calculations, results are sensitive to analysis parameters, such as frequency resolution, time step, simulation length and others. The selection of parameters has been based on sensitivity studies. The document review has not revealed any important issues that have not been handled properly. The most important parameters used include:

- 1 hour simulation time
- Time step 0.1 sec
- 10 seeds
- Number of elements between pontoons: 6
- Structural damping: The ratio is set to 0.5% for two frequencies. The angular frequencies chosen are 0.0785 rad/s and 2.094 rad/s, corresponding to period range 3-80 seconds. This means that the damping at the intermediate frequencies are less than 0.5%, which is conservative.

Polyester lines have complex visco-elastic and non-linear response characteristic. For mooring analyses, this can be modelled by a non-linear quasi-static characteristic applicable for mean environmental loading, and a linear, stiffer characteristic for dynamic loading.

The mooring design analyses have been based on the linearized quasi-static stiffness. This means that the effect of the mooring lines on the eigen modes is under-predicted, giving too large natural periods. This will result in conservative results with respect to the response in the bridge girder, while mooring loads will be under-predicted, both for ULS and FLS

Inhomogeneous wave conditions have been accounted for as a base case.

In the design basis it is said that the characteristic response from environmental loading shall be taken as the following fractiles from the 1 hour extreme value distribution.

- ULS: 90%
- ALS: 95%
- SLS: 50%

Load combination factors are given in the Design Basis. For strong axis bending moment it is the 100 year environmental condition that has been found governing. For this condition, the following load factors apply:

- Permanent load: 1.2
- Temperature load: 0.84
- Environmental loads (wind, waves, current, tide): 1.6

For the time domain simulations, analyses are done for different directions of wind, wave and current according to the metocean specification. The 90% value from the short-term extreme response is taken as the 100 year response from 10 different seeds as found from the fitted Gumbel distributions.

During each coupled analysis with traffic, cars are sent from one side of the bridge to the other at a speed of 70 km/h.

A number of different sensitivities have been analyzed. Key findings are given below:

- Influence of swell waves on total response. Contribution from swell waves may give a small increase to the governing design loads.
- Conditions with return period of 10000 years could be governing with regards to design loads.
- Evaluation of critical wind direction. The southernly winds give the largest forces in critical positions along the bridge due to the large turbulence intensity. The lateral responses for southern winds are comparable with the load effects from the westerly winds.

The review has not identified any major issues with respect to the analysis strategy

4.7 Global analysis results comparisons

Reference is made to the independent analysis report, (SBJ-32-C5-DNV-62-RE-024-A). The main findings are:

4.7.1 ULS

The ULS capacity made as a von-Mises stress check is exceeded at Axis 3 to 9 and close to the abutment North. Independent buckling checks are not carried out, but it is expected that reinforcement at these cross-sections will also make the buckling capacity acceptable. The stress check is based on beam theory and that stress increase due to local stiffening and shear lag is not accounted for.

The available free movement space for the bridge girder at the tower is not sufficient to avoid contact from the bridge girder into the tower for ULS loads. The risk of clash will be drastically reduced by narrowing the girder to the width without the wind nose.

4.7.2 FLS

The independent analyses carried out by DNV GL determine the contribution to damage from environmental loads in the bridge girder. The results from the screening analysis show a minimum fatigue life of 148 years. This number should be reduced with the local stress increase and it is expected that will bring the fatigue damage from environmental loads significantly below the required life of 250 years.

The contributions from traffic and tidal variation is not part of the independent analyses by DNV GL. The damages will add only at certain details in the bridge. Tidal variation will only lead to damage close to the ends and traffic will predominantly give damage in the bridge deck. However, the fatigue loading as determined by DNV GL seems to be above the required capacity for large part of the structure.

4.7.3 Mooring

The size of the bottom chain needs to be increased and this can be included at a small cost increase. Thereby, the strength of the polyester lines will become governing with a safety factor just above the requirement.

The fatigue in the bottom chain is below the requirement, but this will be changed if the dimension of the bottom chain is increased due to the strength requirement.

The increased dynamic loads in mooring lines may also affect the out-of-plane bending of the top chain. This should be further evaluated.

4.8 Structural design for various components

4.8.1 Pontoons

4.8.1.1 General

DNV GL has reviewed the report SBJ-33-C5-OON-22-RE-018 rev. B including the Appendices A, B and C. In addition, the document SBJ-33-C5-OON-22-RE-025 rev. 0; K12 “Anslagsnotat” was briefly reviewed. In lack of setups and drawings forming basis for the weight calculations as well as uncertainties related to the design assessments, the information given in the “anslagsnotat” in way of pontoon weights are difficult to assess.

4.8.1.2 ULS

The pontoon structure including the stiffening system is not sufficiently documented and hence it is difficult to verify the selected plate and stiffener dimensions.

It is stated in the document SBJ-33-C5-OON-22-RE-018 that the ultimate limit states includes gravity loading, permanent static water pressure and dynamic loading caused by waves, wind, current, traffic loads and structural response from the entire bridge structure. The static and dynamic external pressure loading (wave loading) was outlined in the above report and the methodology is considered reasonable.

Nevertheless, the remaining basic loads cases were generally not outlined, nor how the basic loads were combined and implemented.

4.8.1.3 FLS

It is understood from the design document that analysis of fatigue performance is not fully finished. Without detailed drawings it is found difficult to give relevant comments. For the connection to the pontoons it is questioned if it would be easier to achieve good fatigue details using bulb sections as stiffeners both in the pontoon and the columns as it would be possible to install brackets in between these sections which may reduce the dynamic hot spot stresses.

4.8.1.4 ALS

The current pontoon/column design is not fulfilling the Accidental Limit State condition with respect to boat impacts. The boat impact studies are commented more in other parts of this report see 4.5.

4.8.1.5 Pontoon drawings

The drawings which are reviewed are listed in Appendix B. The drawings of the pontoons look reasonable. Nevertheless, the design is not substantiated with adequate documentation as pointed out above.

4.8.2 Mooring lines

4.8.2.1 General

The mooring system is a taut polyester system with 16 mooring lines, located in two groups with 8 lines each. Within each group there are 4 pontoons with two lines each, one line at each side of the bridge.

The mooring lines are terminated in chain stoppers at the deck level and go through a moonpool in the pontoon close to pontoon center and a fairlead at the keel level. Issues that need to be evaluated by this solution are:

- Corrosion in the splash zone.
- Out of Plane Bending (OPB) fatigue and wear at the fairlead
- Need for retensioning of lines in operation

Advantages with this mooring solution are:

- Mooring lines are protected from ship collision.

The mooring lines are composed of a bottom chain, a middle polyester line and a top chain. The desired stiffness of the mooring system is obtained by adjusting the length and diameter of the polyester lines, keeping the ratio of cross-section area to length close to constant, considering favorable anchor positions.

The diameter of the bottom chain is governed by strength, the polyester by required stiffness and the upper chain by fatigue caused by out-of-plane bending.

The global response analyses have been performed with the mooring lines implemented as linear springs. Line dynamics and damping from the lines is thus neglected. This simplification has been checked by independent analyses in SIMA and has been found acceptable.

It is claimed that it is favorable to connect the mooring lines to the pontoon close to the center of the pontoon to reduce the moment from the vertical components of the mooring. DNV GL questions this statement as the line angles are close to horizontal and a mooring connection point close to the center will increase the overturning moment on the bridge. This should be further evaluated through a parametric study.

The damping from the polyester mooring lines is much smaller than for a mooring system with more catenary shape and could be unfavorable with respect to limit the response to parametric resonance.

4.8.2.2 ULS

ULS checks have been performed by including pretension and dynamic loads (including temperature and tide), all multiplied with a load factor of 2.2 according to ISO 19901-7, Annex B2. The maximum utilizations are 0.9 for the bottom chain (line 5), 0.86 for polyester (line 10) and 0.76 for top chain (line 1).

These results are non-conservative since the quasi-static stiffness of polyester has been used in the calculations.

4.8.2.3 FLS

Tension-tension fatigue is calculated according to DNVGL-OS-E301 and out-of-plane bending is calculated according to BV Guidance Node NI 604. In addition, fatigue due to VIV of the mooring lines has been evaluated.

The fatigue life of the top chains, governed by out-of-plane bending, is above 25 years, while the fatigue life of bottom chain, governed by tension-tension fatigue is above 100 years. For both, fatigue factors have been included. Fatigue due to VIV is small.

These results are non-conservative since the quasi-static stiffness of polyester has been used in the calculations.

4.8.2.4 ALS

An ALS check has been done for the case with 2 missing lines and a required safety factor of 1.5. Further, the response in the mooring lines due to ship impact has been analyzed and will not be governing for mooring design.

4.8.3 Subsea anchors

4.8.3.1 General

DNV GL has not put much effort into reviewing the documentation of the subsea anchors in this phase as it is not considered critical for the feasibility. The selected mooring system for K12 consist of 16 mooring lines and anchors put into two groups along the bridge. A combination of suction (8) and gravity (8) has been proposed. Water depths for the suction anchors are 380m – 560m and for the gravity anchors 123m – 411m. Suction anchors are 9m in diameter and 11m – 12.5 high. The gravity anchors are square steel boxes (15m x 15m) split in 4 ballast chambers with depths around 5m.

The level of design documentation for the subsea anchors are somewhat limited hence for upcoming design phases it is required to go into more details both with respect to structural (ULS, ALS, FLS) as well as geotechnical design for the 16 anchors.

4.8.4 Cable stayed bridge

4.8.4.1 General

The cable stay bridge part is documented in report SBJ-33-C5-OON-22-RE-019 Rev. 0 dated 15.08.2019. The design is building of well proven technology and it is judged that the current design is developed to a stage that will make cost and schedule estimates to be within required tolerances. The difference to ordinary cable stayed bridges that are built in recent years is the increased dynamic loading imposed from the connection to the floating bridge, cables that are bundled and the somewhat unconventional design of the tower.

The girder is not laterally supported at the tower, leading to increase in the dynamic loading in the upper part of the tower. However, the effect is not expected to give requirement for increased dimensions in later phases.

4.8.4.2 ULS

The ULS design of the tower and the concrete viaduct girders are made according to well-known methods. Detailed review of the calculations is not made at this stage. The ULS checks for the steel girder is carried out as for the floating bridge and reference is made to comments given in 4.8.7.2.

4.8.4.3 FLS

The dynamic stresses from environmental loads in the cables are larger than for conventional cable stayed bridges. However, it is assessed that the cables have long fatigue lives and FLS is not governing for the design of the cables. Comments to the fatigue of the bridge girder is commented in Section 4.8.7.3.

4.8.4.4 ALS

The girder of the cable stayed bridge is checked for effects from ship impacts and is commented in 4.5. Other accidental load conditions e.g. car fire, sudden loss of cable etc. are not studied in this phase.

4.8.5 Bridge abutments

4.8.5.1 General

Both abutments are founded on prepared bedrock base. 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.

4.8.5.2 ULS

Reference is made to document SBJ-33-C5-OON-22-RE-020. In section 2.1 the general design principles are presented. The following section is quoted:

"To assure a predictable transfer of base shear and normal pressure, only the walls in the front corner parts and the rear part of the abutment are cast directly onto bed rock whereas the base slab is cast onto a sand/gravel layer. The sliding capacity is determined from base friction only. The contribution from post-tensioned rock anchors to the base friction capacity and to the overturning resistance is well within the limits prescribed by N400 11.6.2.2 [2]. The rock anchors are distributed in the front part of the abutment".

In section 2.4 on foot prints the following explanation is given:

"The configuration of the foot print is developed based on the criteria that no uplift is allowed in SLS load conditions in areas with concrete to bed rock contact. The analytical model has been set up with the restriction that the boundary condition is removed upon tension in the vertical direction (uplifting)".

It is further said that: "Some iterations have been necessary to find an arrangement that eliminates uplifting in SLS while keeping the geometry of the abutment. Figure 2-21 to Figure 2-24 shows the development sequence and illustrates the philosophy behind the chosen foot print configuration. For the final analysis the contact surfaces to rock is modeled as fixed in all directions".

Based on this explanation it is understood that prestressed rock anchors are used to avoid uplift in SLS. However, if the vertical reaction force at the front only meets an SLS criterion with respect to uplift, it is questioned if the fixed connections can be assumed for transfer of moment from the bridge around the outer concrete walls of the foundation as the contact pressure may be lost in ULS (or be small). Thus, it is questioned if the moment must be carried as a force couple along the longitudinal walls in addition to the contribution of the normal force from the bridge. This should be further checked in the next phase of the project.

The design has so far been performed and reported with respect to ULS. It should be checked that significantly larger forces will not occur during ALS such that the prestressed cables to the rock anchors will not be significantly damaged during a potential ALS situation.

In section 2.6.5 it is said that "The abutment has good capability for redistribution of forces". It is important that this applies also for transfer of shear load transverse to the rock anchors that are close to the outer edge of the abutment and closest to the action forces from the bridge.

4.8.5.3 FLS

Fatigue of assessment the connections between the bridge girder and the abutment has not been presented. It is assessed that this can be a more detailed design of these connections can be made at a later stage.

4.8.5.4 ALS

The design has so far been performed and reported with respect to ULS. It should be checked that significantly larger forces will not occur during ALS such that the prestressed cables to the rock anchors will not be significantly damaged during a potential ALS situation.

In section 2.6.5 it is said that "The abutment has good capability for redistribution of forces". It is important that this applies also for transfer of shear load transverse to the rock anchors that are close to the outer edge of the abutment and closest to the action forces from the bridge.

4.8.6 Bridge columns

4.8.6.1 General

The report SBJ-33-C5-OON-22-RE-018 rev. B including the appendices A, B and C were reviewed.

In addition, the document SBJ-33-C5-OON-22-RE-025 rev. 0; K12 "Anslagsnotat" was briefly reviewed. In lack of setups and drawings forming basis for the weight calculations as well as uncertainties related to the design assessments, the information given in the "anslagsnotat" in way column weights are difficult to verify.

4.8.6.2 ULS

The column structure including the stiffening system as well as the bolted connections are not sufficiently documented in the report, and it is therefore difficult to verify the selected dimensions.

It is stated in the document SBJ-33-C5-OON-22-RE-018 that the ultimate limit states includes gravity loading, permanent static water pressure and dynamic loading caused by waves, wind, current, traffic loads and structural response from the entire bridge structure. The static and dynamic external pressure loading (wave loading) was outlined in the report and the methodology is considered reasonable. Nevertheless, the remaining basic loads cases were generally not outlined, nor how the basic loads were combined and implemented.

4.8.6.3 FLS

It is understood from the design document that the structural details at the connection between the bridge girder and the columns are not fully finished. Without detailed drawings it is found difficult to give relevant comments. For the connection to the pontoons it is questioned if it would be easier to achieve good fatigue details using bulb sections as vertical stiffeners. By using bulbs both in the pontoon and the columns it would be possible to install brackets in between these parts which may reduce the dynamic hot spot stresses.

It is understood that bolted connections are planned to connect the columns to the girder in the main part of the floating bridge. It is not clear from the design documents if bolted connections are also planned to connect the high columns in axes 3-6 as the area of flanges here will be as large as 12 x 12 meters (Ref. drawing SBJ-33-C5-OON-22-DR-146 Rev. 0).

It is claimed that the bolted solution will reduce the assembling time, but no documentation was implemented supporting this statement. Consideration regarding installation challenges (machining, tolerances, preloading of bolts) and maintenance should be considered (preload levels/ loosening of bolts from eventual vibrations). The bolted connections must also be assessed especially with respect to fatigue including also affected parts of the bridge girder. The use of 8.8 bolts versus 10.9 bolts should also be assessed.

4.8.6.4 ALS

The current pontoon/column design is not fulfilling the Accidental Limit State condition with respect to boat impacts. The boat impact studies are commented more in other parts of this report.

4.8.6.5 Column drawings

The drawings which are reviewed are listed in Appendix B.

The column design is not substantiated with adequate documentation as pointed out above. Furthermore, we have noted the following lack of information.

- The external plating of the columns was analysed with a thickness of 25mm. The drawings give no information about thicknesses.
- Stiffener dimensions and stiffener spacings are indicated on the drawings, but documentation of buckling to applicable standards remains to be documented.
- Application of HP stiffeners are recommended to be considered as an alternative to the trapezoidal stiffeners in order to achieve smoother connections towards the pontoons.
- The transitions areas have not been detailed and the bolted connections are not indicated on the drawings.
- The thickness of the internal bulkheads in the columns were difficult to capture from the information provided in the report.

4.8.7 Bridge girder

4.8.7.1 General

The bridge girder is designed as a box section with generally using closed (trapezoidal) stiffeners. It should be considered in the design development to use open stiffeners for all plate panels with exception of the deck plate that is exposed to wheel loads. The reason is that it is easier to design details that are efficient to fabricate and give better fatigue capacity than closed stiffeners.

4.8.7.2 ULS

The box girder cross-section is made as stiffened plates made with trapezoidal stiffeners. The height of the cross-section is typical 3.5 m and the span between pontoons is 120 m. Longitudinal bulkheads are included at columns and extend up to 12 m into the span. In order to vary the capacity against the applied forces and moments the thicknesses of the stiffeners are varied.

Eurocode (NS-EN-1993-1-5:2006, Eurocode 3:Design of steel structures, Part 1-5: Plated structural elements) is used to account for shear lag effects and to check for buckling of the stiffened panels. The values used for assessing the effectiveness of the cross-section outside the area with additional longitudinal bulkheads is, in the view of DNV GL, too optimistic partly because β_1 and not β_2 are used and partly because shear lag effects from environmental loads are neglected. It is thus not agreed that the effective width calculations are conservative for these parts of the girder. Close to the column where additional longitudinal bulkheads are introduced it is agreed that the effect of shear lag is small and determination by the methods in Eurocode is not suitable.

It is stated that all the stiffeners are designed to be so stocky that local buckling can be ruled out as a possible failure mode. This seems not be fulfilled for the web of Stiffener 1 A and 2A where the c/t ratio exceeds the limit. These stiffeners should be checked as class 4 cross-sections or increase the plate thickness.

A study of the shear leg effects is also presented in Appendix A of the report SBJ-33-C5-OON-22-RE-017 where a simplified part of the low bridge is analysed for a case where the bridge girder at one column is deflected 1 m relatively to the neighbour columns. It is found shear lag effects at the column but no effects in the remaining part of the bridge. This result seems to be interpreted to be due to the imposed deflection used in the analysis but is rather due to the long distance between inflection points in the girder and the concentration of forces at the columns that is located at the centre of the girder.

The use of Eurocode for determination of shear lag has clear limitation for the actual bridge. The method is based upon single vertical web supported in its plane. For the concepts for Bjørnafjorden-crossing the girder is supported on columns in the middle of the cross-section with additional webs close to the support. It is therefore proposed that the documentation of ULS and particularly FLS for the future design developments are based on local shell models for the dominant load cases to account for effects not represented in the beam model.

It is stated in the report that if the shear utilization is less than 50% it can be neglected in the interaction with other stress components in the capacity check. It is referred to EN-1993-1-1 paragraph 6.2.8 (NS-EN-1993-1-1, 2005). This paragraph yields capacity of members and is not applicable for stiffened panels. It is recommended to include buckling effects from shear when checking highly utilized panels for buckling.

It is not clear from the document how shear stresses are included in the ULS checks.

Recommendations

As a recommendation for future design development it should be considered to investigate if open stiffeners could be used for the bottom and side plates where wheel loads are not present. It is judged that such stiffeners may ease the development of details that are easy to fabricate and have good fatigue performance. The benefit of closed stiffeners compared with open stiffeners is that for the same number of welds the unsupported plate between stiffeners is less. However, as most of the plate thicknesses are larger than 12 mm sufficient stiffening of the plate will be provided even with about the same center to center distance of open stiffeners as for closed stiffeners.

In the final design the capacity of stiffeners resisting both environmental and traffic loads should be checked.

4.8.7.3 FLS

Reference is made to Doc. No SBJ-33-C5-OON-22-RE-016 Rev. B Fatigue Assessment.
For fatigue assessment of the bridge girder it has been assumed that it is the butt welds in the trapezoidal stiffener sections that are most critical with respect to fatigue. These butt welds are made against backing bars that results in S-N class F with a SCF = 1.0. This corresponds to a SCF = 1.27 relative to the D-curve. The D-curve is typically used for butt welds made from both sides of the plate.

Here, thickness transition will likely result in stress concentration factor that are larger than 1.27. In the design documentation a thickness transition between plates with different thicknesses equal 1.0 mm has been assumed to achieve sufficiently low SCFs. This is a rather small thickness transition to be selected at an early design stage. The calculated fatigue life depends also on what stress concentration for tolerance is included in the S-N curve D and on what fabrication tolerance can be achieved. In the design basis it is said that a tolerance equal 0.1t is included in the design S-N curve. In the latest revision of DNVGL-RP-C203 (2019) this tolerance is reduced to 0.05t.

It should be noted that this SCF is related to a beam analysis model where plane strain sections are assumed. In the actual concept the strain distribution over the bridge section cannot be expected to be fully linear due to difference in longitudinal stiffeners and stiffness at connections to the columns. Based on this it is assessed that the fatigue design is very marginal with respect to selection of stress concentration factors.

Furthermore, it must be assumed that doubling plates will be welded to the bridge deck plating. Even with a size between 120 and 300 mm this results in the F1 curve with an inherent stress concentration factor equal 1.41. Thus, it may be practical to aim for a SCF for the girder not less than 1.41 when using the D-curve for fatigue life assessment.

It is noted that sufficient fatigue life has been calculated for the bridge girder for longitudinal stress ranges and stress concentration factors as large as 1.5 for most of the bridge when using the D-curve. However, it is expected that it is the local traffic loads that will be governing for the deck plate thickness. Based on experience with fatigue cracking from the weld root through the deck plate as shown in Figure 4-3 it is questioned if the minimum plate thickness should be increased to 16 mm at the heavy traffic lane. Reference is made to the following articles Bohai et al. (2013), (Bohai, Rong, Ce, Hirofumi, & Xiangfai). Reference is also made to (Cheng, Ye, Cao, Mbako, & Cao), (Guo, Liu, & Zhu), (Kainuma, et al.), (Wang, Zhai, Li. H., & Guo), (Yokozaki), (Zhu, et al.), (Maljaars, Bonet, & Pijpers) (Yokozaki) (Wang, Zhai, Li. H., & Guo) and (Zhu, et al.): (Publication No. FHWA-HRT-17-020 Optimization of Rib-to-deck Welds for Steel Orthotropic Bridge Decks. US Department of Transportation. February 2017.) and (Manual for design, construction, and maintenance of orthotropic steel deck bridges. Publication No. FHWA-IF-12-027 , February 2012). The reason for this question is also fatigue assessment due to local traffic load that leads to large hot spot stresses in the transverse direction of the bridge with potential fatigue cracking along the welds between the trapezoidal sections and the bridge plate (Figure 4-3b). So far, the fatigue life at these hot spots have not been reported.

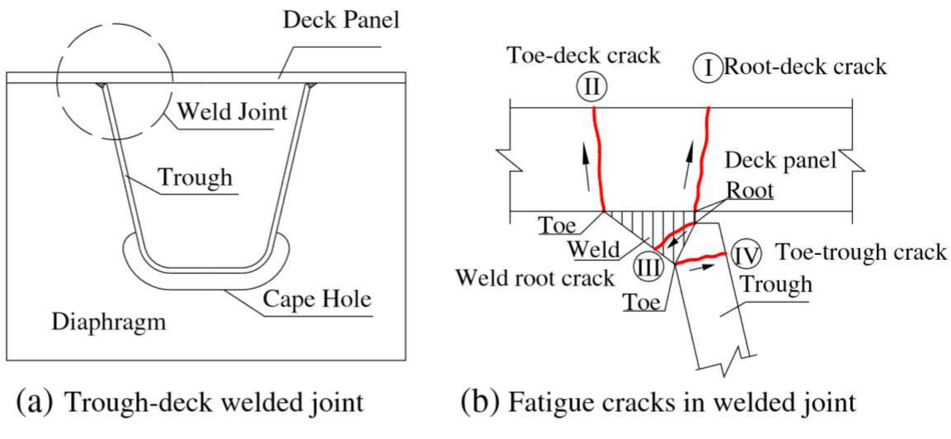

Figure 4-3 Fatigue cracks of trough-deck welded joint

Reference is made to drawing no SBJ-33-C5-OON-DR144 rev 0 where a transition between a trapezoidal section and a T-section is shown. This is a detail that should be further assessed in a next phase to check if the calculated hot spot stress meets the target value.

With the assumption of very low stress concentration factors for the butt welds it is more likely that the transfer of the longitudinal trapezoidal sections through the transverse frames will be governing for the fatigue design of the bridge girder. It is difficult to avoid some local bending stresses in the walls of the trapezoidal sections at the cut-outs in the transverse frames because of the Poisson's ratio. For this reason, it may be preferred to use open sections as longitudinal stiffeners away from the traffic loaded plates.

Assessment of concept with respect to development of next design phase

Characteristic long-term loads and safety factors

From the reported fatigue analyses it is observed that the calculated fatigue lives are sensitive to the values of the traffic loads. It should be noted that the recommended value of safety factors is related to how the long-term loads are defined. In the design basis for the bridge a Design Fatigue Factor (DFF) is being used as a safety factor on number of cycles during the design life. The use of DFF has a long tradition in design of offshore structures. The target safety level is achieved through use of this DFF together with a characteristic S-N curves (also denoted as design S-N curves) which are derived as mean minus two standard deviations from a normal distribution of the test data in a logarithmic format. The long-term stress range distribution used in the fatigue analyses is derived as expected values of the response due to environmental actions. This means that in fatigue analysis of offshore structures the mean value of the response can also be defined as the characteristic value to be used for fatigue analysis.

For land structures such as Eurocode the definition of long-term stress ranges has been different. It is understood that the long-term stress ranges should be determined to be upper bond values or more precisely determined as mean plus two standard deviations. Reference is made to EN 1993-1-9. By this definition rather low additional safety factors on the stress range are required to achieve the target safety level. Similar guidance has also been used in design of British land structures. For example, under fatigue loading in BS 7608:2014, (BS 7608:2014 Guide to fatigue design and assessment of steel products. BSI standard publication.), it is said that "The design load spectrum should be selected on the basis that is an upper bound estimate of the accumulated service conditions, including both loading and number of cycles, over the full design life of the product. The adoption of mean plus two standard deviations data for applied load levels or an upper bound estimate based on knowledge of the actual or predicted loading environment and applied number of cycles, when used with the design S-N data, usually results in an acceptably low probability of failure during the design life, commensurate with safe-life design principles".

When proposing equations for combination of stress ranges and calculating fatigue damages in the design basis for the Bjørnafjorden floating bridge it was assumed that the stress ranges from different sources were defined on the same basis as expected values when values for DFFs were recommended. However, it is likely that the traffic load model is based on another definition of characteristic load than that of the environmental response. Furthermore, it is understood that more relevant information from long-term traffic data are being achieved from measurements being performed. As these data become available it is proposed that the requirements to analysis procedure with definition of characteristic long-term loads/responses and DFFs are revisited before a more detailed design of the bridge is performed.

From measurement of stresses in bridge girders it is observed that the response distributions or spectra are broad as might be expected from the number of different vehicles passing, ref. eg. Guo et al (2015).

Thus, to get representative long-term design spectra for fatigue assessment it is recommended to perform local measurements of the stress response in addition to counting vehicles and axles for some months.

<u>Design of connections between longitudinal trapezoidal sections and transverse frames</u>

There is a cut-out around the corners of the trapezoidal sections below the bridge top plate. The purpose of these cut-outs is to reduce the stress concentration factors at the corners of the trapezoidal sections with radius 40mm when the sections are subjected to an axial force. The resulting axial force in the sections is due to the axial force in the bridge girder in addition to the forces resulting from the vertical and horizontal bending moments. Without this cut-out there will be a significant stress concentration at the small radius corners of the trapezoidal sections due to the Poisson ratio. The stress concentration factor is a function of the height of the cut-out and restraint from the transverse frames. Therefore, it is recommended that a study on optimal cut-out is performed in a next phase of the project to assure that this detail will not be more critical with respect to fatigue than the transverse butt welds in the bridge girder.

There are significant dynamic axial forces in most of the trapezoidal sections in the bridge girder and similar cut-outs around all trapezoidal sections that are welded to the transverse frames will be required However, it is probably easier to arrive at an acceptable geometry for cut-outs at sections that are not subjected to transverse loads than for the cut-outs below the deck plate.

The welded connections between the longitudinal stiffeners and the transverse frames need also to be further assessed also for dynamic stresses in the transverse frames. Experience from reported fatigue cracking in suspension bridges shows that this needs to be further assessed in the next design phase.

4.8.7.4 ALS

Reference is made to section 4.5 for ship impacts.

4.9 Fabrication and installation

4.9.1 General comments/observations

4.9.1.1 Description of construction including marine operations

The construction and marine operations are described in the document SBJ-33-C5-OON-22-RE-023, K12 – Execution of construction. Descriptions of the various operations are given, and some details are included. Various types of marine operations will be performed during this project, but most operations could be considered as standard operations. Hence, for these DNV GL does not see the missing detailing as a feasibility issue, but it imposes uncertainties in schedule and cost for these operations.

The most challenging operation DNV GL considers to be the towing and installation of the complete floating bridge. Some more details, e.g. strength calculations of a temporary securing system at the abutments, are included for this operation. Anyhow, for the connection of the complete bridge the documentation is still largely incomplete, see 4.9.2.8. Also, for the towing, including holding, further documentation is required, see 4.9.2.7.

Some operations may be critical to be planned as weather restricted operations, see 4.9.1.2. Further detailing of methods is hence required to fully document compliance with the requirements in DNVGL-ST-N001 (DNVGL-ST-N001, Marine operations and marine warranty, 2018) to weather restricted operations.

4.9.1.2 Weather restricted operations

Many (most) marine operations are not feasible to execute safely independent of the weather conditions. Hence, such operations need to be carried out from one safe to another safe condition within a reliable weather forecast period. "Safe condition" is defined as a condition where the object is considered exposed to normal risk (i.e. similar risk as expected during in-place condition) for damage or loss in any possible environmental conditions. In DNVGL-ST-N001 such operations are defined as weather restricted operations. The key elements to safe execution of such operations are:

- As short operations as practically possible and always within the maximum time period limits given in DNVGL-ST-N001.
- Well documented operational schedules with ample contingency time.
- Robust (e.g. adequate back-up) equipment, structures and procedures allowing for incidents without severely impacting the operational schedule.
- Appropriate weather forecasting (and monitoring) services, but anyhow duly consider the inherent uncertainty in the weather forecasts, see 4.9.1.3.

4.9.1.3 Weather forecast uncertainty

The uncertainty in the weather forecasts could be accounted for by use of alpha factors according to DNVGL-ST-N001. It should be noted that these alpha factors are derived based on forecasted wave heights in the North Sea. Local conditions could influence the uncertainty in forecasted wave heights including swell, and wind. Reliable current predictions may be important, and there is no general approach to account for the uncertainty in current predictions. Hence, the forecast uncertainty could be more accurately accounted for if weather forecasts/current predictions for the bridge location were systematically compared with measured data.

4.9.1.4 Risk management

An active risk management is required for all marine operations. For challenging operations, the risk management is considered vital for a successful execution. As a part of the risk management plan, risk assessment of the marine operations as well as the various construction phases is needed. The risk management needs to start at an early phase of the detailed engineering phase. As a part of the planning, risk assessment to define relevant loading conditions and accidental load cases should be performed. Typical accidental cases could be ship collision, unintended water filling, mooring/pull-in line failure. Robust/well proven equipment (with back-up as relevant), vessel and procedures should be used to minimize the risk of unacceptable (operational) delays. Generally, the received documentation neither includes detailed risk assessments of marine operations nor accidental load cases.

4.9.1.5 Received calculations

The received calculations, e.g. SBJ-33-C5-OON-22-RE-023 – Appendices A and B, have not been verified in detail. DNV GL has evaluated the reported results and done some spot checks. Where found appropriate we have indicated conclusions from our evaluations/checks in the text.

4.9.1.6 Reinforcements and temporary steel

Local reinforcements will be required to transfer concentrated loads in temporary phases, for example at lift/support points, towing/pull-in fittings and to lock structures while welding. Any technical implications from the temporary steel should be considered, e.g. if local reinforcements may affect the fatigue life of the bridge, etc. The additional steel, and whether it must be removed after the operation, should be accounted for in the planning of weights and cost.

4.9.2 Evaluation of the various (marine) operations

4.9.2.1 Construction alternatives

Two alternative construction workflows are indicated, see sketch below.

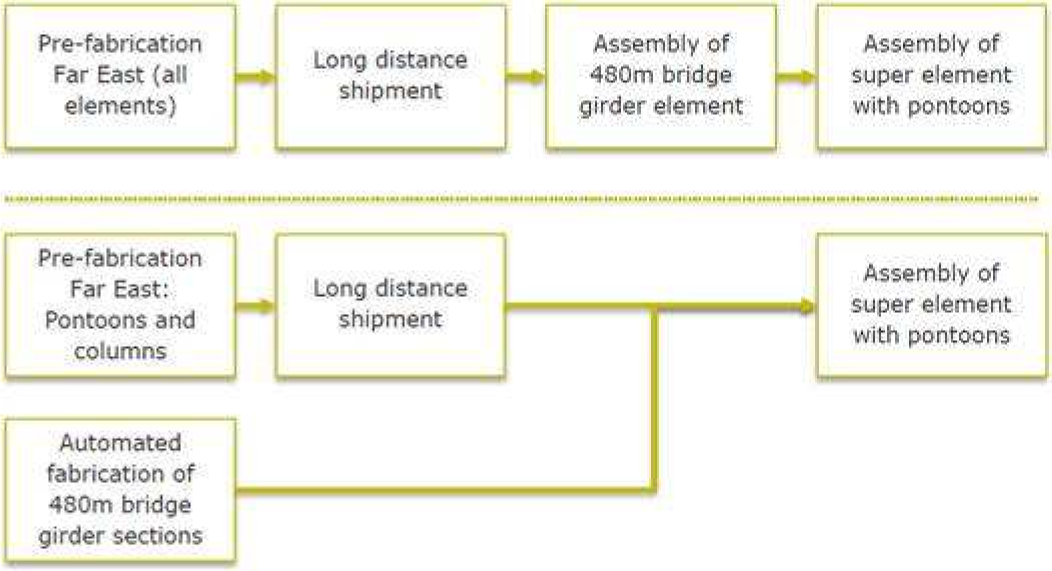

Figure 4-4 Diagram taken from SBJ-33-C5-OON-22-RE-023

DNVGL find both alternatives viable. Cost and risk evaluations regarding quality and schedule will be governing. The evaluations regarding fabrication capabilities in the far east we find reasonable.

The pontoons are assumed delivered from the fabricator with the columns installed. It is mentioned that high bridge may be completed in Norway due to stability limitations. It might be questioned if this will introduce additional requirements to that mentioned in SBJ-33-C5-OON-22-RE-023, sec.3.10.3 for the construction of the high part of the floating bridge.

In order to ensure a "perfect" fit between girder segments it is proposed to cut off one piece by e.g. diamond cutting and start the next segment with that piece. See 4.9.3.2 for our evaluations.

Sea transport of pontoons with columns and eventually girder segments for the far east is proposed done by HLV. We find the indicated transport method/layout and precautions reasonable, but we also note that lot of detailing remains. According to OON the next phase of the project (i.e. 'Forprosjekt') will include a thorough transport engineering and seafastening design for the heavy lift transports.

4.9.2.2 Bridge elements assembly

It is proposed to connect the bridge elements into "super-elements" that will be up to 480m long. Transport of these "super elements" will have strict wave height limitations. Hence, open sea towage should be avoided and generally the tow route should be as short as possible. Possible construction locations are pin-pointed, but DNV GL anyhow finds that there may be significant uncertainties related to yard availability and (possible high) preparation cost should be closely looked at.

The simultaneous lifting by 4 cranes will require a tight control of the lift operation, but we do not see this as a very complicated lift operation. DNV GL assumes cost of/time for installing and removing required lift points and temporary stability outriggers on 4 pontoons have been considered.

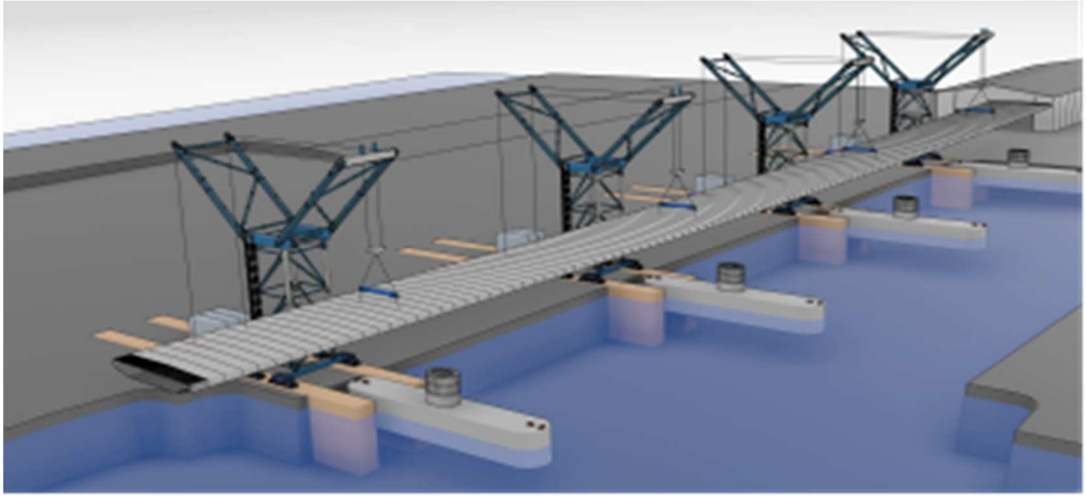

Figure 4-5 Lifting operation taken from SBJ-33-C5-OON-22-RE-023 Figure 3-15

For the ramp/high bridge element there will be additional challenges related to higher and different lift heights at the pontoons. We have not found any specific documentation other than some drawings regarding these lifts.

4.9.2.3 Cable-stayed bridge

The final erection is done by crane vessels and skidding. Both could be considered as standard marine operations and we do not foresee any critical technical challenges. However, these are weather restricted operations and especially the installation of the 100m outer girder elements with a shear leg crane needs proper planning to keep the required weather window within acceptable limits. It should be noted that a pre-laid anchor system probably is required for the shear leg crane due to the water depth.

4.9.2.4 Transportation to inshore assembly site

The towed object height and draft are rather limited considering the object size and we do not see these tows as critical assuming:

- No open sea towage required
- Towing in a weather window as indicated (BF-5)
- A sailing route with reasonable clearances (e.g. not significantly smaller than indicated passing Vatlestraumen) is obtained/applied.

4.9.2.5 Inshore assembly

The indicated method using a tailormade semi-sub for connecting the super elements is considered feasible. General descriptions of method and required equipment are included, and SBJ-33-C5-OON-22-RE-023 - Appendix B contains load calculations for extreme weather conditions. However, DNV GL is not able to see that the indicated pull-in equipment is designed for these loads. Hence, limiting operational weather conditions (acceptable weather windows) for these operations should be established.

The mooring layout seems reasonable, but no calculations other than estimated environmental forces are received. DNV GL does not foresee any feasibility issues with establishing an acceptable mooring system during the bridge assembly. However, we have noted several aspects that could significantly influence the detailing and hence cost:

- The text indicates 26 mooring lines, while the layout in shown in SBJ-33-C5-OON-22-RE-023 has 20 lines.

- Equipment (winches) for checking and maintaining adequate mooring line loads (distribution) are not mentioned.
- Possible design limits/precautions due to requirements for vessels being able to pass the lines safely.
- Requirements to accidental cases (one line broken).
- Available hook-up time to be able to connect (pull-in) new bridge segments within a weather window.
- Some of the anchors are indicated placed with downhill slope in the pull direction. In such cases "special" anchors could be required to obtain adequate holding capacity.
- Loads from/on the semi-sub are not mentioned/included.

4.9.2.6 Installation of north floating bridge segments

This installation is proposed done in two operations; lift installation by crane vessel of a 10m long section and floating-in a 290m long section. The former operation we consider as a standard operation, while the latter includes several elements that need further detailing:

- Operation schedule and required weather windows.
- Control systems/procedures (bridge girder deflection/forces, jack/pull-in forces, ballasting/tide)
- Capacity and functionality of the pull-in system/vessels.
- Jacking system on barge.
- Ballasting of pontoons if required.

4.9.2.7 Complete bridge towing, including holding

Towing of the main part of the bridge from the assembly site in Søreidsvika to the destination is planned as a weather restricted operation. However, it may be required to hold the bridge in a waiting position prior to start of the installation, awaiting acceptable weather conditions for the installation. This holding operation is planned as a weather unrestricted operation, and the specified required force/tow fleet based on a seasonal storm seems acceptable.

During the towing and holding there is a limited sea room, and control of the position of the bridge, as well as deformation control to avoid overloading of bridge section, is vital.

The received documentation includes a rough check of bridge girder stresses due to tug pulling forces. It may be concluded from the check that to control tug pulling forces to avoid structural damage to the bridge girder should be well within the capability of the proposed towing fleet. Anyhow, proper control of the tug direction and pulling forces are required for proper maneuvering of the bridge within its structural limitations. We assume the operational aspects will be evaluated by experienced personnel (including tow masters and tug captains), that will advise regarding detailing of procedures and equipment. Note that in the present documentation both 6 and 8 AHT are indicated.

Regarding the structural strength of the bridge we have also noted the following text in SBJ-33-C5-OON-22-RE-023 – Appendix B – Quote: *Analyses shows that the holding/survival operation of the entire bridge pre-installation might be feasible. However, further investigations and analyses should be* *perform*(ed) *in order to verify the feasibility.* – Unquote. We are awaiting these investigations and analyses as input to our (final) conclusion regarding feasibility.

The documentation shows relocation of the tugs in case of holding in bad weather. We find such relocation fully feasible, but again; detailing is required.

4.9.2.8 Installation of the main bridge section

The North end approach is controlled by tugs (of which two are connected to land bollards) and winch lines. It is stated that the semi-sub used during bridge assembly will be used, but no details are shown. Finally, the two bridge parts are held together by prestressing cables.

The South end connection approach and pull-in is done in a similar way as the South North side but without the semi-sub. Vertical alignment is done by ballasting temporary tanks both on the floating bridge and on the CSB. The final connection will be done as in the North end with prestressing cables.

The described operation seems feasible, but considerably more detailing is required to establish realistic weather limitations and operational schedules.

We have noted that some support calculations for an alternative installation procedure, i.e. installing the bridge in smaller pieces, are included. However, as no procedural descriptions are found we have not considered/evaluated this alternative.

4.9.2.9 Safe condition main bridge section

Calculations for safe condition of the bridge after both ends have been connected are included both in SBJ-33-C5-OON-22-RE-023 – Appendix A and in Appendix B, Sec.5. None of these analyses are considered particularly relevant for the bridge pull in operation itself. Hence, DNV GL assumes calculations documenting/supporting the pull-in equipment and procedure will be made later.

The two analyses have not been reviewed in detail, but we have the following observations/comments:

- It seems relevant to discuss/consider additional forces during pull in (and for securing after pull-in) of the bridge due to:
    - Construction tolerances
    - Temperature variations
    - Tidal variations (mentioned in appendix A, but unclear if included in the applied basis 100y condition).
- The force scale factor (0.56) to find 10 years summer design loads based on the 100 years results seems adequate. However, if the considered 100 years analysis is done including the mooring system the effect of this shall also be quantified.

4.9.2.10 Mooring system

The mooring system pre-installation and hook-up are considered as standard marine operations. Hence, we do foresee any feasibility issues. We have noted that welding in the bridge end connection can only be done in "nice weather" due to possible deflections caused by environmental loads. Hence, it should be evaluated if also mooring hook-up loads can cause undesired defections.

The mooring chain tensioners are indicated in SBJ-33-C5-OON-22-RE-023 Figures 3-52. A few comments are given:

- DNV GL assumes the tensioners will be designed to account for all relevant loads, e.g. any horizontal pulling force from the AHTS while paying-in the slack of the chain during tensioning.

- The stability of the tensioner, buckling of members etc. and the connection to the pontoon, accounting for any eccentric loads from the chain during tensioning.
- The tensioners shall be moved to the next pontoon after the chain tensioning (Sec. 3.17.2). We assume some (skidding) arrangement is provided (because the tensioners are located below the bridge girder and cannot be lifted directly).

4.9.3 Fabrication details

4.9.3.1 Bolted Connections

The connections between the columns and the bridge girder are proposed to be bolted. This requires machining of the contact surfaces after the welding of the parts to be connected are finished. The width of area to be machined is large and the machining must be performed on top of fabricated columns and on the underside of the bridge girder. This will require special considerations with respect to tools to be used for machining.

The bolts must be put in from below of the flanges in the columns. Thus, access platforms for this will be needed both on outside and the inside of the columns. As the bolts cannot be inserted from the bridge girder there needs to be treads in the 100 mm thick plate that is welded into the bridge girder. It might be questioned if the holes and threads in the thick plate can be made before the flanges are put together or it is planned that these will be made after the fabrication of the two parts to be connected are fully finished.

In the design documentation the time period needed for installation of all the bolts at one tower connection is presented. Experience from bolting of large diameter flanges in wind turbine support structures show that loosening of bolts is a problem if only pretensioned once. A procedure for pre-tensioning of the bolts has so far not been presented. Based on this it is questioned if the time needed to connect a tower to the bridge girder is sufficient.

It is noted that distance sleeves on the bolts are proposed to keep the prestressing over time. It might be questioned if this is sufficient to avoid need for pre-tensioning during service life. One need a system to assure that pretension will not be lost due to rotation of the bolts due to dynamic loading on the bridge.

4.9.3.2 Bridge girder element fabrication tolerance control

Reference is made to Doc no SBJ-33-C5-OON-22-RE-023 Rev. 0 K12 – Execution of construction.

It is agreed that control of fabrication tolerances is important to achieve a long fatigue life of the bridge girder. In section 3.3.1 of the report K12- Execution of construction it is mentioned that by using a diamond cut through the full section is a method to get good alignment between the different sections. It might be added that this can be effective for the main plates. However, to achieve welding from both sides of the butt welds, the longitudinal stiffeners cannot be present. These can be installed afterwards in the normal way as typically filled in afterwards by welding against backing bars. (Reference is also made to section 1.5.3 in Anlagsnotat Doc no SBJ-33-C5-OON-22-RE-025 Rev 0). The use of jigs as described in section 3.3.2 in Doc no SBJ-33-C5-OON-22-RE-023 Rev. 0 may be a good alternative to achieve accurate alignments. Also, here short stiffener elements may be inserted after the main butt welds have been made from both sides as specified in section 3.10.2.

4.10 Material selection and corrosion protection

A rather detailed document on corrosion protection is presented in document SBJ-33-CS-OON-40-RE-001. This document gives a good overview over existing practise and no further comments are made at this project stage.

5 COMMENTS TO CONCEPT RANKING

5.1 General

The designer presents the results from the ranking process during the design development in report SBJ-32-C5-OON-22-RE-002 rev C. dated 24.05.2019. Comments from the review is given in the following sections.

5.2 Concept K11

5.2.1 Structural design

The conceptual design for K11 has been developed in earlier phases and is further detailed in Phase 5. The difference to K12 is that K11 is without anchor lines. The structural design seems to be developed to the same level as for K12. The lack of anchor lines leads to larger environmental forces in the bridge girder leading to increased dimensions in the bridge girder especially towards the abutments. For the K11 bridge concept it will represent an engineering challenge to overcome all issues related to building a so slender structure exposed to dynamic environmental loads. All these issues cannot build on proven technology and the K11 concept will need more research to be realized. See also Section 4.4.

5.2.2 Need for maintenance

The overall maintenance for the bridge is judged to be less than the other concepts as there is no anchor lines. The need for maintenance of the bridge girder is regarded to be higher as the dynamic loading causing fatigue is larger.

5.2.3 Robustness

Compared with the concepts that include anchor lines the robustness of the K11 concept is regarded to be less as there is only one load carrying system and severe damages to the bridge girder may mean loss of the entire bridge. The bridge girder for K11 will be stronger than the other concepts and will locally have increased capacity against ship collisions, but in general the anchor lines are judged to add robustness compared to a bridge designed without anchor lines.

5.2.4 Movements

No comparisons between the various concepts on movements that may influence the comfort to personnel is made. The conceptual design of K11 meets the requirements in the design specification and is consequently judged to give adequate conditions for the traffic on the bridge.

5.2.5 Fabrication and installation

The K11 concept is described in Doc. No SBJ-30-C5-OON-22-RE-001, *Alternative K11 - Consolidated technical report*, Rev. A dated 29.03.2019. It does not have mooring lines. Otherwise, the K11 concept is installed similarly to concept K12.

5.3 Concept K13

5.3.1 Structural design

The K13 bridge concept is a straight bridge with anchor lines. The concept is a development of the previous concept K7 which was verified by DNV GL last year. The structural design of the girder is further developed and is judged to be able to meet all requirements. The thermal expansions will induce large stresses in the bridge or a dilation joint need to be introduced at the north end of the bridge.

The dilation joint will require a complex structure to allow for the large deformations induced in such a long bridge and at the same time provide full speed driving conditions for the traffic.

5.3.2 Need for maintenance

The need for maintenance will be larger for the case with a dilation joint in the North end compared with the other concepts.

5.3.3 Robustness

The robustness of K13 is regarded to be similar to the other concepts with anchor lines (K12 and K14).

5.3.4 Movements

No comparisons between the various concepts on movements that may influence the comfort to personnel is made. The conceptual design of K13 meets the requirements in the design specification and is consequently judged to give adequate conditions for the traffic on the bridge.

5.3.5 Fabrication and installation

The K13 concept is described in Doc. No SBJ-31-C5-OON-22-RE-001, *Alternative K13 - Consolidated technical report*, Rev. A dated 29.03.2019.

The main difference from K11 is that the main floating bridge is towed and installed in several sections.

5.4 Concept K14

5.4.1 Structural design

The structural arrangement for concept K14 is a S-shaped bridge with 3 clusters of anchor lines. The structural behaviour is quite similar to K13 with the exception that thermal expansion is taken by deformation of the bridge as for K11 and K12. The concept gives somewhat added flexibility towards road lining compared with K12. But as K12 fits well with the approaching roads it is reasonable that the girder dimensions and anchor line arrangement is not as optimized for K14 as for the others.

5.4.2 Need for maintenance

The need for maintenance for this bridge concept is larger than for K12 due to fewer anchor lines, but less than for K13 as the dilation joint is omitted.

5.4.3 Robustness

The robustness of K14 is regarded to be similar to the other concepts with anchor lines (K12 and K13).

5.4.4 Movements

No comparisons between the various concepts on movements that may influence the comfort to personnel is made. The conceptual design of K14 meets the requirements in the design specification and is consequently judged to give adequate conditions for the traffic on the bridge.

5.4.5 Fabrication and installation

The K14 concept is described in Doc. No SBJ-31-C5-OON-22-RE-001, *Alternative K14 - Consolidated technical report*, Rev. A dated 29.03.2019.

The main difference from K11 is that the main the floating bridge is towed and installed in several sections.

5.5 Comments to designer's selection of preferred concept alternative

The designer's arguments for the selection of the preferred concept is presented in their report SBJ-32-C5-OON-22-RE-002 Rev. C dated 24.05.2019. DNV GL concur with K12 as the preferred alternative but would rank K11 behind K13 and K14 due to the uncertainties in dynamic behavior of such a slender bridge and the risk of losing the entire bridge as there is no redundant load carrying system. The robustness is hence judged to be less than for the other concepts.

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APPENDIX A
Verification comments issued during Conceptual Phase 5

Project name:

Bjørnafjorden floating bridge concept development phase 5

Project no:	**Subject**	
12777	Verification comments to OON status 2	
Meeting no.:	**Day of meeting:**	**Location:**
	2019-05-09	Lysaker
Document no.:	**Distribution date:**	**Minutes taker:**
SBJ-32-C5-OON-63-MM-001	2019-07-05	

Participants:

Names

Next meeting:

Date, time, location

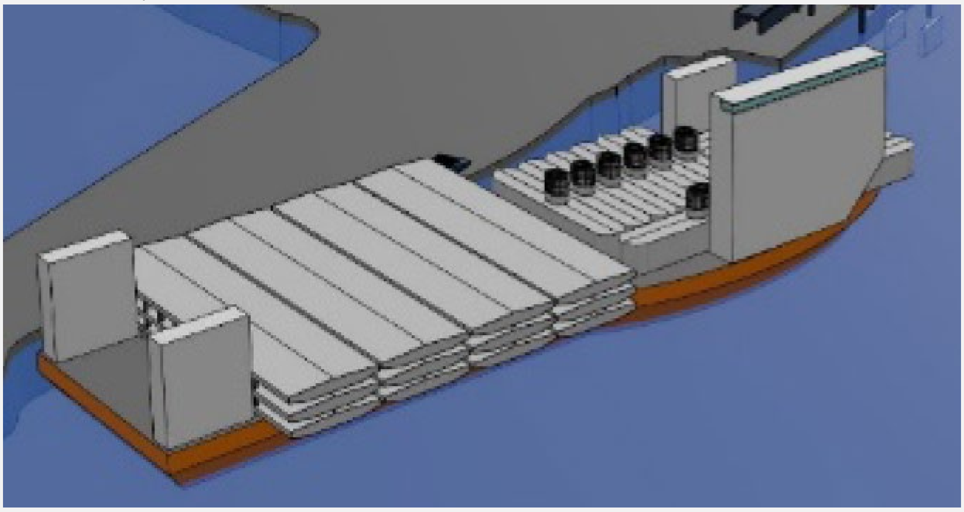
Item	Topic	Status
Slide 1.		
Slide 2.		
Slide 3.		
Slide 4. Comments to concept ranking	DNV GL concur with the technical evaluations made as basis for the concept ranking ref: 10205546-02-RAP-172 Appendix Q: Concept ranking, however in the final selection the following points should also be considered: • K12 (and K13 and K14) has less dynamic loading than K11 hence reduced fatigue load (possibility to reduce dimensions or less inspection and risk for repair during service) that should also be considered in the ranking between the concepts. Included in robustness evaluation in the concept ranking. • The risk loosing the entire bridge by a high energy ship impact is larger for K11 than the other having mooring lines. Included in robustness evaluation in the concept ranking. • The factors established from static push-over analyses as reported in SBJ-32-C5-OON-22-RE-001 CONCEPTS DEFINITION AND DESCRIPTION are not judged to be useful for ranking of the concepts as the results totally depends on the assumptions made and that the capacity of the structure is most likely limited by cyclic loads. Agreed. This was a part of preliminary results.	Open
Slide 5. Comment to analyses. Hydrodynamic interaction	• Summary of Hydrodynamic interaction: • "Hydrodynamic interaction between pontoons is significant for weak axis bridge girder moments, while the impact is limited for strong axis- and torsional moments on most load cases." - Have analyses been performed with a damping lid? This could possibly reduce interaction effects.	Open Jørgen

Slide 6. Comments to the structural details	• So far mainly details in the bridge girders have been assessed with respect to fatigue as more drawings are needed for assessment of other details like connection between girder and pontoon columns and pontoons. We are working on these details, and this will be included in the drawings for June 30th.	Open
Slide 7. Shear lag	• Ref. section 4.5.3 of the main report. It is noted that the effect of the shear lag will be included in the final report. However, all concepts as presented will expect reductions in stiffness and increase in calculated stresses when shear lag is accounted for. It is not clear to us that there are sufficient margins in the present design to allow for this without changes that may lead to cost increase. Effect of shear-lag has been investigated in FEM-analyses, and will be reported in final report. 360 meters of the girder has been modelled with shell elements. The model includes additional longitudinal bulkheads at column supports. Two typical load conditions have been analyzed, an equally distributed load representing permanent loading, and a forced displacement at one pontoon representing a typical response from environmental loads. We clearly see shear-lag effects in the analyses. However, the strong reinforcements at column supports compensates the reduction from shear-lag effects. It is showed that stresses calculated from global analyses, based on general cross sections without shear lag effects and without reinforcements at columns supports, gives conservative results.	Open
Slide 8. Sideway support at the column for the cable stayed bridge	• The sideway supports at the column will have to be designed for different forces for the different concepts. A description of the design of these should be given as this is now the only mechanical parts remaining in the concepts. The sideway support is meant to stabilize in construction of the stay-cable bridge and the assembly of the floating bridge to the stay-cable bridge, not in the final setup of the bridge. Design and description of these details is going to be delivered in the report 30. June. Short description for now: The support can only take transversal forces from the beam. This is done trough free moving bearings mounted on the side of the girder and to the side of the column. The design of the bearing set-up going at this moment.	Open
Slide 9. Fatigue analyses. Stiffener details	• Reference is made to drawings SBJ-33-C5-OON-22-DR-001 rev B and SBJ-33-C5-OON-22-DR-002 rev A. It is noted that there is no cut-out around the outer trapezoidal sections. The main reason for having cut-outs at the rounded corners of the trapezoidal sections is to relieve stress concentrations due to axial forces in the sections. Thus, it is questioned if also cut-outs at the trapezoidal sections at the bottom plates in the bridge girder may be needed. In this respect reference is also made to Table 4-3 of document no SBJ-33-C5-OON-22-RE-001 Rev. Alternative K12 – consolidated technical report. Here SCFs are listed; however, it is not clear if these are related to the different connections based on analyses or not. There will mainly be cut-outs around all trapezoidal stiffeners but it could be evaluated in the next phase of the project if it is required other places than under local traffic loading.	Open
Slide 10. Fatigue cont'd	• Reference is made to section 4.5.2 of document no SBJ-33-C5-OON-22-RE-001 Rev. Alternative K12 – consolidated technical report. It is said that a misalignment $\delta_0 = 0.1t$ has been used for the butt welds. This is according to the existing DNVGL-RP-C203. However, for fatigue assessment the following should be noted: DNVGL-RP-C203 is being revised and that the δ0 value for butt welds in plates will be reduced from 0.1t to 0.05t to be in line with test data and other fatigue design standards. It has been indicated that 5% on stress will also be used for butt welds in the revised EN 1993-1-9; however, here it is a process before a revised standard is finished. The	Open

	revision of DNVGL-RP-C203 means that at a plate transition from a thickness t = 12 mm to a thickness t = 14 mm with a fabrication tolerance of 2 mm, the stress concentration factor is increased from 1.40 for $\delta0 = 0.1t$ to 1.53 for $\delta0 = 0.05t$. It is noted that the DNVGL-RP-C203 is being revised, hence increased SCF's will have to be accounted for. This will not be included in 'todays' calculations as we have to stick to the given Design Basis. It should be evaluated in the next phase of the project.	
Slide 11. Abutment design	• Reference is made to section 4.5.2 of document no SBJ-33-C5-OON-22-RE-001 Rev. Alternative K12 – consolidated technical report. Reference is made to section 2.8.1 second paragraph. It is indicated that with post-tensioned cables one will get a connection that can undergo large non-linear bridge end deformation without yielding. However, by a proper post tensioning of the cables it is expected that a linear behavior can be assumed without any gapping in the connection. This design philosophy refers to non-grouted post-tensioning tendons spanning over the entire length of the abutment. In earlier design phases, especially for K11 that did not have any lateral stabilization (e.g. anchoring) except in the ends, there were certain uncertainties related to instability phenomena affecting the global lateral behavior such as 'dynamic buckling' and parametric excitation. Hence, a connection that allows for large non-linear bridge end deformation was part of a robustness design. With the present design concept, these uncertainties are reduced, and this extra robustness is not deemed necessary. Consequently, the tendons are now much shorter and will be assumed grouted. Hence, this is no longer relevant.	Open
Slide 12. Fatigue cont'd	• So far, a fatigue design due to local wheel loads have not been reported. For this purpose, the thickness of the asphalt layer should be given and assumption of how the wheel loads are transferred to the steel structure. The local traffic, asphalt thickness and assumptions of wheel loads is given in the report; SBJ-33-C5-OON-22-RE-016-K12 - Appendix B - Local traffic fatigue methodology.	Open
Slide 13. Fatigue cont'd	• Reference is made to document no SBJ-32-C5-OON-22-RE-009 rev. A, fabrication. Ref. Weld 5 page 28. It is questioned if the shown connections at the lower part of the bridge girder without cut-outs in the transverse frames by the longitudinal trapezoidal sections shown has sufficient fatigue capacity. The same question is asked with respect the two double bottom plates shown in page 31. Both of these solutions were preliminary and not evaluated in detail. Weld 5, page 28 has cut-outs in transverse frames but It is not connected to the bottom plate. The fatigue loading in the bottom part of the bridge girder is mainly longitudinal. The vertical loading from traffic will be distributed over a larger area and the stress level will be small, so there is no need for much welding between transverse frames and the bottom part of the bridge girder. For the bottom part of the girder, the solution is evaluated to be a good building method, but for this project a traditional solution is chosen to avoid several new solutions. On page 31, the longitudinal stiffener with double bottom plates will reduce the weight and make the bending process easier. It will be more challenging to make a proper weld, which has the same fatigue properties and that is why a traditional stiffener profile is chosen.	Open
Slide 14. Bolted connections to ease fabrication	• Reference is made to document no SBJ-32-C5-OON-22-RE-009 rev. A fabrication. Ref. section 7.8.2. The type of bolt should be specified. Part of a	Open

	sentence is missing in first paragraph of section 7.8.1. This is updated in the next report, rev B, released 24.05.2019.	
Slide 15. Boat impact	General comments from OONO: The ship impact results presented in the status 2 reports focuses on what separates the bridge concepts. This is mainly global behaviour, which is why girder impacts are the main result presented. In the final report there will be focus on the local details as well, and better correlations between local (explicit) impact analysis and global dynamic implicit analysis. All the scenarios presented in the design basis will be discussed and documented. • Impact from submarine ref section 6.4.1.1.4 in the Design Basis has not been considered or evaluated. • *The energy to be dissipated from impact with submarine is low, maximum 23 MJ (submerged, 7 MJ surfaced). The submarine structure is assumed to be like the bulb of the ship bows. Based on impact with the ship bows, the indentation is expected to be maximum 2 m. This is if the energy is assumed to be dissipated only locally and only in the pontoon, which is conservative. The indentation is lower than impact with the ship bows and further evaluation is not conducted.* • It is stated that material modelling is based on low fractile values. This is not in accordance with DNV-RP-C208 and should be explained. This can also be seen in conjunction with the conclusion that an impact of 8 meter ship displacement into the bridge girder only reduces the capacity of the strong axis by 5.6%. This is believed to be a too optimistic value. *Updated to also include mean values in report in May. The residual capacity of strong axis at 8 m ship displacement is 7.7 % in status report 2. Will be updated to the report in May with the following strong axis residual capacities (impact location changed, material properties changed):* *Intact bridge girder* – ***100 %*** *4 m ship displacement* – ***92.3 %*** *8 m ship displacement* – ***88.2 %*** *12 m ship displacement* – ***87.4 %*** *16 m ship displacement* – ***84.7 %*** • Several possible scenarios have not been investigated yet, and an evaluation of these are expected. These are, but not limited to: Impact from both east and west, impact on pontoon including eccentric impact, on pontoon with and without anchor line etc. *All the impulse impacts (girder impact only) in the report of march are from both east and west -> the global response of the bridge is well documented for impacts in both directions.*	Open

	These scenarios will be included in the final report, as well will other scenarios that have been investigated. • It is estimated that an impact on the pontoon will give an indentation of 4-5 meters into the pontoon. An evaluation of the damaged condition due to flooding of the pontoons will have to be given. *Design basis states that the bridge will have to survive a 100-years storm post impact, so this will be included in the final report.*	
Slide 16. Corrosion protection during construction	• The review has in this phase not focus on corrosion protection, but we would like to rise the issue of corrosion protection during the long construction period before the permanent arrangement for the inside protection come into service? Construction in yard; The construction of the bridge parts done in yard, will be using shop primed steel. External side; Shop primer will be removed by blasting and system no. 2 (TSZ/Epoxy/Polyurethane) applied. Internal side; Zinc rich shop primer will be used. Weld areas will require appropriate surface preparation and application of a zinc silicate primer, in order to avoid excessive rusting. Automated construction; Automated construction of the bridge parts, carbon steel without shop primer will be used. External side; Surface preparation by blast-cleaning and application of system no. 2 (TSZ / Epoxy / Polyurethane) Internal side; Surface preparation by blast-cleaning and application of a zinc rich primer. The shopprimer / zinc primer will be able to provide satisfactory protection during the installation period even with box girders open. Typical performance life of the zinc rich shopprimer, min. 12 months. During operation, dehumidification units will be installed Inside the box girders / columns, keeping the relative humidity < 40% at all time.	Open
Slide 17. Geological stability checks	• Tower in south and stability of slope in rock mass: Please allow for geological stability checks of the steep rock outcrop/ rock cliff for the placement of the tower in south at Svarvhelleholmen. As DNV GL understand it, a geological survey shall be conducted, and stability analyses will be performed. It is recommended to perform this survey and the stability assessment as soon as possible to ensure the feasibility of the placement of the tower. Reference is given to NS-EN 1997-1 section 11.5.2. Please state design philosophy on this issue as moving into detailed engineering. This issue is commented in the Engineering geological assessment report, section 2.4 (same text for all 4 concept reports; K11, K12, K13 & K14): *"Another important element related to foundations for a bridge tower is evaluation of the potential for land slide, either debris from higher ground striking the structure, or resulting in loss of stability of bearing capacity for the foundation. At the location of the K13 bridge tower there are no higher grounds in the vicinity of the planned structure. Further, the foundation is located well inside a broad subsea shelf, with water depths less than 30 m, and about 130 m horizontal distance to the nearest steeper sub-marine slope (se Figure 4). Hence the issue of slope stability is not considered relevant for this location.* *For more detailed information on the geological conditions, degree of jointing*	Open

	and rock mechanical properties, it is recommended to perform ground investigations in this area, preferably core drilling to depths of 40 – 60 m below the planned foundation." Our evaluation suggest that the topographical layout of the tower foundations for all 4 concepts suggest that the issue of sub-sea sliding is not relevant – hence detailed slope stability analyses is not considered necessary. If required, this issue may be further discussed in the next phase of the project, along with more detailed structure geology mapping and investigation program for the local area of the chosen site for the tower foundation.	
Slide 18. Transport of 100 m bridge elements from Pre-fabrication site to Assembly Site 1	The transport is performed on heavy transport vessels from the far East to the Assembly site The following comments are given: ▪ The bridge girders are stored three elements in height. We assume that the appurtenances are installed prior to this transport, and the height of the cribbing elements must account for this. ▪ The sea fastening design must consider the effect of transport vessel accelerations and the stability of the cribbing elements, etc. ▪ The space between elements must allow for installation of the sea fastening. ▪ For cargo extending over the ship side, slamming may be a design case depending on wave height, vessel motions, outreach etc., and must be checked.  Transport of bridge elements (cut from Figure 8-2 in *Consolidated Technical report*, K12) In this stage, only preliminary cribbing and seafastening design has been performed to confirm transport feasibility and assess specific cost implications. In the next phase of the project (i.e. 'Forprosjekt'), a thorough transport engineering and seafastening design for the heavy lift transport will have to be undertaken.	Open
Slide 19. Transport of 100 m bridge elements from Pre-fabrication site to Assembly Site 1, continued	Load-in from the transport ship is proposed by Self-Propelled Modular Transporters (SPMTs) (*Consolidated Technical report, K12*, Sec. 8.2.5). It must be checked that the assembly site is fitted for such transports with regard to strength of mooring bollards, quay at the link span bridge, general ground capacity etc.	Open

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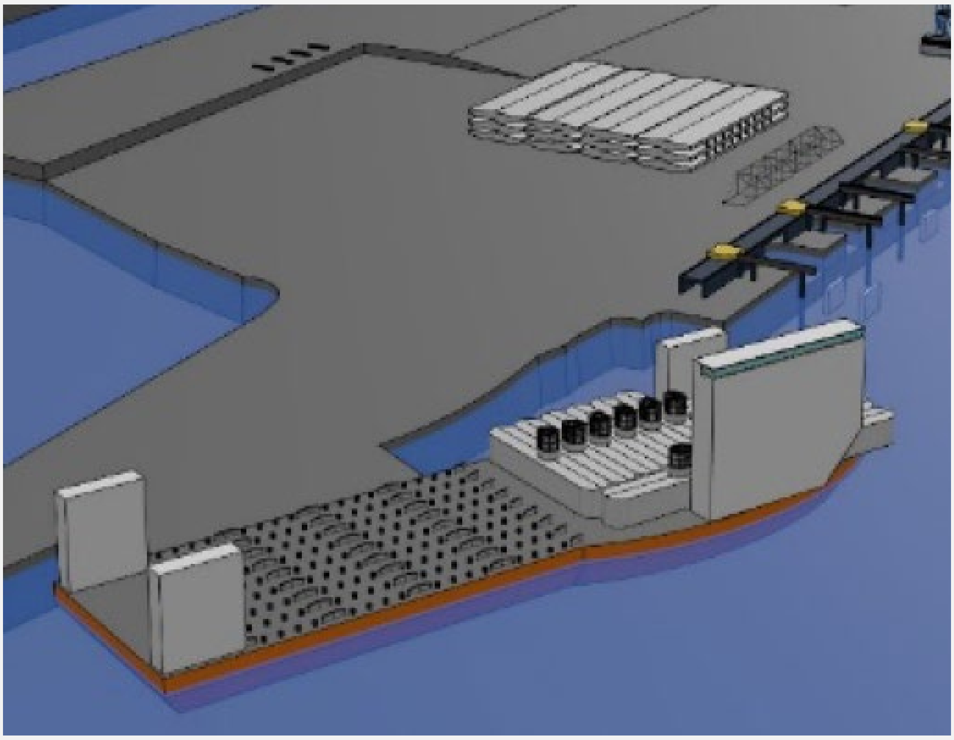
<td>
Load-in of bridge elements (cut from Figure 8-2 in Consolidated Technical report, K12)
Correct. The use of SPMTs involves special requirements for the load bearing capacity of quay and storage areas. Although Hanøytangen has been used as a demonstration case, a specific Assembly site has not been selected in this phase. The axel loads and ground bearing pressures have been included in the requirements specification for the Assembly site. Allowances for potential site upgrading have been reflected in the cost estimate.</td>
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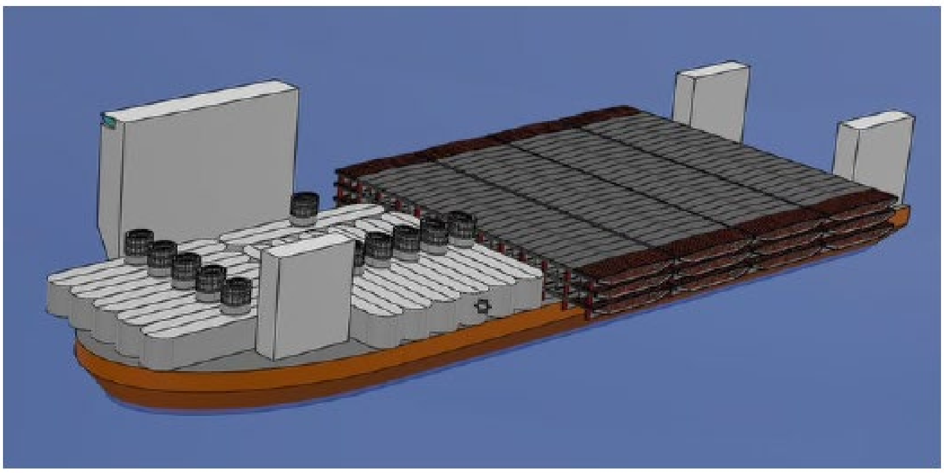
<td>Slide 20.
Transport of pontoons from Pre-fabrication site to Assembly Site 1</td>
<td>A large number of pontoons and bridge sections are transported on the same HTV transport. The vessel utilization seems somewhat optimistic e.g.:
Space is required for a guiding system for positioning of the pontoons, to get them into correct position
The space between elements must allow for installation of the sea fastening.
Practical/stability aspects related to individual ballasting of the pontoons must be evaluated.

Transport of bridge elements and pontoons
(Figure 15 in Steel Fabrication report)
The cargo arrangement has been altered and improved, refer rev. B of report SBJ-33-C5-OON-22-RE-023 Execution of construction.</td>
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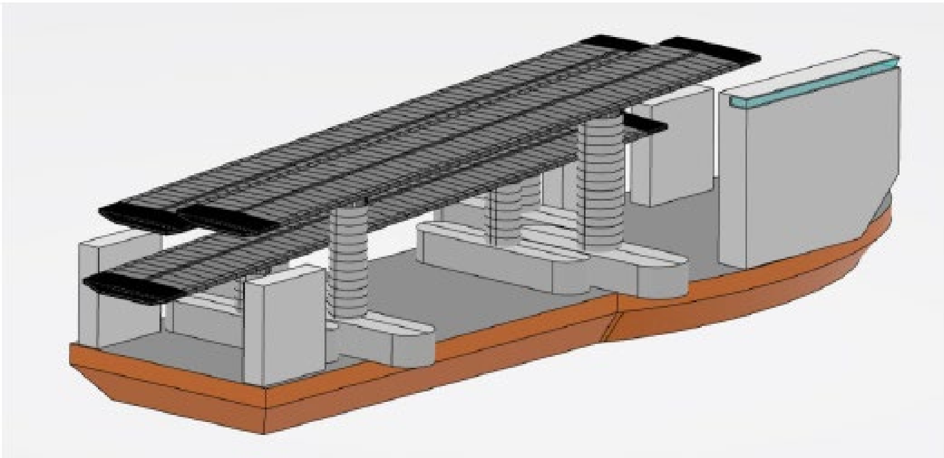
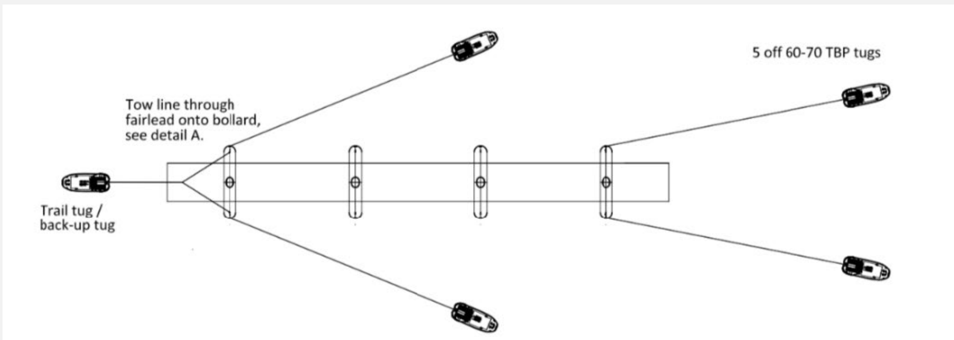
<tr><td>Slide 21.
Transport of 200 m bridge elements Pre-fabrication site to Assembly Site 1</td><td>One option is to fabricate 200 m long bridge elements on pontoons in the Far East, and transport three structures on a heavy transport vessel. This solution would require calculations/evaluations related to:
▪ Fatigue damage during transport, in particular the transition between column and bridge girder
▪ Positioning aids (guide structures) to avoid collisions during float-on and float off.

> Figure 19: High bridge transport
From Steel fabrication report
This transport option is waived, among other things because of such considerations.</td><td>Open</td></tr>
<tr><td>Slide 22.
Accidental scenarios</td><td>• Stability of the elements and super elements in an accidental condition (flooding) must be checked during tow and fabrication/storage.


From Consolidated report, K12, Fig. 8-9
Adequate stability and buoyancy for two-compartment damage will be documented or all transit conditions.</td><td>Open</td></tr>
<tr><td>Slide 23.
Connection of bridge girder</td><td>We have noted that main bridge girder connections is still under development. Because these connections are rather critical, we recommend that further development is done as soon as possible, e.g.
▪ Connecting the main bridge to the cable stayed bridge is described on an overall level. No information regarding design forces during the welding period is included.
▪ In the north end, a 260 long element is included to reduce the effect of tide. However, more calculations/ detailing are outstanding.
▪ A securing arrangement in top of the bridge girder. We assume that securing in both top and bottom of the bridge cross section is needed to keep the relative position fixed during welding.</td><td>Open</td></tr>
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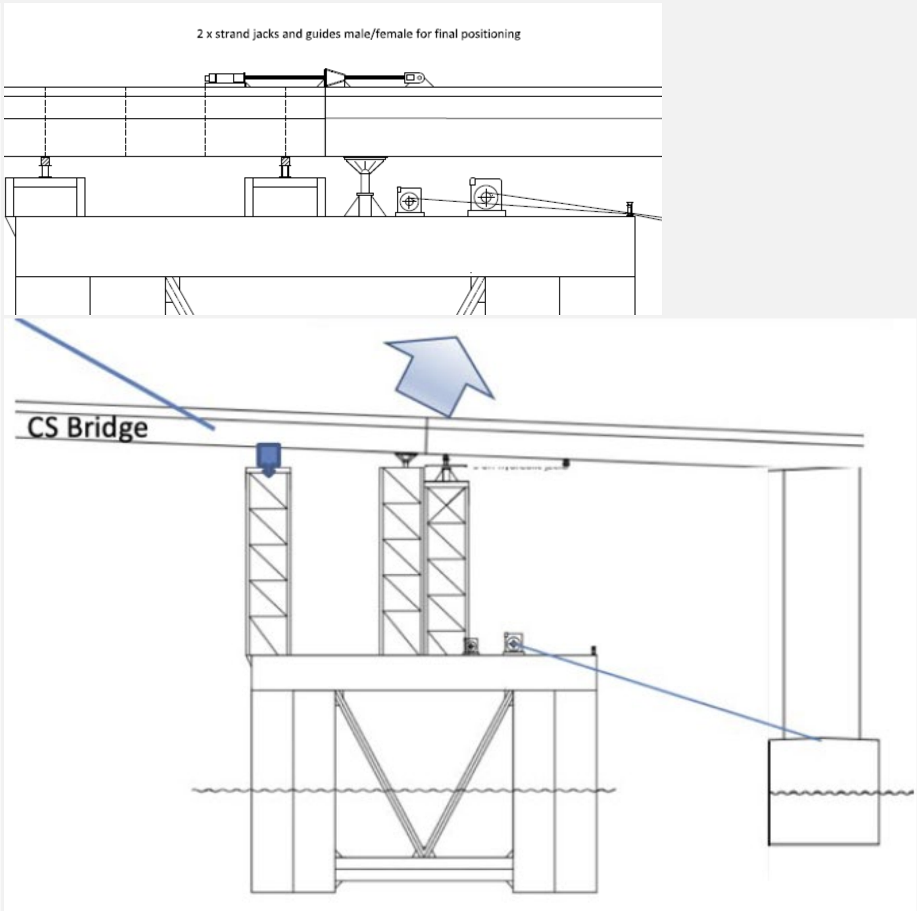

From Consolidated report, K12, Fig. 8-16
(and drawing BB-1107-0314 Rev. C)
The temporary connection for the bridge closure joints consists of a set of post-tensioned steel tube clamps distributed along the periphery of the bridge deck cross-section. The connection has been subject to thorough development and assessment and is reported technical note SBJ-3X-C5-OON-22-TN-002-A included as Appendix in rev. B of report SBJ-33-C5-OON-22-RE-023 Execution of construction.
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<td>Slide 24.
Tensioning of the mooring chain on the pontoons</td>
<td>The mooring chain tensioners are indicated in Figures 2-7 and 2-8 in Alternative K12 - Consolidated technical report. A few comments are given:

We assume the tensioners will be designed to account for all relevant loads, e.g. any horizontal pulling force from the AHTS while paying-in the slack of the chain during tensioning.
The stability of the tensioner, buckling of members etc. and the connection to the pontoon, accounting for any eccentric loads from the chain during tensioning.
The tensioners shall be moved to the next pontoon after the chain tensioning (Sec. 8.2.12). We assume some kind of skidding arrangement is provided (because the tensioners are located below the bridge girder and cannot be lifted directly).

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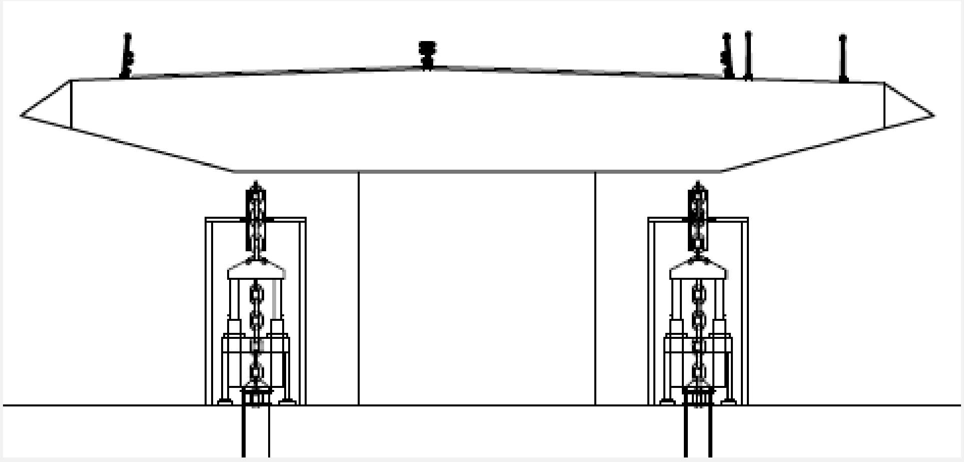

From drawing BB-1107-0462 Rev. A

Slide 25. Installation costs

The installation costs are indicated to be very different for the K11-K14, as shown in the table. Even if
there is an explanation given to Figure 3, all the differences are not understood.

Activity	K11	K12	K13	K14	Unit X
Sea bed prep. and anchor installation	0	511	918	770	MNOK
Towing to Bjørnafjorden (complete bridge)	137	138	0	0	MNOK
Towing and positioning in Bjørnafjorden (sections)	0	0	96	76	MNOK
Mooring system hook-up (during installation)	0	0	302	239	MNOK
Bridge installation Bjørnafjorden (complete bridge)	43	39	0	0	MNOK
Mooring system hook-up (after installation)	0	176	0	0	MNOK
Towing to Bjørnafjorden (infills)	0	0	236	236	MNOK
Bridge complete installation and assembly (infills)	0	0	190	187	MNOK
Finalization	970	970	923	932	MNOK
Total cost	13 521	13 424	13 691	13 605	MNOK

Figure 3: Main cost elements and activities for all concepts, compared

From *Concept Selection and Risk Management,*
Doc. No. SBJ-32-C5-OON-22-RE-002, Appendix A12

Closed – Discussed with SVV and DNV in follow-up meeting. No need to detail beside "anslags data"

APPENDIX B
List of drawings and reports

Document & Drawing No	Rev.	Title	Date document issued
SBJ-32-C5-OON-22-RE-003	0	Analysis method	15.08.2019
SBJ-32-C5-OON-22-RE-003 App.A	0	Appendix A - Hydrostatic and hydrodynamic coefficients	15.08.2019
SBJ-32-C5-OON-22-RE-003 App.B	0	Appendix B - Air loads and drift forces in 3DFloat	15.08.2019
SBJ-32-C5-OON-22-RE-003 App.C	0	Appendix C - Comfort requirements method	15.08.2019
SBJ-32-C5-OON-22-RE-004	0	Verification and validation of global analysis, instability phenomena and interaction effects	15.08.2019
SBJ-32-C5-OON-22-RE-004 App.A	0	Appendix A - Flutter instability	15.08.2019
SBJ-32-C5-OON-22-RE-004 App.B	0	Appendix B - Benchmark of global analyses	15.08.2019
SBJ-32-C5-OON-22-RE-004 App.C	0	Appendix C - Validation workshop	15.08.2019
SBJ-32-C5-OON-22-RE-005	0	Sensitivity studies	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.A	0	Appendix A - Influence of swell waves	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.B	0	Appendix B - K11 Evaluation of 10000 RP conditions	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.C	0	Appendix C - Evaluation of wind spectrum uncertainty	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.D	0	Appendix D - Horizontal Curvature K14 Mode Shape 2	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.E	0	Appendix E - Horizontal curvature K14 mode shape 3	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.F	0	Appendix F - Anchor stiffness study K12 and K14	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.G	0	Appendix G - Number of anchor groups K12 and K14	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.H	0	Appendix H - Tower girder connection study	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.I	0	Appendix I - Uniform anchor distribution K13	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.J	0	Appendix J - Simplified anchor loss K12, K13 and K14	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.K	0	Appendix K - Eigenmode anchor stiffness sensitivity	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.L	0	Appendix L - Pushover analysis	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.M	0	Appendix M - Skew wind from traffic	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.N	0	Appendix N - Evaluation of critical wind directions	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.P	0	Appendix P - Discretization of mooring lines	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.Q	0	Appendix Q - Sensitivity of wavespread	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.R	0	Appendix R - K12 Evaluation of 10000 RP conditions	15.08.2019
SBJ-32-C5-OON-22-RE-005 App.S	0	Appendix S - Evaluation of directional grid	15.08.2019
SBJ-32-C5-OON-22-RE-006	0	Parametric Resonance	15.08.2019
SBJ-32-C5-OON-22-RE-006 App.A	0	Appendix A - Investigations in the time	15.08.2019

		domain model 3DFloat	
SBJ-32-C5-OON-22-RE-006 App.B	0	Appendix B - Effect of conservation of energy in a 2 degree of freedom system	15.08.2019
SBJ-32-C5-OON-22-RE-006 App.C	0	Appendix C - Eigenfrequency sensitivity study	15.08.2019
SBJ-32-C5-OON-22-RE-006 App.D	0	Appendix D - Verification of stability criterion calculation procedure	15.08.2019
SBJ-32-C5-OON-22-RE-006 App.E	0	Appendix E - Time domain axial force estimation for swell loads	15.08.2019
SBJ-32-C5-OON-22-RE-006 App.F	0	Appendix F - Probabilistic approach	15.08.2019
SBJ-32-C5-OON-22-RE-006 App.G	0	Appendix G - Stochastic loading of SDOF system with quadratic damping	15.08.2019
SBJ-32-C5-OON-22-RE-007	0	Wind loads and aerodynamic optimisation	15.08.2019
SBJ-32-C5-OON-22-RE-008	0	Hydrodynamic optimization	15.08.2019
SBJ-32-C5-OON-22-RE-008 App.A	0	Appendix A - Higher and lower order panel representation	15.08.2019
SBJ-32-C5-OON-22-RE-008 App.B	0	Appendix B - Pontoon geometry variations - Local analyses	15.08.2019
SBJ-32-C5-OON-22-RE-008 App.C	0	Appendix C - Pontoon geometry variations - Global analyses	15.08.2019
SBJ-32-C5-OON-22-RE-008 App.D	0	Appendix C - Preliminary assessment of global interaction effects and structural damping	15.08.2019
SBJ-32-C5-OON-22-RE-008 App.E	0	Appendix E - Hydrodynamic pontoon interaction - Local analyses	15.08.2019
SBJ-32-C5-OON-22-RE-008 App.F	0	Appendix F - Hydrodynamic pontoon interaction - Global analyses in time domain	15.08.2019
SBJ-32-C5-OON-22-RE-008 App.G	0	Appendix G - Wind wave analysis along floating bridge towing route	15.08.2019
SBJ-32-C5-OON-22-RE-008 App.H	0	Appendix H - Second order loads and moments	15.08.2019
SBJ-32-C5-OON-22-RE-008 App.I	0	Appendix I - Wave-current interaction	15.08.2019
SBJ-32-C5-OON-22-RE-008 App.J	0	Appendix J -Pontoon properties K12 Model 30	15.08.2019
SBJ-32-C5-OON-22-RE-009	0	Automatic steel fabrication	15.08.2019
SBJ-32-C5-OON-40-RE-001	0	Corrosion protection	15.08.2019
SBJ-32-C5-OON-40-RE-002	0	Operation and maintenance considerations	15.08.2019
SBJ-33-C5-OON-22-DR-201	0	Concept development floating bridge-Reksteren – Os - Fabrication Bridge Girder Part 1	15.08.2019
SBJ-33-C5-OON-22-DR-202	0	Fabrication Bridge Girder Part 2	15.08.2019
SBJ-33-C5-OON-22-DR-203	0	Fabrication Bridge Girder Part 3	15.08.2019
SBJ-33-C5-OON-22-DR-204	0	Fabrication Bridge Girder Part 4	15.08.2019
SBJ-33-C5-OON-22-DR-205	0	Concept development floating bridge-Reksteren – Os - Fabrication Bridge Girder Part 5	15.08.2019
SBJ-33-C5-OON-22-RE-001	0	Alternative K12 - Consolidated technical report	15.08.2019
SBJ-33-C5-OON-22-RE-011	0	K12 - Architectural design	15.08.2019
SBJ-33-C5-OON-22-RE-012	0	K12 Structural response analyses	15.08.2019
SBJ-33-C5-OON-22-RE-012 App.A	0	Appendix A - K12 Geometry input	15.08.2019
SBJ-33-C5-OON-22-RE-012 App.B	0	Appendix B - K12 Modal analysis	15.08.2019

SBJ-33-C5-OON-22-RE-012 App.C	0	Appendix C - K12 Coupled analyses	15.08.2019
SBJ-33-C5-OON-22-RE-012 App.D	0	Appendix D - Global stability evaluations	15.08.2019
SBJ-33-C5-OON-22-RE-012 App.E	0	Appendix E - Characteristic and limit state response	15.08.2019
SBJ-33-C5-OON-22-RE-013	0	K12 - Ship impact, Global assessment	15.08.2019
SBJ-33-C5-OON-22-RE-013 App.A	0	Appendix A - K12 - Abaqus terminology	15.08.2019
SBJ-33-C5-OON-22-RE-013 App.B	0	Appendix B - K12 - Complementary model data	15.08.2019
SBJ-33-C5-OON-22-RE-013 App.C	0	Appendix C - K12 - Complementary results	15.08.2019
SBJ-33-C5-OON-22-RE-013 App.D	0	Appendix D - K12 - Post impact evaluations	15.08.2019
SBJ-33-C5-OON-22-RE-013 App.F	0	Appendix F - K12 - Girder-column connection	15.08.2019
SBJ-33-C5-OON-22-RE-014	0	K12 - Ship impact, pontoons and columns	15.08.2019
SBJ-33-C5-OON-22-RE-014 App.A	0	Appendix A - K12 - Abaqus terminology	15.08.2019
SBJ-33-C5-OON-22-RE-014 App.B	0	Appendix B - K12 - Mesh and material sensitivity study	15.08.2019
SBJ-33-C5-OON-22-RE-014 App.C	0	Appendix C - K12 - Sensitivity of ship impact response	15.08.2019
SBJ-33-C5-OON-22-RE-015	0	K12 - Ship impact, Bridge girder	15.08.2019
SBJ-33-C5-OON-22-RE-015 App.A	0	Appendix A - K12 - Abaqus terminology	15.08.2019
SBJ-33-C5-OON-22-RE-015 App.B	0	Appendix B - K12 - Mesh and material sensitivity study	15.08.2019
SBJ-33-C5-OON-22-RE-015 App.C	0	Appendix C - K12 - Sensitivity of ship impact response	15.08.2019
SBJ-33-C5-OON-22-RE-016	0	Fatigue assessment	15.08.2019
SBJ-33-C5-OON-22-RE-016 App.A	0	Appendix A - Environmental fatigue loads Lumping of scatter diagrams and realization sensitivity	15.08.2019
SBJ-33-C5-OON-22-RE-016 App.B	0	Appendix B - Local traffic fatigue methodology	15.08.2019
SBJ-33-C5-OON-22-RE-016 App.C	0	Appendix C - Detailed results, section data and pairplots	15.08.2019
SBJ-33-C5-OON-22-RE-017	0	K12 Design of bridge girder	15.08.2019
SBJ-33-C5-OON-22-RE-017 App.A	0	Appendix A - Shear lag study	15.08.2019
SBJ-33-C5-OON-22-RE-017 App.B	0	Appendix B - Forces for stress calculation	15.08.2019
SBJ-33-C5-OON-22-RE-017 App.C	0	Appendix C - Optimalization study of bottom plate stiffeners	15.08.2019
SBJ-33-C5-OON-22-RE-018	0	Design of pontoons and columns	15.08.2019
SBJ-33-C5-OON-22-RE-018 App.A	0	Appendix A - Column ULS response	15.08.2019
SBJ-33-C5-OON-22-RE-018 App.B	0	Appendix B - Abaqus results ULS	15.08.2019
SBJ-33-C5-OON-22-RE-018 App.C	0	Appendix C - Bolted column connection	15.08.2019
SBJ-33-C5-OON-22-RE-019	0	Design of cable stayed bridge	15.08.2019
SBJ-33-C5-OON-22-RE-019 App.A	0	Appendix A - Technical note - ballasting	15.08.2019
SBJ-33-C5-OON-22-RE-019 App.B	0	Appendix B - Stay cables	15.08.2019
SBJ-33-C5-OON-22-RE-019 App.C	0	Appendix C - Design of temporary girder support	15.08.2019
SBJ-33-C5-OON-22-RE-019 App.D	0	Appendix D - Analysis results	15.08.2019
SBJ-33-C5-OON-22-RE-019 App.E	0	Appendix E - Fatigue design	15.08.2019

SBJ-33-C5-OON-22-RE-019 App.F	0	Appendix F - Shelldesign	15.08.2019
SBJ-33-C5-OON-22-RE-019 App.G	0	Appendix G - Foundation design	15.08.2019
SBJ-33-C5-OON-22-RE-020	0	K12 - Design of abutments	15.08.2019
SBJ-33-C5-OON-22-RE-021	0	K12 - Design of mooring and anchoring	15.08.2019
SBJ-33-C5-OON-22-RE-021 App.A	0	Appendix A - Low cycle fatigue	15.08.2019
SBJ-33-C5-OON-22-RE-021 App.B	0	Appendix B - Interlink stiffness modell	15.08.2019
SBJ-33-C5-OON-22-RE-021 App.C	0	Appendix C - High cycle fatigue	15.08.2019
SBJ-33-C5-OON-22-RE-021 App.D	0	Appendix D - Alternative mooring concept	15.08.2019
SBJ-33-C5-OON-22-RE-021 App.E	0	Appendix D - Pontoon Interface	15.08.2019
SBJ-33-C5-OON-22-RE-021 App.F	0	Appendix F - Gravity anchor design	15.08.2019
SBJ-33-C5-OON-22-RE-022	0	K12 - Marine geotecnical design	15.08.2019
SBJ-33-C5-OON-22-RE-022 App.A	0	Appendix A - Design brief	15.08.2019
SBJ-33-C5-OON-22-RE-022 App.B	0	Appendix B - Seismic evaluation	15.08.2019
SBJ-33-C5-OON-22-RE-022 App.C	0	Appendix C - Static slope stability calculations	15.08.2019
SBJ-33-C5-OON-22-RE-022 App.D	0	Appendix D - Pseudo Static slope stability calculations	15.08.2019
SBJ-33-C5-OON-22-RE-022 App.E	0	Appendix E - PLAXIS Slope calculations	15.08.2019
SBJ-33-C5-OON-22-RE-022 App.F	0	Appendix F - Plaxis - Suction anchor, failure modes	15.08.2019
SBJ-33-C5-OON-22-RE-022 App.G	0	Appendix G - Map of anchor groups	15.08.2019
SBJ-33-C5-OON-22-RE-100	0	K12 - Summary report	15.08.2019
SBJ-33-C5-OON-24-DR-001	0	Marine Operations – Tug Moored at Pontoon	15.08.2019
SBJ-33-C5-OON-24-DR-002	0	Marine Operations – Towing Configuration 480 m section	15.08.2019
SBJ-33-C5-OON-24-DR-003	0	Marine Operations – Tow 480 m section - Vatlestraumen	15.08.2019
SBJ-33-C5-OON-24-DR-004	0	Marine Operations – General Arrangement Assembly Rig	15.08.2019
SBJ-33-C5-OON-24-DR-005	0	Marine Operations – GA High Assembly Rig	15.08.2019
SBJ-33-C5-OON-24-DR-006	0	Marine Operations – Bridge mooored Søreidvågen	15.08.2019
SBJ-33-C5-OON-24-DR-007	0	Marine Operations – Assembly sequence Søreidvågen	15.08.2019
SBJ-33-C5-OON-24-DR-008	0	Marine Operations – Assembly section 1 & 2 Søreidvågen	15.08.2019
SBJ-33-C5-OON-24-DR-009	0	Marine Operations – Assembly section 1 & 2 Søreidvågen	15.08.2019
SBJ-33-C5-OON-24-DR-010	0	Marine Operations – Assembly section 1 & 2 Søreidvågen	15.08.2019
SBJ-33-C5-OON-24-DR-011	0	Marine Operations – Assembly 480 m sections	15.08.2019
SBJ-33-C5-OON-24-DR-012	0	Marine Operations – Towing Configuration	15.08.2019
SBJ-33-C5-OON-24-DR-013	0	Marine Operations – Towing Hold Configuration	15.08.2019
SBJ-33-C5-OON-24-DR-014	0	Marine Operations – Towing Hold Configuration, alt. B	15.08.2019
SBJ-33-C5-OON-24-DR-015	0	Marine Operations – Towing Søreidvåg to Bjørnafjorden	15.08.2019
SBJ-33-C5-OON-24-DR-017	0	Marine Operations – Assembly sequence	15.08.2019

		Bjørnafjorden	
SBJ-33-C5-OON-24-DR-018	0	Marine Operations – North abutment installation	15.08.2019
SBJ-33-C5-OON-24-DR-019	0	Marine Operations – North abutment installation	15.08.2019
SBJ-33-C5-OON-24-DR-020	0	Marine Operations – Bridge arriving Bjørnafjorden	15.08.2019
SBJ-33-C5-OON-24-DR-021	0	Marine Operations – Floating bridge at North abutment	15.08.2019
SBJ-33-C5-OON-24-DR-022	0	Marine Operations - Connecting at high bridge end	15.08.2019
SBJ-33-C5-OON-24-DR-023	0	Marine Operations - Pre-installed anchors	15.08.2019
SBJ-33-C5-OON-24-DR-024	0	Marine Operations - AHTS connecting mooring lines - Top view	15.08.2019
SBJ-33-C5-OON-24-DR-025	0	Marine Operations - AHTS connecting mooring lines - Side view	15.08.2019
SBJ-33-C5-OON-24-DR-026	0	Marine Operations - Tensioner details - Side view	15.08.2019
SBJ-33-C5-OON-24-DR-027	0	Marine Operations - AHTS tensioning mooring lines - Top view	15.08.2019
SBJ-33-C5-OON-24-DR-028	0	Marine Operations - Mooring lines installed	15.08.2019
SBJ-32-C5-OON-22-RE-002	C	Concept Selection and Risk Management	24.05.2019

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