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FATIGUE ASSESSMENT

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Prodtex IFE Pure Logic HEYERDAHL ARKITEKTER AS H&BB MIKO MARINE AS BUKSÉR OG BERGING FORCE TECHNOLOGY SWERIM

REPORT

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CONCEPT DEVELOPMENT FLOATING BRIDGE E39 BJØRNAFJORDEN

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Summary

General

The Bjørnafjorden floating bridge concept K12 (the project groups (OON) proposed concept; curved, end-anchored with mooring system) has been evaluated for fatigue life. This report includes calculations of fatigue life for multiple points on the cross-section along the entirety of the bridge girder from global and local loads. It also includes calculations of fatigue life for the top and bottom of columns.

Method

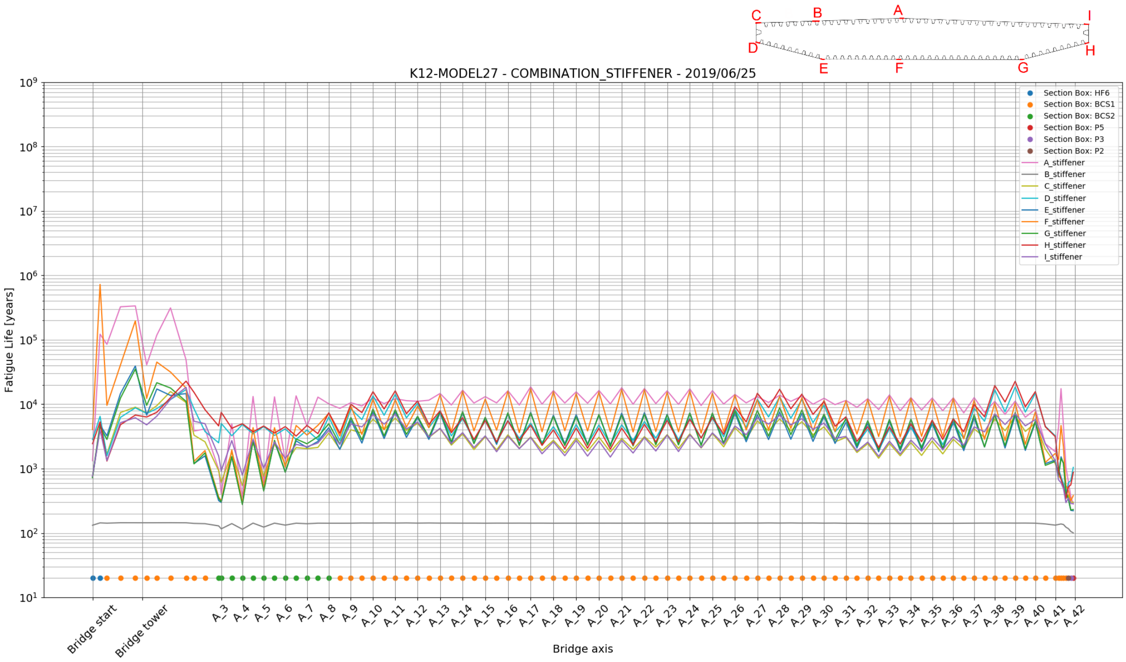
Analyses of dynamic response to environmental loads (swell, wind, wind-sea, tide) are performed in the frequency domain using DynNO/ABAQUS. From the complex response spectral density matrix, corresponding section force time-series are generated to create stress time-series for specific points on the cross-section. Traffic and tidal loads are analysed using quasi-static analyses in Sofistik.

The environmental stress histories are then combined and the rainflow counting method is used to calculate stress ranges and blocks for both environmental and quasi-static stress histories. The results from the rainflow count are used in a combination formula (developed by DNVGL) to account for all actions on the bridge, to calculate the fatigue damage and thus the fatigue life for the bridge.

Local traffic actions have been evaluated in combination with global actions. A welded box stiffener was designed to achieve sufficient fatigue life (>100 years) under wheel loads in outer slow lanes.

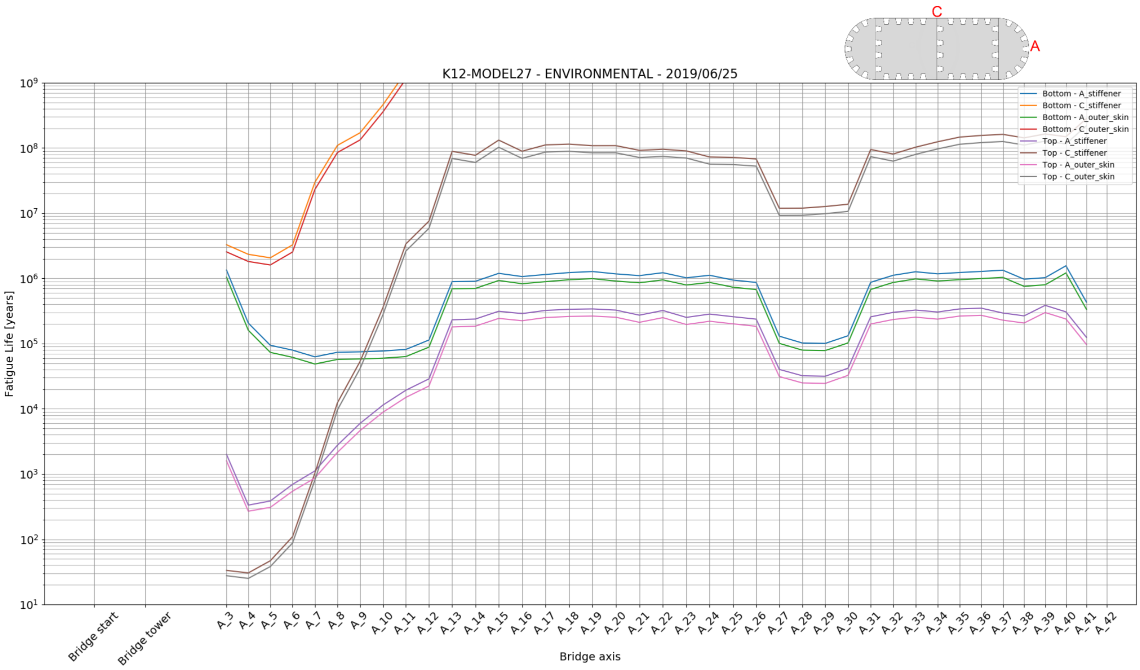
Results

Figure 1 shows fatigue life for various points on the bridge girder (see reference points A-I). The butt weld between longitudinal trapezoidal stiffeners is the detail that was found critical in most cases which is shown here (butt welds between outer plates/skin have also been checked, see chapter 5). Points A C D E F G H and I are checked for global loads (wind, swell, wind-sea, tidal and traffic) and point B is checked for a combination of global environmental loads and local traffic loads. DFF = 2,5 and detail category F is used for this detail. All areas have sufficient fatigue life across the entire bridge (generally over 300 years). The north end needs to be detailed further due to section transitions from a general bridge girder to a stronger girder connected to the abutments.



> *Figure 1: Fatigue life for various points along the bridge girder. Dots (e.g. "Section Box: BCS1") indicate the different sections along the length of the bridge.*

Columns have sufficient fatigue life at all axes except for close to the cable stayed bridge (see figure below). Weak axis bending moments at the top end of the tallest columns have insufficient fatigue life in three axes both when stiffeners and outer skin is checked. This is based on simplified stress concentration factors and needs further work and detailing. Ship impact is governing for most of the columns.



> *Figure 2: Fatigue life of columns. The same cross-section is used in all axes due to ship impact being governing.*

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APPENDICES

A - Environmental fatigue loads – lumping of scatter diagrams and realization sensitivity

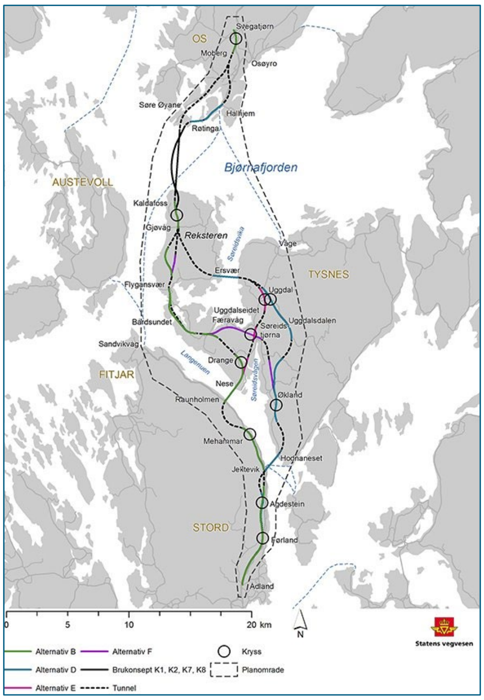
B - Local traffic fatigue methodology

C - Detailed results, section data, stress transfer factors and pairplots

1 INTRODUCTION

1.1 Project context

Statens vegvesen (SVV) has been commissioned by the Norwegian Ministry of Transport and Communications to develop plans for a ferry free coastal highway E39 between Kristiansand and Trondheim. The 1100 km long coastal corridor comprise today 8 ferry connections, most of them wide and deep fjord crossings that will require massive investments and longer spanning structures than previously installed in Norway. Based on the choice of concept evaluation (KVU) E39 Aksdal Bergen, the Ministry of Transport and Communications has decided that E39 shall cross Bjørnafjorden between Reksteren and Os.



SVV is finalizing the work on a governmental regional plan with consequence assessment for E39 Stord-Os. This plan recommends a route from Stord to Os, including crossing solution for Bjørnafjorden, and shall be approved by the ministry of Local Government and Modernisation. In this fifth phase of the concept development, only floating bridge alternatives remain under consideration.

1.2 Project team

Norconsult AS and Dr.techn.Olav Olsen AS have a joint work collaboration for execution of this project. Norconsult is the largest multidiscipline consultant in Norway, and is a leading player within engineering for transportation and communication. Dr.techn.Olav Olsen is an independent structural engineering and marine technology consultant firm, who has a specialty in design of large floating structures. The team has been strengthened with selected subcontractors who are all highly qualified within their respective areas of expertise:

- Prodtex AS is a consultancy company specializing in the development of modern production and design processes. Prodtex sits on a highly qualified staff who have experience from design and operation of automated factories, where robots are used to handle materials and to carry out welding processes.
- Pure Logic AS is a consultancy firm specializing in cost- and uncertainty analyses for prediction of design effects to optimize large-scale constructs, ensuring optimal feedback for a multidisciplinary project team.
- Institute for Energy Technology (IFE) is an independent nonprofit foundation with 600 employees dedicated to research on energy technologies. IFE has been working on high-performance computing software based on the Finite-Element-Method for the industry, wind, wind loads and aero-elasticity for more than 40 years.
- Buksér og Berging AS (BB) provides turn-key solutions, quality vessels and maritime personnel for the marine operations market. BB is currently operating 30 vessels for

harbour assistance, project work and offshore support from headquarter at Lysaker, Norway.

- Miko Marine AS is a Norwegian registered company, established in 1996. The company specializes in products and services for oil pollution prevention and in-water repair of ship and floating rigs, and is further offering marine operation services for transport, handling and installation of heavy construction elements in the marine environment.
- Heyerdahl Arkitekter AS has in the last 20 years been providing architect services to major national infrastructural projects, both for roads and rails. The company shares have been sold to Norconsult, and the companies will be merged by 2020.
- Haug og Blom-Bakke AS is a structural engineering consultancy firm, who has extensive experience in bridge design.
- FORCE Technology AS is engineering company supplying assistance within many fields, and has in this project phase provided services within corrosion protection by use of coating technology and inspection/maintenance/monitoring.
- Swerim is a newly founded Metals and Mining research institute. It originates from Swerea-KIMAB and Swerea-MEFOS and the metals research institute IM founded in 1921. Core competences are within Manufacturing of and with metals, including application technologies for infrastructure, vehicles / transport, and the manufacturing industry.

In order to strengthen our expertise further on risk and uncertainties management in execution of large construction projects Kåre Dybwad has been seconded to the team as a consultant.

1.3 Current report

This report describes fatigue assessment of the Bjørnafjorden floating bridge, recommended concept K12; end-anchored bridge with mooring system.

The focus of this report has been on the bridge girder and the columns, i.e. the main elements that are subjected to fatigue loads. Reference is made to respective design reports for fatigue design of other structural elements such as cable stayed bridge, mooring system and abutments.

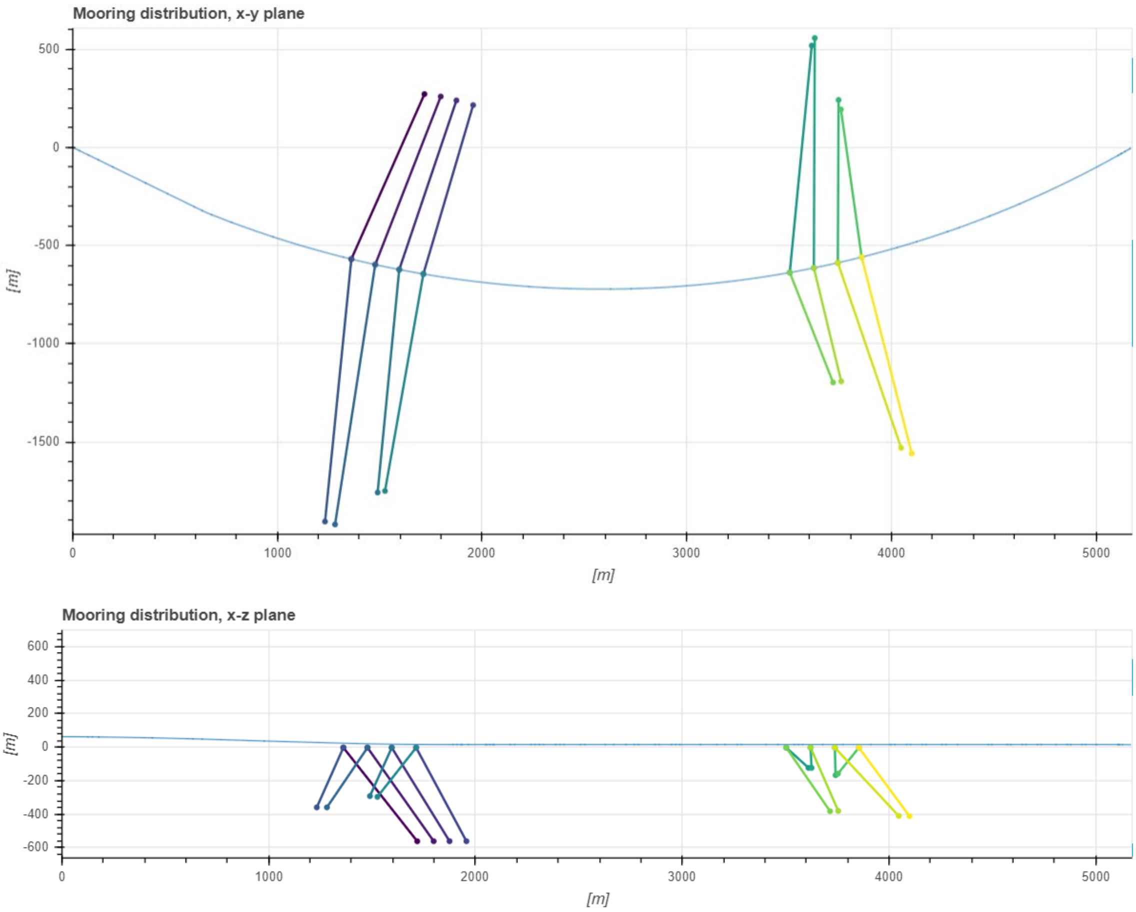
It includes

- Calculation of fatigue life for critical points on the bridge girder from global loads, i.e. environmental, tidal and (global) traffic loads for the entire length of the bridge.
- Calculation and evaluation of fatigue life for critical details from local traffic loads.
- Calculation of fatigue life at top and bottom of columns.
- Evaluation of stress concentrations along bridge girder.
- Evaluation of shear lag effect.
- Evaluation of temporary phases

2 BASIS FOR WORK

2.1 Overview

The Bjørnafjorden floating bridge concept K12 is the recommended concept, ref. [1], that is assessed in this report. It consists of an end-anchored curved bridge with a mooring system as well as a straight cable-stayed part in the south end, as shown schematically in Figure 3. The mooring lines were initially thought to be used as a safety feature for progressive failure of the girder in an accidental limit state, but it was early on decided that the mooring lines should be active also for other limit states, hence, it will affect fatigue life of the bridge.



> *Figure 3: Overview of concept K12 (horizontal plane above, vertical elevation below). (See mooring report, [2], for details on mooring lines)*

As the concept selection phase (pre 2019-03-29, ref. [3]) showed, the K12 concept performed similarly or better than the other concepts with regards to fatigue life.

This report focuses on the specific details that are susceptible to greater loads and load variations for this bridge type in comparison to conventional suspension and cable-stayed bridges, which is the increased amount of stresses in the longitudinal direction. This is due to environmental loads being more prevalent for this bridge type compared to conventional suspension bridges. The main focus is therefore on the longitudinal load carrying details such as the transverse welds between the outer plates and the transverse welds between stiffeners.

2.2 Design rules

The main design rules and guidelines that have been used are listed below. Further references can be found in the reference list in chapter 9.

	Issued/edition	*Ref. no.*
Design basis Bjørnafjorden Rev 0	19.11.2018	[4]
DNVGL-RP-C203 Fatigue design of offshore steel structures	April 2016	[5]
DNVGL Fatigue Design Methodology for BJF floating bridges	2018-05-22	[6]
NS-EN 1991-2 Actions on structures: Traffic load on bridges	2003+NA:2010	[7]
NS-EN 1993-2 Design of steel structures: Steel bridges	2006+NA:2009	[8]
NS-EN 1993-1-9 Design of steel structures: Fatigue	2005+NA:2010	[9]

2.3 Software

Software that has been used in the fatigue calculations (other than standard MS Office tools) are listed below.

FE-analyses

Abaqus/CAE 2018: Global analyses models of environmental loads, local shell and beam models.

Sofistik 2018: Global analyses models of traffic and tidal loads.

Scripting

Calculations and result plots are performed by in-house python script with packages and versions:

Python 3.6.8
Numpy 1.15.4
h5py 2.9.0
Rainflow 2.1.2
Matplotlib 3.0.2
Seaborn 0.9.0

Dynamic analyses

DynNO:

DynNO/ABAQUS: Dynamic response analyses for environmental loads are performed in the frequency domain using DynNO [10].

2.4 Fatigue design basis

2.4.1 Detail categories and S-N curves

Detail categories have generally been chosen according to DNVGL-RP-C203 [5]. When choosing detail categories, it has been assumed traditional manufacturing methods for the bridge girder.

The outer plates/skin of the bridge girder and columns are assumed to be welded with two-sided butt welds and detail category D is chosen.

Detail category for the butt to butt connection of the trapezoidal stiffeners is chosen with basis from Eurocode 3:1-9 table 8.8 [9]. A detail for welds between stiffeners is given in this standard with detail category 71. This is equivalent to category F in RP-C203, hence F is chosen. It should be noted that the S-N curve for category F is more conservative in the high-cycle/low-stress region than for category 71 in Eurocode.

S-N curves in air is used with the corresponding detail category in all cases.

2.4.2 Design fatigue factor

Design fatigue factors (DFF) have been chosen according to the design basis [4]. A DFF of 2,5 has been chosen for all details on the bridge girder. All details checked on the bridge girder are regarded as low consequence for failure and are open for inspection from the inside of the bridge girder.

The outer deck plate is less accessible for inspection and a crack will not be exposed until it has developed to the area between the stiffeners. In addition, it may result in closure of one or more lanes to repair. It is however regarded as unlikely that it will result in full closure of the bridge and a DFF of 2,5 is adopted for the deck plates.

Furthermore DFF = 2,5 is also used for column-bridge girder and column-pontoon connections. Inspection of these areas are relatively easy to identify and critical points that need to be inspected more often can be specified in the detail phase.

All DFFs are shown in Table 3-1.

2.4.3 Stress concentration factors

Stress concentration factors (SCFs) for butt welds are calculated in accordance with DNVGL-RP-C203 [5], see Table 3-1 below. A misalignment, δ_m, of 2 mm and δ_0 of 0,1*t is used for all plate thicknesses*.

It should be noted that according to the Eurocode [9], it is not common practice to include an additional SCF for the splicing of stiffeners (ref. made to detail category 71). This is because normal tolerances are included in this category (if EXC 3 acc. to NS-EN 1090-2 [11] is followed).

At the north end of the bridge there may be geometric stress concentrations due to a change in bridge cross-sections. These have not been calculated but an estimated SCF of 1,5 has been used to account for this. Thicker plates are assumed in this area. See Figure 4 for an overview of the location of all cross-sections.

For transition between different plate thicknesses, it is assumed that this is done in several steps (1 mm step with smoothed edge 1:4) to avoid significant additional SCFs. It is assumed a stricter control for this splicing and a maximum misalignment of 1,5 mm. The

SCFs will, with this approach, not exceed those of butt welds between same-thickness plates (a comparison is shown in APPENDIX C).

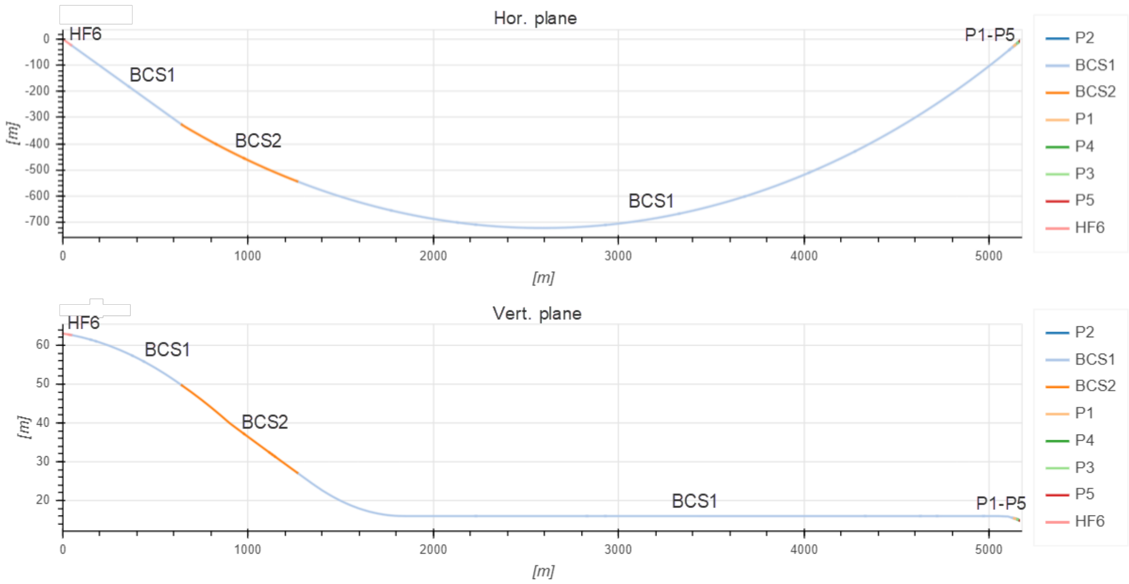
SCFs for columns have not been established from plate thicknesses, but rather estimated SCFs based on the geometry of the connection between column and bridge girder were used. This is because detailing of the columns and column/girder/pontoon connections are heavily governed by ship impact at this stage.

All SCFs are shown in Table 3-1.

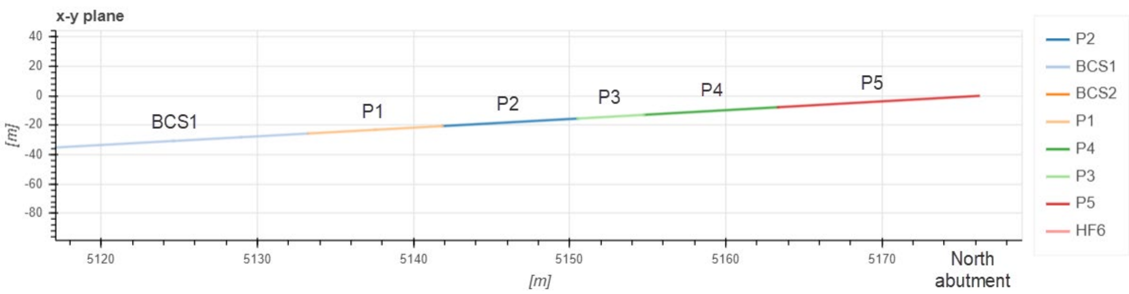
** During this concept phase DNVGL has commented that a revised RP will include δ_0 of 0,05*t, but in this report the current design rules are used according to the Design basis* [4].

2.4.4 Bridge girder cross-sections

Different types of cross-sections have been used in the global analysis models and each cross-section have been checked for fatigue.



> *Figure 4: Overview of cross-sections*



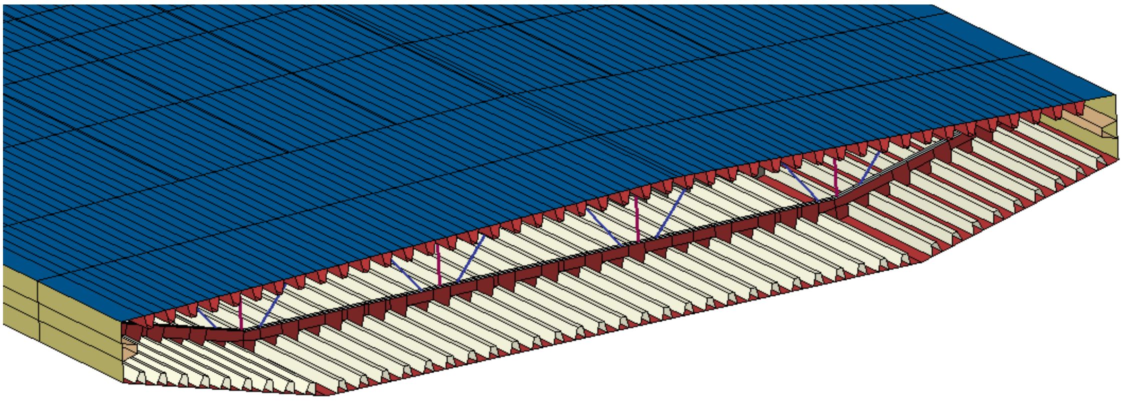
> *Figure 5: Zoomed in view of north end*

Section data bridge girder:

There are two main cross-sections: cross-section 1 - BCS1 and cross-section 2 - BCS2.

BCS1 is the main cross-section which spans most of the bridge, except for the inclined part near the cable-stayed bridge, where a reinforced cross-section, BCS2, is used.

Reinforced cross-sections are used near the abutments, section data for these are shown in APPENDIX C. At columns a thicker bottom plate is used in addition to longitudinal stiffening hull plates. See report Design of bridge deck girder [12] for further details.



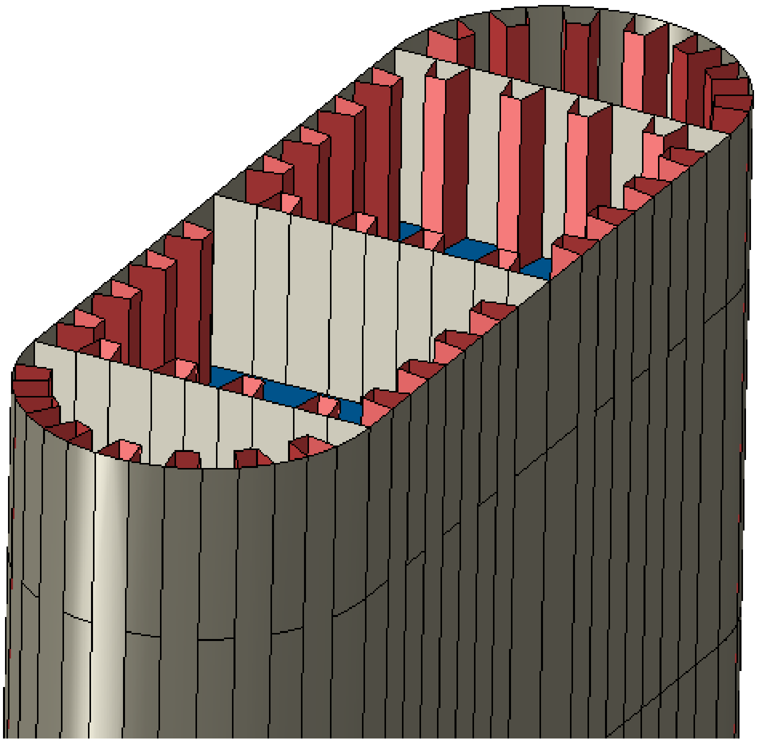
> *Figure 6: Bridge girder in FE-model*

> *Table 2-1: Section data and thicknesses*

	BCS1 **Cross-section 1**	**BCS2** **Cross-section 2**	**BCS1 at column**	**BCS2 at column**
Iy (weak axis)	2,714 m^4	3,201 m^4	3,919 m^4	4,425 m^4
Iz (strong axis)	114,926 m^4	132,013 m^4	133,19 m^4	148,959 m^4
Area	1,4709 m^2	1,7429 m^2	2,409 m^2	2,658 m^2
Top plates	14 mm	14 mm	14 mm	14 mm
Top trapezoidal stiffener	8 mm	12 mm	8 mm	12 mm
Bottom plates	12 mm	12 mm	20 mm	20 mm
Bottom trapezoidal stiffener	8 mm	12 mm	8 mm	12 mm
Side plates	40 mm	40 mm	40 mm	40 mm
Side trapezoidal stiffener	20 mm	20 mm	20 mm	20 mm

Section data column:

Columns that have been checked are elongated 4 m x 12 m, see Figure 7 and section properties in Table 2-2. The same cross-section is assumed in all axes.



> *Figure 7: Column in FE-model*

> *Table 2-2: Section data column*

	Column 4 m x 12 m
Iy (strong axis)	20,1 m^4
Iz (weak axis)	3,9 m^4
Area	1,38 m^2

2.4.5 Shear lag

Reference is made to report SBJ-33-C5-OON-22-RE-017-K12-Design of bridge deck girder [12] for in-depth assessment of shear lag. It is concluded that with a combination of internal longitudinal stiffeners and trusses the shear lag effect is reduced to being negligible. It is therefore not included in the fatigue calculations.

2.5 Environmental dynamic loads

2.5.1 General definitions

Environmental load data for wind, wind sea and swell are provided in the design basis [13]. Scatter diagrams for significant wave height and peak wave period are given for 12 wind sea heading angles with 30-degree shifts. The design basis direction definitions are such that waves and wind coming from north is direction 0/360°, 90° from the east, 180° from south and 270° from west. The swell scatter diagrams provided is used for the incoming swell direction giving the largest fatigue damage within the sector 300°-330°, where 300° is worst for the chosen K12 concept. Probability density functions are not given explicitly for wind loads but is given through combination definitions with wind sea cases.

2.5.2 Combination of load processes

Wind and wind sea are correlated processes and should be combined accordingly. In Table 2-3 combinations of wind sea cases and mean wind velocity are given. The wind and wind sea directions are not necessarily the same, but they can deviate. Input on confidence intervals for directional deviation are provided in the design basis [13]. This information is however not utilized in the fatigue calculations at this stage. All wind/wind sea combinations are run with the same heading angles. In [14] the directional sensitivity of wind loads is investigated for ULS cases. The response is sensitive for changes in mean wind velocity and turbulence definitions but seems to be less sensitive to small directional changes. For the fatigue calculations the changes in mean wind velocity are defined from Table 2-3 so it is not expected that including directional deviations would change the fatigue life estimate significantly.

> *Table 2-3 Combination definition of wind and wind sea [13]*

Direct ion	**0°**	**30°**	**60°**	**90°**	**120°**	**150°**	**180°**	**210°**	**240°**	**270°**	**300°**	**330°**
Hs [m]	345°-15°	15°-45°	45°-75°	75°-105°	105°-135°	135°-165°	165°-195°	195°-225°	225°-255°	255°-285°	285°-315°	315°-345°
0-0.1	2.12	2.14	2.39	2.59	2.49	2.35	2.11	2.18	2.21	2.19	2.24	2.2
0.1-0.2	2.73	2.49	3.25	3.86	3.93	4.06	3.61	3.82	3.96	3.89	3.6	3.13
0.2-0.3	5.15	4.93	4.72	5.38	5.7	6.06	5.63	5.64	6.07	5.59	4.97	4.95
0.3-0.4	6.28	5.95	6.04	6.63	7.17	7.44	6.91	7	7.66	7.11	6.3	6.18
0.4-0.5	7.44	8.64	7.2	7.75	8.53	8.94	8.33	8.52	8.93	8.27	7.51	7.25
0.5-0.6	8.67	9.13	8.19	8.6	10.03	10.47	9.77	9.81	10.44	9.59	8.58	8.32
0.6-0.7	11.43		9.7	9.42	10.95	11.79	11.01	11.47	11.83	10.85	9.77	9.52
0.7-0.8	10.78			10.47	12.1	13.33	12.65	12.62	13.44	12.19	10.63	9.95
0.8-0.9	12.86			11.28	12.73	14.16	13.79	14.02	14.41	13.59	11.78	10.69
0.9-0.9				12.2	14.01	14.66	14.4	14.94	15.26	14.71	12.77	
1-1.1				13.68	14.27	16.43	14.73	16.08	17.22	16.33	13.9	
1.1-1.2				14.01	16.69	16.85		19.57	16.11	16.83	14.66	
1.2-1.3				15.95				20.06		20.18	15.56	
1.3-1.4				14.88				15.7		21.05	17.25	
1.4-1.5				15.94						17.48	18.64	
1.5-1.6				17.79							17.02	
1.6-1.7											19.88	
1.7-1.8											19.8	
1.8-1.9											23.28	

The swell process is defined as uncorrelated with wind and wind sea. In the damage calculations these loads are randomly combined with the wind sea cases. To be able to maintain the total probability of the combinations the randomization does however have a constraint. The swell cases are randomly assigned to the wind sea combinations. If the swell case has a lower probability than the assigned wind sea case, a combination is defined with the probability according to the swell case going into the combination, and the remaining probability of the wind sea case is not combined with any swell. If the swell probability is larger than the assigned wind sea case, then the swell case is split up and combined with more than one wind sea case.

The full case matrices and the combination matrix is shown in APPENDIX A.

2.5.3 Lumping scatter diagrams

In APPENDIX A an optimized lumping of the environmental scatter diagrams is shown. The investigations are performed on the concept model K12-model7 [15]. In the investigations a fatigue damage analysis of the scatter diagrams with no lumping was first performed. Then the damage ratio of each case was calculated (the fatigue damage from one case divided by the total damage). Based on this simulation the scatter diagrams are lumped according to the damage density of the scatter diagrams following the guidelines from DNV-GL's recommended practise for riser fatigue analysis where the fatigue damage from one block should not exceed 5-10 % of the total damage.

In APPENDIX C pairplots describing the fatigue contributions for the chosen lumped environmental cases for swell, wind and wind sea are shown.

2.5.4 Realization sensitivity

In APPENDIX A, the time series realization fatigue life sensitivity is investigated by comparing the fatigue life from 5 different time series realizations. No significant variation from seed to seed is found, so a single time series realization is deemed enough to estimate the fatigue life properly.

2.6 Quasi-static loads

2.6.1 Tidal loads

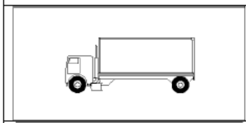
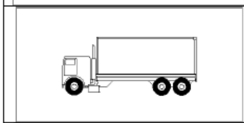
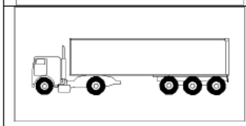
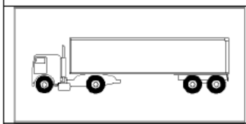
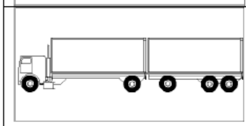
Tidal loads are calculated as static with 730 cycles each year.

For tidal specifications see [16].

2.6.2 Traffic loads

Fatigue load model FLM4 from NS-EN 1991-2, ref. [7] with 500'000 passing's each year have been used for fatigue calculations on traffic actions. Stress ranges have been calculated using influence lines for the 5 different lorry types. Moment around both axes and axial force are considered.

Table 4.7 - Set of equivalent lorries

VEHICLE TYPE			TRAFFIC TYPE			
1	2	3	4	5	6	7
			Long distance	Medium distance	Local traffic	
LORRY	Axle spacing (m)	Equivalent axle loads (kN)	Lorry percentage	Lorry percentage	Lorry percentage	Wheel type
	4,5	70 130	20,0	40,0	80,0	A B
	4,20 1,30	70 120 120	5,0	10,0	5,0	A B B
	3,20 5,20 1,30 1,30	70 150 90 90 90	50,0	30,0	5,0	A B C C C
	3,40 6,00 1,80	70 140 90 90	15,0	15,0	5,0	A B B B
	4,80 3,60 4,40 1,30	70 130 90 80 80	10,0	5,0	5,0	A B C C C

> *Figure 8: Equivalent lorries as per FLM4, ref. [7]*

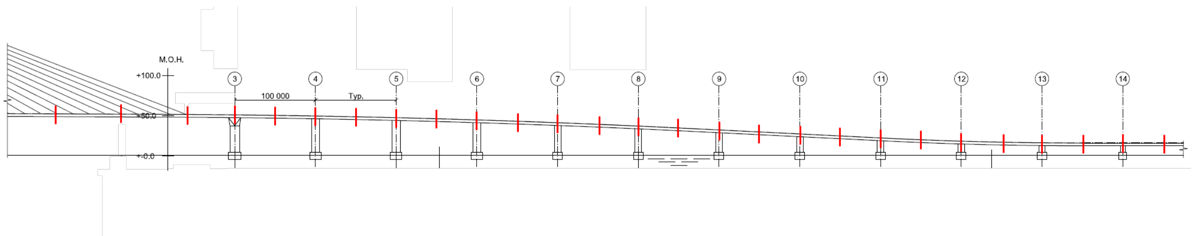
3 METHODOLOGY

3.1 General overview

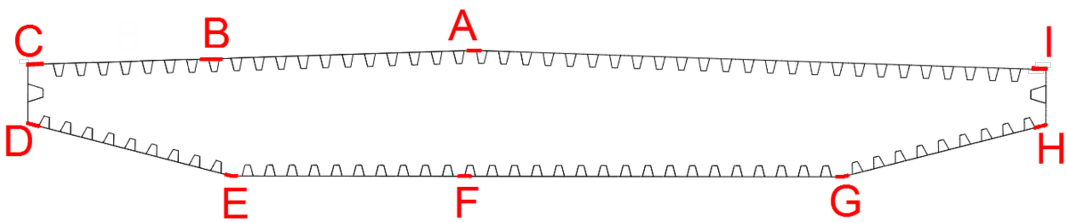
The general workflow for calculating fatigue life is shown below:

1. Creation of relevant FE analysis models:
   a. Global analysis models of the bridge to calculate section forces from the different fatigue load cases; environmental loads, tidal loads and traffic loads.
   b. Local FE models and hand calculations for calculation of stress transfer factors (see ch. 3.4.1) from unit loads at specific points, see Figure 10.
2. Establish fatigue specific parameters, i.e. detail categories, design fatigue factors and stress concentration factors, see Table 3-1.
3. Create script that calculates fatigue life for specific points for the entire length of the bridge girder based on the abovementioned points:
   a. Calculation of local stress ranges from global loads based on stress transfer factors from unit load model.
   b. Rainflow count of the stress data for all load cases
   c. Damage/fatigue life calculation for load types separately
   d. Combination of stresses from environmental, tidal and traffic loads according to design basis and DNV-GL Fatigue methodology to calculate combined damage/fatigue life.

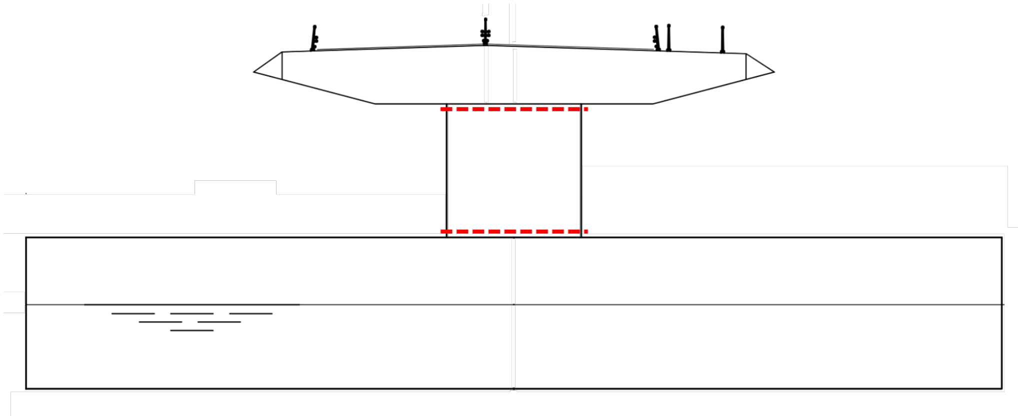
This procedure has been used to calculate fatigue life at midspan between all axes and at each axis, see Figure 9, for the entire bridge length. Points checked on the girder are shown in Figure 10. Stresses calculated for these extremal points are conservatively used for both outer plates as well as trapezoidal stiffeners. The same is done for top and bottom of columns, see Figure 11 and Figure 12.



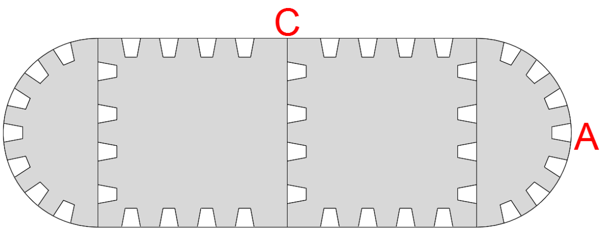
> *Figure 9: Areas checked for fatigue damage along the entire length of the bridge*



> *Figure 10: Points that have been checked for fatigue damage at all midspans and axes.*



> Figure 11: Cross-sections checked at all columns, ref. point A and C shown below



> Figure 12: Points checked for strong and weak axis bending (A and C respectively)

3.1.1 Local vs. global fatigue calculations

Point B on the bridge girder is checked for local wheel stresses in combination with global loads. All other points are checked for global loads only. This is because point B is governing for local traffic due to significant stresses from both weak and strong axis moments from environmental loads in addition to being located at the outermost edge of the slow lane directly under lorry wheels.

A methodology for fatigue calculations for local traffic has been established, see APPENDIX B. Using this method, a modified stiffener geometry was found that has sufficient fatigue life acc. to the combination formula (ref. chapter 3.5.2). This stiffener is used at the wheel positions in slow lanes.

3.1.2 Overview of design parameters

> *Table 3-1: Overview of fatigue design parameters trapezoidal stiffeners and outer skin*

Point	Detail type	Detail category**	DFF	Girder section	Plate thickness	SCF
A C I	Trapez. stiffener top	F	2,5	BCS1	8 mm	1,0*
D E F G H	Trapez. stiffener bottom	F	2,5	BCS1	8 mm	1,0*
A C I	Trapez. stiffener top	F	2,5	BCS2	12 mm	1,0*
D E F G H	Trapez. stiffener bottom	F	2,5	BCS2	12 mm	1,0*
A C D E F G H I	Trapez. stiffener top and bottom	F	2,5	P1-P5, HF6 (north and south end)	Varying	1,0*
B	Modified trapez. Stiffener top	F	2,5	All	Varying	1,0*
A C I	Top deck plate	D	2,5	BCS1	14 mm	1,13
D E F G H	Bottom plate	D	2,5	BCS1	12 mm	1,2
A C I	Top deck plate	D	2,5	BCS2	14 mm	1,13
D E F G H	Bottom plate	D	2,5	BCS2	12 mm	1,2
A C D E F G H I	Outer plates top/bottom	D	2,5	P1-P5, HF6 (north and south end)	Varying	1,5
Column A	Trapez. stiffener	F	2,5	Column		1,5
Column C	Trapez. stiffener	F	2,5	Column		1,5
Column A	Outer plates	D	2,5	Column		2,0
Column C	Outer plates	D	2,5	Column		2,0

* *detail category chosen from Eurocode* [9] *includes SCF for stiffener welds within normal tolerance levels.*

** S-N curves in air is used for all considered points.

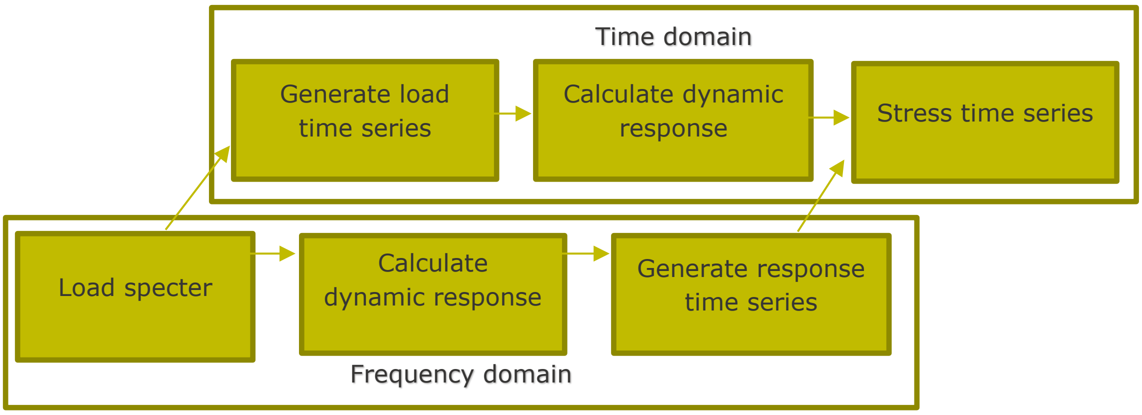
3.2 Dynamic analyses

3.2.1 Methodology

Fatigue calculations from dynamic environmental loads such as wind and wave loading are computationally demanding since a large number of environmental loads need to be simulated. This make frequency domain calculations attractive for such calculations as it is computationally effective compared with time domain analyses. The system is expected to behave quite linear for the load cases dominating the fatigue life, so frequency domain calculations are deemed applicable.

A challenge arises when estimating damage from frequency domain calculations when the response is multimodal and/or wide banded, as it is for the Bjørnafjord floating bridge. Several methods for estimating fatigue damage from response spectral densities are presented in the literature, but they all come with significant simplifications. The established preferred method to calculate fatigue damage from dynamic environmental loads is the Rainflow cycle counting method. This method is based on stress time series from the dynamic response.

The dynamic response calculations are performed with DynNO [10], which calculate the multimodal response in the frequency domain. From these analyses corresponding section force time series can be simulated by Cholesky decomposition of the complex response spectral density matrix. In this way the Rainflow cycle counting method can be used for fatigue damage calculations for frequency domain analyses as well.



> *Figure 13: Calculation overview*

3.2.2 Aerodynamic loads

Wind loads are applied to the bridge girder and columns of the bridge. Wind loads on the tower and stay-cables are neglected in the analyses. The wind loads on the pontoon columns used are for a simple circular shape with 10 m diameter. This is a simplification of the columns in the final version of the bridge but is considered a conservative choice at this stage. The static load coefficients and member dimensions used in the fatigue analyses are shown in Table 3-2.

> *Table 3-2: Static coefficients for aerodynamic forces on the structural elements*

Structural member	Cross wind dim. [m]	Along wind dim. [m]	C_D	C_L	C_M	$dC_L/d\alpha$	$dC_M/d\alpha$
Girder	3.5	31	0.866	-0.521	-0.037	3.825	1.217
Columns	10	10	1.05	0	0	0	0

3.2.3 Hydrodynamic loads

Hydrodynamic forces from linear potential theory effects are described in [17] for K12-model27, and the reader is referred to this report for details.

Hydrodynamic viscous damping is included in the buffeting response analyses by an iterative approach based on stochastic linearization [18, 19]. The viscous damping is calculated for all wind direction sectors, with a basic wind speed of 10 m/s in 10 m height. The damping effect is then scaled to other wind speeds based on the ratio $(V/V_{ref})^2$.

> *Table 3-3: Hydrodynamic drag coefficients used for all pontoons*

Pontoon motion	C_D*A_{ref}
Surge	0.4*60=24
Sway	0.6*290=348

3.3 Finite element analyses

3.3.1 Global fatigue calculations

This chapter regards points A, C, D, E, F, G, H and I (in bridge girder) that have been checked for global loads.

Nominal stresses are calculated for all points for each bridge girder section type by hand with the section modulus and cross-section area.

To accommodate for smaller variations in stresses from bending moments due to the relatively complex geometry, an Abaqus model has been created to verify hand calculated stresses for a base line bridge girder section. Some variations were found, hence an SCF was calculated as follows:

SCF = (stresses in FE-model)/(hand calculated stresses)

In some areas the hand calculated stresses were larger than stresses in the FE-model, but in that case the SCF was conservatively set to 1,0.

This SCF was multiplied with hand calculated stresses for all bridge girder sections.

Stress transfer factors and SCFs are shown in APPENDIX C.

Stresses for trapezoidal stiffeners are set equal to stresses in the outer deck plates at the same point. This is considered to give a slightly conservative stress level for stiffeners.

3.3.2 Local fatigue calculations

Point B has been checked for local wheel stresses in combination with stresses from global loads (i.e. wind, wind-sea, waves and tide).

Local fatigue is assessed using beam models in combination with shell models to calculate stress ranges from each of the passing vehicles acc. to FLM 4. Different shell models have been used to evaluate the level of global vs local stresses from traffic. This is explained in detail in APPENDIX B.

3.4 Calculation of stress ranges

3.4.1 Global stresses

Stress ranges are calculated by multiplying section forces with corresponding stress transfer factors from local shell model subjected to unit forces.

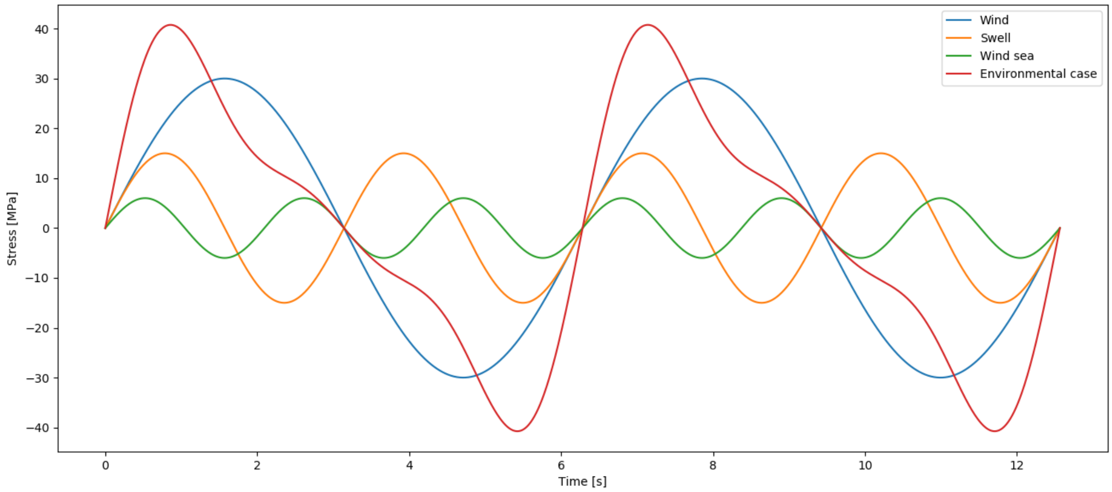
$$\sigma_{xx} = \frac{N_{xx}}{A} + \frac{M_{yy}}{W_{yy}} + \frac{M_{zz}}{W_{zz}} = N_{xx} * \mathrm{Unit}(N_{xx}) + M_{yy} * \mathrm{Unit}(M_{yy}) + M_{zz} * \mathrm{Unit}(M_{zz})$$

Where:

σ_{xx} = Normal stress
N_{xx} = Axial force
M_{yy}, M_{zz} = Weak and strong axis bending moment
A = Section area
W_{yy}, W_{zz} = Weak and strong axis section modulus for considered point
$\mathrm{Unit}(N_{xx}/M_{yy}/M_{zz})$ = Unit stress transfer factor for $\mathrm{N_{xx}/M_{yy}/M_{zz}}$ respectively

Static envelope stresses have been calculated for traffic and tidal effects. Stress ranges are calculated as max(stress) – min(stress).

Time-stress series have been generated for swell, wind and wind sea. Time increment of 0.38 sec is used. Environmental case is the sum of swell, wind and wind sea.



> *Figure 14 Example of time-stress plot for wind, swell, wind sea and environmental*

3.4.2 Local stresses

Peak stress range (as calculated in APPENDIX B) from each vehicle is summed with environmental stresses using the combination formula. Damage is calculated using the combination formula from combined stresses and summed with damage from the remaining stress ranges for each vehicle, acc. to DNV Fatigue Methodology [6].

This is considered a relatively conservative method because the greatest stress-ranges from each separate load type (i.e. Environmental stress + traffic stress + tidal stress) are added together for all load cases. In reality, some of the stresses will act at times where they have different signs and periods and may cancel each other out.

Combination of local and global effects are best analysed using time series analysis of vehicles randomly passing over the bridge during environmental loading. This was assessed early on to be too time consuming at this stage and should be performed at a later stage when there are less variables in the project.

3.4.3 Columns

Stress ranges for columns are calculated based on hand calculated stress transfer factors based on section data (ref. Table 2-2 and APPENDIX C). A general section at the top and bottom of the column is checked for fatigue. Details like voutes etc. are not accounted for as these are subject to change.

3.5 Fatigue damage calculations

Stresses and cycles are carried out from Standard Practices for Cycle Counting in Fatigue Analysis [20]. Damage is then calculated according to DNVGL-RP-C203 using the Palmgren-Miner rule:

$$\mathrm{D} = \sum_{k=1}^{k} \frac{n_i}{N_i}$$

Where:

D = fatigue damage
k = number of stress blocks
n_i = number of stress cycles in each stress block
N_i = number of cycles to failure by stress range $\Delta\sigma_i$

Each environmental case consists of wind-sea, wind and swell with corresponding possibility for appearance. For some cases swell are equal to zero.

One-hour damage:

$$D_{env} = \sum_{Enviromental\ case} p_{case} D_{case}$$

Where:

p_{case} = Possibility for occurrence within one hour (ref. APPENDIX A)

D_{case} = Damage after one hour

3.5.1 Damage from swell, wind, wind sea and environmental

Swell, wind and wind sea conditions are provided with separate case matrices given by the met-ocean specifications. Each case has a given probability for occurrence within one hour. By summing damage multiplied with probability for all these cases, damage contribution from one hour is calculated.

Environmental is the combination of swell, wind and wind sea given by the met-ocean specifications. Each combination state has a probability for occurrence. Damage is then calculated in the same way as swell, wind and wind sea.

$D_{yearly} = D_{env}$ * 24 hour * 365 days

3.5.2 Combination formula

From DNVGL Fatigue Design Methodology [6], chapter 7.3 - equation 5, as stated in the design basis:

$$D_{case} = f_t \sum_{i=1}^{5} p_i \sum_{j=1}^{k} \frac{n_j}{a} \left(\Delta\sigma_{wj} + \Delta\sigma_i + \Delta\sigma_{tide}\right)^m + \sum_{i=1}^{5} (f_i - f_t p_i) \sum_{j=1}^{k} \frac{n_j}{a} \left(\Delta\sigma_{wj} + \Delta\sigma_i\right)^m + \left(1 - \sum_{i=1}^{5} f_i\right) \sum_{j=1}^{k} \frac{n_j}{a} \left(\Delta\sigma_{wj}\right)^m$$

a = intercept of the design S-N curve with the log N axis

m = negative inverse slope of the S-N curve with the log N axis

n_j = number of cycles in stress block j within one hour

f_t = fraction of tidal cycles relative to the number of environmental cycles

f_i = fraction of lorry type i relative to the number of environmental cycles

p_i = fraction of lorry type i relative to the total number of different lorries

$\Delta\sigma_{wj}$ = Stress range at considered point due to environmental action j

$\Delta\sigma_i$ = Stress range at considered point due to lorry i

$\Delta\sigma_{tide}$ = Stress range at considered point due to tidal variation

k = number of stress blocks in environmental case

Environmental cycles are calculated as the sum of cycles in stress block k. For one hour this sum is around 900 cycles. Tidal count within one hour = 2 / 24h. Lorry count within one hour = yearly count / (24h*365days) * 2 directions.

The total damage from all cases for a one-hour time period is calculated as follows:

$$D_{main} = \sum_{case=1}^{r} D_{case} p_{case}$$

n_t = number of tidal cycles within one hour

n_l = number of lorry cycles within one hour

r = number of cases

p_{case} = possibility for case to occur within one hour

The total environmental damage over one year is then equal as:

$$D_{yearly} = 24h \cdot 365days \cdot D_{main}$$

3.5.3 Local traffic stresses

A method has been developed to calculate stress ranges for passing vehicles on local plate structure in the slow lanes. It is focused on trapezoidal stiffeners as these were shown to have the greatest stress ranges from each passing lorry. The method is described in APPENDIX B.

The method was used to study stress ranges from local traffic for a general trapezoidal stiffener. The stress ranges are in turn used in the combination formula combined with stresses from global loads at point B along the entire bridge girder (ref. 3.5.2). To achieve sufficient fatigue life, it was necessary to design a welded box stiffener.

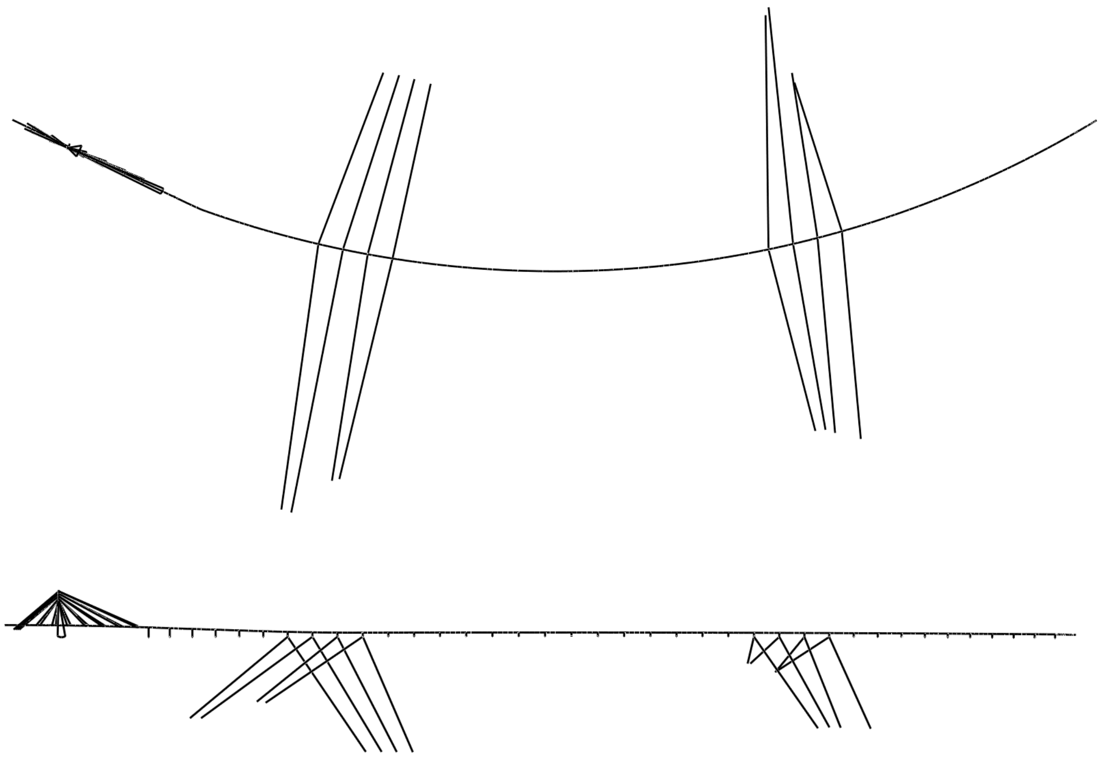
As proposed by DNVGL [6] the largest stress range from each lorry is to be added to the global traffic stresses in the combination formula. The partial damage from the other stress ranges is calculated with the Palmgren-Miner rule and then added to the damage calculated with the combination formula (ch. 7.3 in [6]). Results from this are shown in chapter 0.

4 ANALYSIS MODELS

4.1 Global model

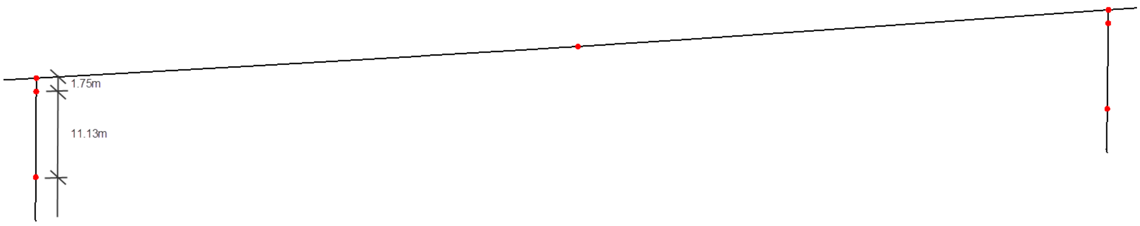
The quasi-static fatigue loads such as traffic and tidal variations are calculated in Sofistik based on the fatigue loads defined in section 2.6.

The dynamic environmental load effects are calculated in the frequency domain with DynNO [10]. DynNO uses an ABAQUS FEM-model as basis. In the calculations presented herein, the concept mode K12-model 27 is used. The ABAQUS model is modelled with B31 elements and the geometry and mesh are identical to the models described in [15].



> *Figure 15: ABAQUS model of K12*

Bridge box nodes are checked over the columns and at the middle span.There are dummy element between bridge box elements and column elements. The distances to the top/bottom column nodes can be seen below.



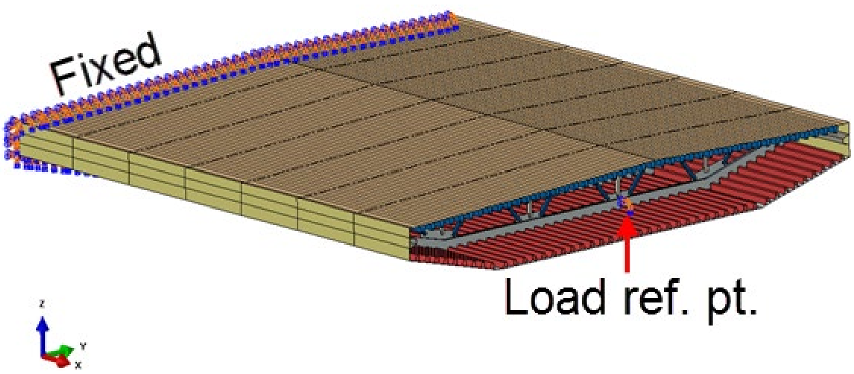
> *Figure 16: Section view of location of nodes (red dot) checked for the columns and bridge boxes*

4.2 Local models

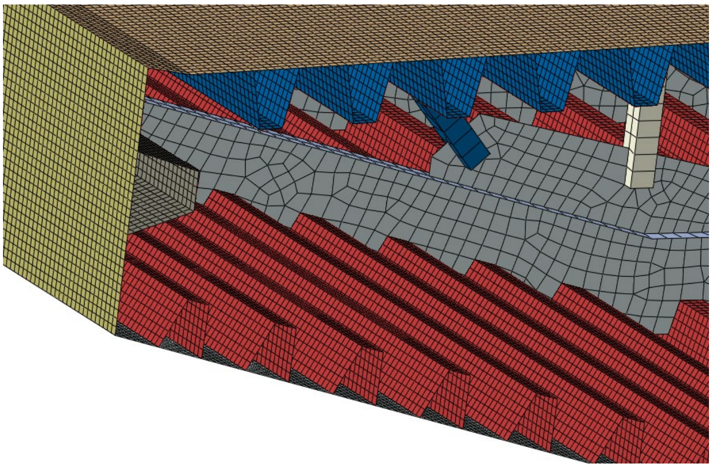
4.2.1 Unit load model

A shell model of the bridge girder has been made to calculate nominal stress transfer factors from unit moments (see APPENDIX C). It is also used to verify and evaluate stresses from hand calculated girder profiles based on 2nd area of moment for different cross-sections during concept development.

A baseline model of the bridge girder has been modelled with shell elements of approx. size of 50 mm (see Figure 18).



> *Figure 17: Unit load model with boundary conditions shown*



> *Figure 18: Illustration of element mesh. Transv. girder has a coarser mesh as shown.*

The model is fixed in one end; unit moments are applied, as shown in Table 4-1, in a reference point in the mass-center at the opposite end that is tied to the edge of the model.

> *Table 4-1: Applied unit loads*

Load case no.	Load direction	Applied load in load reference point
1	MY Weak axis bending	1 Nm
2	MZ Strong axis bending	1 Nm

The unit loads give corresponding stresses (MPa/Nm), see chapter 5.1 which then are multiplied with section forces FX, MY and MZ from global analyses to calculate global stresses (ref. ch. 3.4.1).

Axial stresses from FX are calculated by hand as this was assessed to give sufficient accuracy.

4.2.2 Local traffic models

See chapter 5.2 and APPENDIX B for documentation regarding models used to analyse local traffic stress variations.

4.2.3 Column/girder model

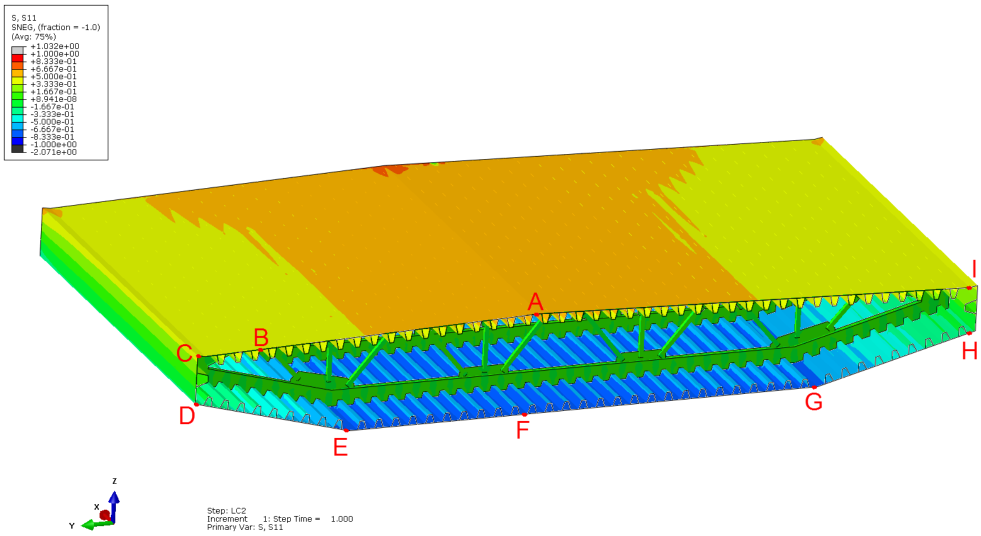
In this chapter it was intended to show models and results from analyses of the column/girder connection. Design of columns was generally governed by ship impact. Due to limited amount of time at the end of this phase, it was not possible to do this, because the columns designed for ship impact were significantly changed at a late stage. It should be noted that the design of the column bridge girder connection needs more attention in the next phase.

5 RESULTS AND CALCULATIONS

5.1 Unit load analyses

Stresses for points shown in Figure 10 are calculated according to chapter 4.2.1.

Figure 19 show stresses from unit moment about weak axis. There is observed small variations in the stresses due to the stiffeners. Stresses are extrapolated from each point A-I by extracting S11 stresses (parallel to bridge length) from a set of elements in each point. This is in turn used to calculate stresses for other cross sections used. Stress transfer factors for cross-section 1 (BCS1), including SCFs from analysis model, are shown in Table 5-1, and the rest of the cross-sections are shown in the APPENDIX C.



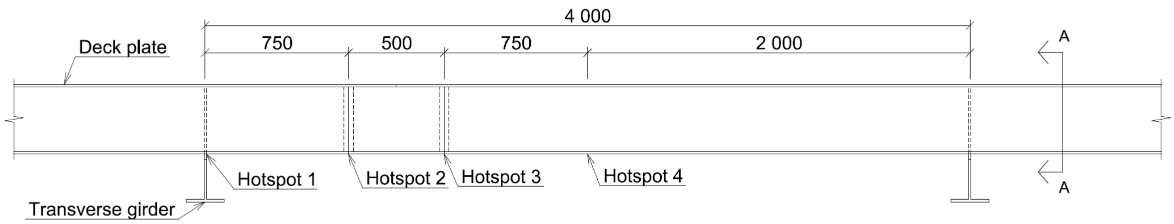
> *Figure 19: S11 stresses (along bridge length) from unit moment about weak axis (1 Nm)*

> *Table 5-1: Stress transfer factors for cross-section BCS1*

Cross-sect.	Point	Unit(Nxx) [MPa/N]	Unit(Myy) [MPa/Nm]	Unit(Mzz) [MPa/Nm]
BCS1	A	0,680	-0,586	0,014
	B	0,680	-0,480	0,083
	C	0,680	-0,438	0,123
	D	0,680	0,196	0,126
	E	0,680	0,704	0,077
	F	0,680	0,722	0,02
	G	0,680	0,704	-0,074
	H	0,680	0,172	-0,125
	I	0,680	-0,402	-0,126

5.2 Fatigue calculations from local traffic

See Table 5-2 below for max. stress ranges as well as calculated corresponding damage for rest of stress ranges. This is for the point with the largest stress variations where stiffeners are welded together, hotspot 3, as shown on Figure 20.



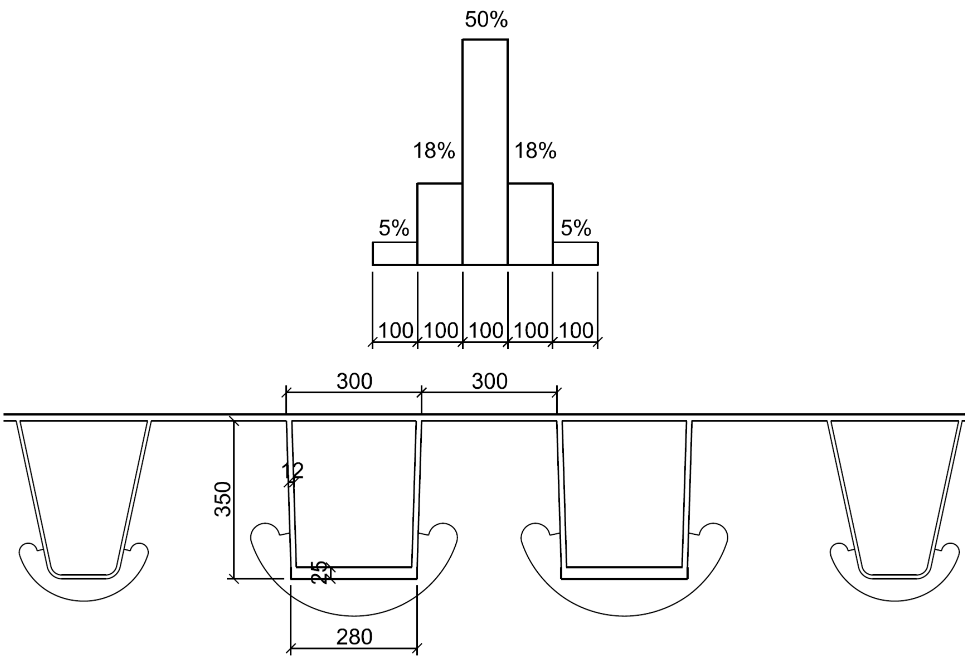
> *Figure 20: Considered areas for local traffic load*

Furthermore, the partial damage from the rest of the stress ranges is calculated as:

Partial damage = (Sum yearly damage all lorries all stress ranges) - (Sum yearly damage max stress ranges)

This partial damage is added to the damage calculated from the combination formula as an initial damage.

The data in the table is valid for the welded box stiffener shown in Figure 21.



> *Figure 21: Principle sketch of welded box stiffener geometry* to be used under each wheel lane (2 per wheel) under both slow lanes, with traffic distribution shown.*

*The cope holes shown have not been detailed accordingly at this stage.

Table 5-2: Input to combination formula for the chosen welded box stiffener

Lorry #	Max. stress range (MPa)	Cycles /year	Corresponding yearly damage
Lorry 1	7,659	50000	0,000001069
Lorry 2	10,472	12500	0,000001277
Lorry 3	11,079	125000	0,000016920
Lorry 4	9,781	37500	0,000002722
Lorry 5	7,144	25000	0,000000377
	Sum yearly damage max stress ranges		0,000022365
	Fatigue life (years) local traffic only		10150 yrs
	Sum yearly damage all lorries all stress ranges		0,00003941
	Partial damage from rest of stress ranges		0,000017043

Fatigue life for the stiffener, point B, is shown in Figure 23.

5.3 Global analyses bridge girder

5.3.1 Bridge girder outer skin

Calculated outer skin fatigue life at point A-F from the combination formula in 3.5.2. Comment: Fatigue life is greater than 100 years all over the bridge. In the lower part, fatigue damage is generally governing in the field, except for at the area closest to the stay cable bridge where bending moments from the tallest columns yield greater fatigue damage than in the field.

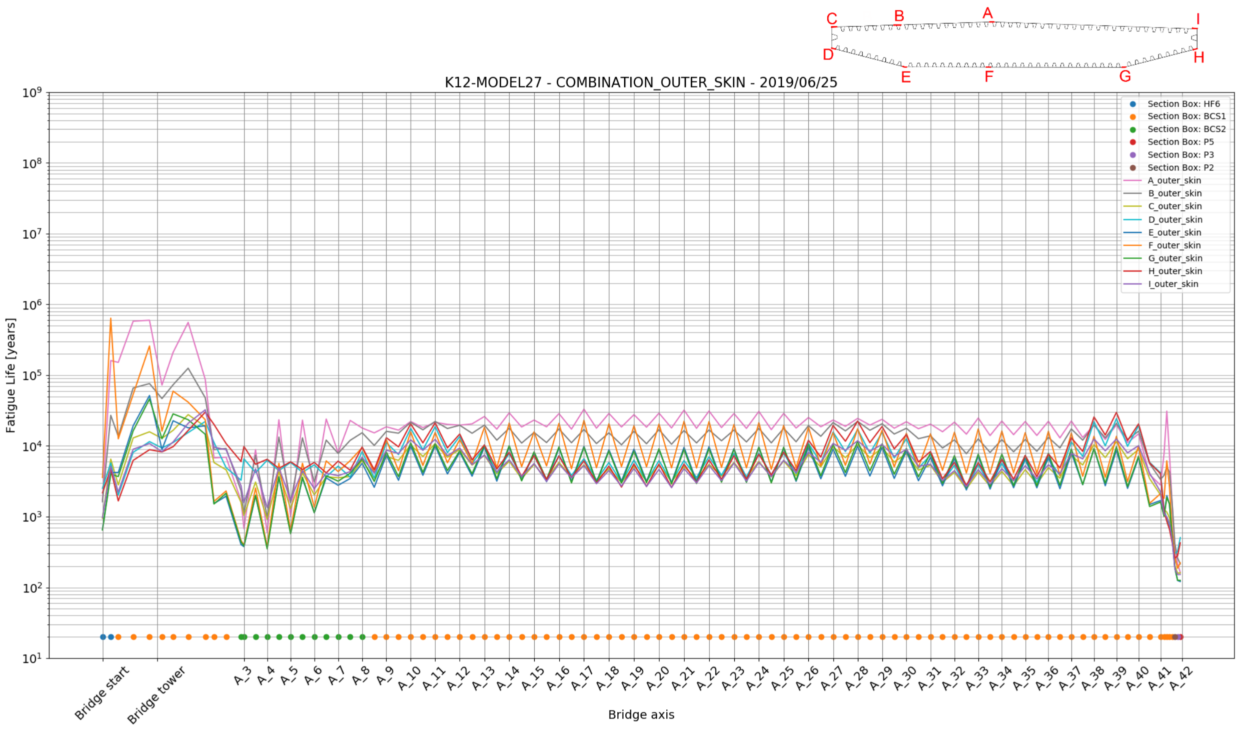
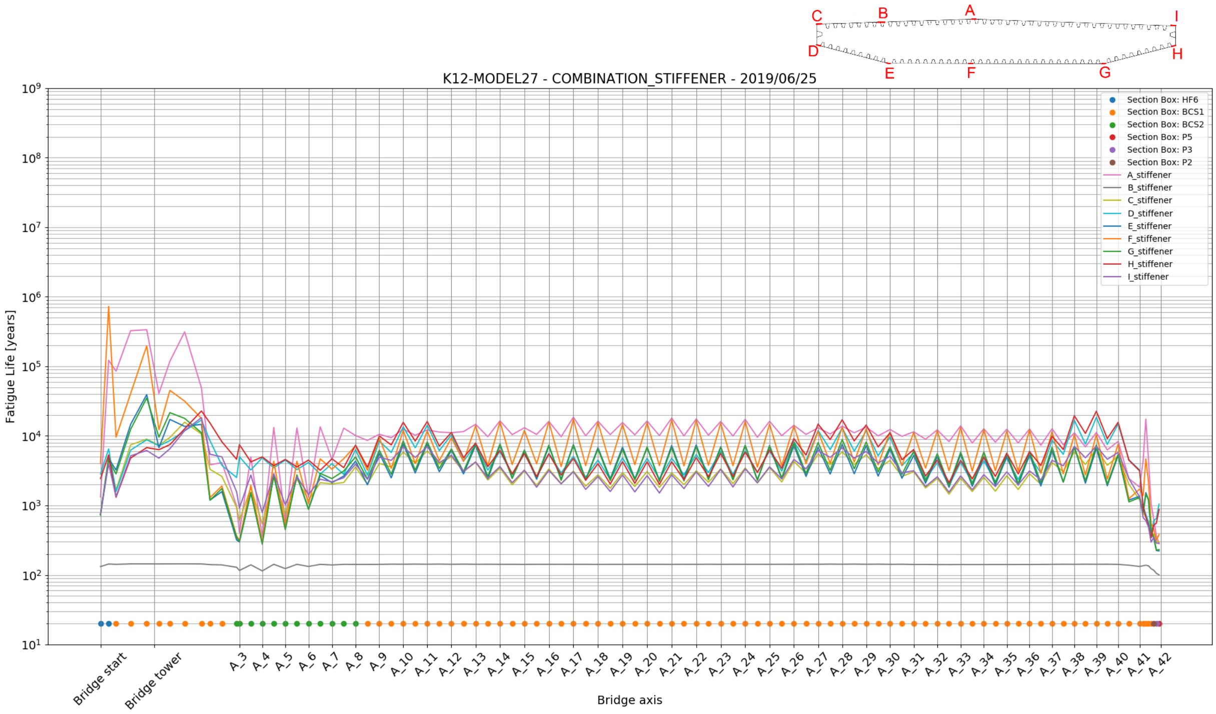


Figure 22: Outer skin fatigue life plot by use of the combination formula

5.3.2 Bridge girder trapezoidal stiffener – combination formula local and global loads

Calculated stiffener fatigue life at point A-F from the combination formula in 3.5.2.

Comment: Fatigue life is greater than 100 years all over the bridge including point B, where local traffic damage is included.

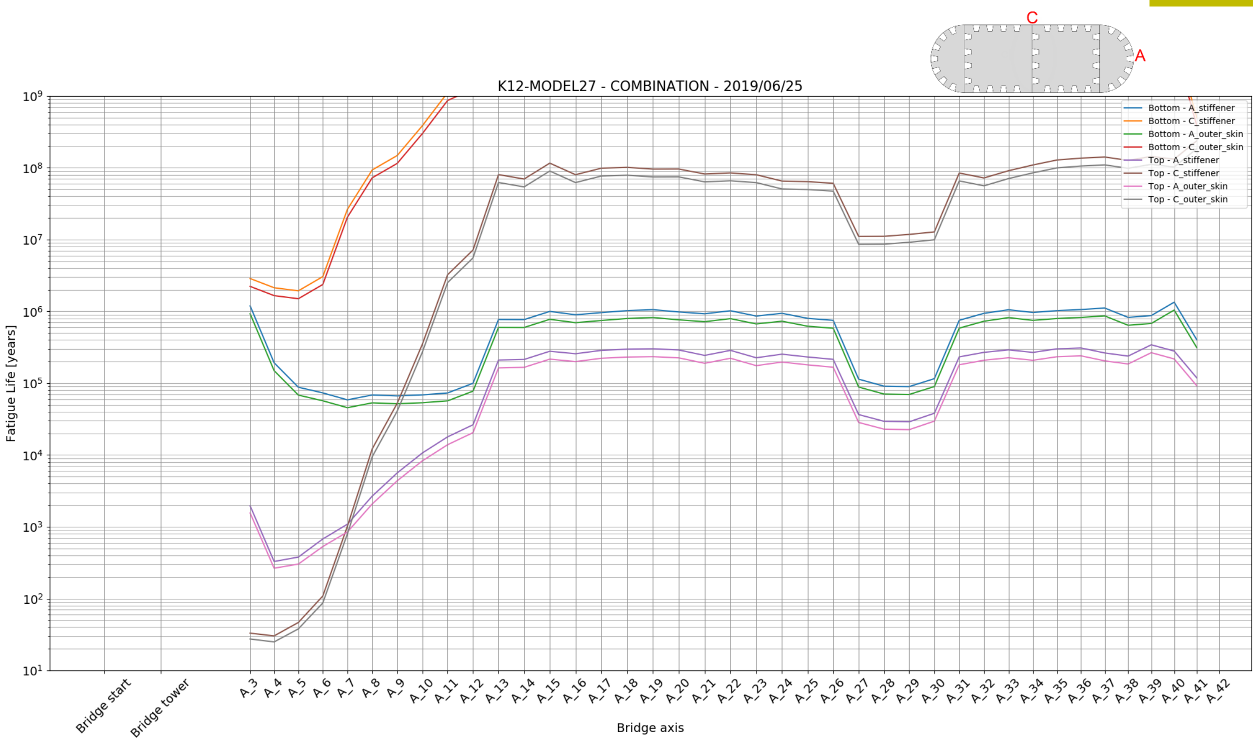


> *Figure 23: Trapez. stiffener fatigue life plot by use of the combination formula*

5.4 Analyses columns

Plots of fatigue life of columns based on detail category D for outer skin and F for trapezoidal stiffeners, SCF 2,0 and 1,5 respectively (butt welds). Fatigue life is shown below.

Calculated column fatigue life at point A and C from the combination formula in 3.5.2. Comment: Fatigue life is greater than 100 years from axis A_6 (south end) to axis A_41(north-end). The top part of the column, at weak axis point A need further detailing.



> *Figure 24: Column fatigue life plot by use of the combination formula*

5.5 Temporary phases

The installation phases have been analysed and were shown to give less stresses in the bridge than a normal operating condition. Ref. SBJ-33-C5-OON-22-RE-023 App. 02 and SBJ-33-C5-OON-22-RE-023-B K12. The reason for not focusing on this is;

- There is no traffic on the bridge
- The environmental loading is less
- There are more side anchors than for the in-place condition (appr. every 500m)

As long as the fatigue life is over 100 years + duration of installation, it is concluded that the fatigue life for installation phases is not problematic.

It should be noted that detail design for temporary phases (including transportation) must be revisited at a later point.

6 CONCLUSIONS AND FURTHER WORK

6.1 Bridge girder

As the results show (Figure 23 and Figure 24), there is sufficient fatigue life (>100 years) for all points along the bridge girder. For most of the lower part of the bridge, fatigue life is well over 1000 years. There is therefore room for increased DFFs and SCFs, if need be, in future detailing.

North of axis 40 there are reinforced sections and an estimated SCF of 1,5 for the butt weld between the outer plates. The details of how the different sections are to be tapered off/connected needs further work. Some measures to achieve sufficient fatigue life in this area include grinding of welds and specific detailing to reduce SCFs. There may also be need for further work on optimising boundary conditions in the global analyses.

6.2 Local traffic

Local traffic is calculated with combined stresses from local and global loads. A fatigue life of more than 100 years is achieved for the entire length.

6.3 Columns

Columns are mostly governed by ship impact, ref. report SBJ-33-C5-OON-22-RE-014-K12-Ship impact, pontoons and columns [21]. This is reflected in the fatigue life which is generally well over 1000 years. There is room for greater SCFs for the lower part of the bridge.

For the tallest columns near the cable-stayed bridge, there is a significant drop in fatigue life from axis A-3 to A-7 at the top of the columns. This is mainly due to weak axis bending moments from wind-sea loads. It may be necessary to reinforce parts of the column at the top or work on improving the connection between the column and the bridge girder to reduce SCFs or improve detail categories.

6.4 Further work

Local traffic should be analysed using time history analyses to evaluate and improve the level of conservatism in the method used in this report. In addition, it may be needed to evaluate local stress-time series for each passing vehicle using shell model instead of beam models to further increase level of certainty and reduce conservatism. Welded box stiffeners may also be improved under the inner wheel lanes due to less damage from strong axis moments in these areas. Other areas may also yield significant fatigue damage in the transverse girder that should be checked.

At the ends of longitudinal main stiffeners in bridge girder above columns there are observed significant SCFs. This area needs improved details for fatigue. By extending the plates out into the span (using plates or trusses), SCFs may be reduced.

S-N curves that are specific to this project and the materials that are to be used may be established to further increase the level of certainty in the fatigue calculations. This may also be done for less traditional production methods like laser welding if applicable.

Tolerances for butt welds and misalignments should be evaluated with a manufacturer. This has a significant impact on the design life for the structure and will affect all details where plates are joined with butt welds.

The yearly traffic and amount of heavy traffic should be studied further to improve the level of certainty. A project specific fatigue load model 5 should be specified.

A detailed inspection plan should be evaluated and established with respect to selection and improvement of DFFs.

Section forces in bridge girder (at columns) is extracted from a beam model directly at the peak of the support moment. Further smoothing of the moment diagram due to the extent of the columns may increase the fatigue life at these areas.

Connections between columns and pontoon/bridge girder need to be evaluated in detail and more detailed models should be studied to evaluate SCFs further.

Transportation phases must be evaluated when this has been specified.

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APPENDIX