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FERJEFRI E39 - RAMMEAVTALE FJORDKRYSNINGSPROSJEKTET

Verification of AMC Floating Bridge Concepts BJF 2019

Statens vegvesen region vest

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1 EXECUTIVE SUMMARY

1.1 General

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

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

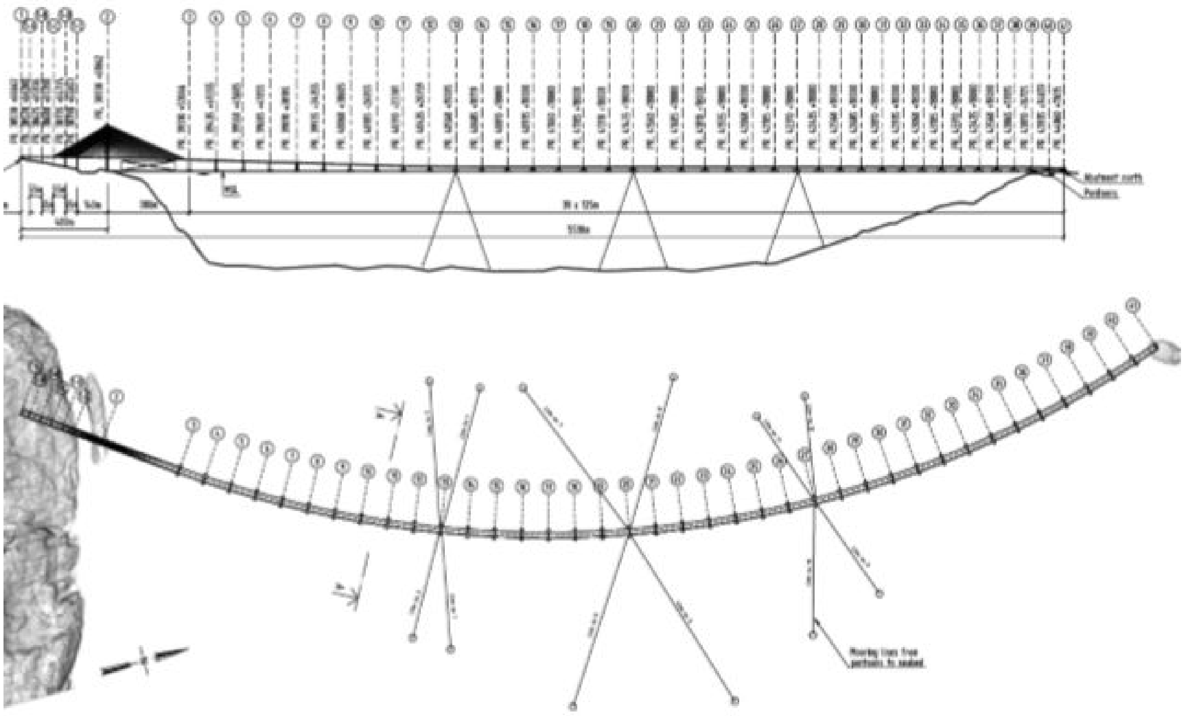

Figure 1-1 AMC chosen alternative K12

1.2 Conclusions

1.2.1 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. The following points are judged to have moderate impact on the cost and schedule estimates:

- The strength of the bridge girder is not fulfilling the imposed actions at the tower and at the North end of the bridge and the need for reinforcements are identified. However, the bridge girder dimensions at these locations are already increased compared with the typical cross-section and further strengthening may lead to more costly details especially since also fatigue loading is high. It may be considered to allow for steel with strength higher than premised in the design specification in order to solve this design task.

- The selected design has not been proven for the specified energies to boat impacts. Capacity to resist half the energy is shown to result in damages that will not lead to loss of the bridge. The bridge will be able to resist larger energies than half the specified for many scenarios, but not at its most vulnerable positions. Even if the high boat impact energies imply large damages to the pontoon, the column and the bridge girder, it is judged that the probability of total loss of the bridge is small consequently the robustness of the K12 bridge concept is considered to be good in this respect. If an ALARP philosophy is followed it is judged that the present design, that has capacity to resist 50% of the impact energy, can be regarded as reasonable.
- It is noted that it is difficult to document enough fatigue life along the longitudinal stiffeners due to local traffic loads without including the stiffness effect of the asphalt layer. It is agreed that one may consider the stiffness of the asphalt layer for this stress direction to avoid a conservative design. It is judged that future technology developments on load modelling, structural detailing and analyses the cost to meet design requirements can be limited.
- 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.

1.2.2 Ranking of concepts

The selection between the four concept that is studied is based on a risk evaluation and it is judged by DNV GL that the evaluations of the risk for the various concepts are fair and we agree with the final ranking presented and the selection of K12 as the preferred alternative.

1.3 Recommendations for future design developments

Some items are considered important for a further development of the concept:

- Review the ship impact methodology to account for the reduced probability for certain impact scenarios and to consider application of ALARP for cases with large cost consequences.
- Traffic model and definition of characteristic traffic loads. The traffic model may be governing for the geometry at significant hot spots and the definition of the characteristic load may influence on the required Design Fatigue Factor.
- The transfer of dynamic forces in the longitudinal direction of the bridge girder leads also to hot spots in the trapezoidal sections at the welded connections at the cut-outs in the transverse girders. It is important to document enough fatigue life at all these hot spots due to the large number of connections between the longitudinal stiffeners and transverse frames.
- It is expected to be simpler to fabricate connections with long fatigue lives using other types of stiffeners than trapezoidal sections such as HP sections. Therefore, one may check if these stiffeners can be efficiently used in other areas than below the traffic loaded bridge plate.
- The calculation methods related to fatigue damage in the deck plate should be calibration as far as possible with experience. There are also hot spots at the connections between the longitudinal trapezoidal sections and the transverse frames that needs to be further assessed with respect to traffic loads. Also, here the stiffness of the asphalt layer may be considered accounted for in the fatigue assessment.

- Assembly of the high floating bridge. Technical challenges are related to jacking of the bridge girder, notably stability of the jacking towers, accidental conditions etc. Manoeuvring and sea room at the proposed assembly location should also be considered further.
- 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, and thereafter be able to transfer the sectional bridge girder forces until the connections have been welded. The documentation regarding this is immature, and 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 AMC 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 AMC. A total of four (4) concepts will be investigated by each of the design groups and one of these considered concepts will be 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	AMC
28/01-19	Status report no 1 with concept ranking issued by AMC	AMC
29/03-19	Status report no 2 with estimates of masses, costs and updated drawings/descriptions for all 4 alternatives issued by AMC	AMC
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 AMC. Interviews to be performed by DNV GL.	SVV
24/05-19	Report from AMC on their chosen bridge concept including evaluations for the three other bridge concepts.	AMC
30/06-19	Documentation basis (drawings and descriptions) for investment estimates of chosen bridge concept	AMC
15/08-19	Final documentation delivery of recommended bridge concept	AMC
31/08-19	Final documentation of the three (3) other bridge concepts	AMC
31/08-19	Resource-diagram prognosis for the period Dec. 2019 – Dec- 2020 (part B)	AMC
31/08-19	Part A completed	AMC

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

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 are 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

This report presents the results from review of the design documentation for the four concepts to the Bjørnafjorden crossing. The scope for the review is related to check that the load carrying capacity and the sustainability of the floating bridge structure 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 of the correctness of the summary tables for the quantities and of the cost estimates.

3.4 Verification objective

The main 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.

3.5 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-023-A, Independent Analyses of AMC Floating Bridge BJF 2019).

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 has an effect on 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 mainly described in AMC documents SBJ-32-C5-AMC-90-RE-106 and SBJ-32-C5-AMC-21-RE-108.

4.2.2 First order wave loads on pontoons

Hydrodynamic input to the AMC Orcaflex model is generated by Wamit. As shown in **Figure 4-1** and **Figure 4-2**, the AMC computed 1st order wave excitations forces are almost identical to the DNVGL results computed by Wadam. This is to be expected since the 3D potential theory in Wadam is based directly on the Wamit program developed by Massachusetts Institute of Technology. The good agreement between the two calculations shows that the geometry modelling is correct and that there are no user errors.

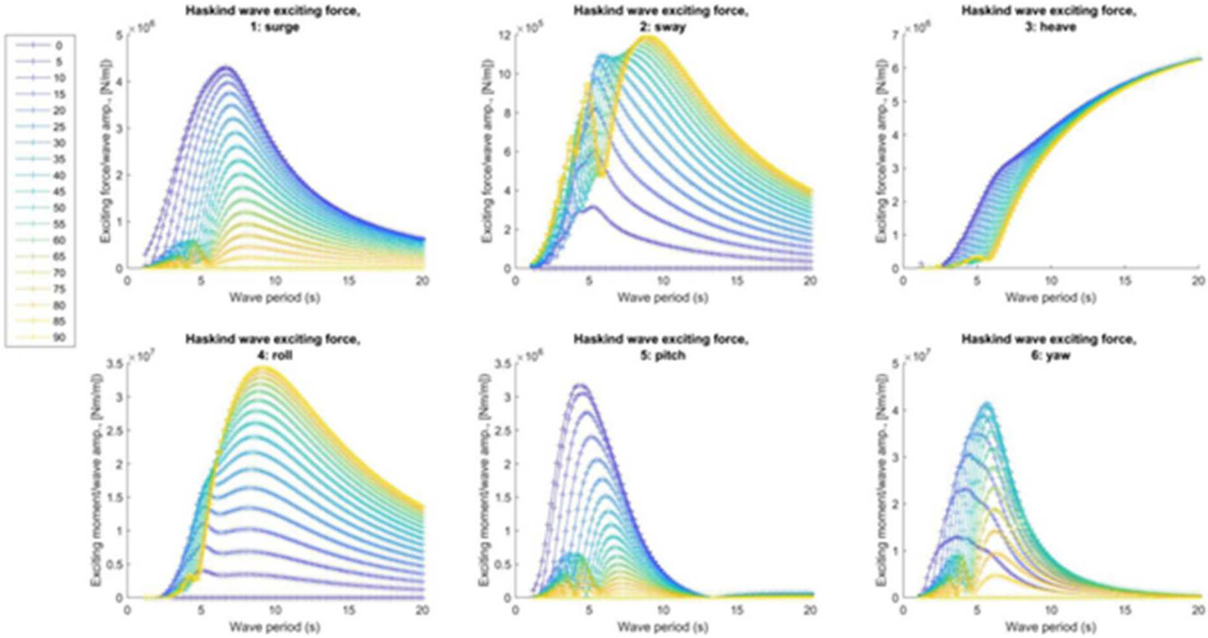

Figure 4-1 Wave excitation forces on pontoons – AMC results (local x-axis in bridge longitudinal direction)

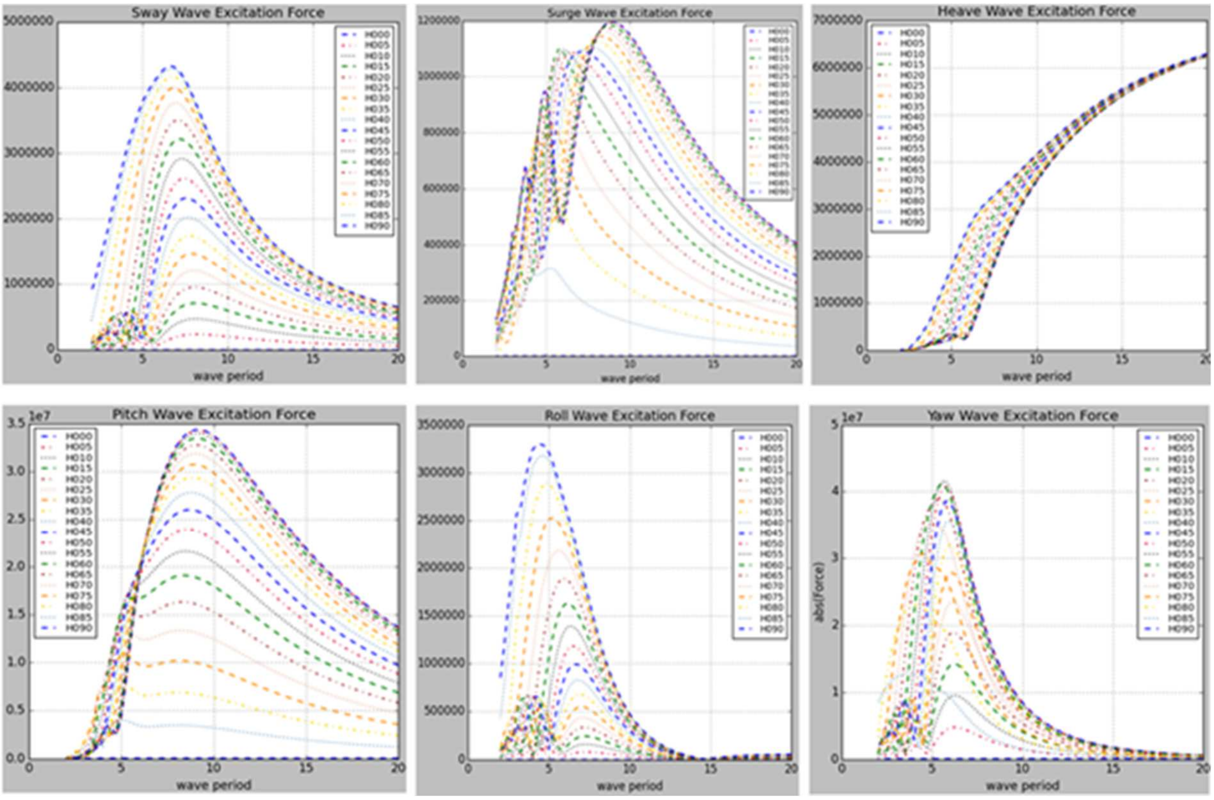

Figure 4-2 Wave excitation forces on pontoons – DNVGL results (local x-axis in pontoon longitudinal direction)

4.2.3 Pontoon added mass and damping

The AMC computed pontoon added mass and damping are also in accordance with DNV GL computed results. Comparisons are shown in **Figure 4-3** and **Figure 4-4**.

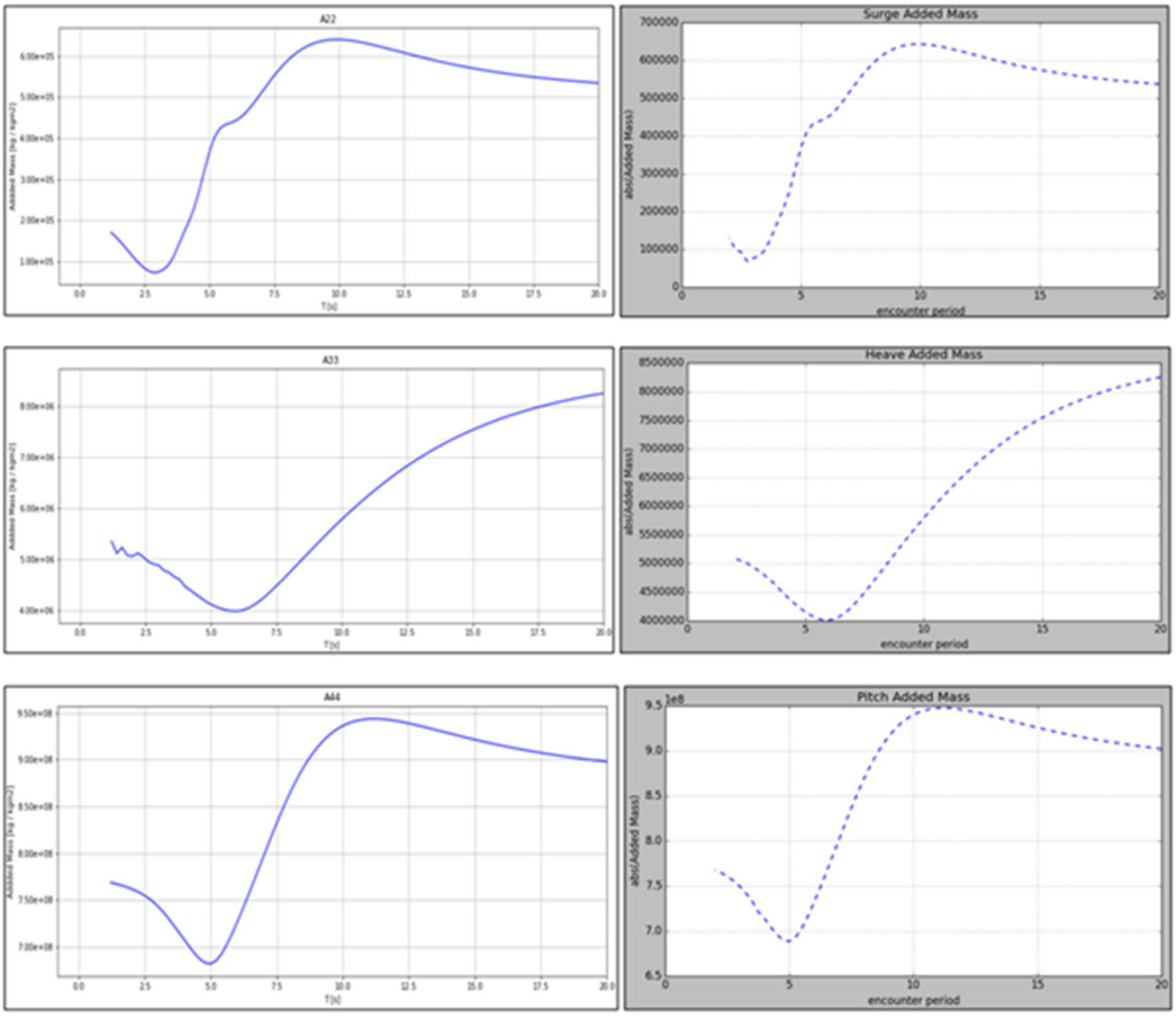

Figure 4-3 Computed added mass in sway, heave and roll (AMC coordinate system). AMC results left, DNVGL results right.

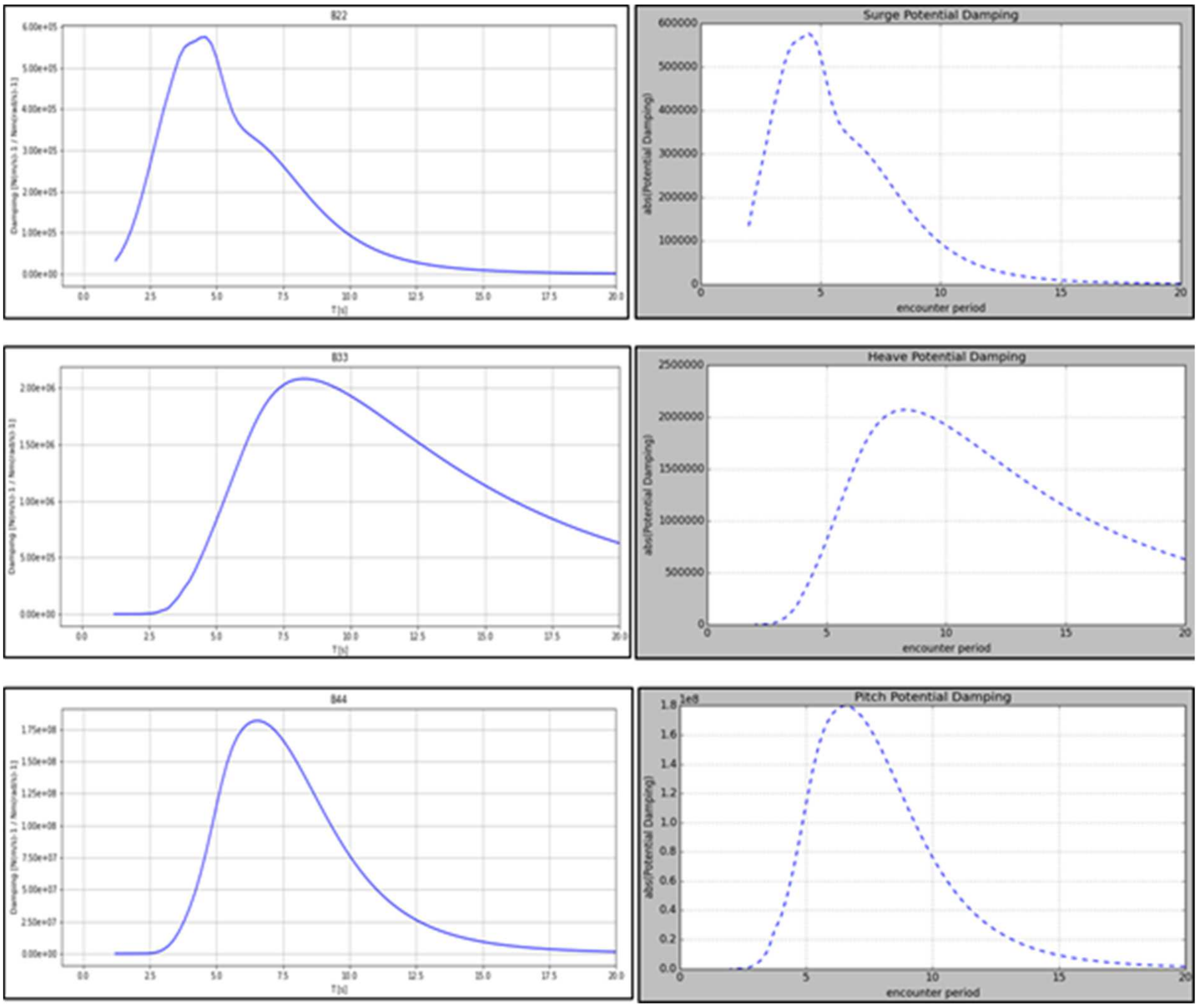

Figure 4-4 Computed potential damping in sway, heave and roll (AMC coordinate system). AMC results left, DNVGL results right.

4.2.4 Hydrodynamic interaction effects on pontoons

AMC 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 5-5 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.

A longer span width reduces the hydrodynamic interaction. AMC shows that the influence is adequately accounted for by modelling three pontoons. Increasing the number of pontoons to more than three does not affect the results significantly.

The interaction effect on the response is found to be largest for the vertical motion and the weak axis moment. The analyses indicate that FLS conditions are more affected than the ULS conditions.

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.

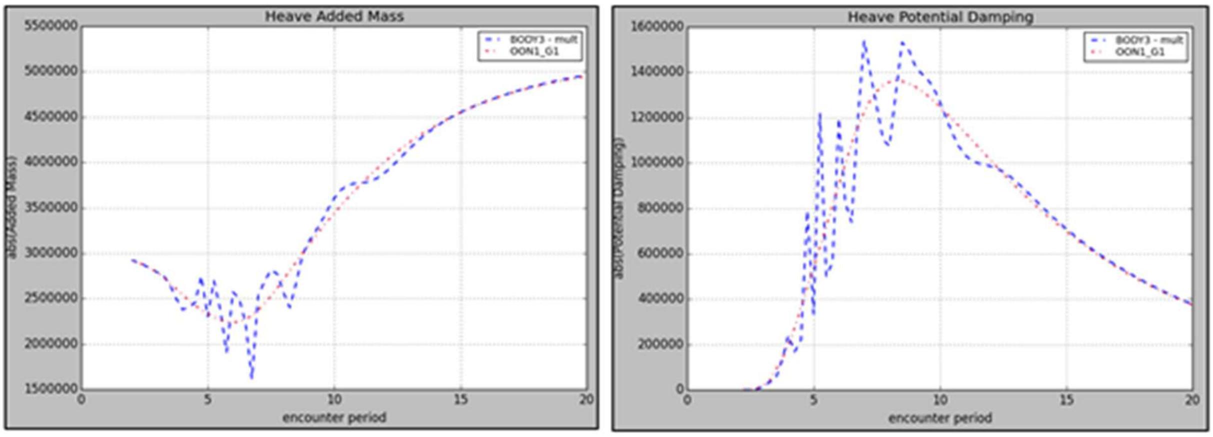

Figure 4-5 Typical differences between single body and multi body analyses – DNVGL computed heave added mass and potential damping for one body (red) and three bodies (blue).

4.2.5 Second order wave effects

In the calculation of the second order wave effects AMC assumes a single fixed pontoon. The difference between applying the full quadratic transfer function (QTF) and applying the Newman's approximation is investigated. AMC concludes that the Newman approximation gives conservative results for shorter wave periods while it may give somewhat lower response values for wave periods in the range 7-10 seconds. Based on their study AMC recommends to not use the full QTF in the global analysis calculations.

Applying Newman's approximation, assuming a single pontoon and assuming the pontoon to be fixed are all simplifications in the calculation of second order wave effects. DNV GL agrees that these simplifications can be made to assess the second order wave effects.

4.2.6 Viscous damping

AMC has estimated pontoon viscous damping based on a literature review. The drag coefficient in the longitudinal direction of the pontoons is taken from the values provided in DNVGL-RP-C205 for rectangular cross-sections with rounded corners.

Based on this reference a drag coefficient Cd=0.3 is applied. It should be noted that the values provided in DNVGL-RP-C205 are for Reynolds number Rn≈10^5 while the flow past the pontoons is in the post-critical flow regime Rn>10^6 where the drag coefficient is somewhat higher.

The preliminary CFD calculations performed by Core-Marine estimates a drag coefficient of Cd=0.41.

AMC applies an analytical approach for estimating the viscous damping from mooring lines. This is described in enclosure 2 in SBJ-32-C5-AMC-90-RE-106. The analytical model is benchmarked against a full mooring line model and found to give a reasonable representation of the damping level.

The applied analytical model is well recognized and is commonly used in the industry.

4.2.7 Pontoon freeboard and wave run-up

Wave run-up and freeboard exceedance on the pontoons have been investigated with respect to possible influence on the global responses of the bridge. A simplified model is applied. The performed simulations show negligible effects. DNV GL considers the proposed simplified model as sufficiently accurate for preliminary estimate of possible effects of freeboard exceedance. AMC provides suggestions for more accurate estimations at a later stage in the project.

4.2.8 Wave-current interactions

The studies performed on the effect of wave-current interaction conclude that this has a large influence on the global responses of the bridge. The encounter frequency for the waves on the pontoon is affected by the wave-current interaction and different eigenmodes are thereby excited.

In the case where a current velocity of 1.5 m/s is combined with the 100-year easterly wave, the resulting motion responses of the bridge is significantly increased. AMC questions the validity of this joint environmental condition. The current velocity is taken as the 10-year current which is according to recommendations given in NORSOK N-003 for combinations of metocean conditions. However, these recommendations are known to be conservative. The actual current velocity to be applied with a highest wave height at an annual joint probability level of 10^{-2} is likely to be significantly lower.

AMC recommends that the metocean design basis SBJ-01-C4-SVV-01-BA-001, is updated with specified combinations of waves and current at the 100-year level. The global analyses of the selected bridge concept should include these combined wave-current load cases. DNV GL agrees with this recommendation.

The wave spectra given in the metocean design basis are based on hindcast data and measurements. These wave spectra are given without current present. It is not clear whether the estimated wave spectra take into account current present during measurements.

4.2.9 Inhomogeneous wave conditions along bridge

The global models used by AMC in the present analyses assume fully correlated short crested sea, i.e. the applied stochastic wave field consisting of a JONSWAP wave spectrum and a wave spreading function does not change across the fjord.

AMC has investigated the effect of inhomogeneous wave conditions by assuming the interaction between each pontoon to be negligible. This is done by applying unrelated individual representations of waves at each pontoon. The performed study indicates that uncorrelated wave field may give a slight increase in the response.

The variation in wave height across the fjord is studied for one wave direction sector. Scaling factors are taken from the metocean design basis. The results show some effect towards the abutments.

The scaling factors vary with wave direction. Other wave directions are not investigated.

In the metocean design basis, scaling factors are also given for the variation in wave peak period across the fjord. This has not been covered by AMC in their studies.

4.2.10 10 000 year conditions

Apart from the freeboard exceedance study, the 10 000-year return period conditions have not been investigated by AMC 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-33-C5-AMC-20-RE-105_0 Appendix E - Aerodynamics, K12, AMC 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. 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. It appears that for the design analyses, the wind input has been taken from N400. Due to the significant sensitivity to the statistical variations of the wind characteristics, it is recommended to consider more unfavourable combinations in the next phases.

4.3.2 Aerodynamic stability

An assessment of aerodynamic stability has been carried out for the K12 concept in report SBJ-33-C5-AMC-20-RE-105. Aerodynamic force coefficients, static coefficients and aerodynamic derivatives have been calculated by use of CFD and calibrated against wind tunnel tests. The four types of aerodynamic instabilities specified in N400 have been investigated, galloping, static divergence, flutter and torsional instability. The bridge system was found to be aerodynamically stable. In addition, the bridge girder and stay cables have been checked for possible vortex induced vibrations (VIV). The conclusion is that vortex shedding is not expected for the bridge system. However, stay cables are susceptible to VIV and mitigations, e.g. external dampers are needed to suppress the cable vibrations. Other three other aerodynamic cable instabilities (dry galloping, ice/sleet galloping and rain/wind galloping) have been evaluated. It is judged that aerodynamic stability is satisfactory investigated at this stage of the project.

4.4 Parametric excitation

The robustness of all four concepts (K11, K12, K13, K14) have been assessed with respect to possible parametric excitation, due to oscillating variations in structural properties of the bridge (SBJ-32-C5-AMC-90-RE-119). For Bjørnafjorden slender bridge girder, the cause of possible parametric excitation is the axial force variation which induces a variation in the geometric stiffness of the system. According to the classical theory of Mathieu instability, this could lead to resonant response in lateral eigenmodes with low damping. The assessment is based on the onset criterion for parametric excitation proposed by NTNU (SBJ-32-C4-NTNU-22-RE-001).

AMC concludes that K11, K12 and K14 are prone to parametric excitation from swell waves since the NTNU criterion is not passed. However, mooring lines for side anchored concepts (K12 and K14) give significant contributions to damping for the critical modes. Due to mooring line damping, the K12 and K14 concepts are considered robust regarding parametric excitation. AMC has also proposed a possible mitigation for the K11 concept, which has low level of damping and therefore significantly higher axial force variation, by releasing the bridge girder at the tower and position dashpots at tower connection. Whether this design measure is feasible, needs further considerations.

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. The NTNU criterion is based on analysis of a simple structure, Hence, parametric criterion may not be a problem even if the criterion is not passed. But such resonant response cannot be excluded and DNV GL recommends further investigations on this problem.

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, ship deckhouse vs bridge girder and rigid crane pedestal vs bridge girder. Based on the calculated impact damage, the following damaged conditions are considered in Global analyses - response SBJ-33-C5-AMC-90-RE-107: flooding of pontoons, loss of anchor mooring lines and damage to the deck girder.

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

DNV GL are aware that the criterions for boat impact might be made less strict in the next phase of the project. In the following, DNV GL 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 observed that the torsional moment is taken from an analysis based on linear material properties. It is also observed that bow impact parallel with the bridge are using same speed as orthogonal to the bridge which is according to design basis. This report is going far to claim that the design does not necessarily fulfil the originally requirements. It is said that if the requirements in the design basis remains, the column must be of 40mm and with a full size of the column. The drawings are stated with 25mm with a note of 40mm for boat impact and with a reduced circumference of the column giving less torsional capacity than what is required to fulfil the requirement in the design basis. Since there is an uncertainty about the requirements, this uncertainty must also be addressed and accounted for in the cost estimate. As DNV GL understand cost estimate SBJ-33-C5-AMC-90-RE-116 rev 0 it is accounted for energy absorption of 50%.

4.6 Global response analyses

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

- RM-Bridge for permanent and traffic loads
- Orcaflex for wave loading in frequency and time domain and wind loading in time domain
- Novaframe – dynamic wind response in frequency domain, input to modal analysis
- LS-DYNA for local and global ship collision simulations

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 0.2 sec
- 10 seeds
- Number of elements between pontoons: 16
- Structural damping: The ratio is set to 0.5% for two frequencies. The angular frequencies chosen are 0.03927 rad/s and 6.28 rad/s, corresponding to period range 1-160 seconds. This means that the damping at the intermediate frequencies are less than 0.5%, which is conservative.

The metocean specification SBJ-01-C4-SVV-01-BA-001 describes inhomogeneous wave conditions. This has been analyzed as a sensitivity, and designer concludes that this may have a significant effect on global response, and that more detailed metocean specification is needed in the next phase.

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%

The designer has based the design check on using the expected maximum from a superposition of uncoupled time series but have demonstrated by coupled analyses that this agrees with the above requirement for ULS.

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

The governing 100 year return period load cases were determined through a screening analysis where all important load components (axial force, bending moments, torsion, mooring forces and displacements were evaluated individually. Based on this, 6 different environmental combinations were identified for detailed analyses, this includes wind sea with directions 75, 105, 195 and 315 and swell with direction 300 degrees, two different peak periods. Our independent analyses indicate that governing cased for strong axis bending at axis 2 may have been overlooked by this procedure.

For fatigue, calculations have been based on frequency domain, and a subset of the fatigue conditions have been analyzed in a coupled time domain simulation. No documentation of the current speed applied in the fatigue calculations has been found.

For design evaluation time series for each load component were combined by adding time series, and the load factors were applied on the expected 1 hour maxima. This proved to give conservative results (except for torsional response close to the high bridge) compared to the 90% fractile when all environmental loads were applied simultaneously.

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

- Response from wind-waves is not sensitive to variation in wave spectrum parameters, whereas swell waves are moderately sensitive to period.
- Mooring line damping is dependent on the line pretension and thereby the static transverse offset of the bridge girder. For a selected condition with extreme offset due to temperature and tide (>10 000-year return period) the mooring line damping was reduced with 70%, resulting in an increased strong-axis moment in swell of about 10%.
- Variations of abutment stiffness in a reasonable range does not affect global bridge behavior.

Second order wave drift forces affect the mean bending moment about strong axis but not the dynamic values. It is considered to not have a significant effect on the bridge response but should be included for completeness in future simulations.

The review has not identified any issues with respect to the analysis method.

4.7 Global analysis results comparisons

Reference is made to the independent analysis report, (SBJ-32-C5-DNV-62-RE-023-A, Independent Analyses of AMC Floating Bridge BJF 2019). The main findings are:

4.7.1 ULS

The calculated stresses exceed the ULS capacity of the box girder at Axis 2 (tower) and need to be reinforced. At the North end the capacity is at the limit and reinforcements may be needed. This agrees with the checks done by the designer. The rest of the bridge girder satisfy the specified requirements.

4.7.2 FLS

The independent analyses carried out by DNV GL determines the contribution to damage from environmental loads in the bridge girder. The results from the screening analysis show a minimum fatigue life of 482 years. This number should be reduced due to the local stress increase which will bring the fatigue damage from environmental loads close to the required life of 250 years.

The contributions from traffic and tidal variation are not part of the independent analyses by DNV GL. The contributions will add to the damage 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 show that it should be expected that in certain areas details as assumed in the fatigue screening with SCF of 1.5 and SN-curve D may not be allowed even from environmental actions alone.

4.7.3 Mooring

The loads in the mooring system give a safety factor well above the requirement of 2.2

The calculated fatigue life for the top chain is just above the requirement of 50 years design life and DDF of 10, while the bottom chain goes below 100 years design life and DDF of 10. These results are with a SCF of 1.15, which may be conservative.

4.8 Structural design for various components

4.8.1 Pontoons

4.8.1.1 General

The report SBJ-33-C5-AMC-22-RE-111 rev. 0 Appendix K and Enclosure 6 were reviewed.

Two different pontoon shapes "*cirtangel*" and "*kayak*" have been assessed in which the first rectangular type was recommended since the fabrication will be simpler.

4.8.1.2 ULS

Two types of stiffened plated pontoon structures with and without mooring have been assessed with relevant loading and to well-known methods as far as we can see without doing a detailed review.

DNV GL noted that the dynamic wave pressures were stated to have been modelled from the Stillwater line up to the pontoon top plate and applied along the entire pontoon. A more exact approach using a design wave with length equal to the pontoon lengths allowing for a more precise calculation of the global pontoon stresses should be considered for future design developments.

The lateral internal tank pressures might have to be considered more thoroughly considering the filling methods.

4.8.1.3 FLS

A general screening of the pontoon plating to check that the dynamic stress levels allow for the expected welding details are generally missing.

The corners from the columns to the pontoons are designed with castings. This simplifies fabrication and makes it possible to achieve long fatigue lives at these areas. Plates with thickness of 50 mm has been indicated at these cast pieces. Then there is a transition in thickness to a 20 mm thick plate. One might question if the 50 mm plate should be extended or if there should be more transitions in thickness down to the 20 mm thick plate. This can be further assessed in the next phase of the project. It can be assumed that this has not a significant influence on selection or cost of concept. In the latest analysis report long fatigue lives are reported and it is indicated that castings may not be needed.

The connections between the columns and the pontoon outside the cast steel areas seems not assessed in detail. DNV GL note that HP stiffeners are applied both in the columns and in the pontoons resulting in a simpler detailing. Nevertheless, stress concentrations might have to be reduced by use of brackets and eventually reducing the use of eventual keyholes.

4.8.1.4 ALS

The current design is not fulfilling the Accidental Limit State condition with respect to boat impacts. The reports states that the resistance can be improved by increasing the column shell plating from 25 to 40mm. In addition, it is stated that a change of the column shapes using a regular form also will improve the collision resistance.

The boat impact studies are commented more in other parts of this report, see Section 4.5.

4.8.1.5 Structural Drawings

The pontoon drawings are listed in Appendix B. The drawings of the pontoons including the stiffening system look reasonable.

The pontoons will be fabricated using S420 steel quality both for the plating and for the stiffening system. In addition, super duplex steel is indicated applied in a certain width covering the splash zones.

4.8.2 Mooring lines

4.8.2.1 General

The mooring system is a conventional semi-taut line mooring system with 12 mooring lines, where 3 pontoons have 4 lines each, two lines at each side of the bridge.

Pontoons with mooring lines attached have an increased draft of 2.5 m. The mooring lines are connected 6 m below the water line, which is outside the splash zone, and also below the keel of most passing vessels that can collide with the bridge.

The mooring lines are composed of a bottom chain, a middle wire rope section and a top chain. The system gives progressive stiffness for displacements beyond 5 m. Typically, maximum offset is 10 m.

Mooring analyses are performed without marine growth included. This is sufficient at present stage. DNV GL previous analyses have shown that the marine growth on the mooring lines will increase the damping, and thus reduce the response in the bridge at the expense of a small increase in line tensions.

An effect that normally can be neglected in mooring system design is the change in pretension due to temperature. For this bridge the effect of temperature is not insignificant and has been included in the design analyses.

The top chain will be terminated in a chain stopper at an articulated arm with low friction, thereby reducing out-of-plane bending effects. However, a SCF of 1.15 has been used for both top chain and bottom chain. In the next phase it should be checked how out-of-plane bending and wear at the anchor must be considered in the mooring design.

4.8.2.2 ULS

The mooring system is designed in accordance with ISO 19901-7, Annex B2. Mooring analyses have been performed in a fully coupled analysis model in Orcaflex. It is stated that extreme tide and temperature is included in the calculations, but no details are presented. The utilization of the mooring system is below 0.76 for all mooring lines.

4.8.2.3 FLS

Tension-tension fatigue of mooring lines has been calculated according to DNVGL-OS-E301, while out-of-plane bending fatigue has been calculated according to BV Guidance Note NI 604. The results are stated to give fatigue life above 100 years for both effects, but few details are presented. Our independent analyses indicate that the fatigue life of bottom chain is below requirements in the mooring design basis.

4.8.2.4 ALS

For ALS, all mooring lines on one side of a mooring cluster has been removed. Results presented show lower line tensions than in ULS, which seems strange and should be checked. However, removal of two lines is not expected to be governing for the design.

The mooring system has also been checked for ship impact on the pontoons.

4.8.3 Subsea anchors

4.8.3.1 General

DNV GL has not put much effort into reviewing the documentation of subsea anchors in this phase. From the documentation it is seen that four different anchor types have been considered by AMC in their efforts:

- Gravity anchors
- Suction anchors
- Plate anchors
- Mixed/combined anchors.

For the K12 alternative suction anchors are suggested for all 12 anchor points. Water depths at anchor points varies from 360m – 560m. Diameter of the suction anchors varying from 6 m – 8m with heights 11m – 19m. Steel material selected as S355.

It is further noted that both ULS and ALS mooring loads acting on the anchors have been established; however, no fatigue analysis has been reported. It is, however, expected that sufficient capacities (ULS, ALS, FLS) can be documented in a later phase of design for the K12 alternative.

It is also assumed that the actual soil properties at the selected anchor locations will be further looked into in upcoming design phases.

4.8.4 Cable stayed bridge

4.8.4.1 General

The cable stay bridge part is documented in report SBJ-33-C5-AMC-22-RE-112 Appendix L. The design is building on 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 and the rather large horizontal loads imposed to the sliding bearings at the tower.

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.

4.8.4.4 ALS

Reference is made to Section 4.5 for ship impacts.

4.8.5 Bridge abutments

4.8.5.1 General

The bridge abutments are a cell structure founded on rock made of concrete and filled with ballast of gravel in the south and iron ore in the abutment North. The suitability of iron ore should be evaluated due to possible forces from expansion due to corrosion. However, alternative fill material is not expected to lead to large cost increase.

4.8.5.2 ULS

The ULS design is carried out according to well established methods and to a level of detail that is seen as sufficient at this stage of the project.

4.8.5.3 FLS

Prestressing cables are used to connect the bridge end to the North abutment. The longitudinal trapezoidal sections are ended at a thick plate where the cables are connected. Here similar cut-outs in this transverse plate cannot be used as the transverse girders. This is expected to result in a larger stress concentration at the corners of the trapezoidal sections. Thus, thicker trapezoidal sections may be needed locally at the bridge girder ends due to this effect. However, this is not expected to significantly increase the amount of steel or cost and is thus not of importance for selection of concept. Similar consideration may apply to the connection between the steel girder and the concrete girder at the south end.

4.8.5.4 ALS

Reference is made to section 4.5 for ship impacts.

4.8.6 Bridge columns

4.8.6.1 General

The report SBJ-33-C5-AMC-22-RE-111 revision 0 Appendix I and Appendix K as well as enclosures 5 and 7 were reviewed.

4.8.6.2 ULS

The column structures including the stiffening system has been assessed for ULS and ALS in the document SBJ-33-C5-AMC-22-RE-11 Enclosure 5. The typical short and the long columns have been assessed with relevant loading and to well-known methods as far as we can see without doing a detailed review.

4.8.6.3 FLS

A general screening of the plating to check that the dynamic stress levels allow for the expected welding details are generally missing, but with the proposed plate thickness, the nominal dynamic stress levels are assumed to be low.

The corners from the columns to the bridge girder are designed with castings. This simplifies fabrication and makes it possible to achieve long fatigue lives at these areas. Plates with thickness of 30 mm has been indicated at these cast pieces. Then there is a transition in thickness to a plate with thickness 14 mm. In the next phase of the project one may assess if it is recommended to make transitions in more steps. It is assumed that this will not be of significance for concept selection or result in significant additional cost.

The connections between the columns and the pontoon outside the cast steel areas seems not assessed in detail. DNV GL note that HP stiffeners are applied both in the columns and in the pontoons resulting in a simpler detailing. Nevertheless, stress concentrations might have to be reduced by use of brackets and eventually reducing the use of eventual keyholes.

4.8.6.4 ALS

The most relevant ALS load for the design of the column is ship impact that is dealt with in Section 4.5.

4.8.6.5 Structural Drawings

The column drawings are listed in Appendix B.

The drawings including plating and the stiffening /framing system look reasonable.

It is indicated on the drawings that the shell thickness shall be increased to 40mm due to ship collisions, but the drawings do not contain any information regarding possible changes of the shapes. As such the design seems not fully set with respect to boat impacts.

The columns will be fabricated using S420 steel quality both for the plating and for the stiffening system.

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 with open stiffeners (bulb) in the bottom plate. 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. The design of the cross-sections is made with stiffener positions at the same locations in the total length of the bridge girder. This is a favourable design principle as it avoids complex transitions between different shape of stiffeners. However, at the end of some longitudinal trusses there is an abrupt ending without a continuing stiffener as shown on **Figure 4-6**. This may be avoided in the next phase by careful distribution of the bridge deck stiffeners.

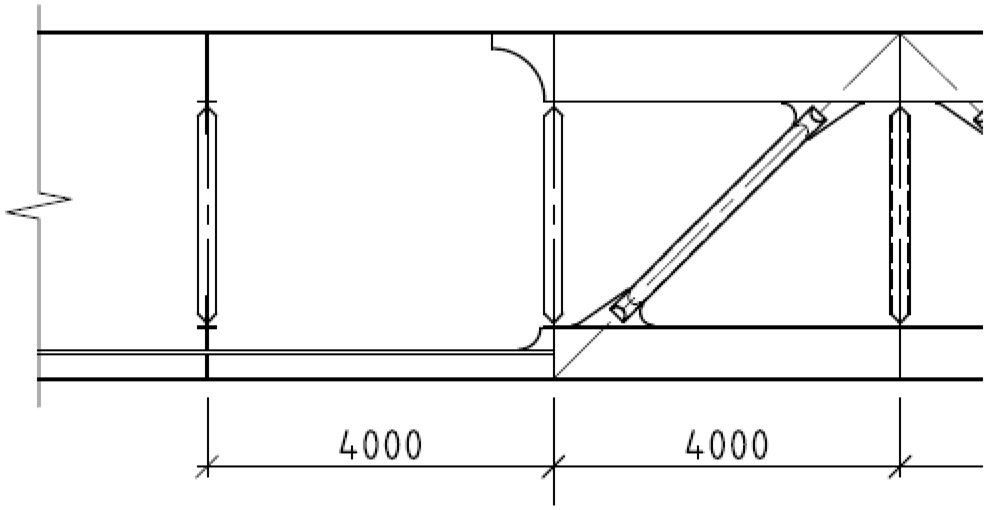

Figure 4-6 Abrupt end of longitudinal truss

4.8.7.2 ULS

The box girder cross-section is made as stiffened plates made with trapezoidal - and bulb stiffeners. The height of the cross-section is typical 4.0 m and the span between pontoons is 125 m. Longitudinal bulkheads are included at columns and extend up to 40 m into the span providing increased stiffness about weak axis and reducing stresses both from traffic and environmental loads.

The check of the bridge girder is made according to EN-1993-1-5 (NS-EN-1993-1-5:2006, Eurocode 3:Design of steel structures, Part 1-5: Plated structural elements) and reported in SBJ-32-C5-AMC-22-RE-111 Appendix K Enclosure 8 when it comes to accounting for shear lag effects and for check of plate buckling. As pointed out in the report application of Eurocode for this type of structure will be conservative. The design of the bridge girder deviates from the typical bridge designs that is intended to be covered by Eurocode. That means that shear lag effects will not be accurately determined as the support condition here is on the columns whereas in Eurocode support directly under the longitudinal shear carrying elements are assumed. It is therefore recommended that future design developments are made with assessing shear load effects from local shell models.

The report gives a comprehensive presentation of the capacity checks made, but there is not given a result overview of the utilization for the various positions along the bridge. It is referred to the calculation of the von Mises stresses in the global report SBJ-32-C5-AMC-90-RE-107 as an additional check. The von Mises checks in this report are exceeding the limits at the supports which indicate that buckling checks may be critical at these positions. It would be of interest to see a summary of the checks. However, it is judged that needed reinforcements will be with limited impact on the total steel weight.

Shear stresses are neglected in the buckling checks of the stiffened panels which is understandable as the contribution of shear to the buckling capacity is not explicitly covered in Eurocode. It is stated that "the shear resistance is considered as full" and it is unclear if shear stresses are influencing the buckling capacity towards other stresses in the checks. Obviously, there may be parts of the cross-sections that for certain load cases can be significantly loaded in shear and the effects cannot be neglected for all areas.

4.8.7.3 FLS

Fatigue due to global and local load response

No significant comments are given with respect to the presented K12 concept with respect to fatigue due to environmental load response. It is noted that the anchoring of the bridges has a positive effect on calculated fatigue lives.

The fatigue of the transverse butt welds in the deck plate are assumed to be governing for determining the plate thickness for stress ranges in the longitudinal direction of the bridge from global and local load responses (detail 2a in Figure 4-7). A fabrication tolerance between the plates equal 2.0 mm is assumed as recommended in (Norwegian public roads adm, 2015). It is assumed that the design is performed using S-N curve D for double sided welds and that a misalignment equal 0.05t has been assumed included in this S-N curve which is according to the 2019 version of DNVGL-RP-C203. A Design Fatigue Factor equal 2.5 is used for the bridge girder.

It is noted that the change in plate thickness is typically larger in the drawings than that used in the calculations of stress concentration factors. However, this can easily be improved at a later stage of the project.

It is noted that several structural details in the bridge girder which are subjected to traffic load are found to have insufficient fatigue life with the applied traffic load model as given in the design basis. A sensitivity study has been reported where necessary reductions in fatigue traffic loads to achieve sufficient fatigue life for these details have been quantified. See also comment on definition of characteristic long-term loads and safety factors in the section below for development of the next design phase.

The selection of traffic model will not influence on concept selection. However, it may have consequence for the total cost of the bridge.

So far in the concept development the fatigue lives of the attachments of the guard rails to the bridge girder has not been presented. These rails may be welded to the bridge deck using doubling plates. Then the S-N curve to be used depends on the size of the doubling plates. If the size of the plates in the direction of the main cycling stress is between 120 and 300 mm, this detail will be classified as F1 in as-welded condition according to DNVGL-RP-C203. The difference in stress between this S-N curve and the D-curve used for design of the butt welds is 1.43. As long as a larger SCF than 1.43 is used for the butt welds, these attachment plates will not be governing for the design.

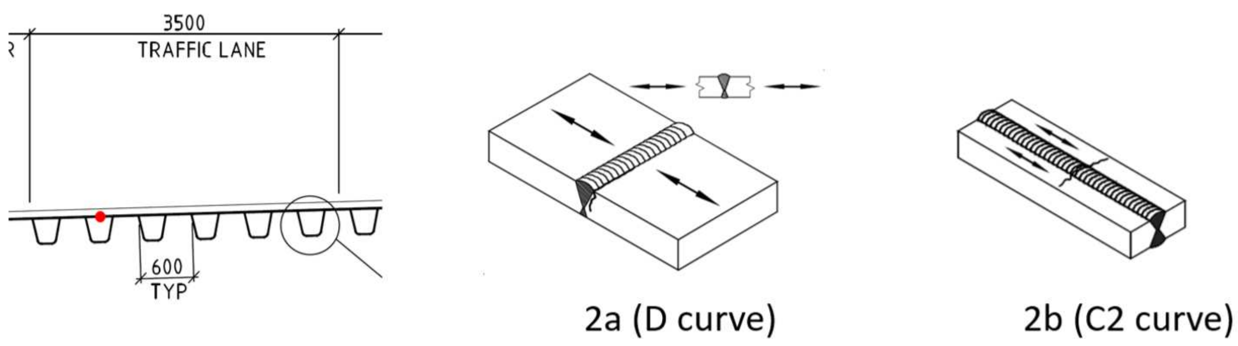

Figure 4-7 Location of hot spot used for assessment of detail type 2a and 2b. This is at the westerly wheel position in the westerly slow lane (From SBJ-32-C5-AMC-22-RE-109)

Reference is made to detail type 5 in section 5.1.6 of Appendix I SBJ-32-C5-AMC-22-RE-109 (Figure 4-8). It is noted that short fatigue lives are calculated for this detail due to global and local load effects. The reason for short calculated fatigue lives for this detail is bending of the stiffener due to wheel loads. It might be questioned if one should consider the stiffness effect of the asphalt layer as considered below for transverse stresses in the deck. However, it may be difficult to accept a stress reduction from global bending effects without including studs to document interaction between the asphalt layer and the deck plate for global beam loads. Another alternative may be to increase the stiffener size. The need for this depends also on the traffic model to be used. With the present traffic model, it is assessed that the target fatigue life can be achieved if the stiffeners areas below the heavy traffic lanes are increased by approximately 0.03 m^2.

The fatigue analysis procedure for dynamic stresses in the longitudinal direction of the bridge is complex when environmental load response is to be combined with stress ranges due to traffic load. Therefore, it is relieving that it in general has been found feasible to document acceptable fatigue lives for these dynamic loads and that this design condition is not leading to significant more fabrication cost than that due to local traffic loads which is of general concern also in design of other similar orthotropic steel girders used in suspension bridges.

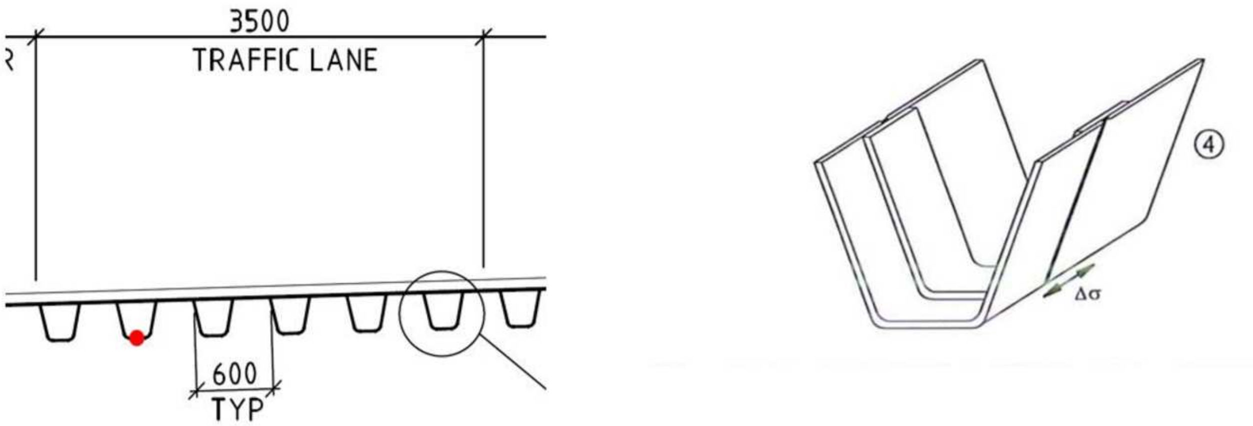

Figure 4-8 Location of hot spot used for assessment of detail type 5. This is at the westerly wheel position in the westerly slow lane (from SBJ-32-C5-AMC-22-RE-109)

<u>Fatigue due to local traffic loads</u>

It is noted that the calculated fatigue lives for butt welds type 2b (Figure 4-7) and trapezoidal longitudinal welds (Figure 4-9) do not meet the target life for local load effects. Reference is made to section 5.1.3 and 5.1.5 in Appendix I SBJ-32-C5-AMC-22-RE-109. This is a similar problem as observed with fatigue cracking through the deck plate as illustrated in Figure 4-10 in other suspension bridges which are described in recent published literature (Bohai, Rong, Ce, Hirofumi, & Xiangfai), (Cheng, Ye, Cao, Mbako, & Cao), (Kainuma, et al.), (Maljaars, Bonet, & Pijpers), (Yokozaki), (Wang) and (Zhu, et al.). According to Bohai et al. (2013) (Bohai, Rong, Ce, Hirofumi, & Xiangfai) it was found that one need to include the stiffness of the asphalt layer to get correspondence between observed fatigue cracking and calculated fatigue lives for fatigue cracking through the plate in a bridge girder with plate thickness 12 mm where fatigue cracking occurred only 5 years after being put in-service. Fatigue crack growth from the weld root through the deck plate in typical girders used in suspension bridges is now a large concern in many countries. Based on this experience it was decided to increase the deck plate thickness to 16 mm in the heavy truck lane in the Taizhou suspension bridge crossing the Yangtze River in China. This is a 3-pylon suspension bridge with 1080 m + 1080 m steel box girder in the main spans. For the other two lanes 14 mm thick deck plates were used.

When comparing past experience with calculations, it should also be mentioned that fatigue test results indicate that the Eurocode is conservative for this detail and type of loading (Root-deck crack in Figure 4-10).

It might be added that the frequency of heavy load traffic on the Bjørnafjorden bridge is expected to be less than on the Taizhou suspension bridge. However, it should be added that the target DFF and the distance between transverse frames in this suspension bridge is not known to us.

Reference is made to drawing no SBJ-33-C5-AMC-22-DR-434. Except for the two longitudinal bulkheads the deck plate with stiffeners in K12 is similar to that in the heavy truck lane in the Taizhou suspension bridge. As the calculated fatigue life due to local loads is sensitive to the transverse stresses (including bending stress) in the deck plate it may be questioned if the longitudinal girders will attract more stress from wheel loads in the midspan between the transverse girders than at the longitudinal stiffeners as the girders may be significantly stiffer than the longitudinal stiffeners. It is difficult to assess this properly without performing a detailed finite element analysis. This needs to be assessed in the next phase of the project.

Also, the bridge above the longitudinal frame is stiffer with respect to vertical forces than the other part of the orthotropic deck structure and the longitudinal girder will attract larger vertical stresses locally due to traffic load. These vertical stresses in the longitudinal wall are compressive; however, there will also be some bending stress above the vertical stiffeners due to eccentricity that needs to be accounted for in further design analyses.

Reinforced concrete of thickness 60 mm has been used to rectify bridge girder decks that have cracked in China. This requires use of studs for documentation of shear interaction between the steel plate and the concrete. Furthermore, it has been proposed to improve the fatigue strength of the deck plate by using steel fibre reinforced concrete below a thin asphalt layer for wear.

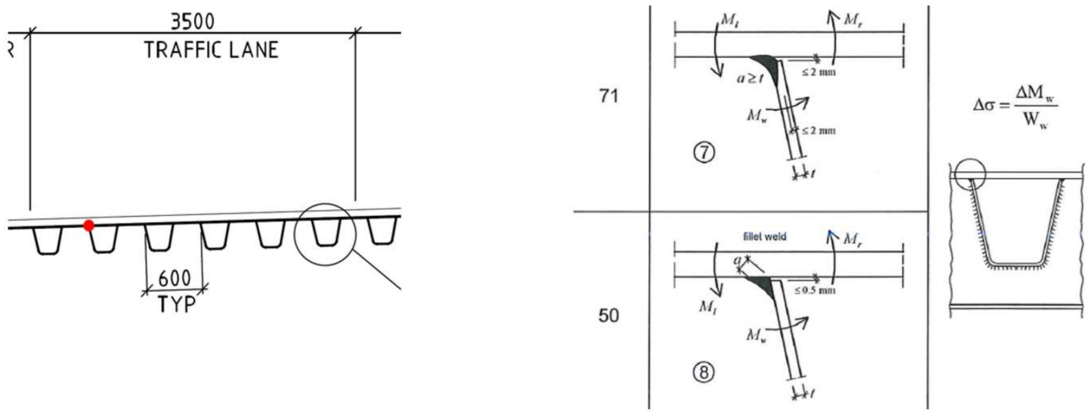

Figure 4-9 Location of hot spot used for assessment of detail type 4. This is at the westerly wheel position in the westerly slow lane (From SBJ-32-C5-AMC-22-RE-109)

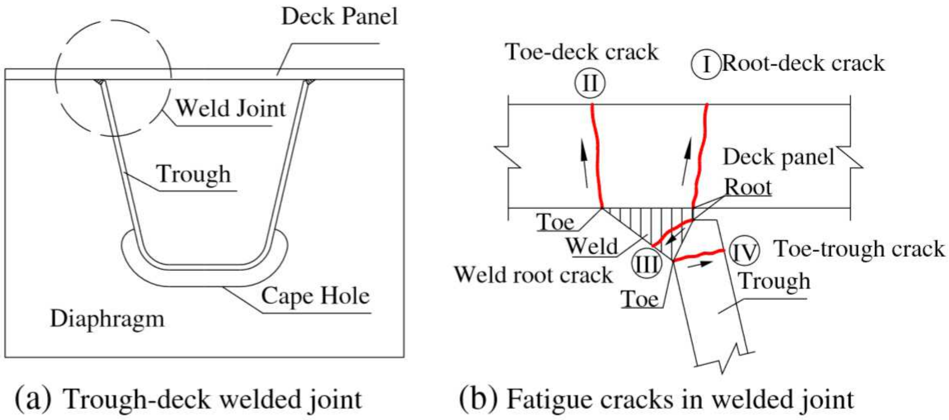

Figure 4-10 Fatigue cracks of trough-deck welded joint

Conclusions

Based on the presented design documentation supported by experience from fatigue cracking of orthotropic steel bridges in other countries it is observed that it is a challenge to document the target design life for this bridge based on the existing design premises and traffic model.

Sensitivity studies have been presented where it is demonstrated how the target fatigue life can be achieved by changing the requirements to the traffic model. It is assessed that an alternative to changing the traffic model significantly can be to strengthen the orthotropic deck locally below the heavy traffic lanes. In addition, to document that fatigue cracking of the deck plate along the longitudinal stiffeners will not occur due to traffic loads one may need to include the stiffness properties of the asphalt layer in the transverse direction of the deck plate. The stiffness of the asphalt layer is depending on the temperature; therefore, relevant stiffness properties need to be assessed in the next design phase. It should also be assessed if other materials such as a layer of epoxy is recommended to achieve the necessary interaction with the deck plate without need for studs.

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 bound values or more precisely determined as mean plus two standard deviations. Reference is made to EN 1993-1-9 (NS-EN 1993-1-9:2005, Eurocode 3: Design of steel structures - Part 1-9: Fatigue). 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.) (BS 7608:2014 Guide to fatigue design and assessment of steel products.) it is said that "The design load spectrum should be selected on the basis that it 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), (Guo, Liu, & Zhu). 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 at these connections 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 cut-outs around all trapezoidal sections that are welded to the transverse frames will be required. Lugs are proposed welded for side way stabilisation at other connections than below the deck plate. It can be noted that other stiffeners such as HP sections can be rather easily connected by welding to the transverse frames. Thus, use of HP stiffeners in other areas than below the deck plate can be recommended to be investigated in the next phase of the project.

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.

<u>Other items to be investigated.</u>

As the concept is being further developed also the connections at the structural parts used to connect and install the bridge will need to be analysed with respect to fatigue.

So far, a fatigue assessment of the structure at the supports between the bridge girder and the pylon has not been performed. Some friction forces will be present, and some investigation is recommended to investigate what friction coefficient can be acceptable.

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 Doc. No. SBJ-33-C5-AMC-28-RE-114, *Preferred solution, K12 – Appendix N Construction and marine operations*, Rev. 0, dated 15.08.2019. Descriptions of the various operations are given, and some details are included. Naturally, at this phase of the project, all details are not yet included.

Various types of marine operations will be performed during this project, but most operations could be considered as standard operations. For example, sea transport of 25 m long bridge elements from fabrication site to the assembly site. No major challenges are foreseen for such operations, but they will still require proper planning. Hence, the missing detailing are not seen as a feasibility issue, but it imposes uncertainties in schedule and cost for these operations.

The most challenging operation is considered to be the towing and installation of the complete floating bridge. Assembly of the 150 m long high floating bridge elements is also considered challenging with respect to weather windows, structural stability and strength, etc.

Some operations may be critical to be planned as weather restricted operations, see Sec. 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, 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 (DNVGL-ST-N001, 2018) 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 limits given in (DNVGL-ST-N001, 2018).
- 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 DNV-GL-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.

The use of alpha factors is discussed in Doc. No. SBJ-33-C5-AMC-28-RE-114, Appendix N, but the planning needs to be further refined in later phases.

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. The risk management plan must include risk assessment of the marine operations as well as the various construction phases. 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 calculations included in Doc. No. SBJ-33-C5-AMC-28-RE-114 have not been verified in detail at this stage. The focus has been on the proposed installation methods, and some comments are given below.

4.9.1.6 Reinforcements and temporary steel

Local reinforcements will be required to transfer concentrated loads in temporary phases, for example at lift points and to lock structures while welding (columns, floating bridge girders, etc.). 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.

4.9.1.7 Construction tolerances

The tolerances during fabrication of each 25 m bridge girder segment and the cumulative tolerances during assembly of the bridge girders need to be estimated and accounted for in order to install the complete bridge girder within reasonable tolerances. The required clearance at construction joint 2 and 3 during installation will depend, among other factors, on the fabrication and assembly tolerances, assumed to be considered in later phases.

4.9.2 Evaluation of the various (marine) operations

4.9.2.1 Cable stayed bridge

The erection of the cable stayed bridge is assumed to follow well proven methods for bridge construction, and no critical technical challenges are foreseen.

The two first bridge girder sections closest to the tower are installed by floating crane. This will be a weather restricted operation and needs proper planning. It should be noted that a pre-laid anchor system probably is required for the floating crane due to the water depth.

4.9.2.2 Assembly of low floating bridge elements

There will be many mooring lines to keep the bridge in position during assembly, and the lines must be reconnected when the bridge is shifted toward north during the assembly work. All details are not included yet. Key elements will be on the control of the mooring line lengths and the force distribution between the mooring lines. This could be controlled by using winches with load monitoring. Monitoring of deformations in the floating bridge section should be considered for control during the assembly period. This assembly method is considered to be feasible when properly planned.

Accidental conditions with typically failure of (at least) one mooring line should be included in the design. Alternatively, tugs on standby may be an alternative. It was confirmed in reply to previous comments (reply from AMC 2019-06-04) that it will be planned to have a standby tug connected to the pontoon until enough moorings are hooked up to survive a seasonal storm condition given a line failure.

At the assembly site, swell, ship traffic, current etc. that could lead to downtime during the assembly period need to be monitored.

During assembly, the bridge girder will be skidded along the assembly platform. It is assumed that some form of load equalizing system is needed to have a distributed load along the bridge girder, hence avoiding large concentrated forces during skidding.

Further, unintended sliding of the bridge girder in an accidental condition during assembly or during skidding, refer to Figure 4-11, must be prevented.

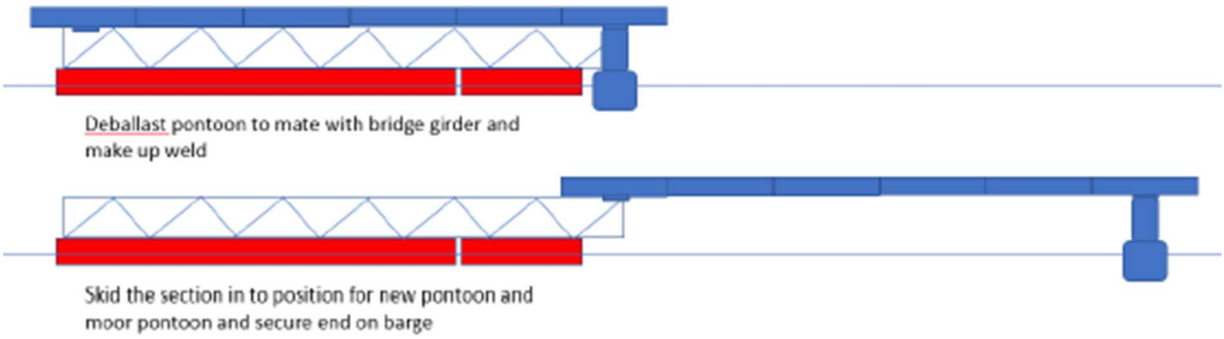

Figure 4-11 Assembly of low floating bridge (from Figure 7-5 in Doc. No. SBJ-33-C5-AMC-28-RE-114, Appendix N, dated 15/8-19)

4.9.2.3 Assembly of high floating bridge elements

A study of the assembly of the high floating bridge is given in Enclosure 2 in Doc. No. SBJ-33-C5-AMC-28-RE-114, *Preferred solution, K12 – Appendix N, Construction and marine operations*, Rev. 0, dated 15.08.2019. This is an early phase study and describes a method for elevating the bridge girder and install the columns and pontoons. A few comments are given below:

- Stability of the jacking towers in transverse direction is mentioned, and transverse support structures are indicated, see Figure 4-12. The longitudinal direction must also be considered in later phases, when relevant load cases have been defined.
- The 150m long bridge girder is supported by four jacking towers. A limited number of supports are indicated; hence a relatively large concentrated force is transferred into the bridge girders, and local reinforcements are required.

- The bridge girder must be kept in position on the jacking beams, i.e. prevented from sliding in case of severe heel or trim of the assembly platform. The supports are indicated to be low friction (teflon and stainless steel). The position keeping is assumed to be further detailed later.
- Stability of the assembly platform in accidental condition and the capacity of the stability masts needs to be checked in later phases.

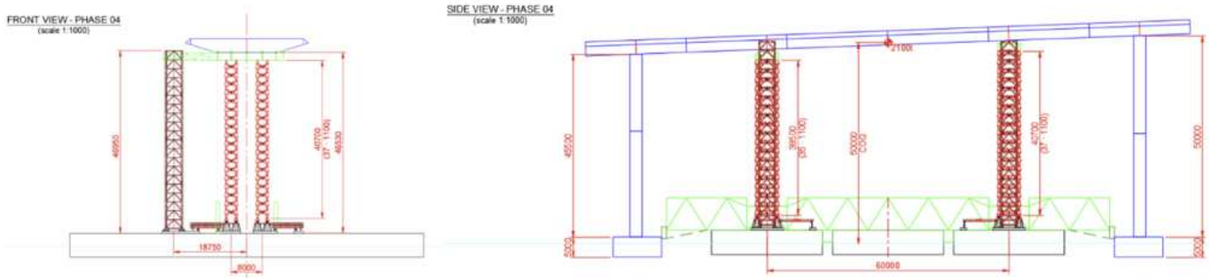

Figure 4-12 Jacking of high bridge elements, (from Enclosure 2 in Doc. No. SBJ-33-C5-AMC-28-RE-114, Appendix N, dated 15/8-19)

The high floating bridge elements are planned to be assembled in the same area as the low floating bridge, see Figure 4-13.

The assembly platform for the high floating bridge supports the jacking towers. Any relative motion between the three barges in the assembly platform would lead to large relative motions between the bridge girder and the jacking towers, refer to Figure 4-12. To avoid this, there must be a rigid connection between the three barges in the assembly platform. In Doc. No. SBJ-33-C5-AMC-28-RE-114, Appendix N, Figure 7-4, the three barges are indicated to be kept in relative position by Yokohama fenders. In Appendix N, Enclosure 2, the barges are said to be connected by hard fenders, and a rigid connection is assumed in the stability check. This is assumed to be more detailed in later phases, but it is noted that a significant amount of steelwork is needed to form a rigid platform from the tree barges.

The plan seems to be to assemble the high floating bridge by welding the 150 m long element to the already assembled part while it is still supported on the assembly platform. When the high floating bridge (including the 150 m long element) is to be shifted towards north, the assembly platform, or the bridge, must be shifted in east-west direction for the pontoons to go clear of the assembly barge fleet (because the assembly platform is between two pontoons). Considering that the barge fleet is 91m wide and the pontoon is 53m long, the strait where the assembly platform is located could seem narrow (Figure 4-13).

Figure 4-13 Areas for assembly of high floating bridge (left side) and low floating bridge (from Figure 7-3 in Doc. No. SBJ-33-C5-AMC-28-RE-114, Appendix N)

The water depth in the proposed area could also be a challenge for the proposed assembly site, see Figure 4-14.

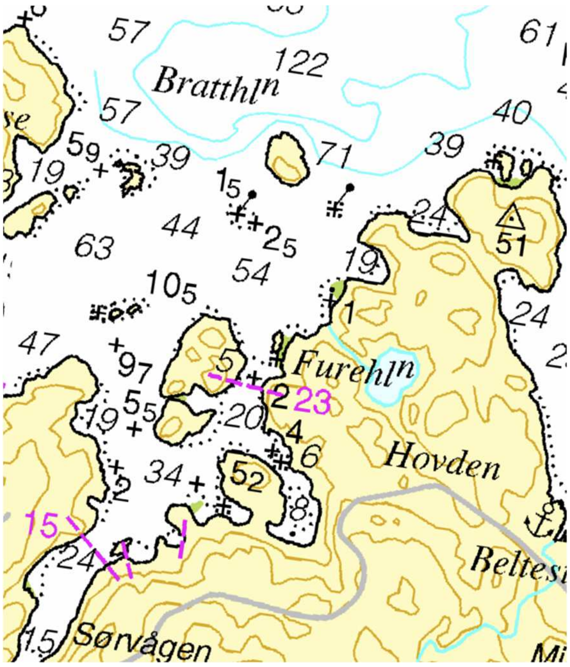

Figure 4-14 Areas for assembly of high floating bridge (chart from www.norgeskart.no)

4.9.2.4 Installation of north floating bridge segments

This installation is proposed done in two operations; lift installation by crane vessel of a 40m long (950t) section and floating in a 292.5m long section. The former operation is considered 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 and the temporary columns
- Method for releasing the temporary columns
- Ballasting of pontoons if required.

If the connection between the 40m long section and the 292.5m long transition sections show high stresses due to tide variation, an alternative is mentioned to be an additional girder segment installed in the similar way on fixed supports and welded to the transition section (Doc. No. SBJ-33-C5-AMC-28-RE-114, Appendix N, Sec. 6.1). This gives one additional lifting operation during this part of the installation, which will affect cost.

4.9.2.5 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 an acceptable weather window 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.

Proper control of the tug direction and pulling force is required for manoeuvring of the bridge section. It is assumed that the operational aspects will be evaluated by experienced personnel (including tow-masters and tug captains). The behaviour should be carefully analysed to ensure that the towing fleet has adequate capacity to keep the bridge section in a controlled position considering the sea room available. The size of the towing fleet should contain extra capacity in the available towing force, see (DNVGL-ST-N001, 2018) Sec. 11.12.1.7, hence allowing for tug or tow line failure and still have enough capacity to maintain the position and the required heading with the remaining towing fleet.

Computer simulation of the towing operation is mentioned in Doc. No. SBJ-33-C5-AMC-28-RE-114, Appendix N, Sec. 7.1. Model tests could also be a useful tool in a later phase.

4.9.2.6 Installation of the main bridge section

The bridge is maneuvered into position by tugs. The mechanical connection of the bridge to the north abutment and the cable stayed bridge is planned to be performed in three steps, called guiding, positioning and locking.

The guiding and positioning systems are planned to be designed for a weather restricted operation. This means that the bridge must be in position and the construction joints locked off within a weather window. Hence time will be a critical parameter. A schedule is included (Appendix N, Table 9.1), indicating that the operation can be performed within one weather window; this is assumed to be further detailed later. For example, installation of the locking system is estimated to take 24 hours. The locking system contains 170 bolts in each of construction joint 2 and 3. In addition to the bolts and nuts, shim packs (indicated with UNP120x60x15 in Figure 4-15) will be fitted. The preloading of the bolts will be an iterative process to achieve the correct load level, and this will take time. On the other hand, if enough personnel and tools are available, then many teams can work in parallel. More details are assumed to be included later to give confidence in the time planning.

The locking system must be able to resist the wave, wind and current induced forces resulting from a seasonal storm. Other relevant load effects should also be considered, e.g.

- forces due to pull in of the bridge during installation, where the construction tolerances require the construction joints to be forced together (e.g. the pull in force in construction joint 3 introduces shear force and bending moment in construction joint 2, etc.)
- forces due to changes in the ambient temperature
- the effect of tidal variations.

It is noted that the plan is to ballast the pontoons to reduce or eliminate the effect of tidal variations (Doc. No. SBJ-33-C5-AMC-28-RE-114, Sec. 10.5.1). However, tidal variations could be difficult to eliminate completely, and some tidal variation should be accounted for. Depending on the redundancy in the pump system, the accuracy of the monitoring etc., it may be considered to include tidal effects as an accidental limit state condition. Such considerations should be a part of the risk assessments in later phases.

For a weather restricted operation, the planned duration and the contingency time must be thoroughly evaluated during the planning. For example, the locking system in the construction joints contain some 170 bolts, and the required time for installation and preloading must be included in the planning. This is particularly important for the installation of the main bridge.

The locking design will be further developed by the designer. A few comments to the current preliminary design are given below.

4.9.2.7 Mooring system

The mooring system pre-installation and hook-up are considered as standard marine operations. Hence, no feasibility issues are foreseen.

Several methods are indicated for tensioning of mooring lines. In addition to the ultimate capacity, the fatigue capacity and the possibility for replacement of components should be evaluated. See also Section 4.8.2.

4.9.2.8 Construction joints

The bridge sections connected in construction joint 1 and 2 are planned to have a perfect match. This is assumed to be possible when the cross sections have been trial assembled during the fabrication (Appendix N, Sec. 9.4.4). In construction joint 3, there is a nominal gap of 800 mm. However, the applied shim packs (875 mm long UNP120x60x15 profile) shown in Figure 4-15 seem to have a fixed length, and the construction joint might not have the flexibility that was intended.

When construction joint 2 is in position, there may still be a gap in construction joint 3 due to construction tolerances and temperature effects. This gap needs to be closed by the positioning system before the locking system is established. The positioning system in joint 2 and 3 need to resist this force.

It could be considered to have a nominal gap also at construction joint 2, to have more flexibility and be able to even out the tolerances between construction joints 2 and 3.

A few comments to the locking system shown in Figure 4-15:

- In the construction joints at the deck plate, infill pieces of the trapezoidal stiffeners (U-ribs) are indicated. It should be checked further how these infills can be fitted, as there seems to be a conflict with the bolts and the flat stiffeners 450x12.
- The eccentricity of the lower bolt from the deck plate needs to be accounted for. As a result of that, the indicated 750 mm long 450x12 stiffeners will probably have to be much longer.
- The RHS 120 x 10 profile seems weak for the actual bolt force

Anyway, such issues can be dealt with during the further design development in later phases.

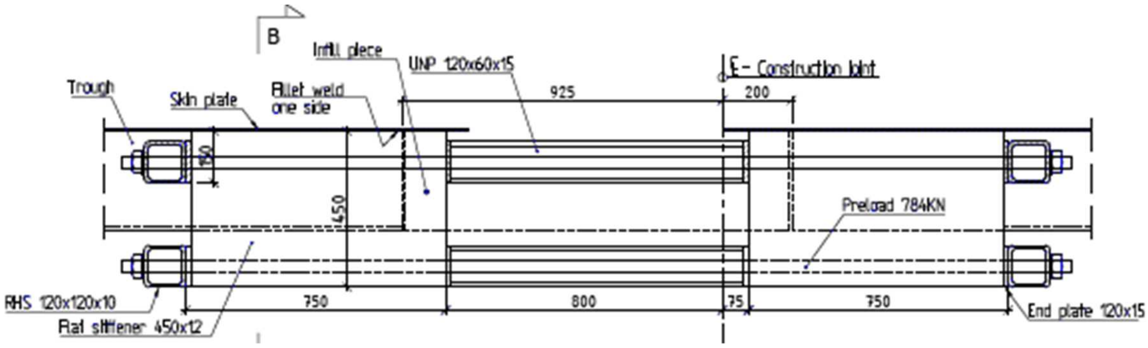

Figure 4-15 Detail 6 from drawing SBJ-33-C5-AMC-22-DR-824, Rev. 0

4.9.2.9 Local structural reinforcements

During the fabrication and marine operations, handling of the object will require local forces to be resisted, for examples the:

- Guides for positioning during installation
- Lift points where large forces are transferred into the structure
- Support points for lifting (jacking) during assembly of the high floating bridge
- Support points for skidding during assembly of the low floating bridge
- Construction joints for installation of the complete bridge.

When the local reinforcement be designed, any side effects of the local reinforcement for the inplace condition need to be considered, see also Sec. 4.8.7.3.

4.9.3 Fabrication details

4.9.3.1 Welding

The welding at the cut-outs between the longitudinal trapezoidal sections and the transverse frames in the bridge girder may be considered more cumbersome than if the welding could be made continuous around the section. Due to the large number of such connections it is recommended that an effective fabrication of these are assessed in the next phase of the project when optimal connections have been decided.

4.10 Material selection and corrosion protection

4.10.1 General

Material selection and corrosion protection are commented based upon review of Document No. SBJ-32-C5-AMC-04-RE-115, Concept evaluations – Appendix O, Material technology and steel in marine environment, Rev. 0. The following review comments to the document are taking into account that this is a concept phase, and that the document lists a number of areas which should be further evaluated (ref. section 10.)

4.10.2 Selection of 25Cr Duplex for splash zone

Reference is made to section 3.2.2, 6.1. and other sections. 25Cr Duplex has been selected for the splash zone area. Although the evaluation and testing presented in the document conclude that this material selection is feasible, it should be commented that also this material requires careful considerations during design and fabrication. For example, marine growth may impose significant crevice effects on the 25Cr duplex, which may contribute to initiation of crevice corrosion. The transition from uncoated to coated area is (as discussed in the document) also a potential region for initiation of crevice corrosion. Sunlight (i.e. temperatures around and above 20 °C) and accumulation of chlorides on the 25Cr Duplex surface above the waterline may contribute to initiation of crevice and potentially pitting corrosion. It should be noted that within a material specification, the susceptibility towards initiation of corrosion may vary significantly as a function of e.g. chemistry and heat treatment process. Hence, a strict specification as well as corrosion testing as part of qualification of the manufacturing process is considered important. It should be further evaluated to apply coating in the splash zone both in order to prevent corrosion and in order to reduce the anode consumption (ref. section 7.3).

4.10.3 Evaluate use of Thermal Sprayed Aluminium (TSA) for structure submerged in mud

Reference is made to section 7.1.1. It should be evaluated if TSA should be applied for additional protection in the part of the structure submerged in mud.

4.10.4 Qualification of coatings for ballast tanks

Reference is made to section 7.2. It should be evaluated if coatings for ballast tanks should be qualified as ballast tank coatings in order to document acceptable performance, ref. e.g. adapted ballast tank coating testing according to IMO MSC 215(82): 2006 Performance Standard for Protective Coatings for Dedicated Sea Water Ballast Tanks in all Types of Ships and Double-Side Skin Spaces of Bulk Carriers (PSPC).

4.10.5 Cathodic protection (CP) in closed compartments

Reference is made to section 7.2. Regarding CP in tank / closed compartment: It should be noted that the cathodic reaction leads to hydrogen gas formation. This needs to be considered in order to prevent formation of an explosive atmosphere internally in closed compartments.

4.10.6 Account for current drain to anchoring chain in CP design

Reference is made to section 7.3. CP calculations in the document does not take current drain to the anchoring chain into account. It should be noted that current drain to the anchor chain the first 60-100 meters (or to the transition to fibre rope section) may be expected and should be considered when updating the CP design / anode calculation in the next phase.

5 COMMENTS TO CONCEPT RANKING

5.1 General

The designer presents the results from the ranking process during the design development in report SBJ-33-C5-AMC-23-RE-118 rev. 0 dated 15.08.2019. 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 are 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. The K11 bridge concept 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 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 includes 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 loads of the entire bridge. The bridge girder 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 Appendix N, Rev. 0, dated 24.05.2019. It does not have mooring lines. Otherwise, the K11 concept is installed similarly to concept K12.

While for concept K12, four mooring lines will be installed during the weather window to be able to resist a seasonal storm, that is not possible for K11. Hence, the locking mechanism at construction joints 2 and 3 must be designed to resist the environmental loads from a seasonal storm.

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. In order to cope with the thermal expansions that will induce large stresses in the bridge girder or need of a dilation joint in the north end of the bridge. For the case without dilation joint it is proposed to avoid buckling of the bridge by prestressing of the bridge with a considerable tension force. This force will load the bridge from abutment to abutment and will add to the stresses from other loads. It is also unfavourable to have large tension forces in the structure if one experience cracks due to fatigue.

The alternative with a dilation joint will require a complex structure that enables the large deformations to take place 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. In case of a prestressed alternative it may be need for shorter inspection intervals due to the imposed tension stress.

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

Alternative K13 is the straight bridge concept and is described in Appendix N, Rev. 0, dated 24.05.2019. The bridge girder is fabricated same as K12. The cable stayed bridge is the same as for K11. A small part of the bridge is installed at the north abutment, this is like K12.

The main part of the bridge is towed and installed in three sections. One challenge is said to be the installation of the final section, because the bridge cannot be curved to reduce the effective length during installation. A sliding support in one end is indicated. The bridge is planned to be pretensioned to allow for temperature expansion of the bridge.

To introduce the correct pretension load in the bridge girder (to pull and lock off the) is considered difficult, because the required force needed to account for the temperature effects would be very large.

5.4 Concept K14

5.4.1 Structural design

The structural arrangement for concept K14 is a S-shaped bridge with 4 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 this alternative as for the others.

5.4.2 Need for maintenance

The need for maintenance for this bridge is larger than K12 due to fewer anchor lines, but less than 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

Alternative K14 is the S-shaped bridge concept, described in Appendix N, Rev. 0, dated 24.05.2019. The bridge is fabricated and installed using the same method as for concept K13. The only difference from the K13 installation is that K14 will not be pretensioned in the longitudinal direction, because deformations due to temperature variation can be compensated by bridge deflection.

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 report 33-C5-AMC-23-RE-118 rev. 0 dated 15.08.2019. The selection is based on a risk evaluation of the four concept and it is judged by DNV GL that the evaluations of the risk for the various concepts are fair and we agree with the final ranking presented and with K12 as the preferred concept.

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APPENDIX A
Verification comments issued during concept phase 5

Reference is made to meeting with AMC May 8th, 2019 and subsequent written replies to DNV GL comments from AMC.

Subject	Comments/Replies
Introduction by DNV GL	▪ DNV GL has a Frame Agreement with SVV in relation to verification efforts etc for the E39 development ▪ For BJF 2019, verification efforts cover both regular document reviews and independent analyses ▪ DNV GL technical reports will be issued both for document reviews (K11 – K14) as well as independent analyses of the selected concept – Independent modelling/analyses of K12 concept are ongoing and will be discussed later when key analysis results are available ▪ The meeting/discussion today will focus on preliminary DNV GL findings and discussion points in relation to AMC 'Status Report No 2'. – Per agreement with SVV, review focus has been on current selected concept K12
Concept ranking	**DNV GL**: K12 has less dynamic loading than K11 hence reduced fatigue load (which might give possibility to reduce dimensions or with less inspection and risk for repair during service life). **AMC**: We will try to assess all significant cost differences to the degree possible. In particular we will focus on the quantity of steel being the main cost of the structure. Fatigue design will hence be a focused issue. The cost of inspection will be a limited cost compared to the cost of building the bridge. Inspection will have little or no impact on the traffic. Hence, differences related to inspection will have limited consequence regarding ranking. **DNV GL**: The risk of losing the entire bridge by a high energy ship impact is less for K12 than for K11. **AMC**: We are aware of the differences regarding redundancy for ship impact and has included this in operational risk #7, #9 and #34, but general focus is on potential challenges regarding fulfilment of the demands in the design basis, and not on extra redundancy. **DNV GL**: In the above report it is stated: "K13 is ranked as no. 1 regarding cost, and the differences regarding risk compared to K12 is limited. The benefits regarding cost and risk however does not outweigh the challenges related to visual impact. K13 is thus ranked as no. 3. "In report 10205546-13-RAP-166 Appendix K: Design of Floating Bridge Partin; Section 3.6 it is stated that the dimensions for K13 need to be increased. Is this in line with the statement in the Concept ranking report? **AMC**: This will be handled for next milestone, but increased quantity of steel for K13 will make the ranking of K13 compared to K12 and K14 clearer.

	DNV GL: Refer to 10205546-02-RAP-172, Rev. 0, Appendix Q, Concept Ranking: It is stated (Sec. 3.3) that "Being superior in terms of construction risks, the overall preferred option is K12". However, this is conditional on K12 being able to resist a seasonal storm without the mooring lines installed (App. N Sec. 9.5). If not, K12 may be installed in three parts, similar to K13 and K14, in which case the superiority may be reduced. **AMC**: We are aware of this and has already included this in the risk register. The option of changing installation concept for K12 can be seen as a way of mitigating critical risks (being aware that other risks may then increase) and is thus adding concept robustness to K12. This will be made clear in the final risk analyses and ranking.
Hydrodynamic interaction	**DNV GL**: Hydrodynamic interaction in the Summary of Appendix H – Global Analyses – Special studies the following is stated: "The hydrodynamic interaction between pontoons has an influence on the response, the effect is found to be largest for the vertical motion and the weak axis moment." Have analyses been performed with a damping lid? This could possibly reduce interaction effects. **AMC**: The wave amplification between the hulls are within reasonable levels, since the wave radiation is still significant. Further, the values on a damping lid will have to be experimentally determined. Therefore, at this stage it is found not feasible to introduce numerical damping lids.
Transverse trusses	Reference to 10205546-13-RAP-166 Design of Floating Bridge Part Appendix K; Section 3 of NOT-083: **DNV GL**: The effective width of the contribution from the girder outer skin when participating in the transverse frame seems to be made by the wrong length between zero moments and without consideration of buckling of the deck plate. **AMC**: The effective flange width will be corrected. **DNV GL:** The load effects from distributing the traffic loads need to be combined with effects from global loads. **AMC:** The effects from global loads in combination with effects from local traffic loads will be discussed. **DNV GL:** The axle load for the notational lane no. 3 seems to be missing in the load plan. **AMC:** The missing axle load will be included.
Structural details	**DNV GL**: 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. **AMC:** Supplementary drawings will be produced.
Fatigue analyses	Ref. 10205546-11-RAP-164 Concept evaluation. Appendix I Fatigue analyses.

	DNV GL: Section 2.3.1 - The SRSS method implies that stress due to wave and swell is independent of the stress due to the wind. Should there not be a correlation that needs to be considered to avoid a non-conservative combination? **AMC:** The current method will be verified/benchmarked against coupled time-domain analyses by the end of the current project phase, for the chosen bridge concept. **DNV GL:** Section 2.4. It is understood that fatigue damage calculation due to traffic is ongoing. 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. **AMC:** Fatigue due to local wheel loads will be reported in the milestone 7 delivery. **DNV GL:** It is noted that different types of longitudinal stiffeners are being used in the bridge deck. It needs to be controlled that the difference in stiffness does not lead to significant differences in local stress distributions due to wheel loads. **AMC:** This was clarified in the meeting. **DNV GL:** It is not clear if stiffness reductions in the bridge girder due to shear lag have been accounted for in the global analysis. Effective cross sections should be confirmed applied in the analysis. **AMC:** This is included. Will be stated in Appendix I. Ref. 10205546-11-RAP-164 Concept evaluation. Appendix I Fatigue analyses. **DNV GL:** Section 3. It should be mentioned that 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 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 δ0 = 0.1t to 1.53 for δ0 = 0.05t. **AMC:** Ok. Will be taken into consideration Ref. 10205546-11-RAP-164 Concept evaluation. Appendix I Fatigue analyses **DNV GL**: For detail no 3 with cut-out at trapezoidal stiffeners all the stress ranges due to axial global and local forces in the stiffeners need to be included. S-N curve F may be used. In addition, a stress concentration factor that depends on geometry needs to be included. It is expected that this will be a less critical detail with respect to fatigue than that of the transverse butt welds in the bridge girder.

AMC: This detail will not be prioritized for the milestone 7 delivery as it is considered to be less severe than the transverse buttweld.

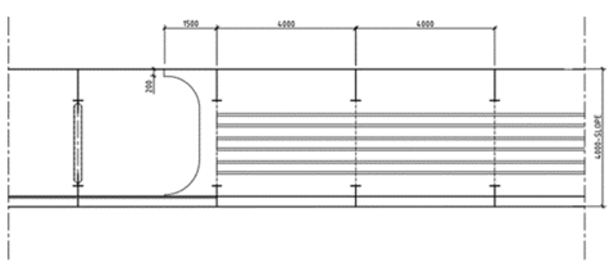
DNV GL: The trapezoidal stiffeners will also be welded against a backing plate where sections are put together. This will also require S-N curve F.
AMC: This detail will be checked for the milestone 7 delivery

DNV GL: In third paragraph in 4.1 it is said that “swell only shall be included in some of the total environmental cases”. It is probably meant “waves” and not “swell” here?
AMC: Will be rephrased.

Ref. 10205546-13-RAP-166 Appendix K: Design of Floating Bridge Part; Section 3.4.
DNV GL: It is noted that a fatigue life longer than 100 years has been calculated for all concepts. This is for a plate splice from 12 mm to 14 mm thickness. This is without effect of traffic loads. Is there sufficient margin to include the traffic loads?
AMC: The effects of traffic load will be included in the fatigue calculations.

Ref. 10205546-13-RAP-166 Appendix K: Design of Floating Bridge Part. Appendix K – Enclosure 4 10205546-13-NOT-084 Shear lag and buckling effects of Bridge girder.
DNV GL: There are longitudinal panels at columns, but not at midspans. Detail of the ends of the panels to avoid stress concentrations?
AMC: The detail will be designed to avoid large stress concentrations.

DNV GL: A longitudinal bulkhead is shown to be ending on the plate. To reduce local bending stresses, one should consider to end this at a transverse frame.
AMC: The longitudinal bulkhead will be transitioned into a longitudinal truss before ending towards a transverse frame.



SECTION I-I - 13-TEG-136/137
TYPICAL LONGITUDINAL BULKHEAD
A1=1:50
A3=1:100

Ref. 10205546-16-RAP-169 Appendix N: Fabrication and Marine Operations.
DNV GL: From Figure 3.13 it is noted that there is no cut-out around the two 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, also cut-outs at other trapezoidal sections may be needed. This may also be related to the sections at the bottom plates in the bridge girder. **AMC:** There will be cut-outs for all longitudinal stiffeners. Ref. 10205546-16-RAP-169 Appendix N: Fabrication and Marine Operations. **DNV GL:** From Figure 3.13 it is noted that there is no cut-out around the two 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, also cut-outs at other trapezoidal sections may be needed. This may also be related to the sections at the bottom plates in the bridge girder. **AMC:** There will be cut-outs for all longitudinal stiffeners. Ref. 10205546-13-RAP-166 Appendix K: Design of Floating Bridge Part. **DNV GL:** Reference is made to section 3.6 discussion of results. It is said that "K13 will be pretensioned for approximately 60 MN which is not included in the analysis. Thus, the final steel amount should be increased for K13 compare to K12 and K14". **AMC:** For AMC Status 3, the steel amount for girder in K13 is increased by 1.5 t/m in the final amount to account for the pretension and due to general higher steel stresses than the other concepts. In status 2 this increase was 1.0 t/m. **DNV GL**: In AMC Status 2 – concept evaluation section 4.8 it is said that the girder weight is increased by 1.0 ton/m. This means that the area of the girder is increased by approximately 0.127 m2 and that the stress range is reduced by approximately 8 %; however, this depends on where the steel area is increased in the bridge girder, and the stress reduction at critical sections might be larger than this number. It is indicated that an increased mean stress will increase the fatigue damage at the hot spots and have an adverse effect regarding crack development during fatigue loading. In Appendix R-Risk Assessment Section 3.2.3 Fatigue it is indicated that the increased stress level can be evaluated through a crack propagation approach. **AMC:** This topic was discussed in the meeting and it was agreed that the tensile stress of this magnitude normally is included in the SN-curves. Evaluation using a crack propagation approach will therefore not be done for K13. Ref. 10205546-13-RAP-166 Appendix K: Design of Floating Bridge Part. **DNV GL:** It is agreed that fracture mechanics could be used to assess the robustness of the concepts with respect to acceptable crack sizes. However, it is difficult to perform reliable fatigue analysis based on fracture mechanics of a

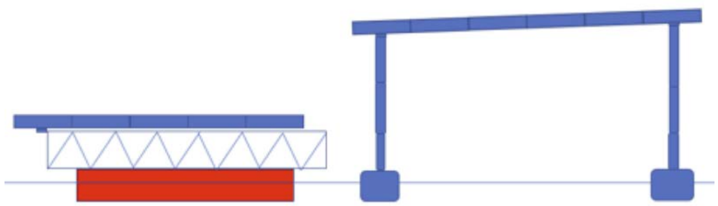
	bridge girder due to residual stresses after fabrication and uncertainties related to the input parameters for such analysis. **AMC:** Evaluation using a crack propagation approach will not be done for K13. **DNV GL:** The referred design S-N curves are considered applicable also for a bridge girder in tension as the fatigue testing as basis for the curves have been derived for a full tensile stress range. Due to the presence of residual stresses at the hot spots it is assessed that an increased mean stress of approximately 40 MPa will not change the situation with respect to crack initiation significantly. A reduction in stress range may be of larger importance. **AMC:** We agree that the static tensile stress from pre-tensioning is included in the SN-curves. Ref. 10205546-13-RAP-166 Appendix K: Design of Floating Bridge Part. **DNV GL:** The failure criterion in the S-N curve is crack growth through the thickness. Thus, the fatigue life for fatigue crack growth through the thickness is not considered to be significantly more severe for K13 than for the other concepts. **AMC**: We agree, and therefore the risk regarding this topic is removed in Status 3. **DNV GL**: However, for further crack of larger cracks that have grown through the thickness, the crack driving stresses in K13 is considered to be larger than for the other concepts which might mean need for a shorter time until repair is performed for this concept than for the other concepts. **AMC:** This can be accounted for in a later stage. Ref. Appendix A - Drawings TEG-141 and TEG150. **DNV GL:** Casting are indicated in the transition between the bridge girder and the columns. Have solutions using brackets been considered as an alternative to the use of cast steel elements? **AMC:** Yes, other solutions has been evaluated and will not be ignored. However, as the design and geometry are today welded plates will create lot of difficult details which are nearly impossible to weld and inspect properly. By design a cast piece all welds will be moved outside the difficult spots and make welding and inspection easy. It is important to form the casting to avoid local SCF. **DNV GL:** A larger DFF may be recommended for casting than for welded connections; ref. DNVGL-RP-C203. **AMC:** This will be accounted for in the casting design. **DNV GL:** Has any additional cost of the castings been included in the cost estimate?

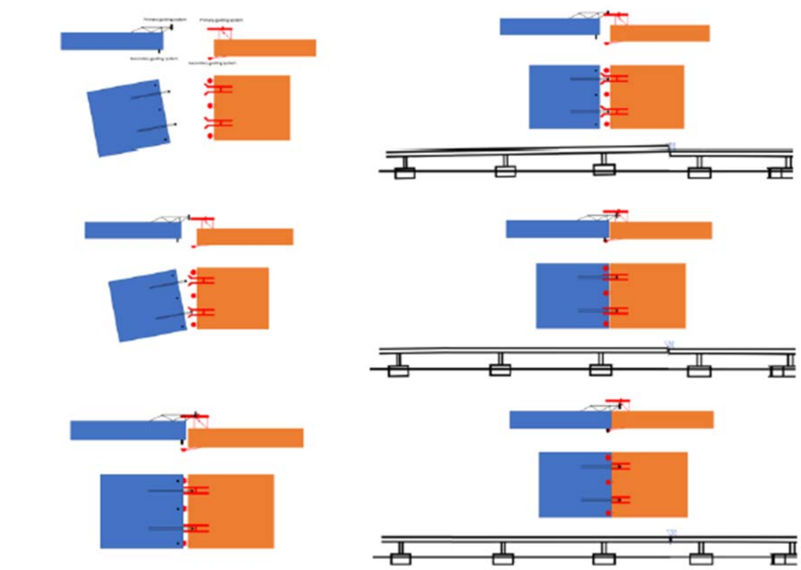
	AMC: Yes, extra cost will be evaluated, but by plan for casting in the beginning of the project the cost is marginal.
Design assumptions vs fabrication 	**DNV GL**: It is recommended to develop a fabrication specification to achieve the intended quality of fabrication that is in line with the design assumptions. **AMC:** We agree that this I required at a certain stage of the project **DNV GL:** The fatigue capacity is depending on the welding method and welding position. Selection of S-N curve D means welding between plates in the horizontal position. Thus, it is assumed that the plates between thickness transitions are welded in this position. **AMC**: Yes **DNV GL:** Will welding at site between sections be made against backing? **AMC:** Yes **DNV GL:** Welding of bulbs? **AMC:** Full penetration weld **DNV GL:** Based on experience with fabrication of other highly utilised structures with respect to fatigue, such as support structures for wind turbines, it is observed that the as-built tolerances are typical close to that allowable in the fabrication standards. **AMC:** Standard fabrication is assumed, supplementary specifications as required to be developed during engineering **DNV GL:** Based on experience it is also observed that the reliability of standard inspection methods for detection of internal defects such as manual UT is low. **AMC:** Noted, should be addressed in later phases **DNV GL:** In a project with long welds it is recommended to develop separate tools for detection of defects during fabrication (in shop and on site). **AMC:** Noted, should be addressed in later phases
Sideway support at tower	**DNV GL:** 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 a mechanical part that may encounter design challenges with large dynamic forces. **AMC:** The sideway supports at the columns are of the same magnitude for all alternatives, and within the range of state of the art bearings used for major long span suspension bridge (e.g.recent design of Canakkale Bridge (Cowi), Izmit Bridge (Cowi) and Chacao Bridge (Aas-Jakobsen)). These are also subjected to major dynamic action from e.g.wind. Large dynamic forces from wind are usual for these types of bearings. The actual forces in ALS, SLS and ULS are comparable to bearings already in operation.

	Ref. Appendix A - Drawings TEG-106 and TEG-107. **DNV GL:** The girder is made in concrete south of axis 1E. The transition details in axis 1E is not shown. Has it been considered, as an alternative, to extend the concrete girder to axis 2 to avoid the sliding supports at the pylon? **AMC**: Yes, as a possible optimization it has been considered to extend the concrete girder to cover the entire side span and pass through the tower and approximately 40m into the main span. This is however not our base case solution and will therefore only be described briefly. See also section 1.10 in app L. Avoiding the sliding support at the tower has also been considered, but here the side- and approach span length becomes too long (400m). Temperature variation makes the monolithic tower/deck solution unfeasible. A solution could be to slice up the tower legs to soften the tower in longitudinal direction. At present this possible solution has however not been detailed further.
Boat impact	**DNV GL:** Impact from submarine ref section 6.4.1.1.4 in the Design Basis has not been considered or evaluated. **AMC:** Local impact from submarine to pontoon will have considerably lower energy than a ship impact, and hence lower damage potential. Impact to a fairlead or mooring line may cause loss of mooring line, which is accounted for as an ALS case. Further assessment of submarine impact has not been considered necessary. **DNV GL:** It is claimed in the main report (page 86) that the torsional capacity of the columns for impact on the pontoons are currently too low and should be a point of focus in further design development. If so, the risk that this represent should be briefly evaluated, e.g. potential weight increase, increased cost. **AMC:** The capacity of the column has now been evaluated directly in a coupled global/local simulation using the appropriate limit-energy mechanism. The response of the column is evaluated with various stages of increasing torsional capacity so that an acceptable limit can be found. This will be documented in the next revision of the report. The current column suffers severe buckling and fracture and is not acceptable for the current energy level. However, with the updated risk analysis with lower energies due to a change in the navigational channel North-West of the bridge (expected fall 2019) the current column may be acceptable but with large deformations. **DNV GL:** Boat impact is only considered from east. It is expected that an evaluation or justification of this limitation is performed. **AMC:** Deckhouse impacts were considered for both directions in the report (see figure 5-35 – 5-37). For K13 and 14 the impact direction did not change the overall global response, whereas K12 have some minor differences locally close to the impact position. A larger difference was seen when the number of impact locations were increased, but this was mainly along the bridge girder and towards Northern abutment. The final calculations for the selected concept will

	include results from both directions also for pontoon impacts and with a more refined selection of impact locations. **DNV GL:** The evaluation of the different bridges is performed by comparing the magnitude of the different internal forces and moments. During an accident it is allowable and absorb energy through plastic deformation. As a consequence, it is recommended to compare the different designs by evaluating ductility and energy absorption. **AMC:** This has been considered, but as the results are now mainly within the ULS capacity of the various response types it has not been given high priority. Local plastic deformations will occur in the pontoon, column and towards the tower connection. Global plastic deformation may occur towards the North abutment.
Geological stability checks	**DNV GL**: 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. **AMC:** The geological stability at the tower location has been evaluated and verified by geologist
Wind spectrum parameters	**DNV GL:** Selection of the wind spectrum parameters is clarified in mail from SVV of April 1 with regard to Table 15: "For the Kaimal parameters "A", the calculated length scales were not used to fit the "A" parameters in the normalized wind spectra, whereas those based on N400 were used instead. Therefore, only the variation of "A" parameters should be evaluated for sensitivity study in generating the single point wind spectra." We understand that this imply that, for generation of wind time series, the A parameters in Table 15 should be applied together with the length scales from N400. In report 10205546-08-RAP-160 Appendix E – Aerodynamics the A-parameters are used with other length scales than given in N400. **AMC:** Our report was submitted in March 2019, before we received this note from SVV. After a discussion with them it has been decided to use N400 values and do sensitivity study on selected parameters.
CP during construction	**DNV GL:** The review has in this phase not had focus on corrosion protection, but we would like to raise the issue of corrosion protection during the long construction period before the permanent arrangement for the inside protection of the bridge girder come into service? **AMC:** System 3G, i.e. a single coat of zinc ethyl silicate, is proposed for the inside surfaces of the bridge girder. Once the bridge is completed a dehumidification system will ensure dry conditions inside the bridge girder and depending on the

	obtained level of humidity the corrosivity is expected to range from very mild (< 60 % RH) to insignificant (< 40% RH). Zinc ethyl silicate is an "active" coating (Zn rich) that provides localized cathodic protection of the steel in case of minor coating damages (scratches, etc.). Typically, this product is used as the primer in a coating system, e.g. NORSOK system 1. However, experience has shown that it also provides very good corrosion protection on its own (without additional coats). In the operational phase of the bridge there is no reason for concern since the corrosivity will be very low. Further, the rate of coating breakdown is expected to be low since the coating is not exposed to sunlight / UV and hardly any mechanical wear. During transport and construction, the coated surfaces will be exposed to corrosive marine atmosphere ranging typically from category C4 to CX (ISO 12944-2). Hence, it is during these phases the worst-case exposure conditions are expected. The severity will depend on the level of preservation, type of transport and actual exposure time, e.g. transport duration. The latter is however expected to be relatively short. Available test documentation to NORSOK M-501 for a typical zinc ethyl silicate primer, Carbozinc 11, shows that the product on its own passes the requirements and without visible signs of corrosion, e.g. cyclic testing involving exposure to UV, salt fog and low temperature for 4200 hours. Such tests are intentionally accelerated and are as such not directly comparable with the exposure during transport and construction. It shows, however, that a single coat of zinc ethyl silicate should provide adequate corrosion protection for all project phases. AMC will include this evaluation in a specific section in the applicable appendix of the concept report.
Marine operations	Refer to 10205546-16-RAP-169, Rev. 0, Appendix N: Fabrication and Marine Operations. **DNV GL:** No major issues are noticed in the planned operation; however, several important items are yet to be analyzed. A 'peculiar' behavior of the analyzed bridge element is reported in Sec. 10.1, but to our understanding, the indicated movement of the south when applying a tangential force seems reasonable. However, the reasoning behind this load case should be elaborated, e.g. the reason for part of the bridge (the approximately 1.5 km part shown in Fig. 10.1) being considered as fixed in the north end should be explained. **AMC:** The presented calculations are a first iteration to get an impression of how the bridge behaves. In the next revision of the report these calculations will be updated to coincide with the method descriptions, with correct boundary conditions and revised loadcases. **DNV GL:** Mooring of first segment during bridge girder installation It is stated that four anchor lines are needed to keep the bridge segment in place

	(Sec. 10.3). Please note that requirement to redundancy in the mooring arrangement, or tugs on standby may be applicable. **AMC:** It will be planned to have a standby tug connected to the pontoon until sufficient number of moorings are hooked up to survive a seasonal storm condition given a line failure.
Floating stability of bridge sections	**DNV GL:** Intact stability yet to be checked (App N, Sec. 6.2.4). **AMC:** Intact stability will be addressed in next revision **DNV GL:** The elements will be moored for some time; hence accidental water filling of pontoons may be a relevant scenario. Accidental conditions should be defined. **AMC:** Accidental condition will be defined as any one compartment filled, and damage stability calculations will be presented  High bridge under construction, from Fig. 6.12 in Appendix N.
Connection of main bridge girders	**DNV GL:** 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. Several options for keeping the bridge positions relative to each other during installation are described on an overall level. Some/one options need to be selected. **AMC:** One option will be presented. **DNV GL:** Calculate design forces during the welding period. **AMC:** Will be included **DNV GL:** An infill section is proposed (App N Sec. 9.3), but not confirmed. **AMC**: Confirmed that a infill section will be planned for **DNV GL:** Ballasting of pontoons during the welding period is proposed, this will require thorough planning and monitoring. **AMC:** Noted.

	 *Figure 9.1: Principals of the connection operation with guiding system* From Appendix N.
Connection of mooring lines	**DNV GL:** Several methods are indicated for tensioning of mooring lines. In addition to the ULS-capacity, FLS-capacity and the possibility for replacement of components should be evaluated. **AMC:** We will investigate with vendor. 
Minor comments, for info	Re: Towing of bridge sections. **DNV GL:** Loss of bridge positioning system is to be accounted for (App. N, Sec. 7.1). We assume that a backup positioning system will be planned for. **AMC:** A back up system will be planned for **DNV GL:** A tug efficiency factor of 0.9 is used (App. N, Sec. 7.2). However, the efficiency should be calculated based on tug size and significant wave height (DNVGL-ST-N001 Sec. 11.12.2.10). Even in calm sea, the factor should not be greater than 0.8. **AMC:** Will be accounted for in the calculations.

APPENDIX B
K12 - List of drawings and reports

Drawing number	Rev	Title	Issue date
SBJ-32-C5-AMC-90-RE-116	0	Concept evaluation - Appendix P - Cost estimates	15.08.2019
SBJ-33-C5-AMC-04-RE-115	0	Preferred solution, K12 - Appendix O - Material technology and steel in marine environment	15.08.2019
SBJ-33-C5-AMC-20-RE-105	0	Preferred solution, K12 - Appendix E - Aerodynamics	15.08.2019
SBJ-33-C5-AMC-21-RE-108	0	Preferred solution, K12 - Appendix H - Global analyses - Special studies	15.08.2019
SBJ-33-C5-AMC-22-RE-109	0	Preferred solution, K12 - Appendix I - Fatigue analyses	15.08.2019
SBJ-33-C5-AMC-22-RE-111	0	Preferred solution, K12 - Appendix K - Design of floating bridge part	15.08.2019
SBJ-33-C5-AMC-22-RE-112	0	Preferred solution, K12 - Appendix L - Design of cable stayed bridge and abutments	15.08.2019
SBJ-33-C5-AMC-23-RE-118	0	Preferred solution, K12 - Appendix R - Risk assessment	15.08.2019
SBJ-33-C5-AMC-26-RE-113	0	Preferred solution, K12 - Appendix M - Mooring system	15.08.2019
SBJ-33-C5-AMC-27-RE-110	0	Preferred solution, K12 - Appendix J - Ship collision	15.08.2019
SBJ-33-C5-AMC-28-RE-114	1	Concept evaluation - Appendix R -Risk assessment	15.08.2019
SBJ-33-C5-AMC-40-RE-090	0	Grunnlag for anslag, K12	30.06.2019
SBJ-33-C5-AMC-90-RE-100	0	Preferred solution, K12 - main report	15.08.2019
SBJ-33-C5-AMC-90-RE-101	1	Concept evaluation - Appendix A - Drawings binder	15.08.2019
SBJ-33-C5-AMC-90-RE-103	0	Preferred solution, K12 - Appendix C - Architectural evaluation	15.08.2019
SBJ-33-C5-AMC-90-RE-106	0	Preferred solution, K12 - Appendix F - Global Analyses - Modelling and assumptions	15.08.2019
SBJ-33-C5-AMC-90-RE-107	0	Preferred solution, K12 - Appendix G - Global analyses - Response	15.08.2019
SBJ-33-C5-AMC-90-RE-119	0	Preferred solution, K12 - Appendix S - Parametric excitation	15.08.2019

APPENDIX C
K11 - K13 - K14 List of drawings and reports

Document & Drawing No	Rev.	Title	Date document issued
	0		
SBJ-32-C5-AMC-04-RE-115	0	Concept evaluation - Appendix O - Material technology and steel in marine environment	24.05.2019
SBJ-32-C5-AMC-20-RE-105	0	Concept evaluation - Appendix E Aerodynamics	24.05.2019
SBJ-32-C5-AMC-21-RE-108	0	Concept evaluation - Appendix H - Global analyses - Special studies	24.05.2019
SBJ-32-C5-AMC-22-RE-103	0	Concept evaluation - Appendix C - Architectural evaluation	24.05.2019
SBJ-32-C5-AMC-22-RE-109	0	Concept evaluation - Appendix I - Fatigue analyses	24.05.2019
SBJ-32-C5-AMC-22-RE-111	0	Concept evaluation - Appendix K - Design of cable stayed bridge and abutments	24.05.2019
SBJ-32-C5-AMC-22-RE-112	0	Concept evaluation - Appendix L - Design of cable stayed bridge and abutments	24.05.2019
SBJ-32-C5-AMC-23-RE-118	1	Concept evaluation - Appendix R - Risk assessment	30.06.2019
SBJ-32-C5-AMC-26-RE-113	0	Concept evaluation - Appendix M - Anchor systems	24.05.2019
SBJ-32-C5-AMC-27-RE-110	0	Concept evaluation - Appendix J - Ship collision	24.05.2019
SBJ-32-C5-AMC-28-RE-114	0	Concept evaluation - Appendix N - Fabrication and Marine Operations	24.05.2019
SBJ-32-C5-AMC-90-RE-100	0	Concept evaluation - main report	24.05.2019
SBJ-32-C5-AMC-90-RE-101	0	Concept evaluation - Appendix A Drawings	24.05.2019
SBJ-32-C5-AMC-90-RE-102	0	Concept evaluation - Appendix B Methods of concept ranking	24.05.2019
SBJ-32-C5-AMC-90-RE-104	0	Concept evaluation - Appendix D Initial evaluations K11-K14	24.05.2019
SBJ-32-C5-AMC-90-RE-106	0	Concept evaluation - Appendix F Global Analyses - Modelling and assumptions	24.05.2019
SBJ-32-C5-AMC-90-RE-107	0	Concept evaluation - Appendix G Global analyses -	24.05.2019

Appendix G1		Response	
SBJ-32-C5-AMC-90-RE-107 Appendix G2	0	Appendix G Enclosure 01 K11 07 PROD load combinations bridge direct expected max	23.05.2019
SBJ-32-C5-AMC-90-RE-107 Appendix G3	0	Appendix G Enclosure 13 K11 07 screening windsea 100yr	23.05.2019
SBJ-32-C5-AMC-90-RE-107 Appendix G4	0	Appendix G Enclosure 14 K11 07 screening swell 10000yr	23.05.2019
SBJ-32-C5-AMC-90-RE-107 Appendix G5	0	Appendix G Enclosure 15 K12 06 screening windsea 100yr	23.05.2019
SBJ-32-C5-AMC-90-RE-107 Appendix G6	0	Appendix G Enclosure 16 K12 06 screening swell 10000yr	23.05.2019
SBJ-32-C5-AMC-90-RE-107 Appendix G7	0	Appendix G Enclosure 17 K13 06s screening windsea 100yr	23.05.2019
SBJ-32-C5-AMC-90-RE-107 Appendix G8	0	Appendix G Enclosure 18 K13 06s screening swell 10000yr	23.05.2019
SBJ-32-C5-AMC-90-RE-107 Appendix G9	0	Appendix G Enclosure 21 10205546-11-NOT-088 Variable static loads	29.03.2019
SBJ-32-C5-AMC-90-RE-117	0	Concept evaluation - Appendix Q - Concept ranking	24.05.2019
SBJ-32-C5-AMC-90-RE-119	0	Appendix S - Parametric excitation	24.05.2019

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