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CONCEPT DEVELOPMENT, FLOATING BRIDGE E39 BJØRNAFJORDEN

Preferred solution, K12

Appendix I – Fatigue analyses

CLIENT

Statens vegvesen

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REPORT

PROJECT	Concept development, floating bridge E39 Bjørnafjorden	DOCUMENT CODE	SBJ-33-C5-AMC-22-RE-109
SUBJECT	Appendix I – Fatigue analyses, K12	ACCESSIBILITY	Restricted
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SUMMARY

This report describes the work performed in consideration of fatigue capacity of the selected concept (K12) in the concept development work of a floating bridge over Bjørnafjorden.

Fatigue calculations have been performed for selected details in the bridge girder, the connection between columns and bridge girder/pontoons, mooring chains and stay cables.

Large parts of the assessed structures have an acceptable fatigue utilization. However, some details are found to have insufficient fatigue life and need additional measures.

Most notably these are the details in the bridge girder deck which are subjected to local traffic loads. A measure that has been applied during the current project phase is to increase the plate thickness in the deck to 16mm along the entire length of the bridge to increase fatigue robustness. Still, additional measures are needed in order to get acceptable fatigue lives. For future fatigue work it is proposed to developed a traffic load model based on historical/forecasted traffic data for the actual bridge location. This is expected to give a less conservative load model which will improve fatigue life.

A sensitivity study has been performed on the traffic load model to identify the load reduction required to achieve acceptable fatigue lives. Several of the typical details in the deck currently have a calculated fatigue life of around 30 years. The required load model reduction to achieve acceptable fatigue life for these details is to use the *medium range* traffic distribution and reduce axle loads to 75% of the full FLM4 axle loads.

Most of the girder deck details are expected to get sufficient fatigue life with a moderate reduction in the traffic load model. However, for the cut-out detail in the transverse frames around the longitudinal trapezoidal stiffeners additional measures are required. For this detail further design optimization remains to get a fatigue friendly design. Another measure that has been proposed is to utilize the asphalt stiffness in the local FE-analyses, which is believed to give reduced stresses due to a more realistic transfer of loads onto the steel deck.

Another detail which currently have insufficient fatigue life is the connection between bridge girder and columns in the high part of the floating bridge. Here, insufficient fatigue life is found at axis 3 and 4 in the vicinity of the cast pieces at the top corners of the columns. At Axis 3 the calculated fatigue life is 44 years and 47 years for the girder side and column side of the corner respectively. At Axis 4 the calculated fatigue life is 89 years (column side). The recommended measure to achieve acceptable fatigue life for these locations is to increase structural dimensions locally. For future fatigue work it is also recommended that this connection is assessed by a more refined calculation method.

0	15.08.2019	Final issue	A. G. Johansen	P. N. Larsen	S. E. Jakobsen
REV.	DATE	DESCRIPTION	PREPARED BY	CHECKED BY	APPROVED BY

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1 Introduction

This report describes the work performed in consideration of fatigue capacity in the concept development work of a floating bridge over Bjørnafjorden. Only results for the selected concept (K12) is presented in this report.

For details regarding concept development and design considerations, see [1].

The steel parts of the floating bridge concepts developed during the current project phase will consist of numerous structural details that will need to be assessed with respect to fatigue. The scope for the fatigue assessment presented in this report is focused on structural details that are considered important with respect to concept selection and structural details/dimensions that will be governed by fatigue, and hence may have a cost impact. Details that are believed not to bring significant increase in structural dimensions or cost has not been prioritized.

The following main structural components has been subject to fatigue checks, where selected details for each component has been assessed:

- Bridge girder
- Mooring lines
- Stay cables
- Bridge girder / column connection
- Pontoon / column connection

The following structural analyses has been utilized to derive stress ranges to be used in the fatigue checks:

- Global analysis
- Bridge girder local analysis (multiple hot spots)
- Bridge girder / column connection local analysis
- Pontoon / column connection local analysis

2 Methodology

2.1 General

An overview of the fatigue calculation procedure is shown in Figure 2-1. Additional local analyses not shown in this overview has been performed to derive stress concentration factors (SCFs).

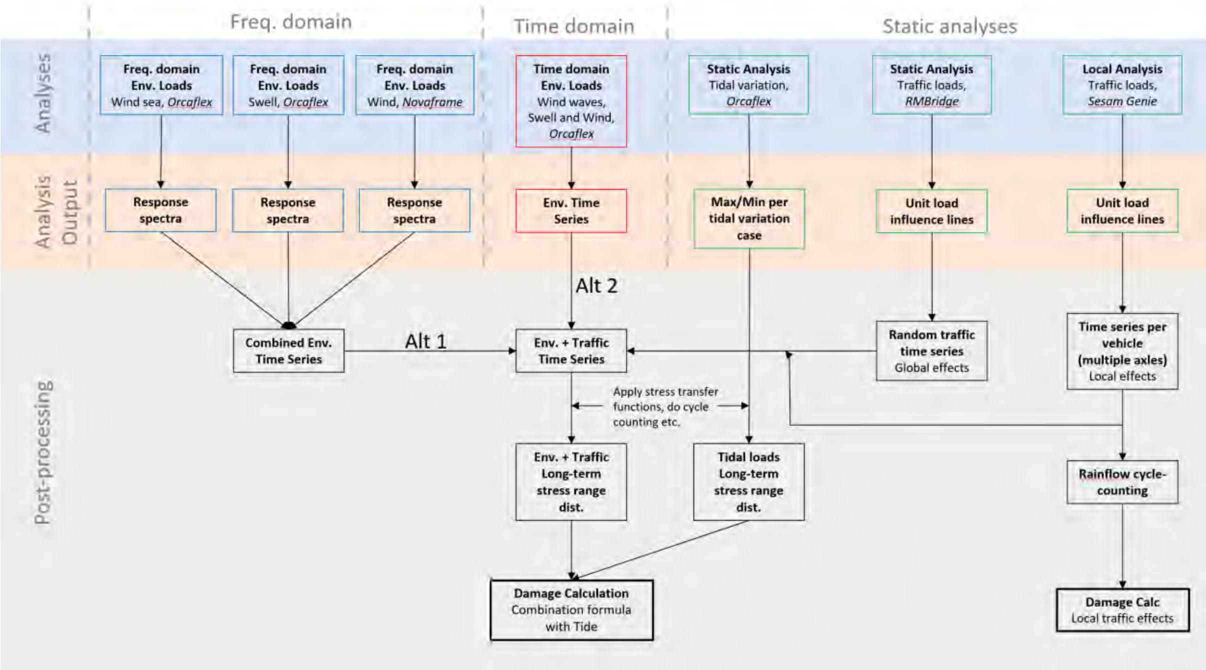


Figure 2-1 Overview of fatigue calculations

2.2 Global vs. local load effects

In the fatigue assessment a distinction is made between global and local load effects, and global and local analysis. The global analysis model is used to derive (nominal) global load effects. A local analysis however, can be used for both global and local load effects:

- Global load effects: A local analysis model is used to derive hotspot stresses due to global load effects, e.g. stress transfer functions/coefficients between global sectional forces and local hotspot stresses. I.e., only local geometric effects are addressed in this type of analysis.
- Local load effects: When load effects due to local loads are not included in the global analysis, a separate local analysis/calculation is used to calculate local stresses (in stiffened panels etc). The local load effects that has been considered in this assessment is from traffic loads on the bridge girder.

2.3 Global load effects

2.3.1 Environmental loads

As can be seen in Figure 2-1 two alternatives are presented for establishing load effects from environmental loads.

The first alternative is based on individual frequency domain analyses for each environmental load group (wind sea, swell and wind). The total load effect for environmental loads are established subsequently by generating random realizations in the time domain for each load group and then taking the sum of these (see Section 4.1 below for how the environmental cases are combined). Coupled effects between the different load groups are hence not included. For the Orcaflex analyses (wind sea and swells) phase information is used when combining stress contributions from each force component. For the Novaframe results no phase information is available and random phase angles are used when combining stress contributions from each force component. The frequency domain method is fast and well suited for rapid design iterations.

The second alternative is coupled time domain analyses where all load groups are applied simultaneously. This is a more accurate method and will primarily be used for verification of the frequency domain approach.

Stiffness reductions in the bridge girder due to shear lag have been accounted for in the global analysis by shear lag factors applied to the stress coefficients, reference is made to [2].

The simulation time for each environmental condition has been taken as 1 hour. Analysis models are not described in further detail here, but reference is made to [3].

2.3.2 Traffic loads

Global load effects from traffic has been accounted for by establishing synthetic timeseries of the sectional forces. This allows for direct combination with environmental loads prior to stress cycle counting and a stochastic combination of vehicle loads in both bridge directions.

The timeseries are established based on unit-load influence lines for the sectional loads. For the global loads it is considered adequate to represent a single lorry with a point load (considered to give results to the safe side). One set of influence lines per lane is used.

A typical influence line for weak axis bending in the low part of the bridge is shown in Figure 2-2. This influence line can be converted to a timeseries for a single lorry driving across the bridge by scaling according to the total lorry weight and resampling according to lorry speed and a chosen timestep. Full traffic simulations for a specific traffic load model and for a given period of time can be established by combination of several single-lorry timeseries. A one-hour simulation of the weak axis bending moment at a fixed position of the bridge is shown in Figure 2-3.

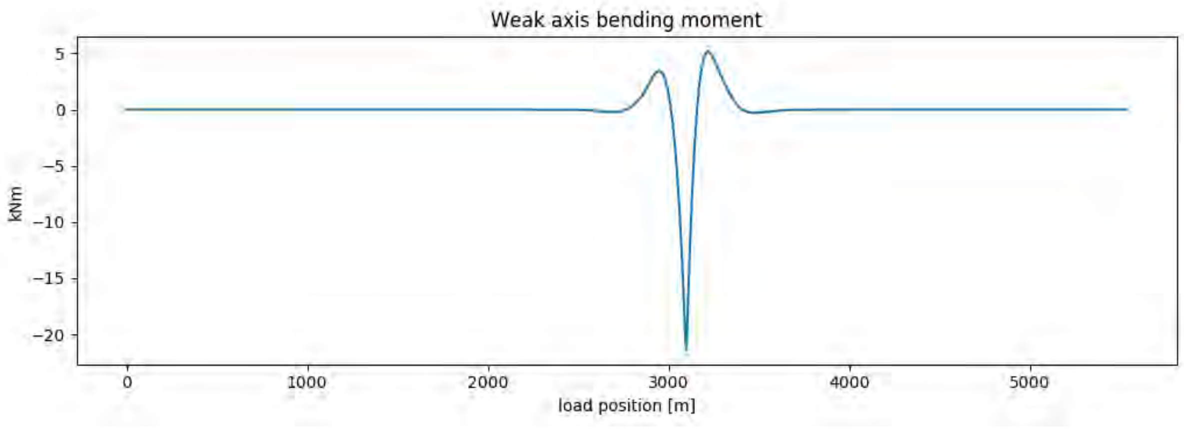


Figure 2-2 Influence line for weak axis bending moment at cross section in low bridge

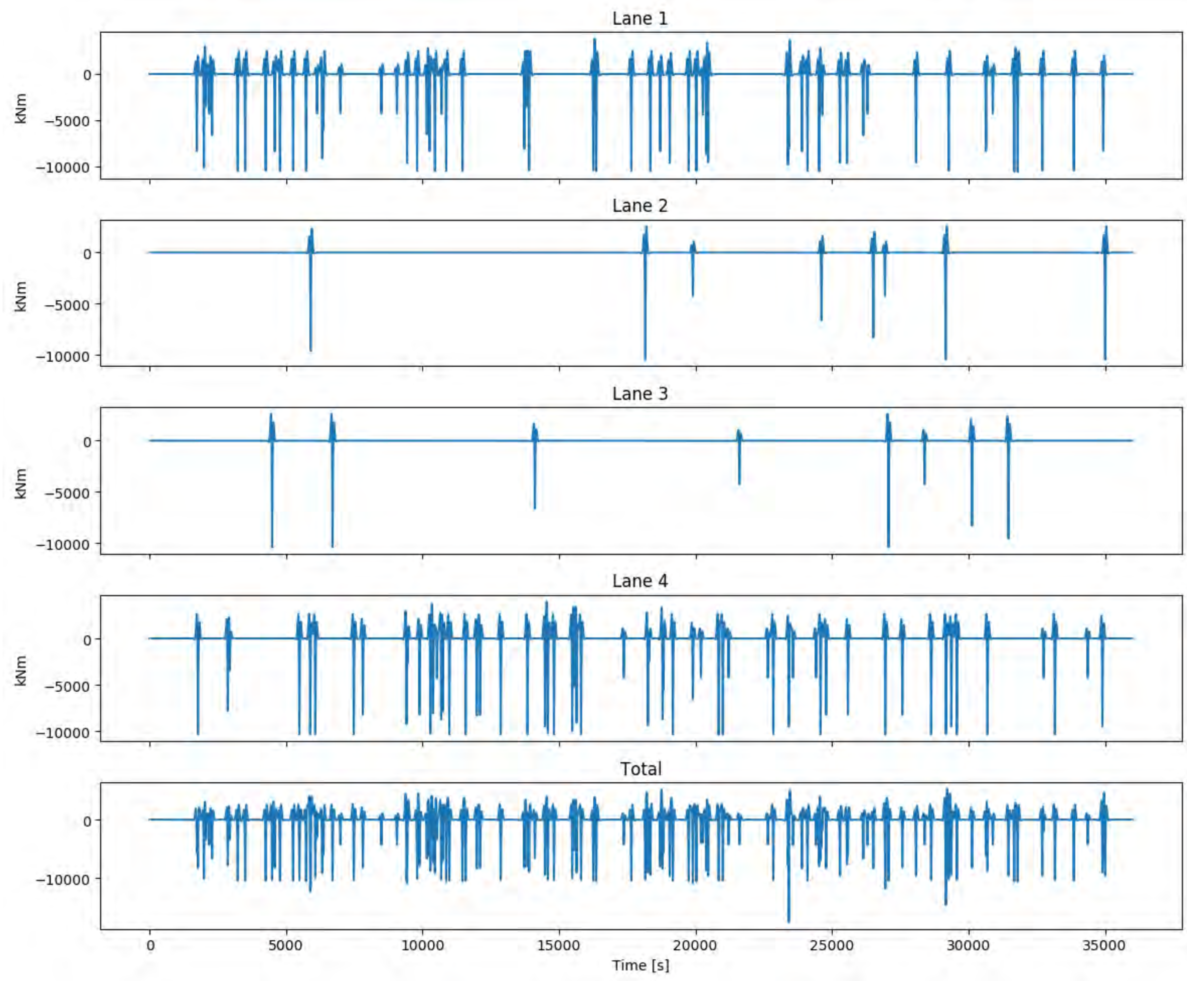
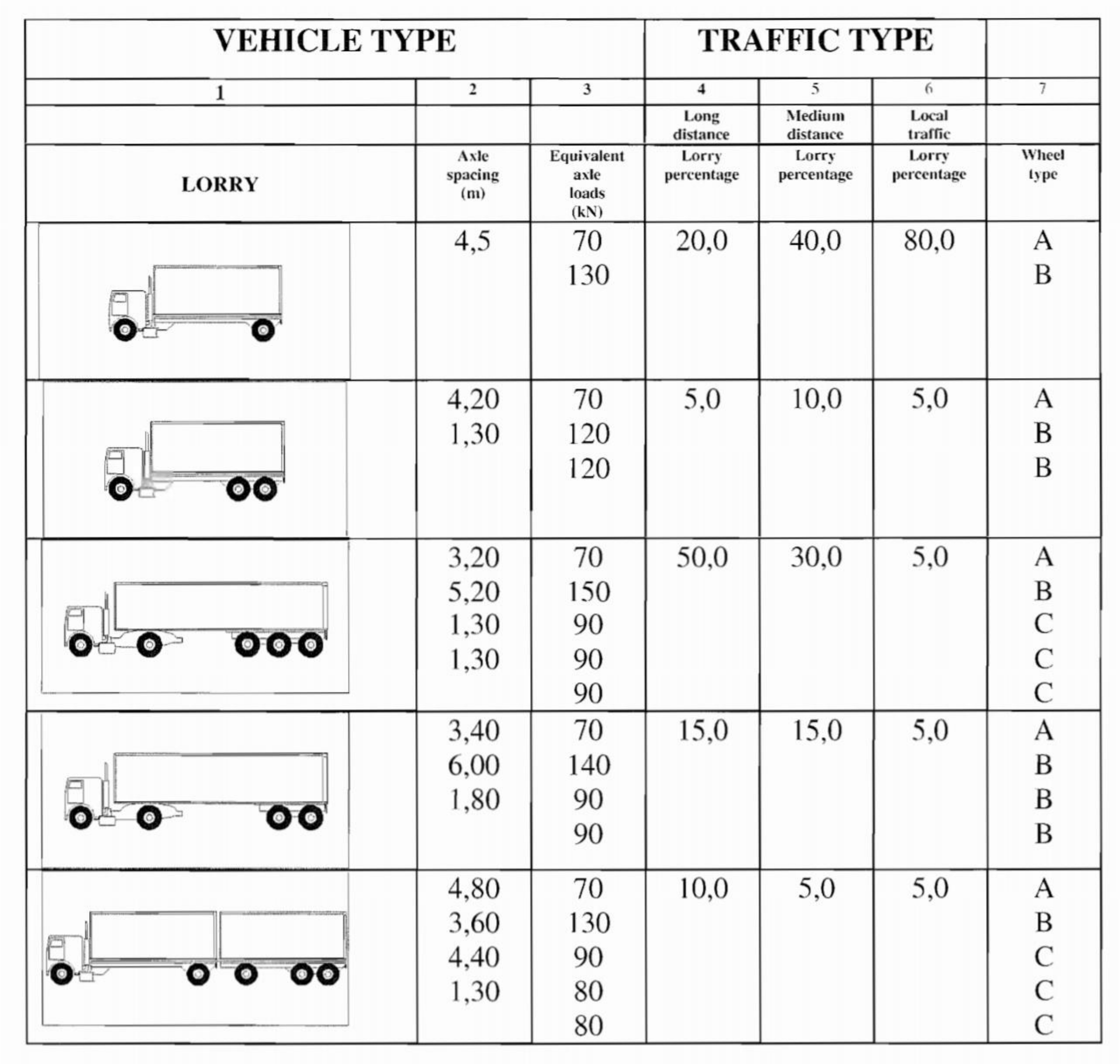


Figure 2-3 Weak axis bending moment from traffic simulation

The traffic simulations have been based on Fatigue Load Model 4 from the Eurocode (FLM4) with parameters taken from the design basis [4]. The load model consists of 5 equivalent vehicles as shown in Figure 2-4. As per the design basis the *Long distance* traffic type is used and the annual number of vehicles per slow lane is taken as N_{obs} = 0.5e6 (Traffic category 2).

10 one-hour realizations of traffic loads are used as basis for the fatigue calculations. When combined with environmental loads a random timeseries is drawn from these 10 realizations for each environmental case.

A constant lorry speed of 100 km/h has been used for all lanes and the minimum distance between lorries in the same lane is taken as 3 seconds (approximately 83 m). Otherwise the order and occurrence of lorries are distributed randomly throughout the realization.



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 2-4 Fatigue Load Model 4

2.3.3 Tidal loads

The distribution of tidal amplitudes given in Appendix D of the metocean design basis [5] is used as basis for establishing stress ranges from tidal variation. Each bin in the distribution is analysed and used as basis for calculating the equivalent stress range from tidal loads. The equivalent stress range is calculated with the following expression.

$$\Delta\sigma_{tide} = \left(\frac{\sum_{j=1}^{k} \Delta\sigma_{tide\ j}{}^{m} \cdot n_j}{\sum_{j=1}^{k} n_j}\right)^{1/m}$$

k – number of stress blocks

$\Delta\sigma_{tide\ j}$ – stress range in block j due to tidal variation

n_j – number of cycles in stress block j

m – negative inverse slope of S-N curve, 3.0 as it is assumed that the stress range due to tidal variation should be combined with the left part of the S-N curve.

2.4 Local load effects from traffic

2.4.1 Method

An influence line approach has been used to establish local load effects from traffic, based on the same philosophy as for the global load effects. A similar FE-model as used by DNVGL [6] has been used to establish the influence lines. The influence lines have been established by stepping a unit axle load along one of the lanes with a sufficiently small step.

The FLM4 traffic model uses three different axle types and one set of influence lines has been established for each axle type. The loads have been applied as surface pressure according to the footprints given in Table 4.8 in EC1 [7], however the areas are adjusted to account for the load spreading effect of the asphalt layer. The thickness of the asphalt layer is taken as 80 mm and the spreading angle is taken as 45 degrees.

Also, the transverse position of the vehicles is taken into account according to Figure 4.6 in EC1 [7]. This results in five sets of influence lines per axle type, which gives a total of 15 sets of influence lines.

Once the influence lines are established, the total timeseries for stresses for a single lorry can be established by scaling and superimposing the influence lines. The principle is illustrated in Figure 2-5 and Figure 2-6 where a dummy unit-load influence line is used to establish a timeseries for Lorry 3 from FLM4.

In order to arrive at correct timeseries for principal hotspot stress the timeseries must first be derived for the component stresses (σ_{xx}, σ_{yy} and τ_{xy} in the case of extreme fibre stress in shell elements) before calculating the principal stress.

The analysis for establishing the influence lines consists of a large number of load cases in order to get sufficient resolution in the influence lines. Scripting has been used both for the pre- and postprocessing for efficient execution of the calculations.

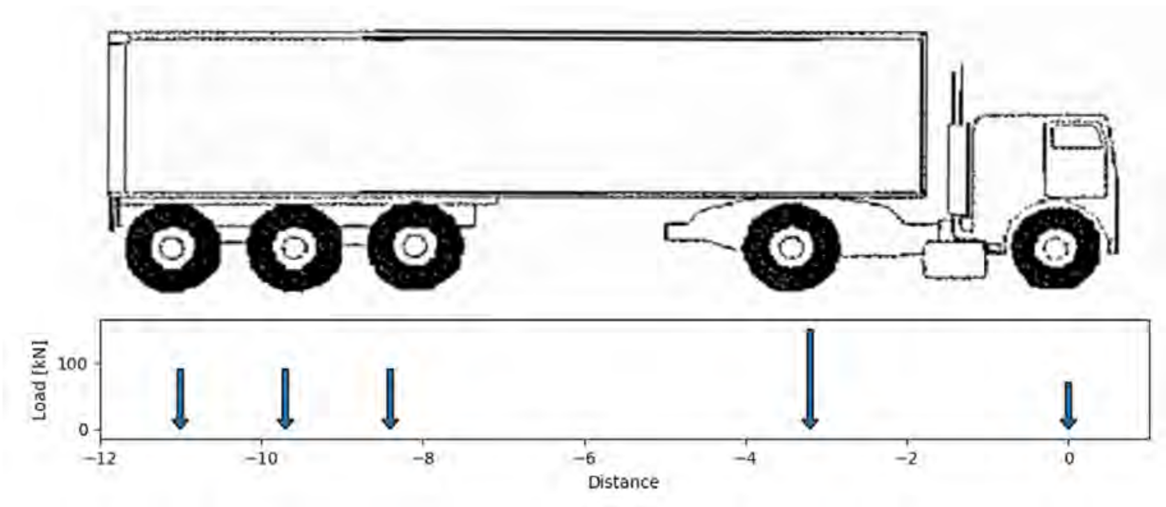


Figure 2-5 Axle loads for Lorry 3 in FLM4

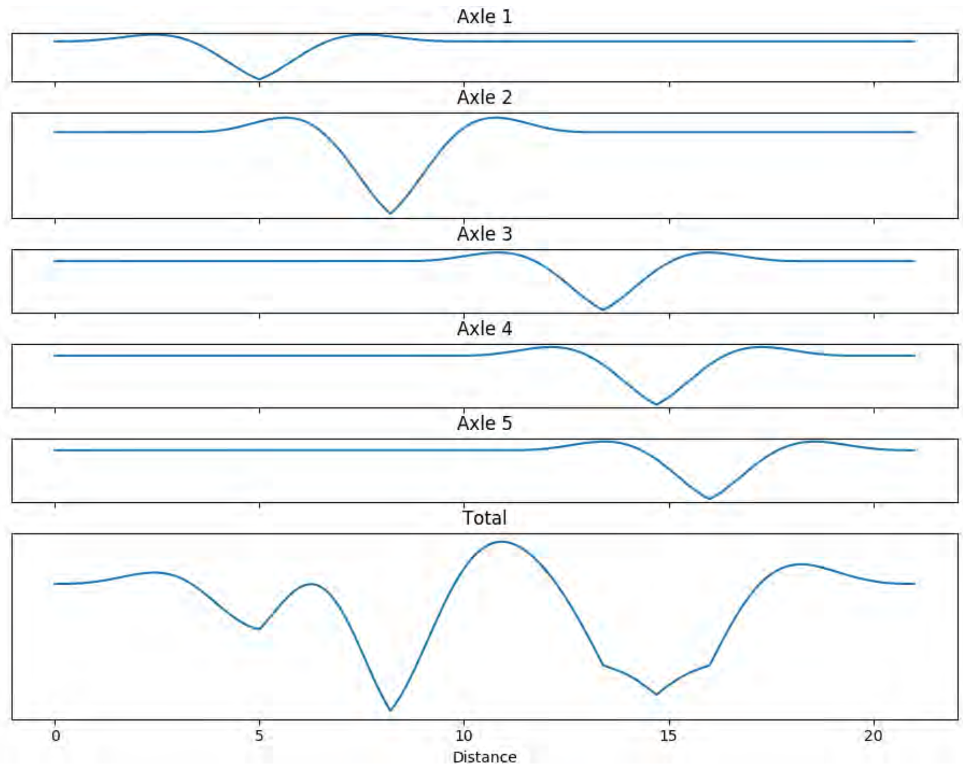
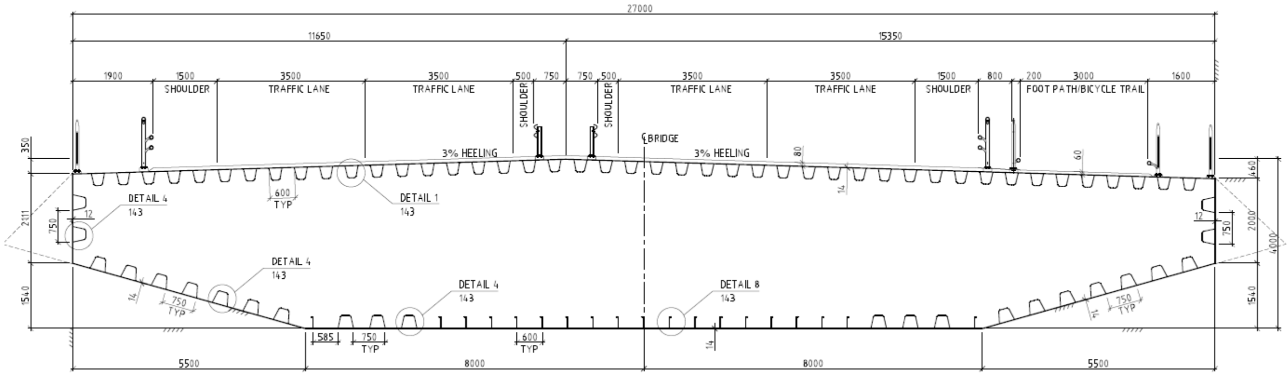


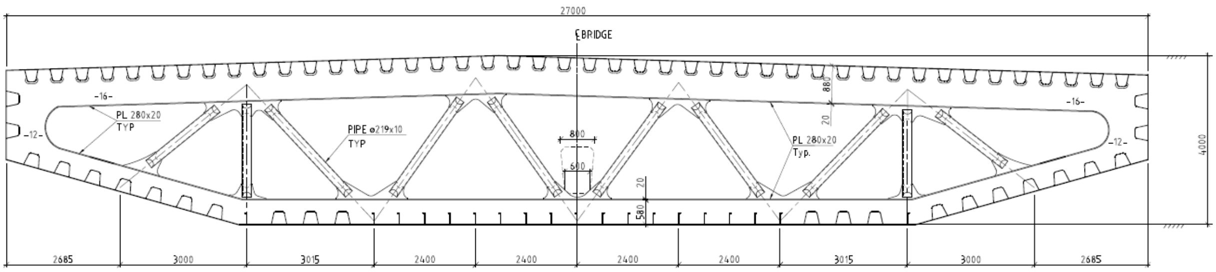
Figure 2-6 Derived stress series for Lorry 3 in FLM4 – The influence line is scaled according to each axle load and shifted according to the distance between axles before they are summed to a total stress series for the passing lorry.

2.4.2 FE analysis model

The local analysis has been performed with Genie/Sestra from the Sesam software package. The model is based on the midspan cross section of the K12 bridge concept as shown in Figure 2-7. The analyses have been performed with 16mm plate thickness in the deck plate. The Genie model is shown in Figure 2-8. An overview of the FE mesh is shown in Figure 2-9, and detailed views of the mesh in the area of interest are shown in Figure 2-10 and Figure 2-11. The chosen mesh size is considered suitable for capturing both membrane and bending stress components in the stiffener and deck plate shells. 2nd order “thick shell” elements are used. A typical footprint of the applied pressure loading is shown in Figure 2-12. Step size used for influence lines is 0.25 m.



SECTION A-A - 13-TEG-136/137
TYPICAL CROSS-SECTION AT MIDSPAN FOR K12
A1=1:50
A3=1:100



SECTION B-B - 13-TEG-136/137
TYPICAL CROSS-SECTION AT MIDSPAN FOR K12
A1=1:50
A3=1:100

Figure 2-7 Cross section used as basis for local traffic analysis

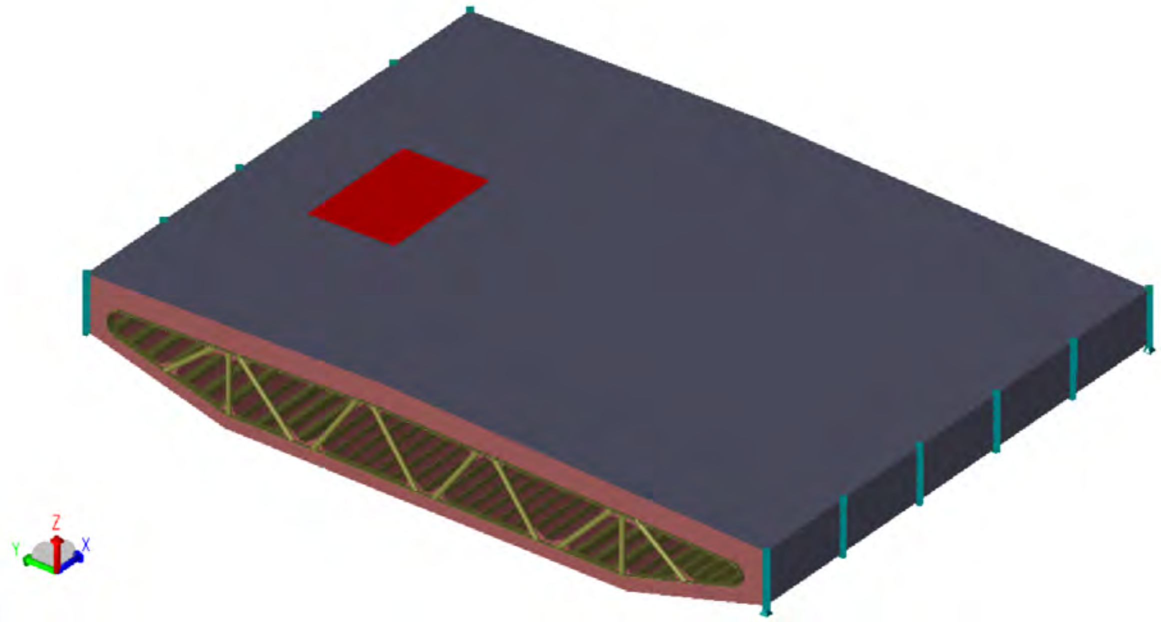


Figure 2-8 Genie model used for local traffic analysis. The area of interest with respect to local load effects from traffic is highlighted in red.

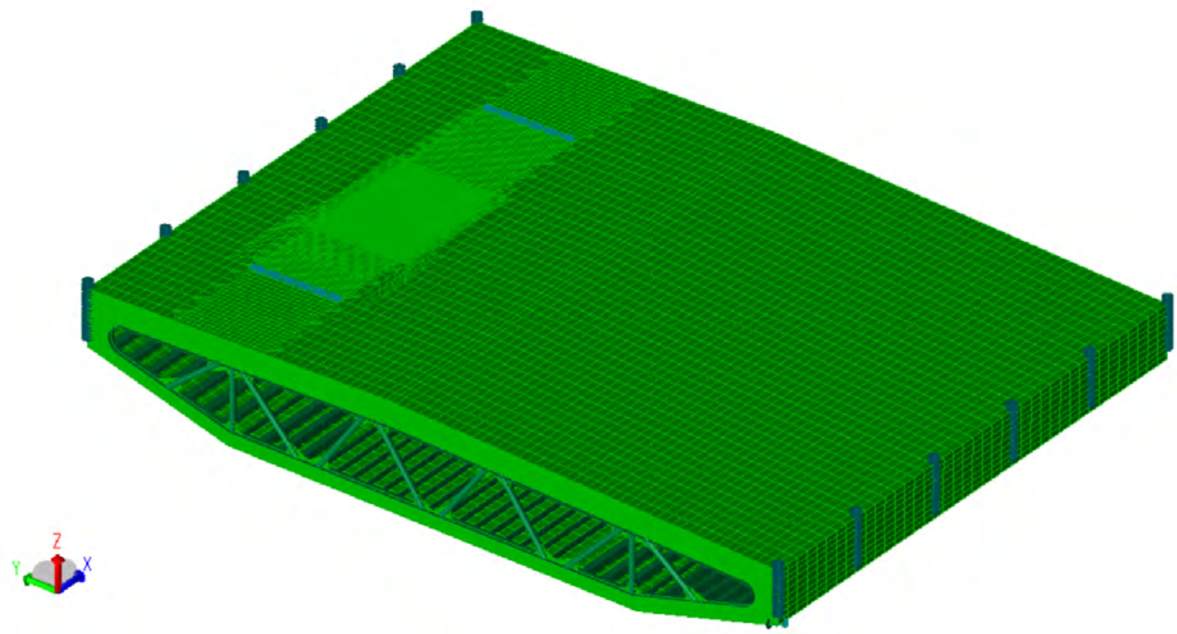


Figure 2-9 Finite element mesh of local traffic analysis model– overview

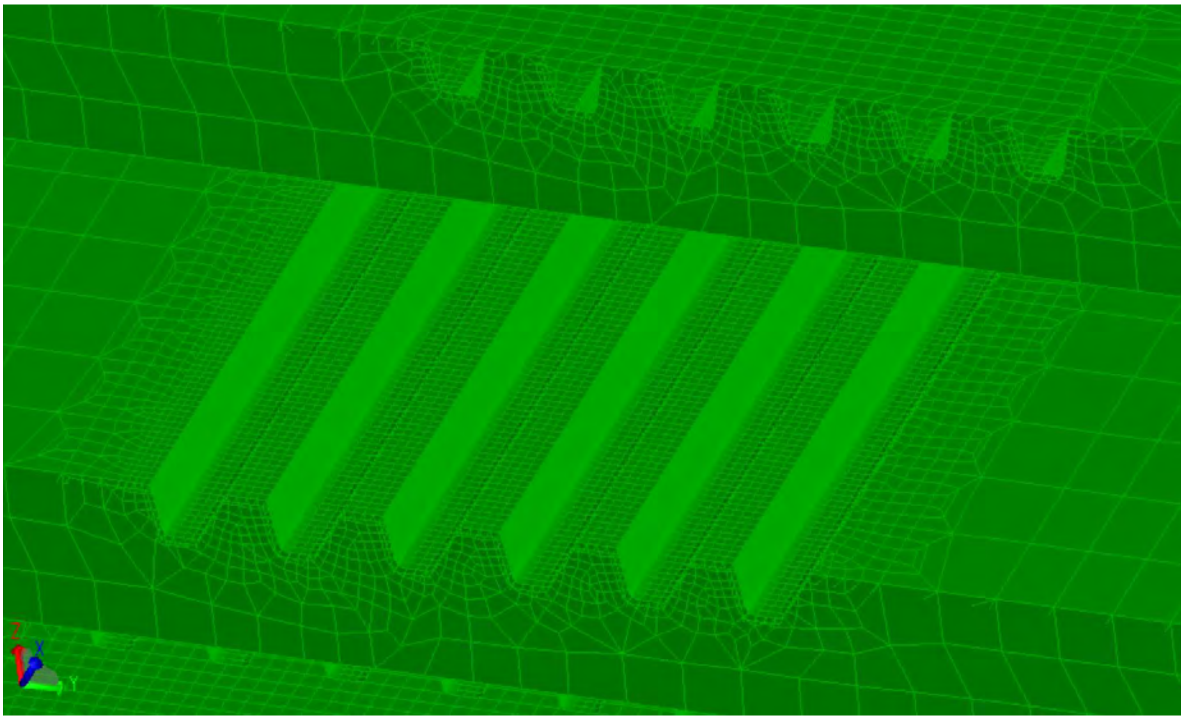


Figure 2-10 Finite element mesh of local traffic analysis model– area of consideration seen from below

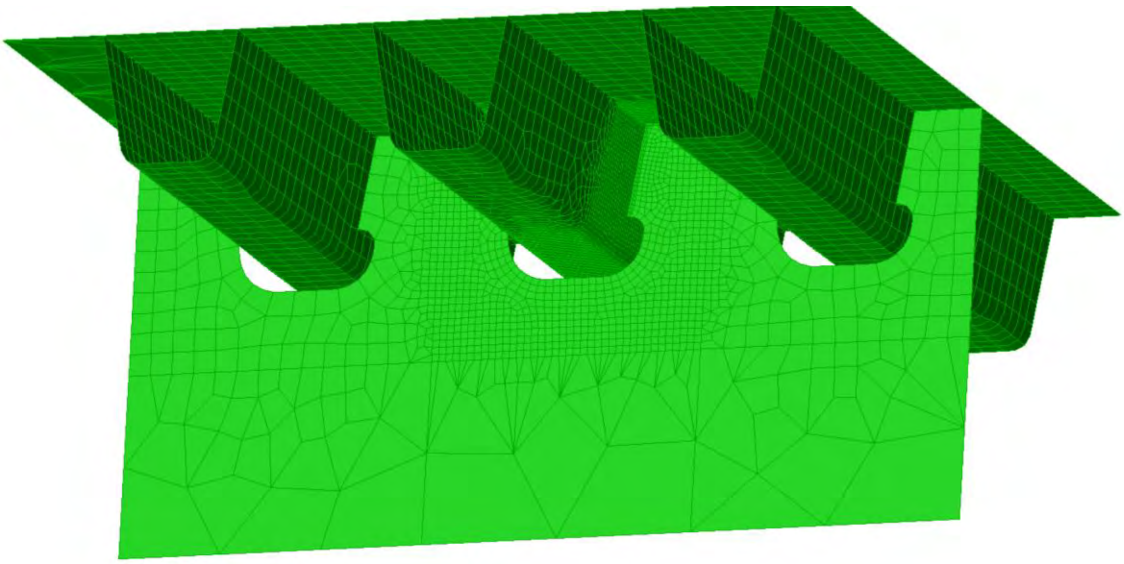


Figure 2-11 Finite element mesh of local traffic analysis model– mesh used when considering fatigue at stiffener to cross-beam connection – mesh size is t x t in area of interest.

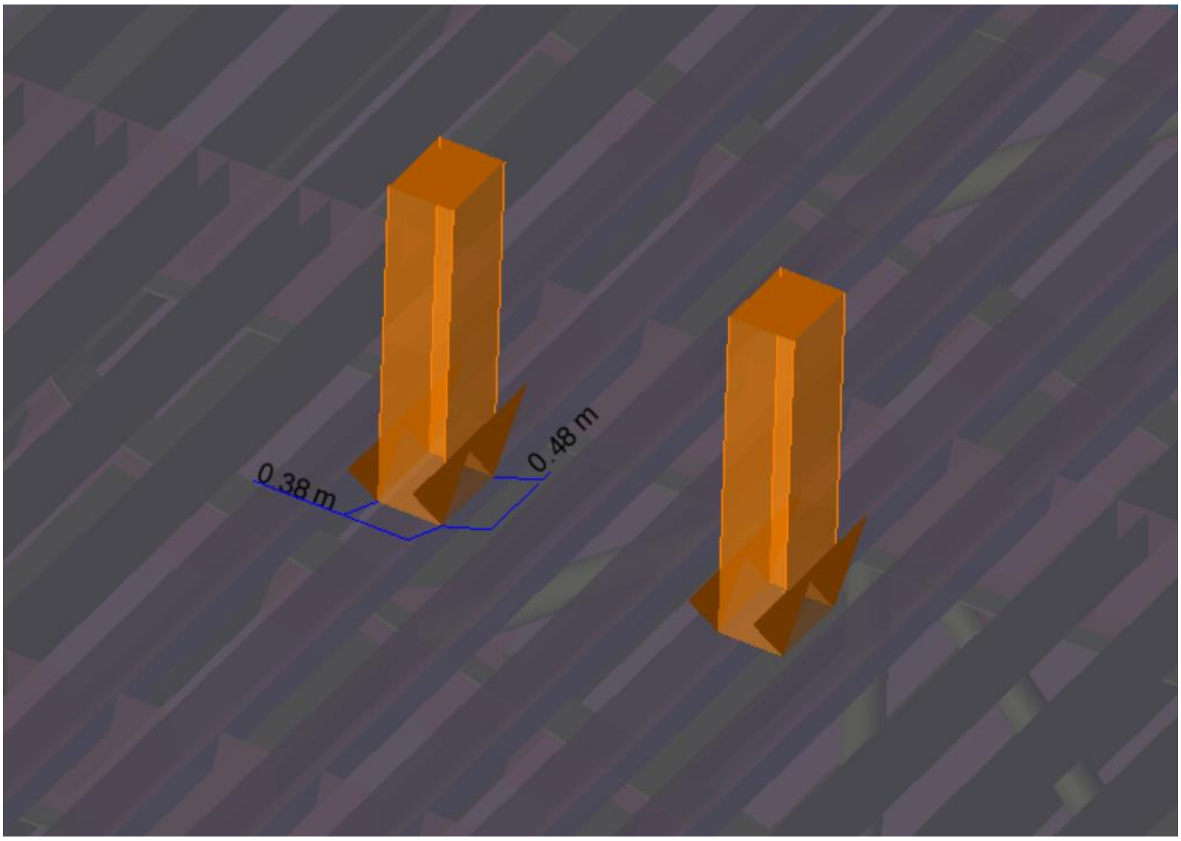


Figure 2-12 Wheel pressure loads for axle type A including load spreading effect of asphalt layer. The shown load is for the mid transverse position of the vehicle and is placed so that the outermost wheel pressure acts centrally above the outermost trapezoidal stiffener in the slow lane.

2.5 Combination of global and local load effects

Stress series for combined global and local effects due to traffic has been established by combining the traffic models for global and local load effects respectively described in Sections 2.3.2 and 2.4.

Once the stress series due to local effects has been established (for each lorry and transverse position) these can be added to the corresponding influence lines used in the global traffic model. This is illustrated in Figure 2-13.

The process of establishing random time realizations for combined global and local traffic follows the same procedure as for the traffic model for global effects described in Section 2.3.2. These traffic realizations can in turn be combined with stress series from environmental loads to get stress series for the combined effect of environmental loads, global traffic loads and local traffic loads.

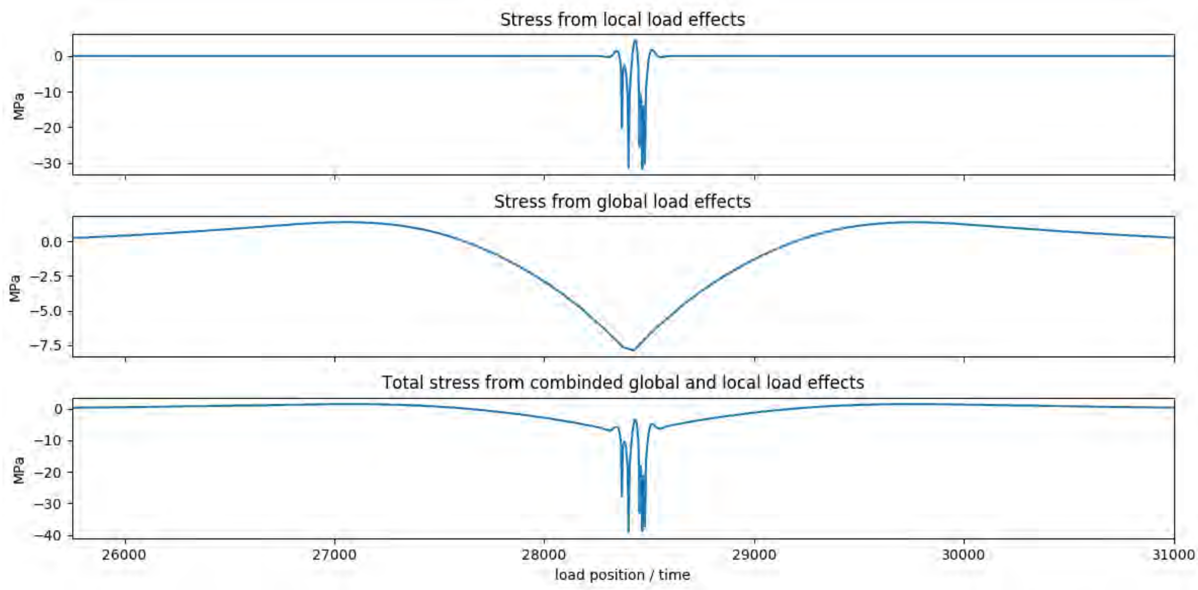


Figure 2-13 Typical stress influence line for combined local and global load effects from traffic

2.6 Fatigue damage calculation

The long-term distribution of stress ranges is established by performing rainflow cycle counting on the stress timeseries from the combined environmental and traffic cases. Based on this a histogram representing the long-term distribution of stress ranges can be established. A histogram for a typical point in the bridge girder subject to both global and local load effects is shown in Figure 2-14. The total annual fatigue damage due to global load effects with contribution from environmental loads, traffic and tide is calculated based on combination formula given in the design basis [4]:

$$D_{yrl} = f_t \sum_{j=1}^{k} \frac{1}{a} n_j \left(\Delta\sigma_{wtj} + \Delta\sigma_{tide}\right)^m + (1 - f_t) \sum_{j=1}^{k} \frac{1}{a} n_j \left(\Delta\sigma_{wtj}\right)^m$$

$\Delta\sigma_{wtj}$ is stress range in block j of stress range histogram, established with combined stress time series from environmental action and traffic.

n_j – annual number of cycles in stress block j, from long-term distribution of combined environmental action and traffic.

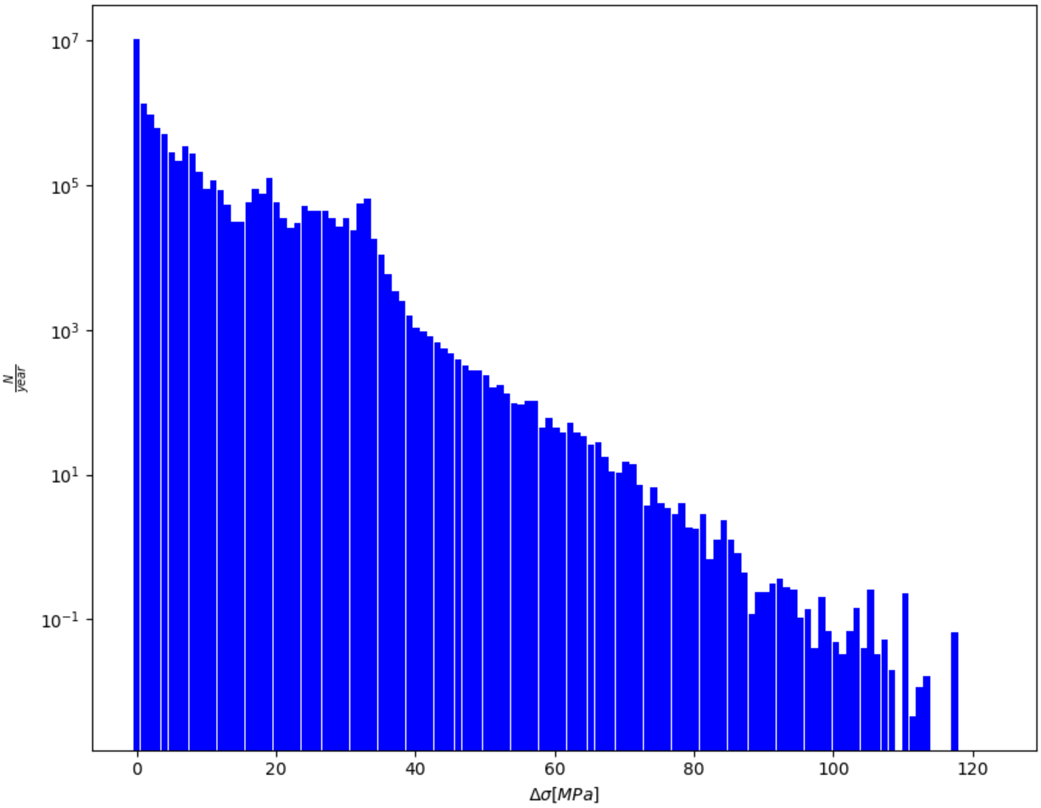


Figure 2-14 Annual stress range histogram for environmental and traffic loads. The contribution from traffic loads can be seen as additional stress cycles between 20 MPa and 40 MPa

3 Selected details

1.1 Bridge girder

The bridge girder has been subject to fatigue assessment by the designer in previous project phases [8] and also by third party DNVGL [6]. Essential findings and recommendations from this work has been taken into account when selecting details to be assessed in the current phase.

The structural configuration of the bridge girder deck with typical welding details is shown below in Figure 3-1. The following details are selected for fatigue calculation:

- Transverse plate welds outside traffic lanes (Detail type 1)
- Transverse plate welds inside traffic lanes (Detail type 2)
- Connection between transverse frame and trapezoidal stiffener (Detail type 3)
- Longitudinal weld in trapezoidal stiffener to deck plate joint (Detail type 4)
- Trapezoidal stiffener splice (Detail type 5)

Note that the longitudinal plate weld has not been considered during the current project phase. It is expected that these welds will not be governing for fatigue as long as they are placed outside of areas with direct traffic wheel loads. This means that the longitudinal welds should be placed near the centre of the traffic lanes or outside the traffic lanes. This is in agreement with the current fabrication plan as described in Appendix N [9].

For plate splices with stress normal to the weld the applicable S-N curve is dependent on the circumstance under which the weld is performed. It is expected that at least three different situations will be relevant:

1. Welding at fabrication yard (in shop). It is expected that welding in this situation will meet the requirements for using a D-curve and that misalignment will be within 2mm.
2. Assembly of larger section in sheltered waters. In this situation, less optimal welding conditions must be expected and an E-curve is considered appropriate. Experience from previous construction projects of similar type bridge girders indicate that misalignment below 2mm can be achieved. 2mm misalignment is hence also assumed for these welds.
3. Final assembly at the bridge location. A similar splicing method as for the sheltered waters situation is assumed and currently an E-curve is used along with 2mm misalignment.

For the current assessment, this differentiation has been taken into account for detail type 1, which has been assessed with both E- and D-curves.

For the trapezoidal to transverse frame connection (detail type 3), three different hotspots have been assessed (See Figure 3-4):

a) Weld between trapezoidal web and deck plate (weld toe on trapezoidal side)
b) Weld between transverse web plate and trapezoidal web at cut-out (weld toe of vertical weld on trapezoidal side)
c) Weld between transverse web plate and trapezoidal web at cut-out (weld toe of horizontal weld on trapezoidal side)

d) Free edge on cut-out

For the transverse weld details (detail type 1, 2a, 2b and 5) it is assumed that the transverse welds will be located close to the transverse frames and no more than 0.5m away.

The selected details are given in Table 3-1. Locations on the cross-section subject to fatigue checks are shown in Figure 3-2.

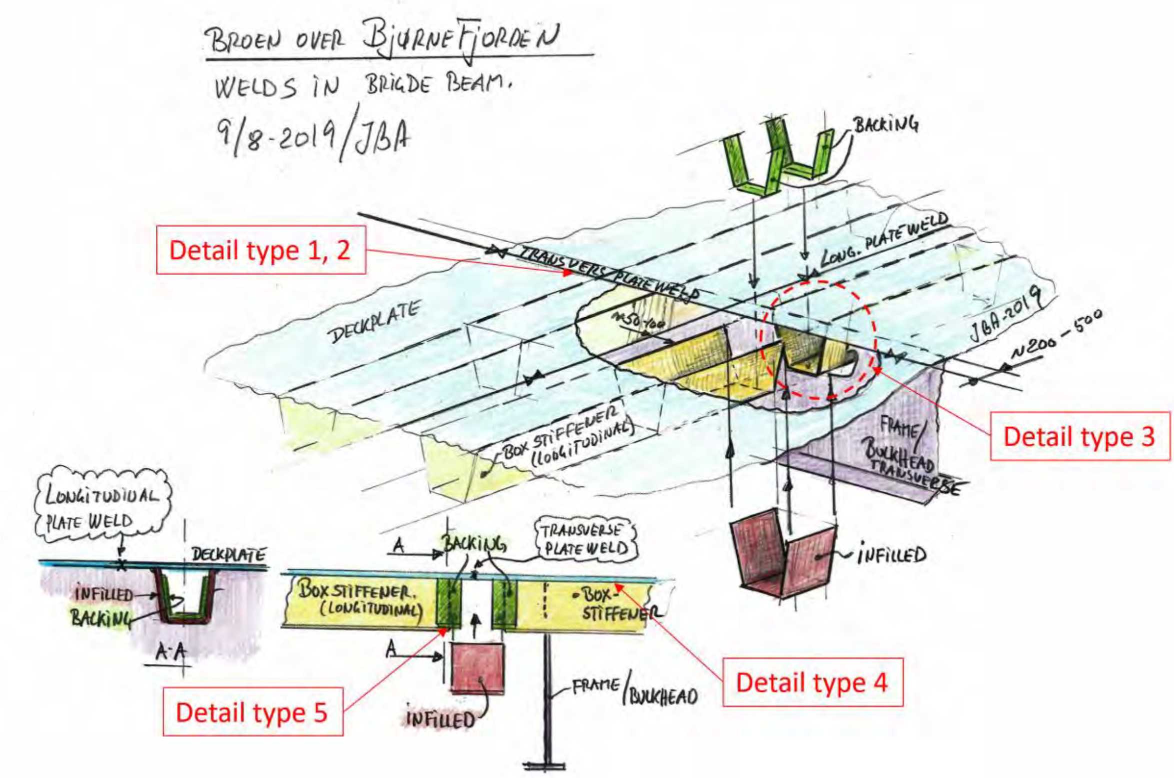


Figure 3-1 Structural arrangement for bridge girder deck showing typical weld details

Table 3-1 Structural details in the bridge girder to be assessed for fatigue

Detail type no.	Description	Load effects	Hotspot method	S-N curve	DFF
1	Transverse Plate splice outside traffic lanes	Global only (environmental, traffic, tide)	Nominal girder normal stress with SCF. For screening the following SCF's has been applied: • 1.20 (12mm to 12mm splice) • 1.40 (12mm to 14 mm splice) The SCF of 1.40 is applied at transition between cross section types	D (shop) E (site)	2.5

Detail type no.	Description	Load effects	Hotspot method	S-N curve	DFF
2a	Transverse Plate splice inside traffic lanes with longitudinal stress. See Figure 3-3.	1. Global (environmental, traffic, tide) and 2. Local (traffic)	1. Nominal girder normal stress with SCF combined with, 2. hotspot stress from local traffic analysis (longitudinal components)	D	2.5
2b	Transverse Plate splice inside traffic lanes with transverse stress. See Figure 3-3.	Local (traffic)	hotspot stress from local traffic analysis (transverse components)	C2	2.5
3	Trapezoidal stiffener to web frame connection. See Figure 3-4.	Local (traffic)	hotspot stress from local traffic analysis	C (free edge) D	2.5
4	Trapezoidal longitudinal weld. See Figure 3-5.	Local only (traffic)	Bending stress in web of trapezoidal stiffener at connection to top plate	F	2.5
5	Trapezoidal splice. See Figure 3-6.	1. Global (environmental, traffic, tide) and 2. Local (traffic)	1. Nominal girder normal stress, 2. Stiffener bending stress at flange from local traffic analysis	F	2.5

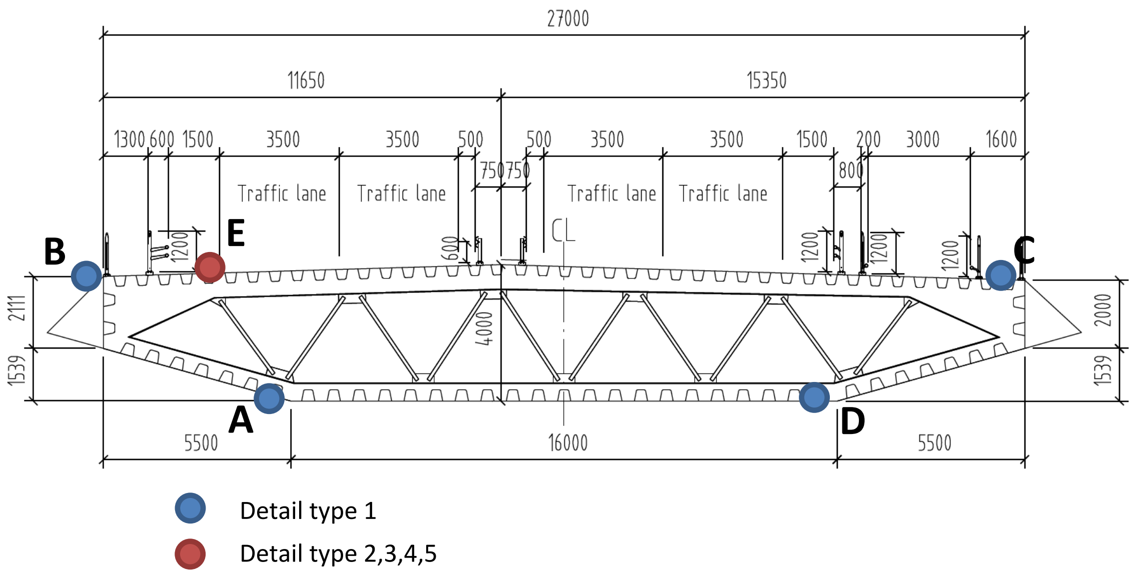


Figure 3-2 Locations on bridge girder subject to fatigue checks – Points are identified by letters A to E

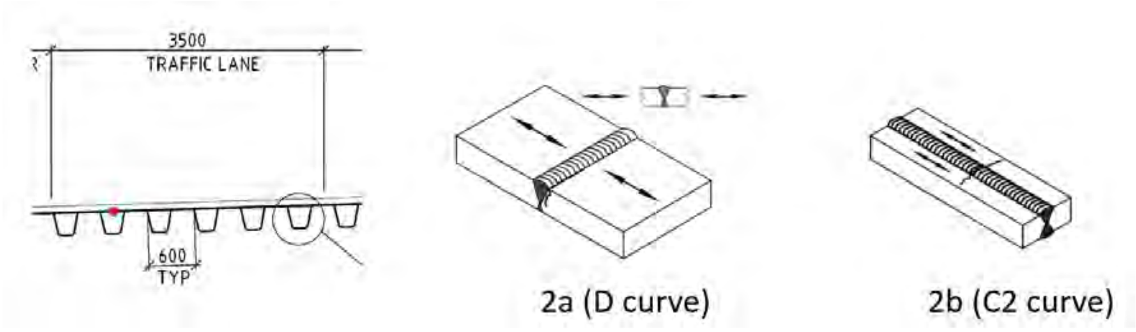


Figure 3-3 Location of hotspot used for assessment of detail type 2a and 2b. This is at the westerly wheel position in the westerly slow lane.

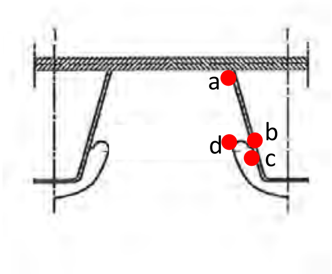


Figure 3-4 Location of hotspot used for assessment of detail type 3

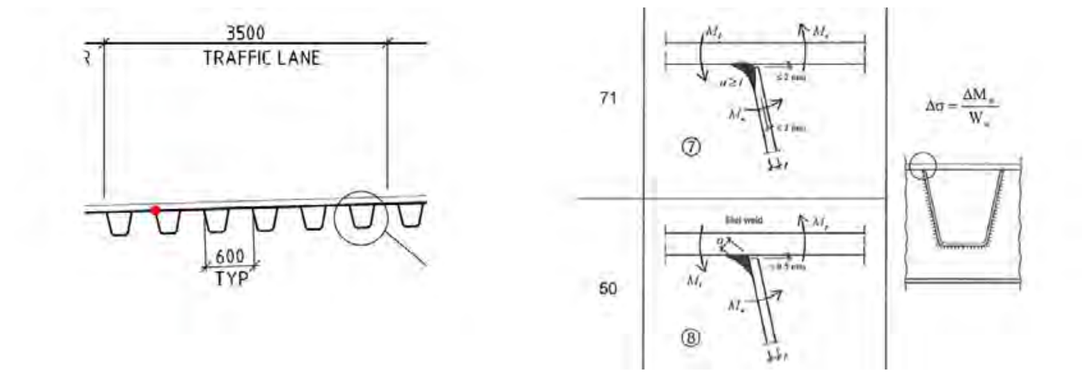


Figure 3-5 Location of hotspot used for assessment of detail type 4. This is at the westerly wheel position in the westerly slow lane

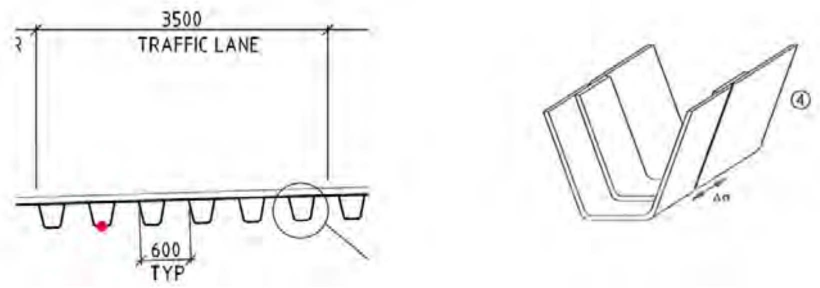


Figure 3-6 Location of hotspot used for assessment of detail type 5. This is at the westerly wheel position in the westerly slow lane

1.2 Mooring lines

Fatigue calculations for the mooring system has been calculated according to DNVGL-OS-E301 [10] and BV NI604 [11]. The selected details are given in Table 3-2.

Cross section area used for stress calculations are based on a nominal chain diameter of 147 mm subtracted by corrosion allowance corresponding to mid-life of the chains. Top chains have been subtracted by 25 years of corrosion allowance (assuming one replacement after 50 years) while the bottom chains have been subtracted by 50 years of corrosion allowance (no chain replacement). The applied corrosion allowance is 0.2 mm/year. This gives an effective chain diameter for fatigue stress calculation of 142 mm and 137 mm for top and bottom chains respectively.

For the tension fatigue calcs, out-of-plane bending (OPB) has been taken into account in the fatigue analysis by an SCF of 1.15 on the nominal axial stress.

OPB/IPB fatigue has been calculated based on simplified method given in BV NI604 [11]. The method is described in Appendix M Mooring system [12].

Table 3-2 Mooring line details to be assessed for fatigue

Detail type no.	Description	Load effects	Hotspot method	S-N curve	DFF
1	Tension fatigue, Mooring line chains	Global only (environmental)	Nominal stress from tension with SCF = 1.15 to account for OPB/IPB	Single slope S-N curve for studless chain with a_0 = 6e10 and m = 3	10
2	OPB/IPB fatigue, Mooring line chains	Global only (environmental)	Simplified BV-method for OPB/IPB	Single slope S-N curve for studless chain with a_0 = 6e10 and m = 3	10

1.3 Stay cables

Fatigue in stay cables have been assessed by DNVGL [8] for the side-anchored alternative from phase 3. There it was found that the fatigue damages in the cables calculated from environmental loads and tidal loads are small, and that sufficient margins for damage from traffic is available. A similar check based on appropriate detail categories given in EN-1993-11 [13] has been performed but with traffic loads included. The applied S-N curve is shown in Figure 3-7.

Table 3-3 Stay cable details to be assessed for fatigue

Detail type no.	Description	Load effects	Hotspot method	S-N curve	DFF
1	Cable end points	Global only (environmental, traffic, tide)	Nominal stress from cable tension	Eurocode detail category 150	2.5

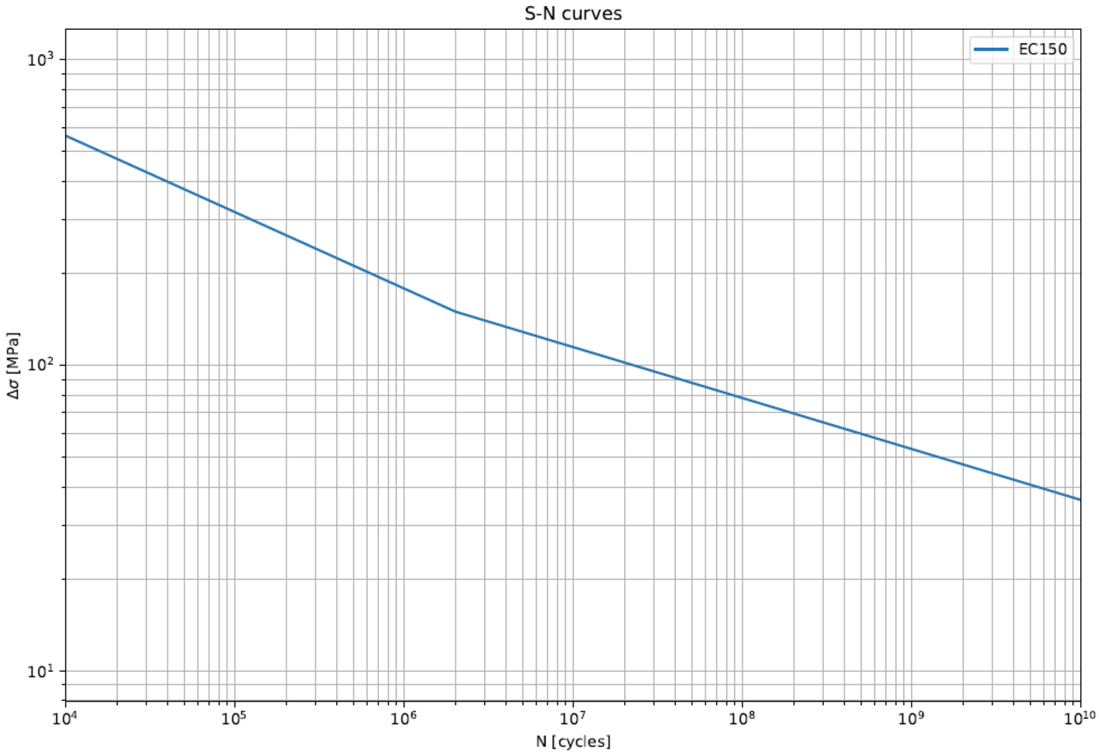


Figure 3-7 S-N curve used for fatigue calculations of stay cables. The curve is a two-slope curve with slope parameter 4 in the upper part and 6 in the lower part. The knuckle is at 2e6 cycles where the stress range is 150 MPa.

1.4 Bridge girder / column / pontoon connections

The bridge girder to column and column to pontoon connections have been assessed by local FE-analysis to derive SCF's for hotspots near the column corners as shown in Figure 3-8.

The assessed locations are where the splice between the cast piece and the girder/column plating is expected to be. For the current assessment a simplified approach has been taken where SCFs are derived from FE-analysis and coupled to the nominal beam stress in the bridge girder or column. Relevant parameters to be applied is given in Table 3-4.

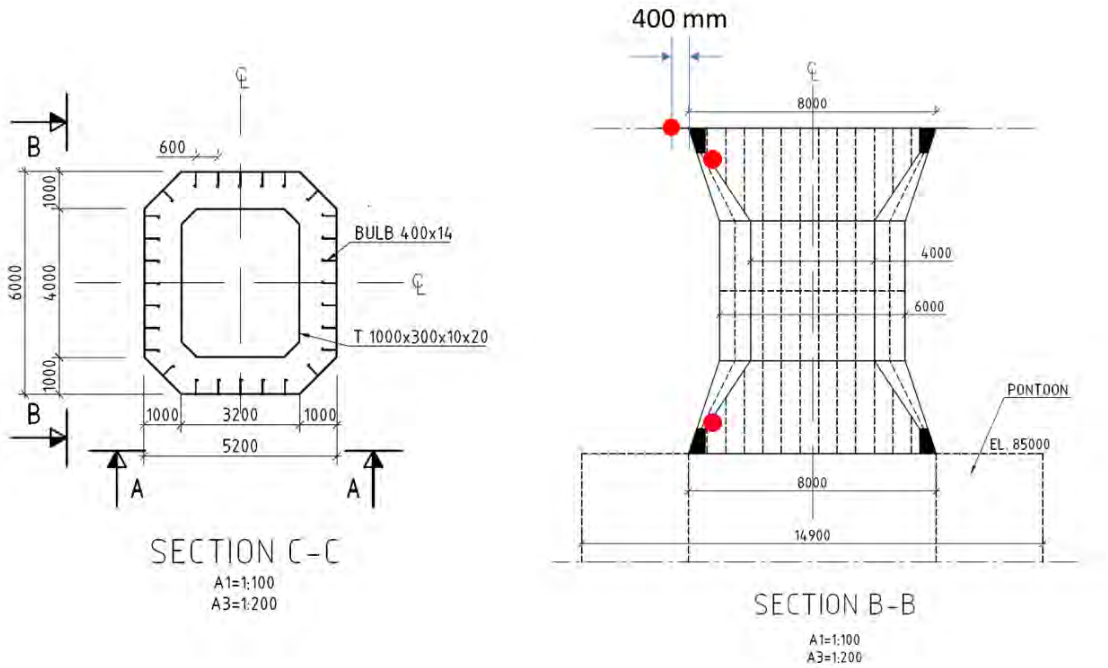


Figure 3-8 Areas of interest for bridge girder to column and column to pontoon fatigue analysis

Table 3-4 Details to be assessed in bridge girder to column connection

Detail type no.	Description	Load effects	Hotspot method	S-N curve	DFF
1	Weld between cast piece and bridge girder	Global only (environmental, traffic, tide)	Nominal girder normal stress at points A and D in the bridge girder with SCFs to account for local geometric effects and misalignment	D	2.5
2	Weld between cast piece and column top	Global only (environmental, traffic, tide)	Normal stress at column top corners with SCFs to account for local geometric effects and misalignment	D	2.5
3	Weld between cast piece and column bottom	Global only (environmental, traffic, tide)	Normal stress at column bottom corners with SCFs to account for local geometric effects and misalignment	D	2.5

4 Loads and assumptions

4.1 Combination of environmental cases

Wind sea and wind are combined according to the correlation table for significant wave height and wind direction given in Table 9 in the Metocean design basis [5]. One wind sea case is defined for each block in the directional wave scatter diagrams, totally 420 cases. The associated wind speed to each wind sea case is then taken from the correlation table.

One swell case is defined for each block in the swell scatter diagram, totally 215 cases. For swells no detailed correlation data is currently provided, but the metocean design basis dictates that swells shall only be combined with wind seas from westerly directions.

According to the swell scatter diagram, swells occur only 39 % of the time, which means that only a subset of the environmental cases will include swell.

The chosen strategy for combination of wind sea and swell is to combine high probability wind sea cases with high probability swell cases. This is achieved by sorting the wind sea and swell cases on probability of occurrence and combining in this order. Wind sea cases are also filtered so that only westerly directions are combined with swell. With the particular long-term distribution applied here, all swell cases are now associated with a wind sea case with higher probability. We now have 215 total environmental cases representing 39% of the time. The remaining total cases will be the 420 wind sea and wind cases where probabilities for the cases already combined with swell is reduced.

The combined environmental cases are given in Enclosure 3.

4.2 Environmental parameters

One environmental case is defined for each bin in the wind sea and swell scatter diagrams. Chosen parameter for individual cases are given in Table 4-1.

Table 4-1 Chosen parameters for individual environmental cases

Env. Parameter	**Wind Sea**	**Swell**	**Wind**
Hs	Bin upper value	Bin upper value	-
Tp	Bin mid value	Bin mid value	-
Direction	Sector mid	Random direction inside 300-330 sector	Sector mid
Wind speed	-	-	Hs correlated value

5 Results

Throughout this chapter, fatigue results are presented as *design fatigue lives*. This means that the presented values include the Design fatigue factor (DFF). The target design fatigue life is 100 years, except for the mooring line top chains which have a target design fatigue life of 50 years (1 planned replacement during the life time of the bridge).

5.1 Bridge girder

5.1.1 Detail type 1: Butt welds - global load effects (Screening)

A full screening has been performed for detail type 1. Below in Figure 5-1 is shown the calculated design lives the K12 concept with contributions from all load groups. In the floating part of the bridge hotspots at support (above columns), midspan and at transition between cross section types have been reported, in the stay cable part of the bridge hotspots at approximately every 40 m have been reported. The screening was carried out with SCF's as given for detail type 1 in Table 3-1. Detailed results from the screening are given in Enclosure 1 where fatigue from the individual load groups are shown along with additional results at the selected sections.

Since the screening was performed, further design development has been performed for the K12 concept. The screening is used as basis for selecting as set of sections along the bridge to perform updated fatigue calculations for the selected fatigue details in the bridge girder. A summary of results at the selected sections is given in Section 5.1.2 for detail type 1 with updated SCF's to reflect the latest design development and based on the most recent analysis models.

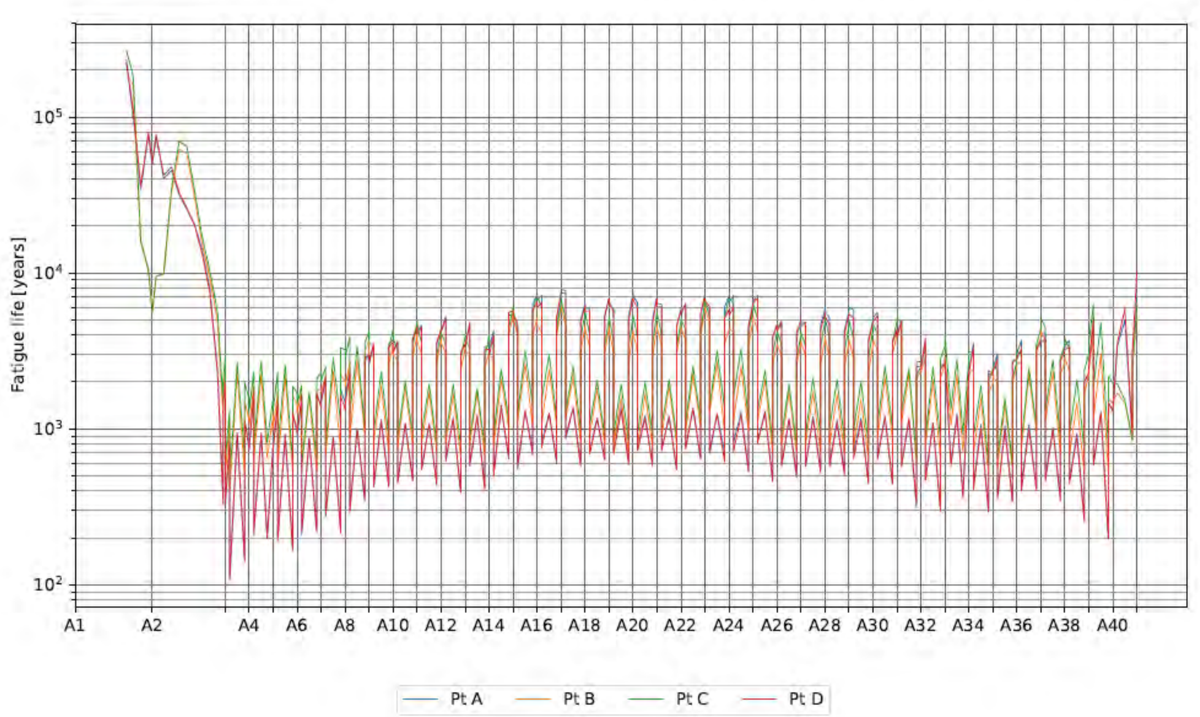


Figure 5-1 Design fatigue life in bridge girder – Screening results

5.1.2 *Detail type 1: Butt welds - global load effects (Selected sections)*

Here is given results for detail type 1 for selected sections along the bridge with updated SCF's to reflect the latest design development. The SCFs are calculated based on formula 3.1.2 in DNVGL-RP-C203 [14] with parameters as given in Table 5-1. The applied SCFs are given in Table 5-2.

Table 5-1 Parameters used for SCF calculations for butt welds

δ_m	2 mm
δ_t	½ (T- t) Note: it is assumed that in the final design the maximum thickness change at a plate splice will be no more than 2 mm. T is therefore corrected in the SCF calculations to be maximum 2 mm larger than t.
δ_0	0.05t

Table 5-2 SCFs for selected section along the bridge

Section location	Stress point				
	A	B	C	D	E
Support A3	1.12	1.23	1.23	1.12	1.23
Transition 1 near A3	1.38	1.44	1.44	1.38	1.23
Transition 2 near A3	1.44	1.28	1.28	1.44	1.23
Midspan A3-A4	1.28	1.28	1.28	1.28	1.23
Support A35	1.15	1.28	1.28	1.15	1.23
Transition 1 near A35	1.38	1.28	1.28	1.38	1.23
Transition 2 near A35	1.53	1.53	1.53	1.53	1.23
Midspan A38-A39	1.35	1.35	1.35	1.35	1.23
Support A40	1.15	1.28	1.28	1.15	1.23
Transition 1 near A40	1.38	1.28	1.28	1.38	1.23
Transition 2 near A40	1.53	1.53	1.53	1.53	1.23
Midspan A40-A41	1.15	1.28	1.28	1.15	1.23

Calculated fatigue lives including contributions from all relevant load effects are shown in Figure 5-2 and Table 5-3. The worst locations are stress points A and D (lower flange) at plate thickness transitions (Transition 2) near axis 40 in the north part of the floating bridge. The calculated fatigue life at this location is 157 years based on S-N curve D. At this location, an SCF of 1.53 is used corresponding to a thickness transition from 14mm to 12mm.

The calculations have also been performed with an E-curve which is relevant for welds performed at site, and the calculated fatigue life at the worst location is then 97 years. All other sections have sufficient fatigue life. If required, only minor design changes will be required to achieve sufficient fatigue life for the plate splices performed at site.

To improve the fatigue life in the upper part of the floating bridge a stronger field cross section has been used with 14mm plating instead of 12mm for the bottom plating of the girder, reducing the SCFs at points A and D to from 1.53 to 1.44.

It is seen that at Axis 3 the worst point on the cross section is point E with a calculated fatigue life of 278 years. This means that fatigue at this section is governed by weak axis bending and that the topmost point near the center of the cross section is likely to be even worse and should hence be checked for detail type 1 in future fatigue work. It also means that when considering combined global and local traffic point E may not be the worst point at Axis 3 and a point under the inner wheel of the slow lanes should also be checked.

Detailed results for the selected sections based on S-N curve D are given in Enclosure 2 where fatigue from the individual load groups are shown along with additional results at the selected sections.

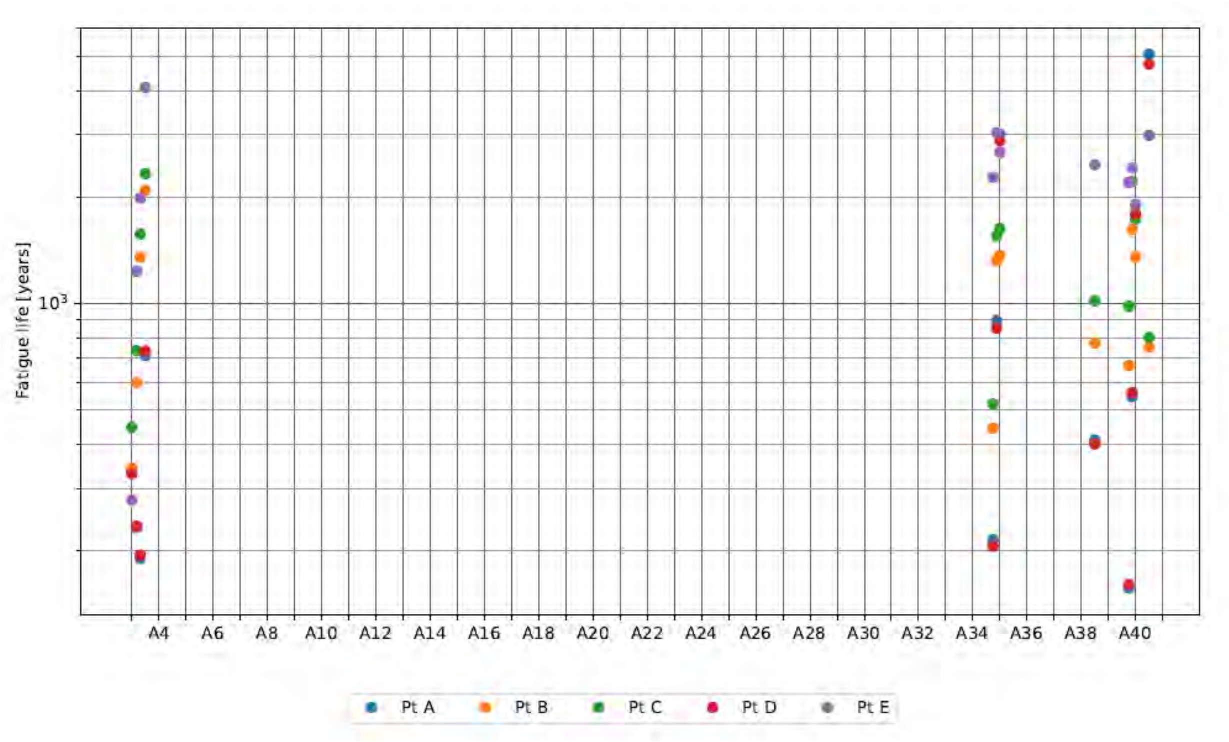


Figure 5-2 Calculated fatigue lives for detail type 1 at selected section along the bridge for K12

Table 5-3 Calculated fatigue lives for detail type 1 at selected section along the bridge for K12 for S-N curve D

Section location	**Design fatigue life [years]**				
	A	**B**	**C**	**D**	**E**
Support A3	332	342	446	331	278
Transition 1 near A3	233	598	736	235	1235
Transition 2 near A3	191	1349	1573	195	1988
Midspan A3-A4	714	2088	2330	734	4082
Support A35	3014	1367	1630	2889	2679
Transition 1 near A35	895	1320	1554	852	3038
Transition 2 near A35	215	444	520	206	2276

Section location	**Design fatigue life [years]**				
Midspan A38-A39	411	773	1018	401	2470
Support A40	1734	1355	1746	1795	1905
Transition 1 near A40	545	1619	2210	560	2417
Transition 2 near A40	157	668	982	161	2200
Midspan A40-A41	5068	754	803	4755	2990

Table 5-4 Calculated fatigue lives for detail type 1 at selected section along the bridge for K12 for S-N curve E

Section location	**Design fatigue life [years]**				
	A	**B**	**C**	**D**	**E**
Support A3	199	204	264	198	168
Transition 1 near A3	141	352	432	142	710
Transition 2 near A3	115	786	918	117	1131
Midspan A3-A4	401	1206	1355	409	2269
Support A35	1719	806	958	1648	1538
Transition 1 near A35	517	784	920	494	1742
Transition 2 near A35	125	272	318	120	1313
Midspan A38-A39	219	453	595	214	1362
Support A40	995	793	1019	1029	1096
Transition 1 near A40	325	958	1301	334	1405
Transition 2 near A40	97	406	593	100	1290
Midspan A40-A41	2926	460	492	2754	1746

5.1.3 Detail type 2a: Butt welds – global + local load effects

First, this detail has been assessed for local traffic loads to identify the effect of the location of the weld in the longitudinal direction. Five points along the stiffener between support and midspan has been assessed and calculated fatigue lives are shown in Figure 5-3 for the top and bottom surfaces of the deck plate. The top surface of the plating is the worst and the fatigue lives improve towards the support of the stiffener. An SCF of 1.225 (16mm plate) is applied on the membrane part of the stress.

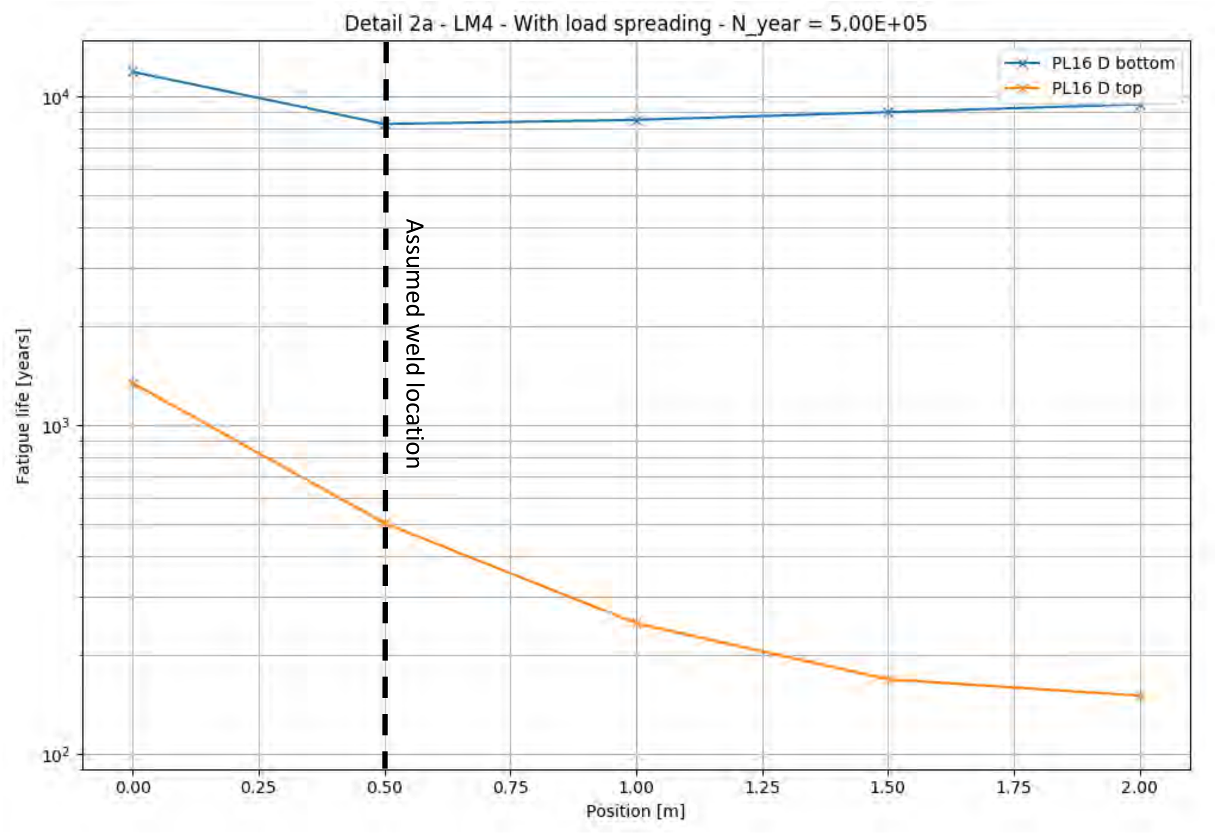


Figure 5-3 Calculated fatigue lives for detail 2a along the length of the stiffener (0.0m is at support and 2.0m is at midspan) – Local traffic loads only

For the combined global and local effect, it is assumed that the transverse weld will be located 0.5m away from the stiffener support location. This is in agreement with the planned method for splicing of bridge girder segments. This means that also during yard fabrication transverse butt welds must be located no more than 0.5m away from the transverse frames.

A fatigue life of 504 years is found at 0.5m away from the stiffener support for local load effects only.

Global load effects are taken from stress point E. Calculated fatigue lives for the combined effect of global and local load effects at the selected sections along the bridge is shown in Figure 5-4. The lowest calculated fatigue life is seen at the support at Axis 3 and at midspan between Axes 38 and 39 with fatigue lives of 137 and 161 years respectively. A histogram of the total stress range distribution for the points at Axis 3 is shown in Figure 5-5.

Contribution from tide is not included in these calculations. It is seen from the fatigue calculations for detail type 1 that contributions from tide is small, and it is hence believed that contribution from tide will only give a small reduction of the calculated fatigue lives. The effect of tide should be included in further fatigue work.

The calculated Stress series and rainflow counted stress cycles from local traffic loads for each vehicle type are given in Enclosure 5.

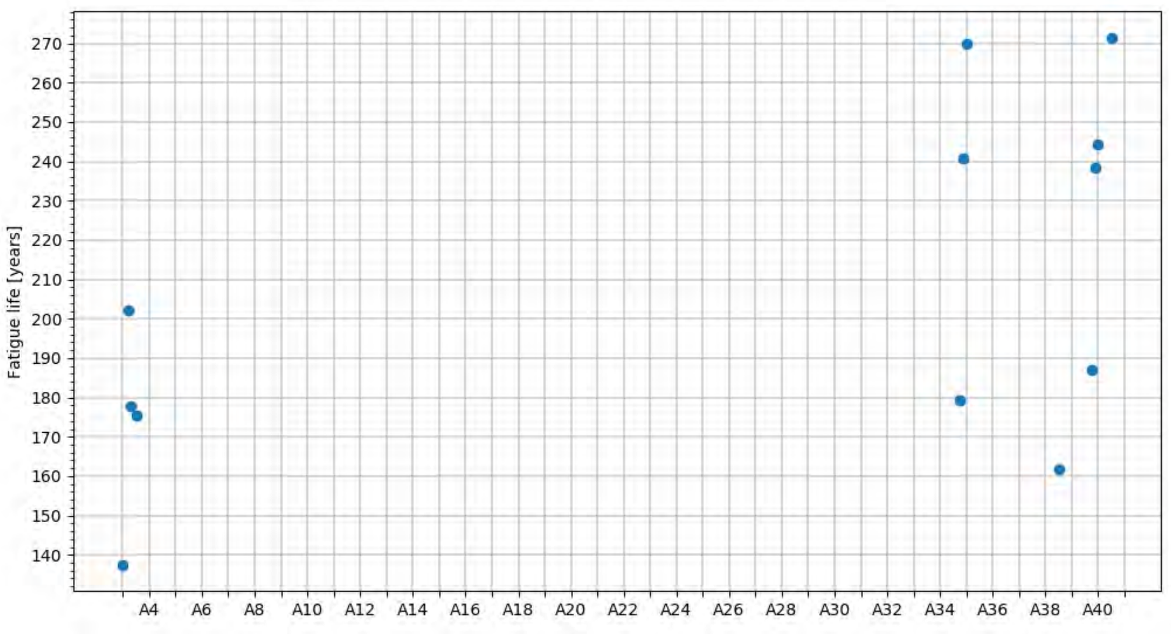


Figure 5-4 Calculated fatigue lives at selected section along the bridge for the combined effect of global and local loads.

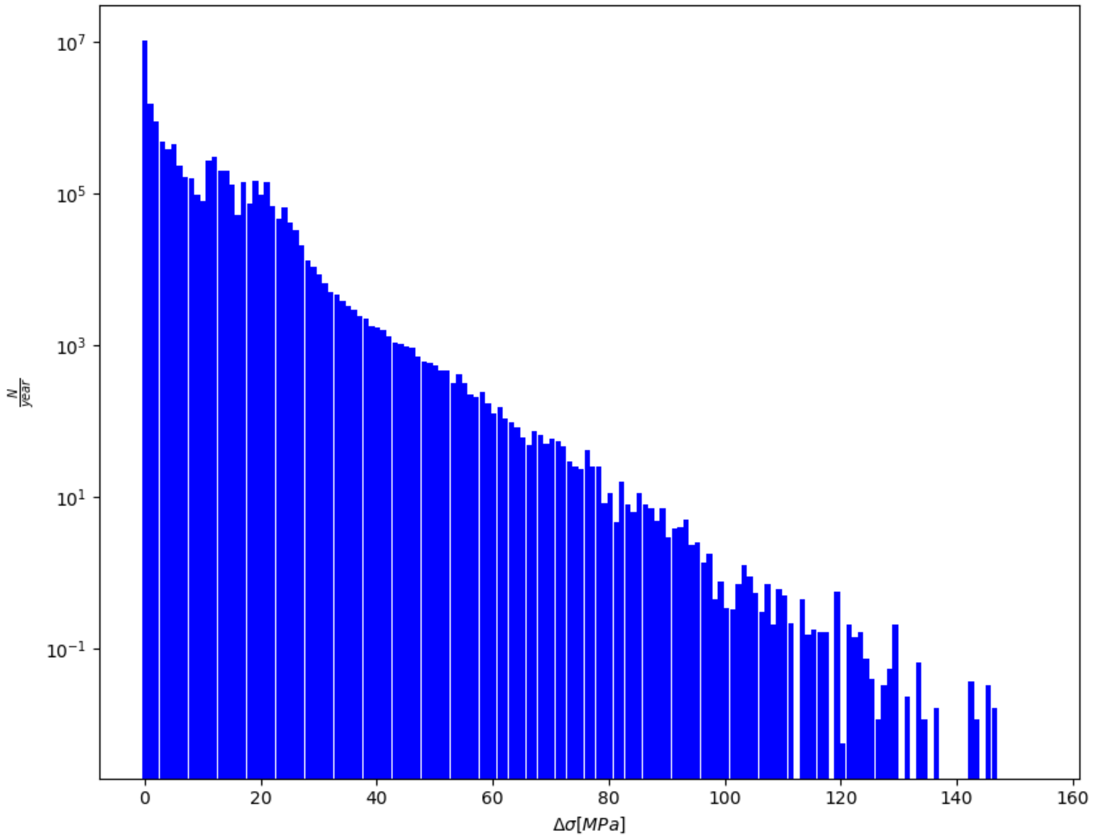


Figure 5-5 Stress range distribution for detail type 2a at Axis 3 with the combined effect of global and local loads

5.1.4 *Detail type 2b: Butt welds – local load effects only*

This detail has been assessed for local traffic loads only and with plate bending stresses parallel to the butt weld. Five points along the stiffener from support to midspan has been assessed and calculated fatigue lives are shown in Figure 5-6. Calculated fatigue lives at 0.5m away from the stiffener support is 70 years. With the applied fatigue load model this detail has insufficient fatigue life. See Section 5.5 for a load model sensitivity study where the required reduction in load level to get sufficient fatigue life is identified.

The calculated Stress series and rainflow counted stress cycles from local traffic loads for each vehicle type are given in Enclosure 5.

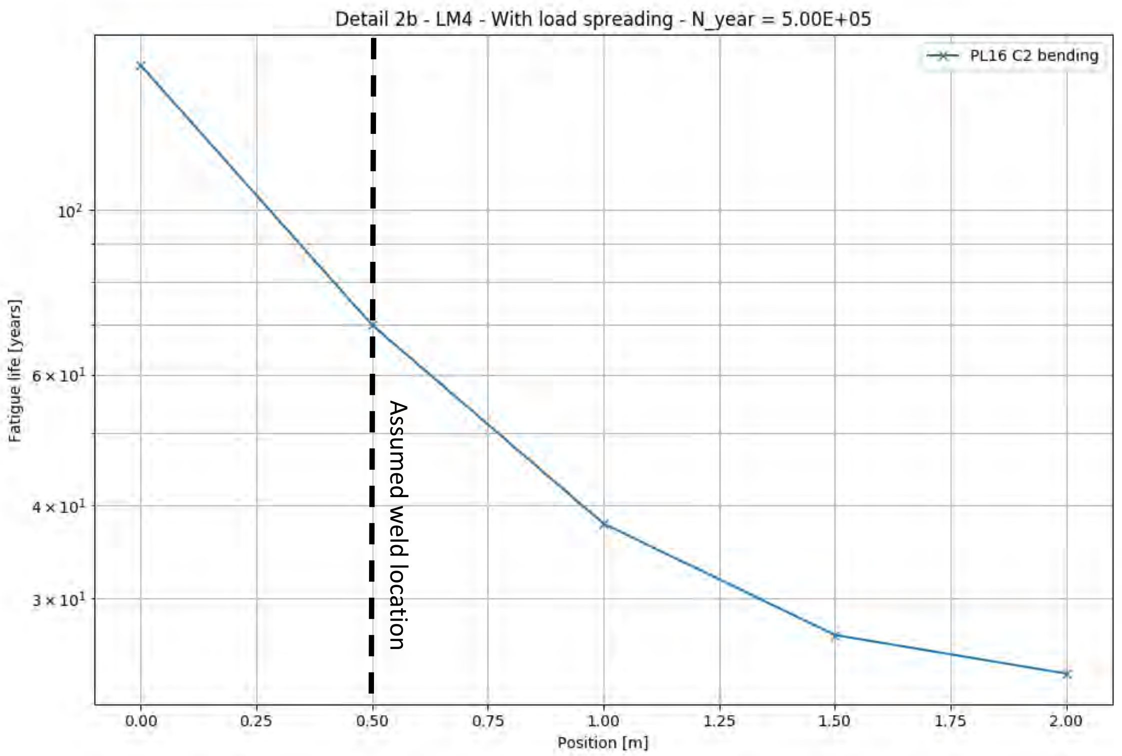


Figure 5-6 Calculated fatigue lives for detail 2b along the length of the stiffener (0.0m is at support and 2.0m is at midspan)

5.1.5 *Detail type 3: Trapezoidal stiffener to web frame connection – local load effects*

Two geometries have been investigated as shown in Figure 5-7. Locations for hotspots a, b and c are shown below in Figure 5-8, Figure 5-9 and Figure 5-10 for cut-out geometry 1, where normal and parallel stress directions are indicated. For these hotspots *Method B* from DNVGL-RP-C203 [14] is used. At the stress readout points for the selected hotspots, stresses normal and parallel to the weld is extracted from the FE-analysis, and stress series are calculated based on three different equations as given in Equation 4.3.5 in [14]. The stress series yielding the highest fatigue damage is taken as the fatigue damage for the hotspot. Locations for hotspot d is shown in Figure 5-11. This hotspot is only assessed for cut-out geometry 2. For this hotspot, fatigue is calculated based on the maximum principal stress at the free edge. The calculated fatigue lives are given in Table 5-5. It is seen that cut-out geometry 2 performs better for the welded details, but with very low fatigue life at the free edge. For hotspots related to the cut-out (3b, 3c and 3d) very low fatigue lives are found and a reduction of the fatigue traffic loads alone is not considered sufficient for this detail with the current structural design. A more fatigue friendly design must be developed. Also, the stiffness effect of the asphalt layer may be considered in the FE-analysis to reduce stresses. These additional measures are elaborated in Section 6.1.

Table 5-5 Calculated fatigue lives for detail type 3

Cut-out geometry	Fatigue life [years]			
	3a	3b	3c	3d
1	11	8	9	-
2	21	13	16	6

Figure 5-7 Detail 3 cut-out geometries

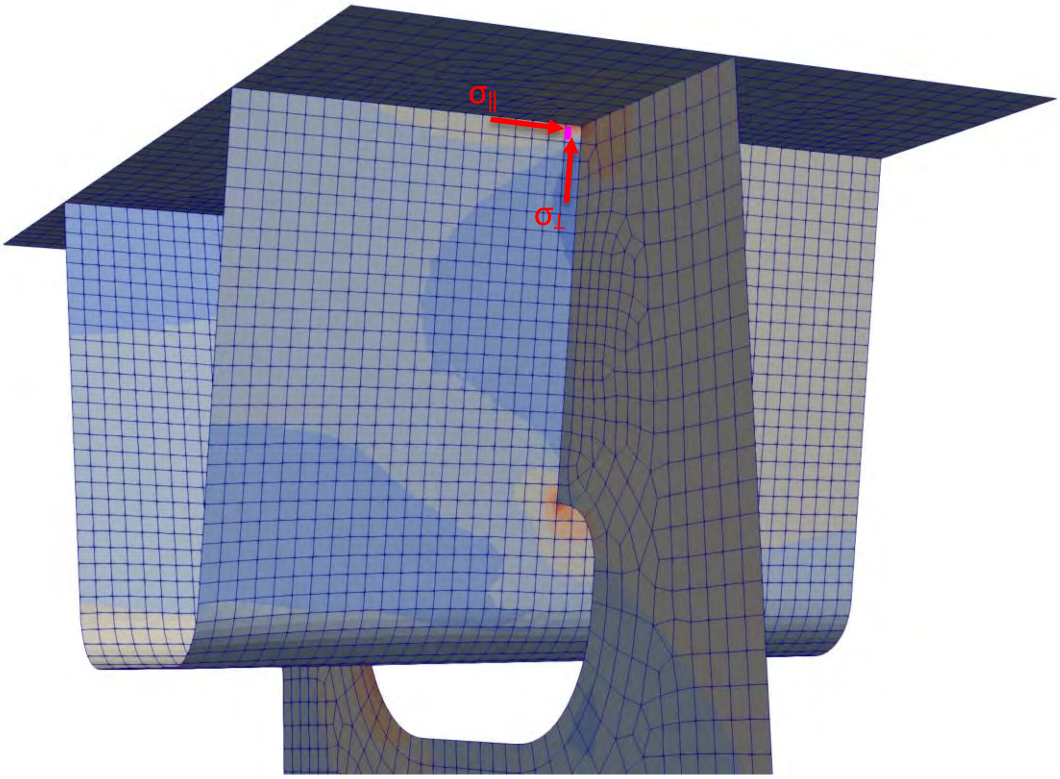


Figure 5-8 Detail type 3a – Weld between trapezoidal web and deck plate (weld toe on trapezoidal side)

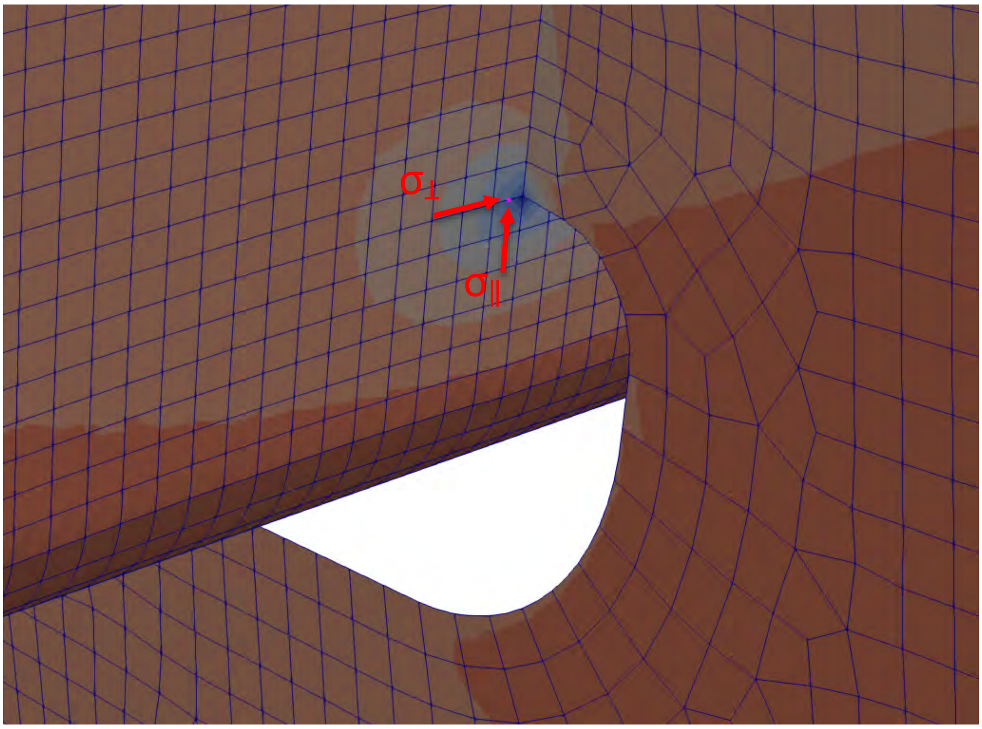


Figure 5-9 Detail type 3b – Weld between transverse web plate and trapezoidal web at cutout (weld toe of vertical weld on trapezoidal side)

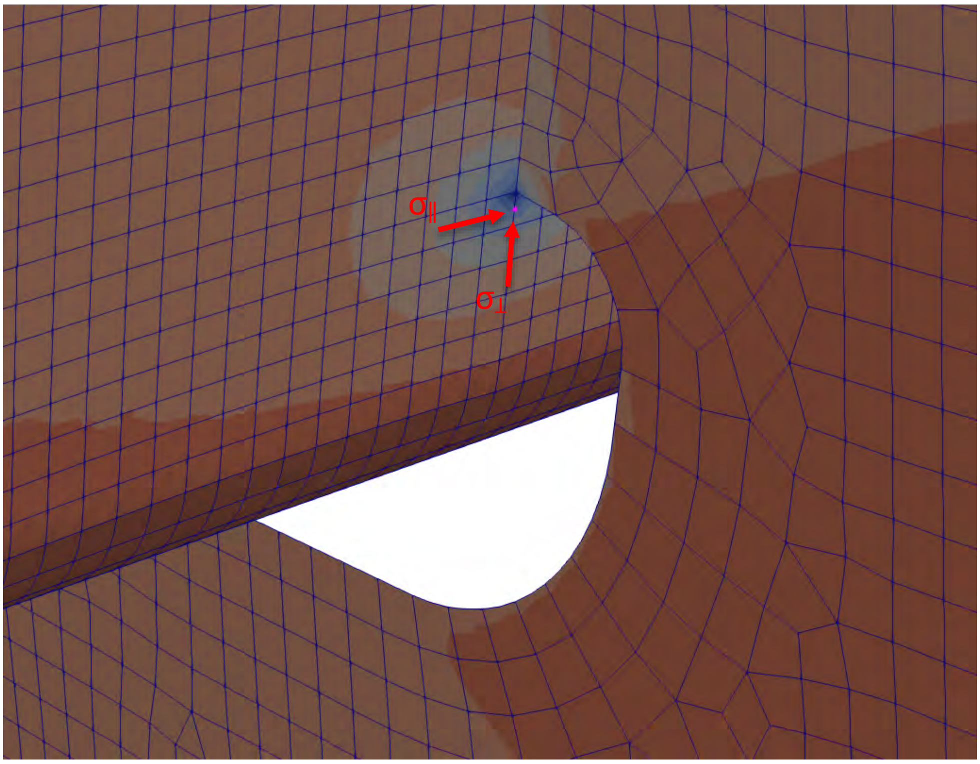


Figure 5-10 Detail type 3c – Weld between transverse web plate and trapezoidal web at cutout (weld toe of horizontal weld on trapezoidal side)

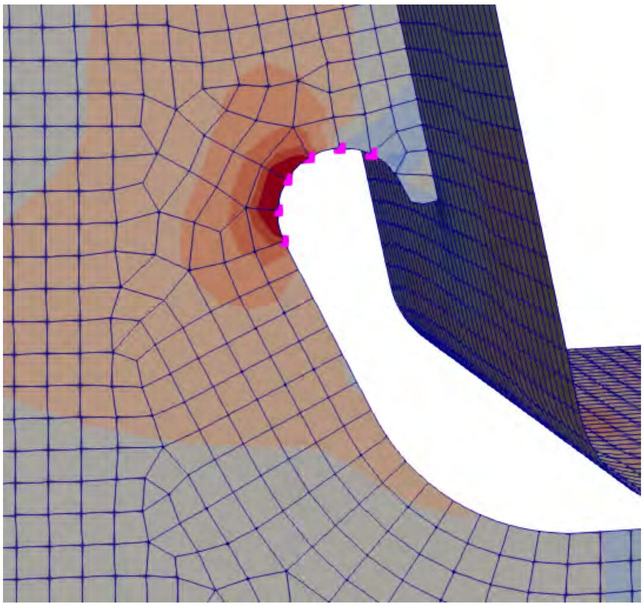


Figure 5-11 Detail type 3d – Free edge on cut-out

5.1.6 *Detail type 4: Trapezoidal longitudinal weld –local load effects only*

This detail has been assessed for local traffic loads only and with bending stress in web of trapezoidal stiffener at connection to top plate. Five points along the stiffener from support to midspan has been assessed and calculated fatigue lives are shown in Figure 5-12. Calculated fatigue lives at stiffener midspan is 27 years. With the applied fatigue load model this detail has insufficient fatigue life. See

Section 5.5 for a load model sensitivity study where the required reduction in load level to get sufficient fatigue life is identified.

Note that the calculated fatigue lives at the stiffener support is 16 years. This location is considered part of the trapezoidal stiffener to web frame connection and is assessed as detail type 3a in Section 5.1.5.

The calculated Stress series and rainflow counted stress cycles from local traffic loads for each vehicle type are given in Enclosure 5.

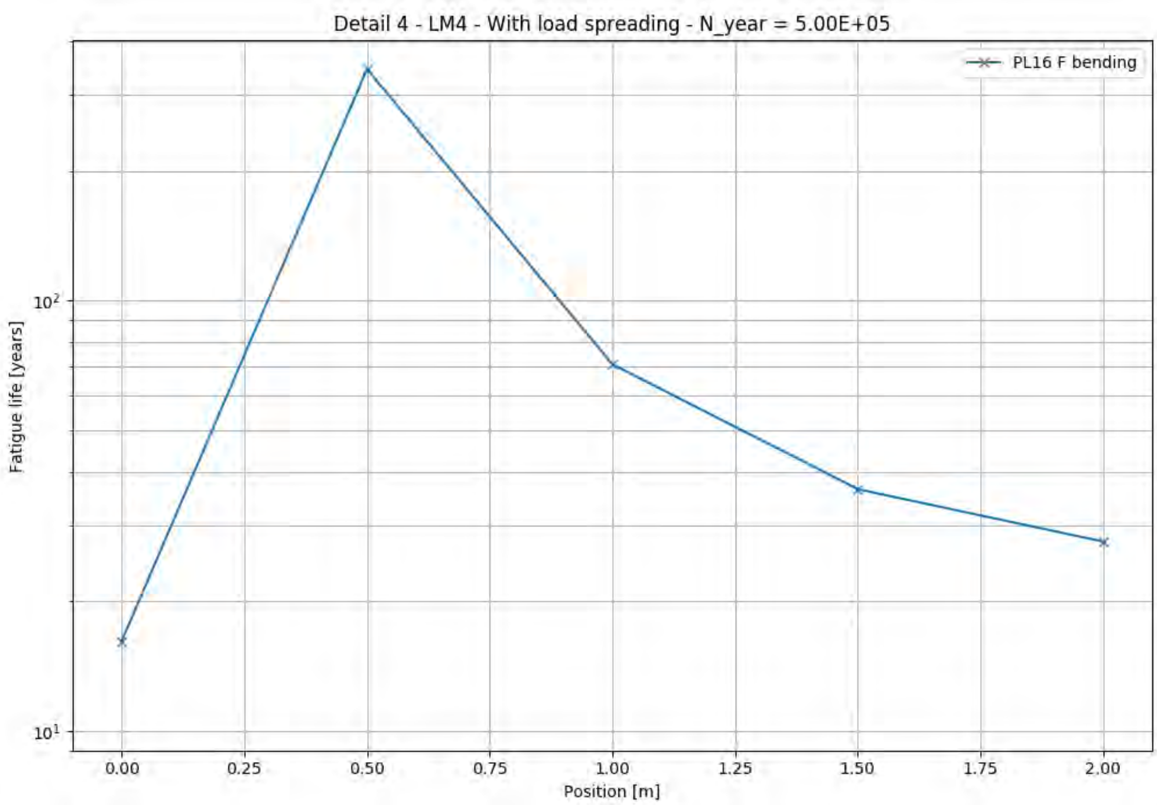


Figure 5-12 Calculated fatigue lives for detail 4 along the length of the stiffener (0.0m is at support and 2.0m is at midspan)

5.1.7 Detail type 5: Trapezoidal splice – global + local load effects

First, this detail has been assessed for local traffic loads to identify the effect of the location of the weld in the longitudinal direction. 5 points along the stiffener has been assessed and calculated fatigue lives are shown in Figure 5-13. Calculated fatigue lives at 0.5m from the stiffener support is 52 years. With the applied fatigue load model this detail has insufficient fatigue life for local traffic loads only. To increase fatigue life for this detail stiffener wall thickness have been increased from 8mm to 10mm in the slow traffic lanes.

Global load effects are taken from stress point E. When accounting for global load effects the calculated fatigue lives at 0.5m from the stiffener support is reduced to approximately 32 years. A histogram of the total stress range distribution for the point at Axis 3 is shown in Figure 5-14.

Contribution from tide is not included in these calculations.

See Section 5.5 for a load model sensitivity study where the required reduction in load level to get sufficient fatigue life is identified.

The calculated Stress series and rainflow counted stress cycles from local traffic loads for each vehicle type are given in Enclosure 5.

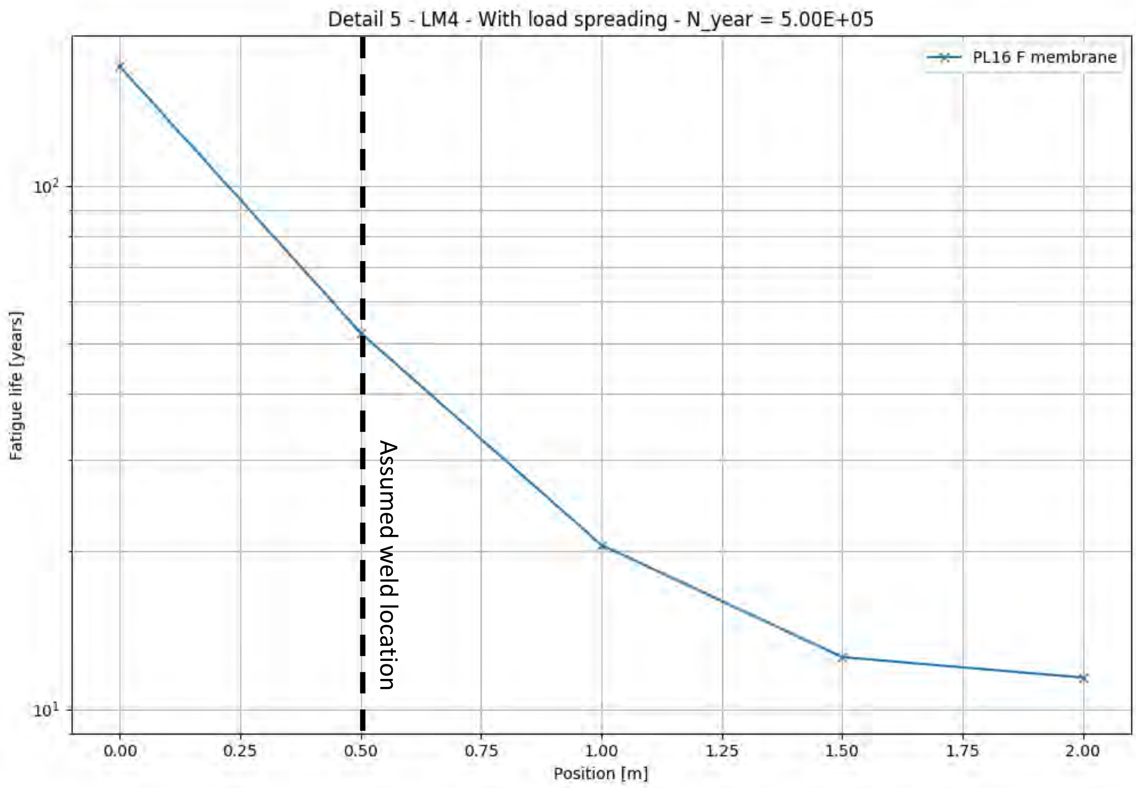


Figure 5-13 Calculated fatigue lives for detail type 5 along the length of the stiffener (0.0m is at support and 2.0m is at midspan)

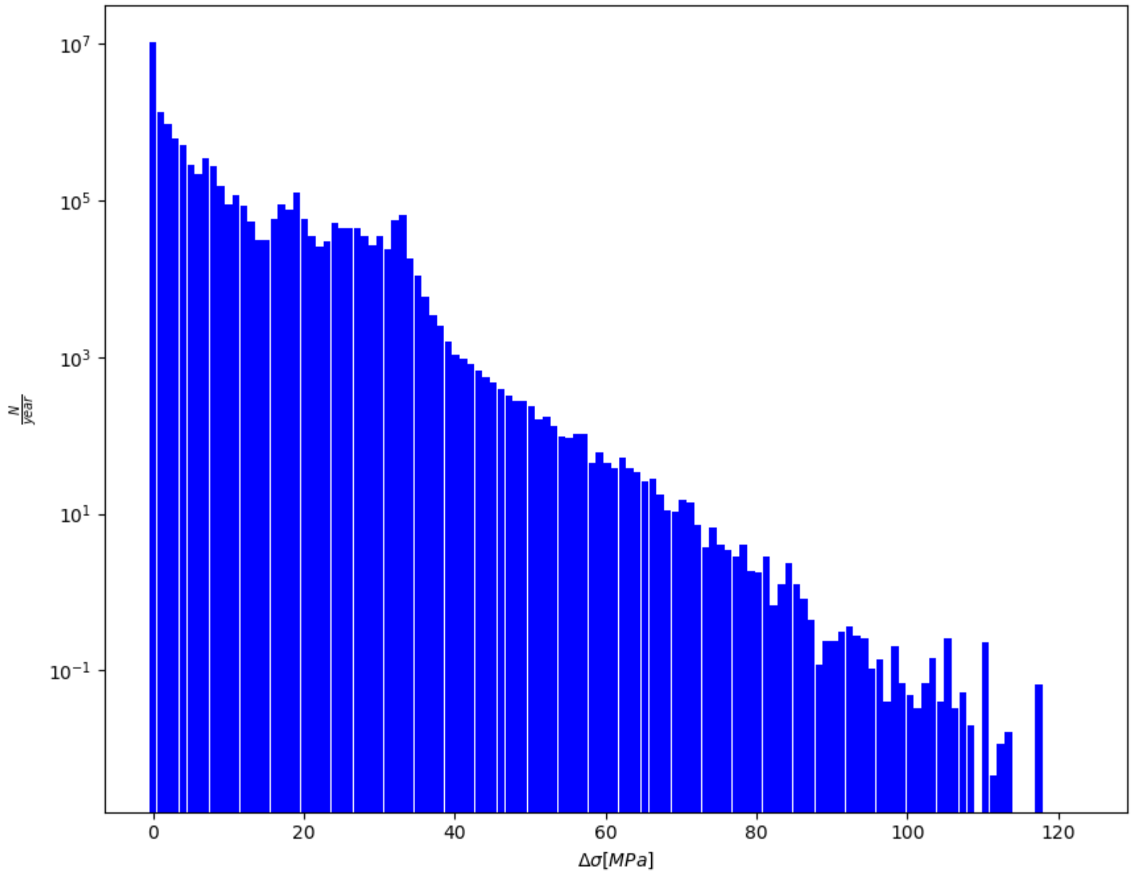


Figure 5-14 Stress range distribution for detail type 5 at Axis 3 with the combined effect of global and local loads

5.2 Mooring lines

5.2.1 Detail type 1: Mooring line chains – top/bottom, tension fatigue

For the mooring lines tension fatigue calculations has been performed at the top and bottom chains. Loads from wind waves, swell and tide has been included. It was shown in the previous phase that the main contribution to fatigue comes from wind waves. Design fatigue lives (including DFF of 10) is shown below in Figure 5-15. The lowest fatigue life is found at the bottom chain of line 2 with 100 years life. The bottom chains are not planned to be replaced, and this chain hence has a fatigue utilization of 1.0. It is noted that the effect of OPB/IPB is expected to be less at the anchor side and the applied SCF of 1.15 is considered conservative for the bottom chain. The most utilized top chain is also at line two with 120 years fatigue life. The top chains are planned to be replaced once, which means that this chain has a utilization of 0.42.

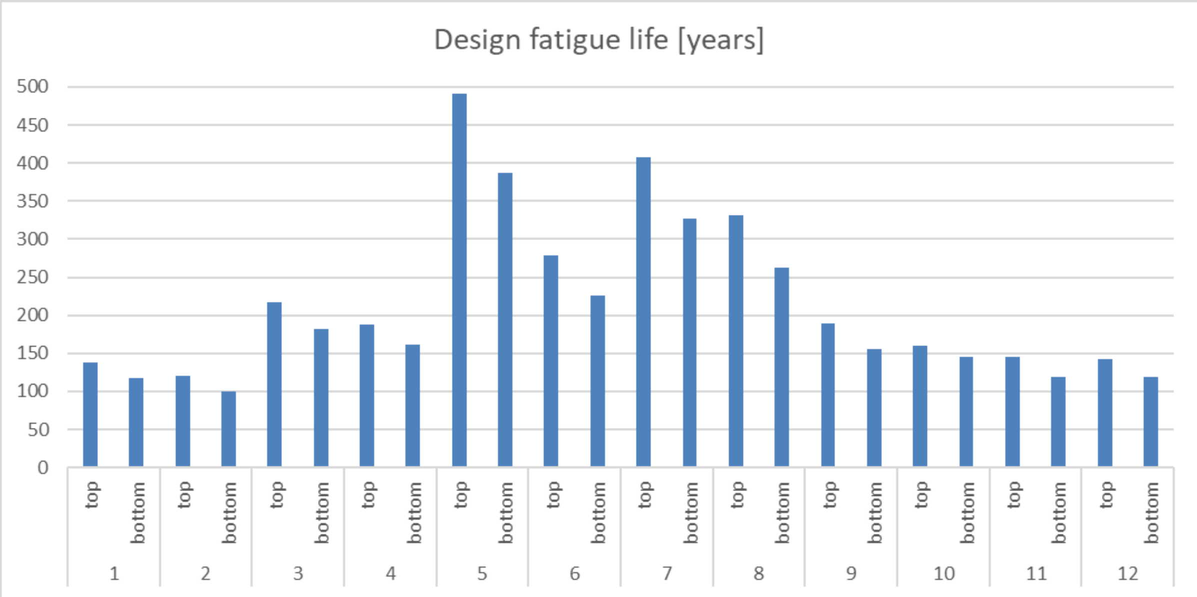


Figure 5-15 Design fatigue life in mooringline chains – Tension fatigue

5.2.2 *Detail type 2: Mooring line chains – top, OPB/IPB fatigue*

OPB/IPB fatigue have been calculated for the upper part of the top chains based on coupled analysis including wind, wind waves and swell. Contribution from tide is also included by use of combination formula.

The coupled time domain analyses have been performed for a subset of the environmental cases (wind, wind waves and swell). 115 cases are selected based on fatigue damage contribution. The selected cases are given in Enclosure 4.

For the mooring lines the selected cases contribute to approximately 80 % of the total fatigue damage. The presented fatigue lives have therefore been multiplied by a factor 0.8 to estimate the total fatigue damage.

The calculated fatigue lives are shown below in Figure 5-16. The lowest fatigue life is found at line 2 with 110 years which gives a utilization of 0.45. In comparison, the tension fatigue calculation gives a fatigue life of 120 years at the same location.

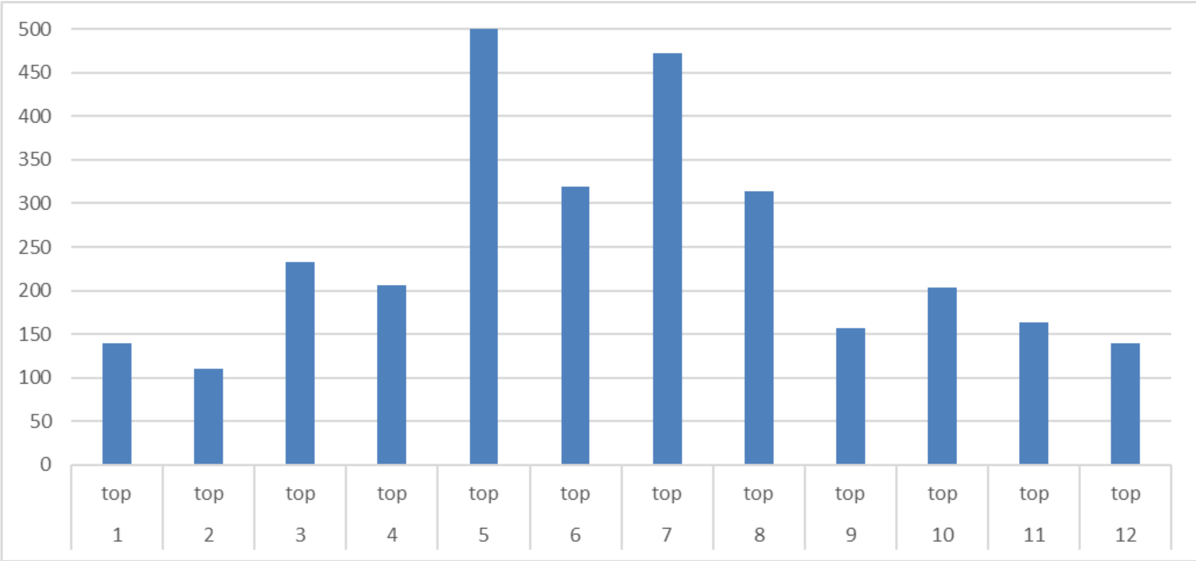


Figure 5-16 Design fatigue life in mooringline chains – OPB/IPB fatigue

5.3 Stay cables

Local vibrations/dynamics are not included properly in the analyses and the fatigue assessment is hence based on dynamic axial stress due to global load effects. It is the intension to prevent local dynamic response by means of damping devices and it is hence considered adequate to disregard these effects in the fatigue assessment. The calculated fatigue lives are shown in Figure 5-17. Fatigue lives are found to be very high. The main contributors to fatigue are traffic and tide. Traffic contributes to most of the fatigue near the tower and tide contributes to most of the fatigue in the northern end near Axis 3.

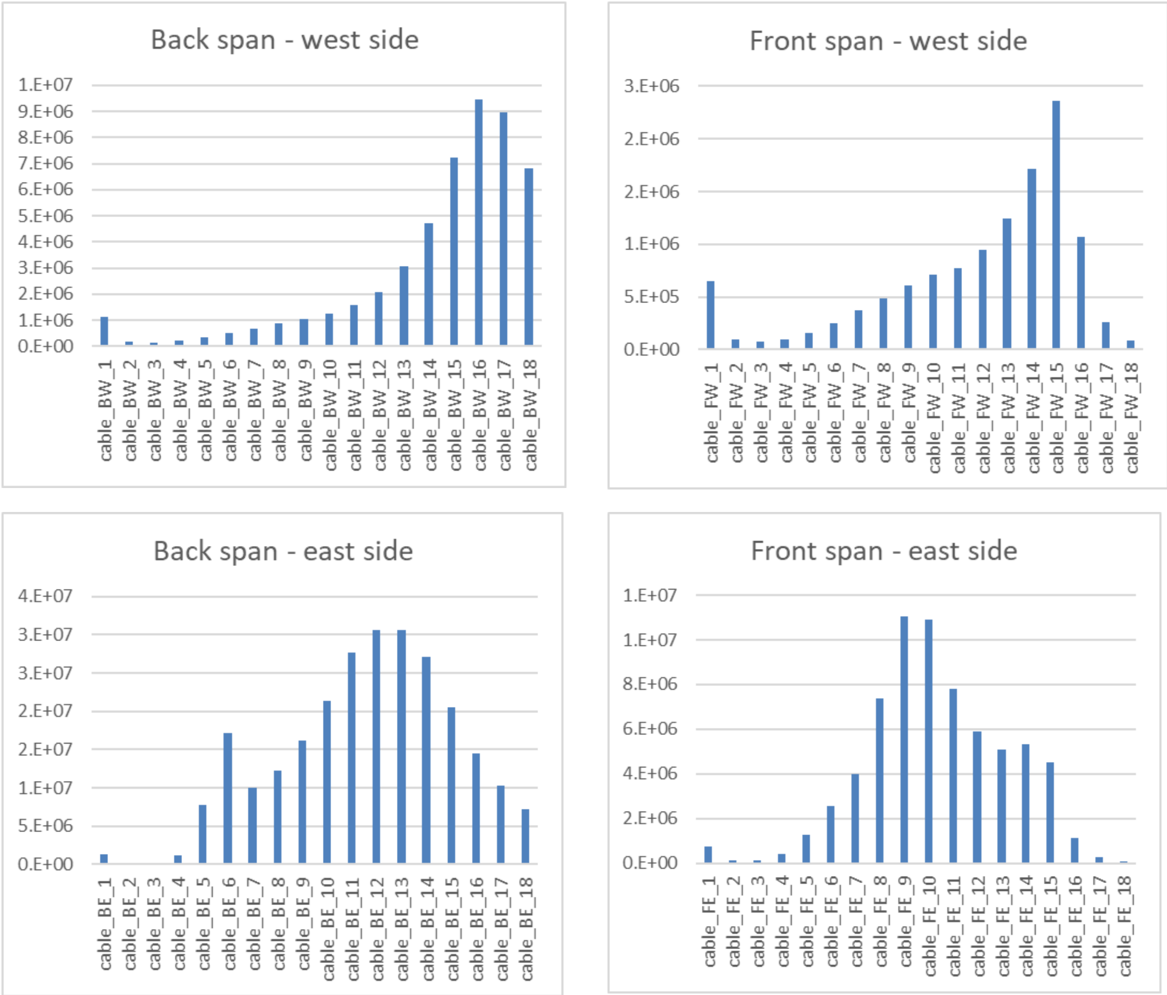


Figure 5-17 Calculated fatigue life in stay cables

5.4 Bridge girder / column / pontoon connections

5.4.1 Detail type 1: Weld between cast piece and bridge girder

The local analysis model used to derive SCFs is shown in Figure 5-18. The SCFs have been calculated relative to the nominal girder stress at stress points A and D. Two different loading situations have been analysed, one where forces are balanced fully between End 1 and End 2, i.e. no forces go into the column (Symmetric). The other situation is for the girder weak axis bending moment where 50 % of the bending moment is transferred in to the column (Asymmetric). The stress distribution for the asymmetric case is shown in Figure 5-19. The SCFs are given in Table 5-6. These SCFs include geometric effects only. In addition, an SCF to account for fabrication tolerances should be included. With a thickness of 22mm of both sides of the weld and a 2mm fabrication tolerance this SCF is 1.12. The total SCF for weak axis bending in the asymmetric case is then 1.86. Calculated fatigue lives for all the column top locations based on an SCF of 1.86 is shown in Figure 5-20 for K12. A fatigue life of 44 years is found at axis 3, all other locations have fatigue life above 100 years. It is expected that sufficient fatigue life can be achieved at Axes 3 by increasing structural dimensions locally.

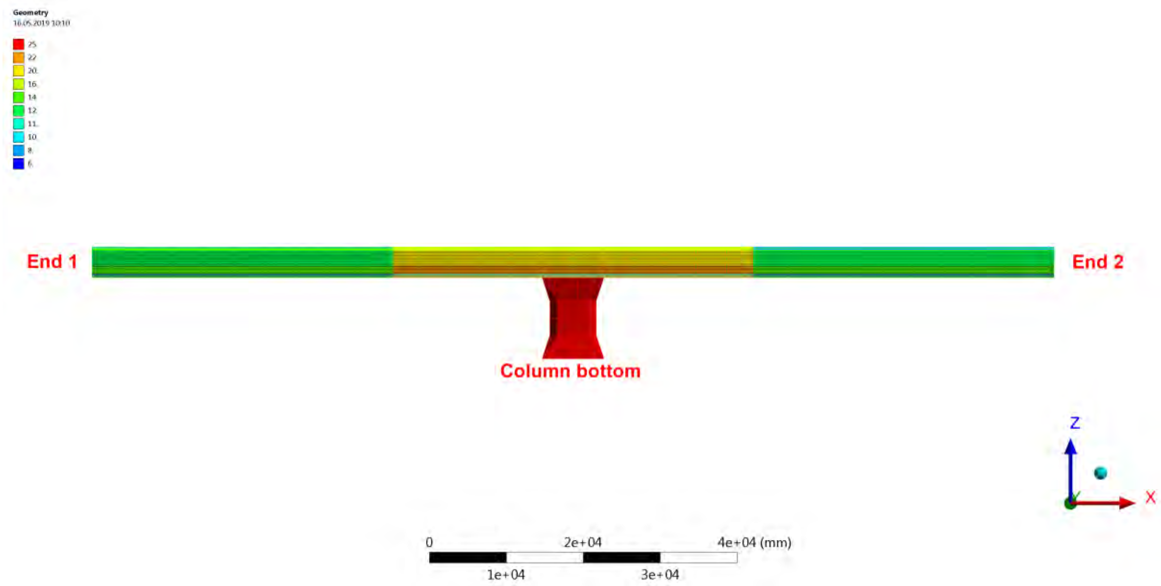


Figure 5-18 Local FE-model used for calculation of SCFs

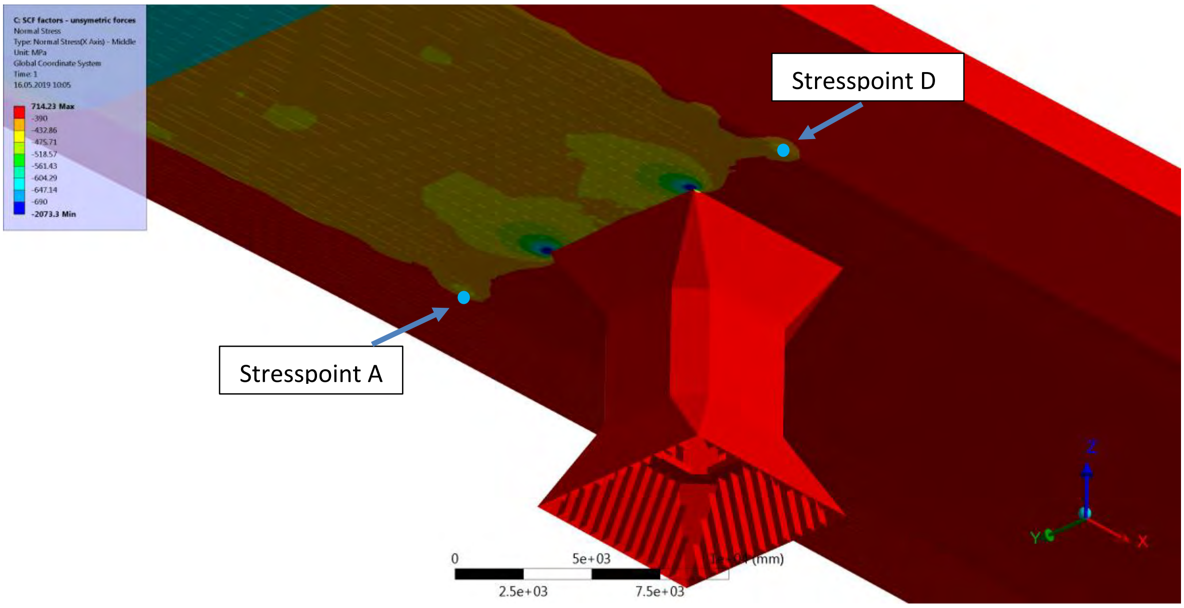


Figure 5-19 Stress distribution at bridge girder to column connection for weak axis bending in the asymmetric case

Table 5-6 SCFs for the butt weld between bridge girder plating and cast piece at column corners

	Applied force		
Boundary condition	N	M_weak	M_strong
Symmetric End 1: 100% End 2: 100%	1.25	1.38	1.41
Asymmetric End 1: 100% End 2: 50% Column bottom: 50%		1.66	

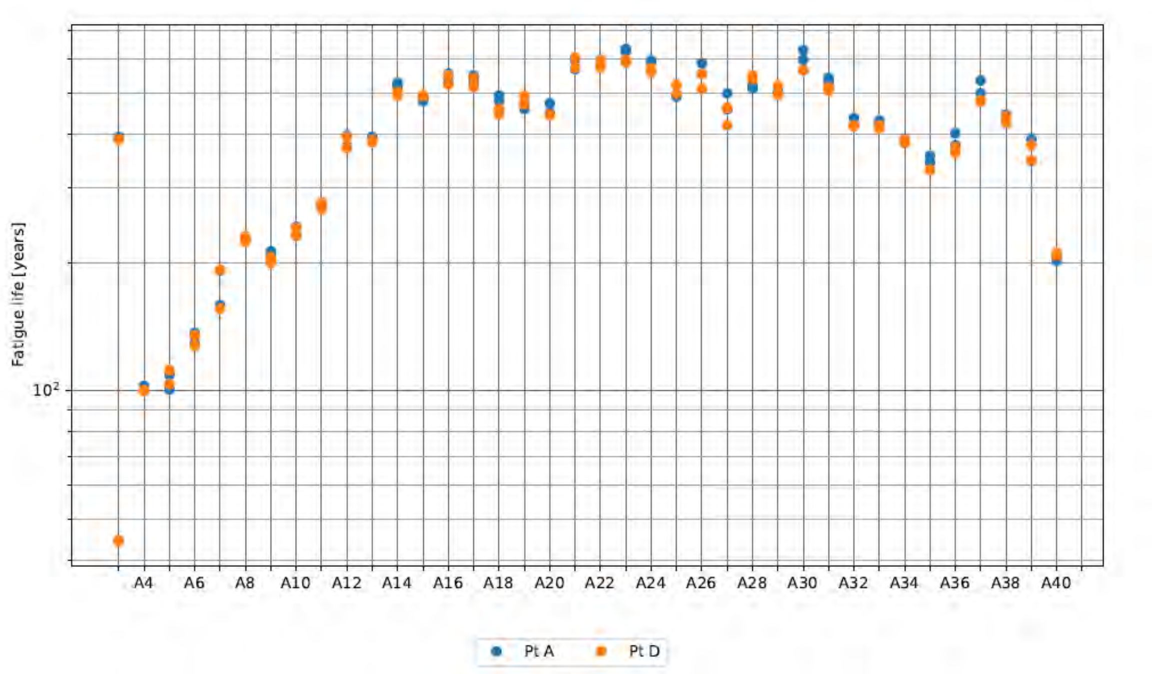


Figure 5-20 Calculated fatigue life at bridge girder to column connection at all column locations

5.4.2 Detail type 2: Weld between cast piece and column bottom

The SCFs have been calculated relative to the nominal column stress at the column corners. The hotspot location is shown in Figure 5-21 along with selected nodes used for stress extrapolation. The SCF is taken as the ratio between the hotspot stress and the nominal stress evaluated a distance away from the hotspot location as shown in Figure 5-22. SCFs have been calculated for pure bending moment about the two cross section axes and for pure axial force. The values are shown in Table 5-7. The largest SCF is found for bending moment with an SCF of 2.96. This value is used for the fatigue calculations. In addition, an SCF to account for misalignment and thickness effects are applied. This SCF is taken as 1.18, which gives a total SCF of 3.5.

Nominal stresses are calculated from the coupled analyses presented in Enclosure 4. This means that only a subset of environmental cases has been assessed. For the calculations performed here, it is assumed that the selected environmental cases contribute to 80% of the total fatigue damage. The presented fatigue lives have therefore been multiplied by a factor 0.8 to estimate the total fatigue damage.

The calculated fatigue lives are high and consequently fatigue utilization is low. The lowest calculated fatigue life is at the mooring pontoons (Axes 13,20,27) with approximately 9000 years at Axis 13. This indicates that it may be possible to avoint cast details for the lower connection.

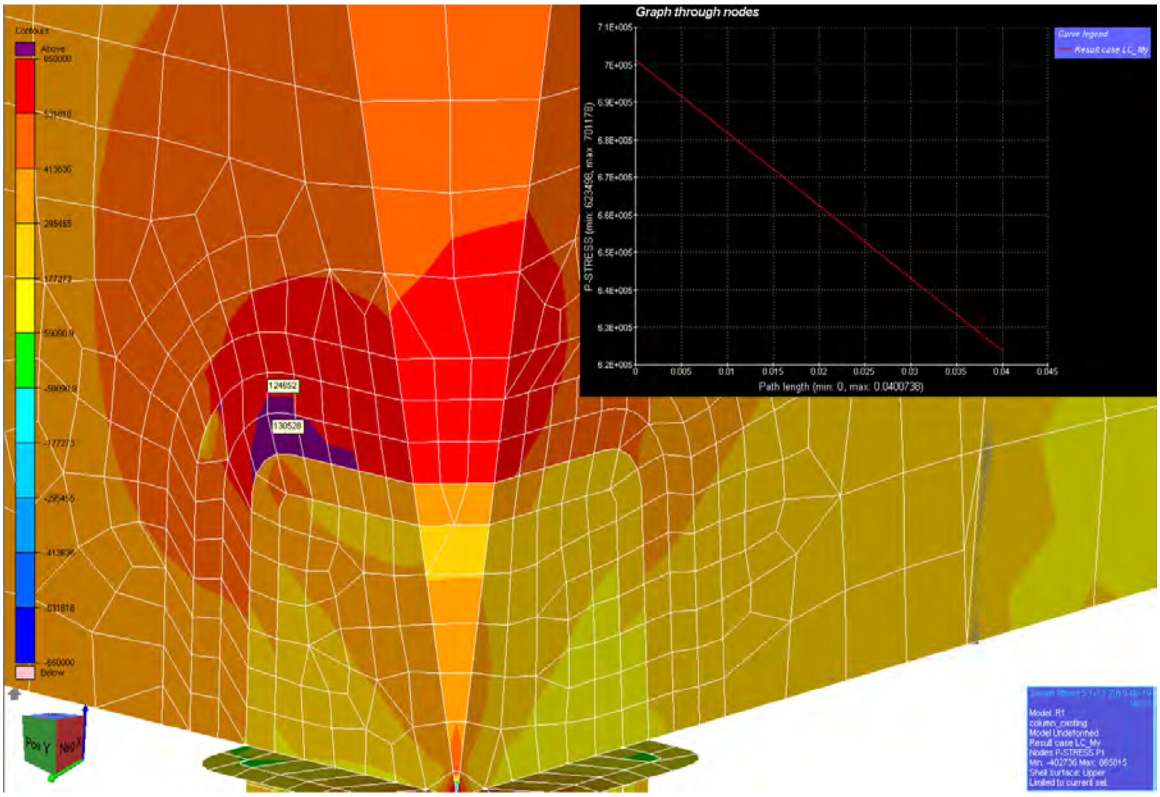


Figure 5-21 Hotspot location in FE-model for weld between column plating and corner cast piece – Stress level at weld is found by extrapolation from t/2 and 3t/2 away from weld.

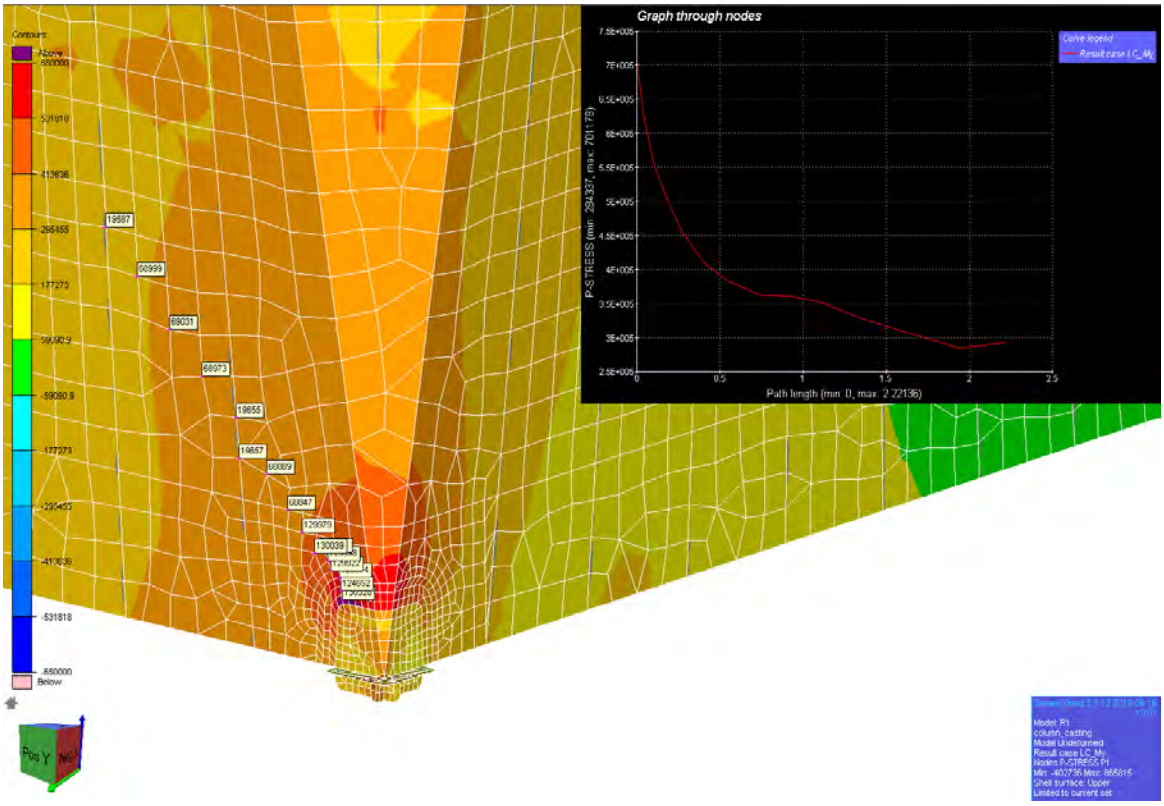


Figure 5-22 Stress levels away from hotspot used to determine nominal stress in the SCF calculation

Table 5-7 SCFs calculated for hotspot stress at weld towards cast pieces at column corners

	sig_{nom}	$sig_{hotspot}$	SCF
M_z	250000	740000	2.96
M_y	300000	740000	2.47
N_x	3000000	7550000	2.52

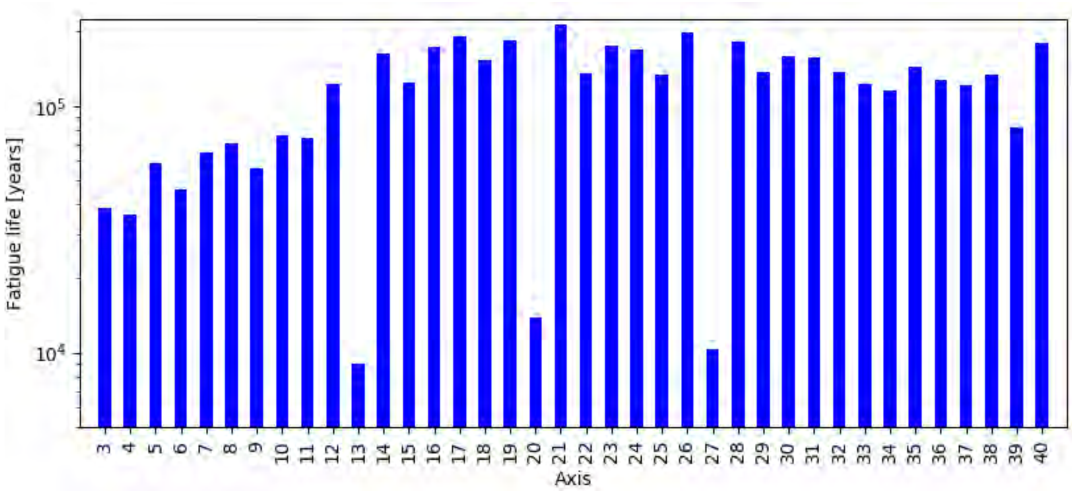


Figure 5-23 Calculated fatigue lives at weld between cast piece and column bottom

5.4.3 *Detail type 3: Weld between cast piece and column top*

The same calculation as for the column bottom has been performed for the column top. All the fatigue calculation parameters are identical. The calculated fatigue lives are shown in Figure 5-24. Fatigue lives of 47 and 89 years are found at axes 3 and 4 respectively. All other axes have fatigue lives above 100 years. It is expected that sufficient fatigue life can be achieved at Axes 3 by increasing structural dimensions locally.

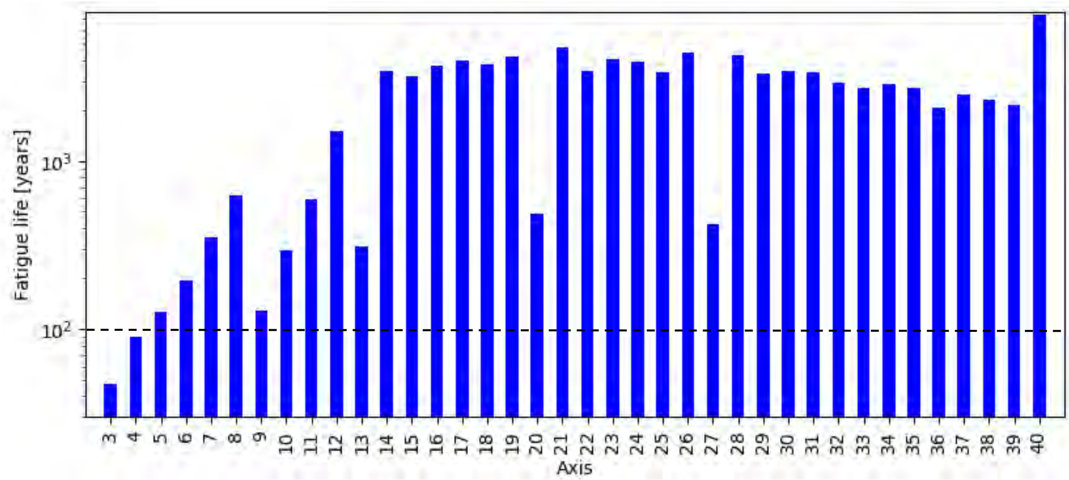


Figure 5-24 Calculated fatigue lives at weld between cast piece and column top – The target fatigue life of 100 years is indicated

5.5 Traffic load model sensitivity

The results presented in this appendix are based on FLM4 with *long distance* lorry distribution and 0.5e6 annual number of vehicles in each slow lane as given in the design basis [4]. In the following a sensitivity study carried out to identify the effect of reducing different parameters of the load model is presented.

Bridge girder details types 2b, 3, 4 and 5 which has insufficient fatigue life with the current load model are chosen for the study. Detail types 2b, 3 and 4 are subject to local traffic loads only, while detail type 5 is subject to combined global and local loads.

The following incremental changes has been done to the load model

1. Changing the lorry distribution to *medium distance*
2. With medium distance lorry distribution to, reduce axle loads to 90 %
3. With medium distance lorry distribution to, reduce axle loads to 80 %
4. With medium distance lorry distribution to, reduce axle loads to 75 %
5. With medium distance lorry distribution to, reduce axle loads to 70 %
6. With medium distance lorry distribution to and 70% axle loads, reduce number of vehicles to 0.25e6

The number of vehicles is proportional to the fatigue life which means that a reduction to 50% of the vehicles (N=0.25e6) will double the fatigue life when considering traffic loads only.

Results are shown in Figure 5-25, Figure 5-26 and Figure 5-27 for detail types 2b, 4 and 5 respectively for local traffic loads only.

For detail type 2b a sufficient fatigue life is achieved at 0.5m from the stiffener support by using the medium range lorry distribution and a reduction to 90 % of the full axle loads.

For detail type 3a a sufficient fatigue life is achieved by using the medium range lorry distribution and a reduction to 75 % of the full axle loads

For detail types 3 b, c and d a relatively large reduction of the traffic loads is required an further design optimization should also be performed to improve fatigue life.

For detail type 4 a sufficient fatigue life is achieved at stiffener midspan by using the medium range lorry distribution and a reduction to 80 % of the full axle loads.

For detail type 5 a sufficient fatigue life is achieved at 0.5m from the stiffener support for local traffic load only by using the medium range lorry distribution and a reduction to 90 % of the full axle loads. For the combined effect of local and global loads this reduction gives as fatigue life of 57 years. When the axle loads are reduced to 75 % of the full axle loads the fatigue life from combined effect of local and global loads is 103 years.

The required measures are summarized in Table 5-8.

Table 5-8 Calculated fatigue lives and reduction measures

Detail type no.	Location	Fatigue life with full FLM4 load model	Load model reduction measures	Fatigue life with reduced load model
2b	0.5 m from stiffener support	70 years	Medium distance 90% axle loads	156 years
3a	Connection trapezoidal / web frame	21 years	Medium distance 75% axle loads	122 years
3b	Connection trapezoidal / web frame	13 years	Medium distance 70% axle loads N=0.25e6	139 years
3c	Connection trapezoidal / web frame	16 years	Medium distance 70% axle loads N=0.25e6	180 years
3d	Connection trapezoidal / web frame	6 years	Medium distance 70% axle loads N=0.25e6	60 years
4	Stiffener midspan	27 years	Medium distance 80% axle loads	123 years
5 (local only)	0.5 m from stiffener support	52 years	Medium distance 90% axle loads	112 years
5 (local + global)	0.5 m from stiffener support	32 years	Medium distance 75% axle loads	103 years

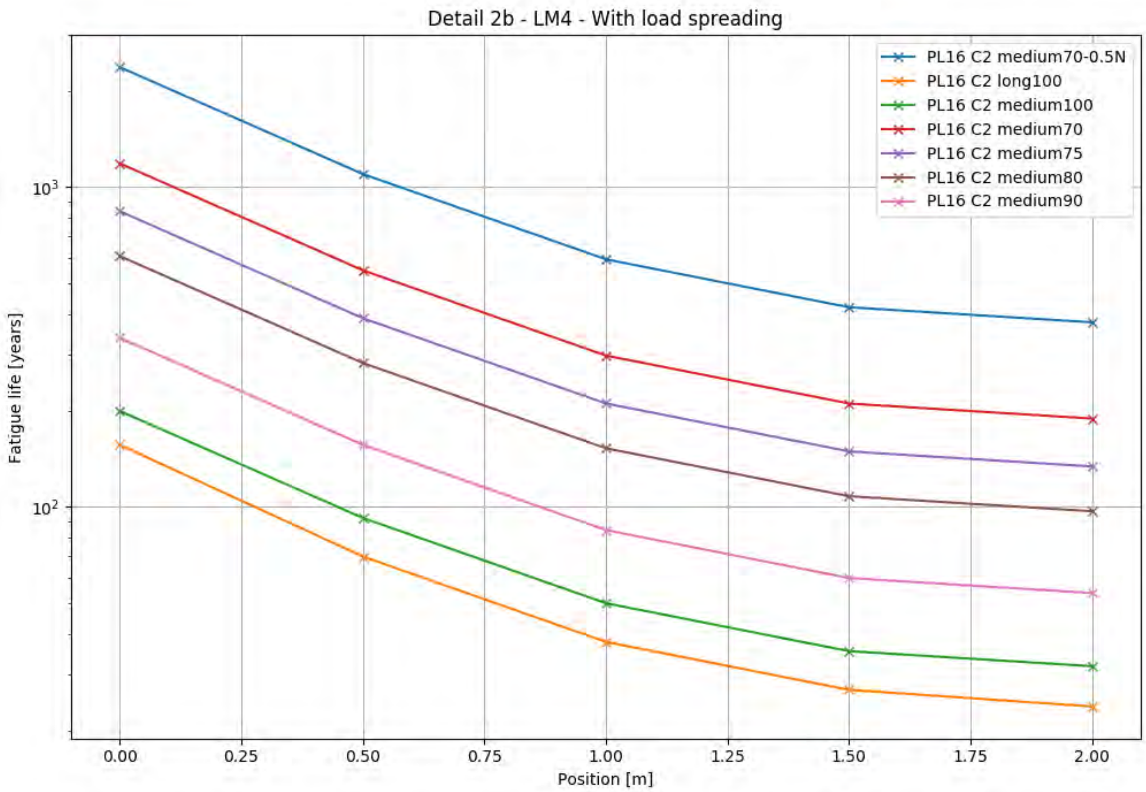


Figure 5-25 Results from load model sensitivity study – Detail type 2b - (0.0m is at support and 2.0m is at midspan)

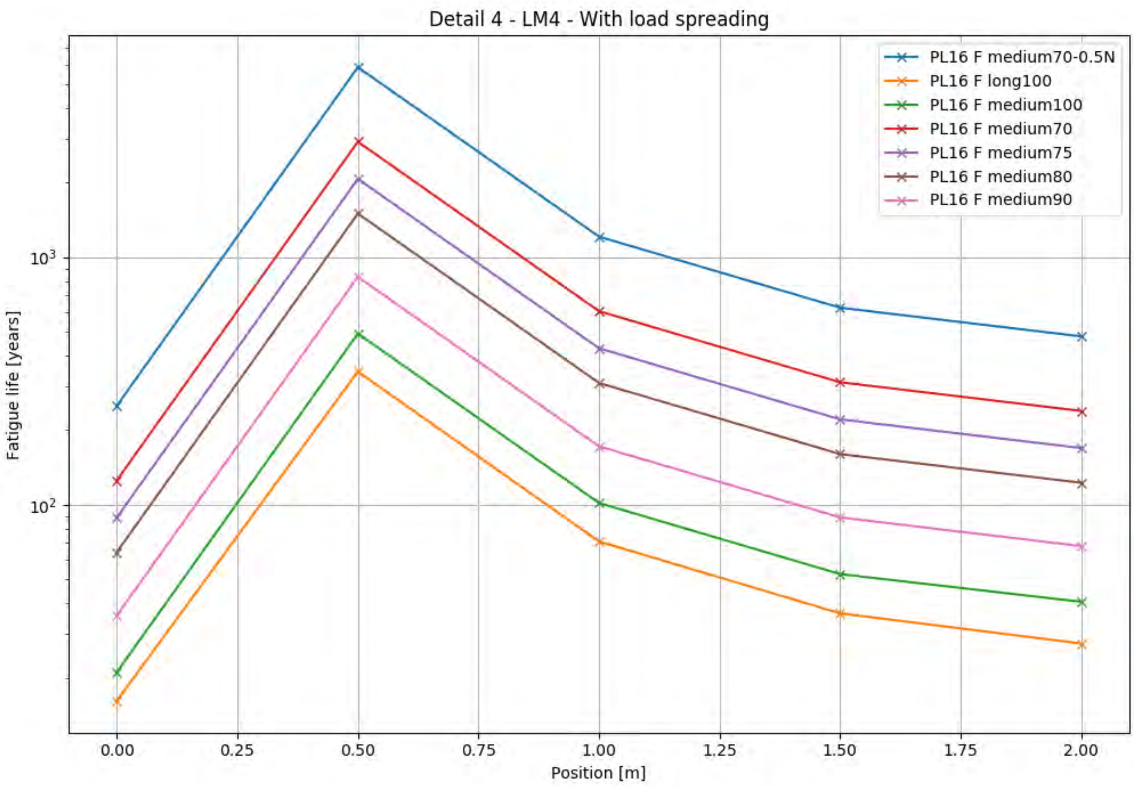


Figure 5-26 Results from load model sensitivity study – Detail type 4 - (0.0m is at support and 2.0m is at midspan)

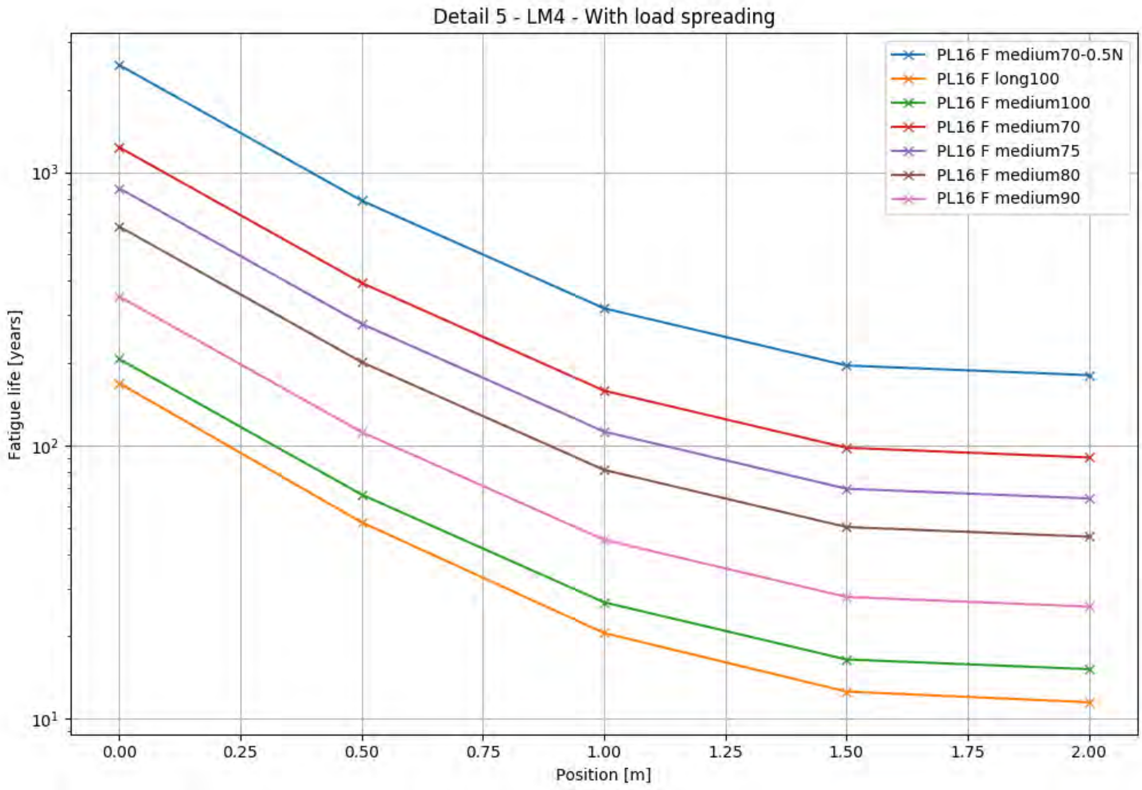


Figure 5-27 Results from load model sensitivity study – Detail type 5 - (0.0m is at support and 2.0m is at midspan)

Traffic load case	Fatigue life [years]			
	3a	3b	3c	3d
Long distance, 100 % load, N=0.5e6	21	13	16	6
Medium distance, 100 % load, N=0.5e6	29	18	23	8
Medium distance, 90 % load, N=0.5e6	49	27	33	12
Medium distance, 80 % load, N=0.5e6	89	42	52	18
Medium distance, 75 % load, N=0.5e6	122	53	66	23
Medium distance, 70 % load, N=0.5e6	173	69	90	30
Medium distance, 70 % load, N=0.25e6	346	139	180	60

5.6 Time domain verification

Coupled time domain analyses has been performed for a subset of the environmental cases (wind, wind waves and swell). 115 cases are selected based on fatigue damage contribution. The selected cases are given in Enclosure 4. For the bridge girder the selected cases contribute to approximately 90 % of the total fatigue damage.

Fatigue lives have been calculated for the bridge girder with both frequency domain and coupled time domain analyses for the selected cases. A comparison between the two methods is shown below in Figure 5-28 and Figure 5-29 for stress points A (bottom flange) and B (top flange) respectively. The comparison confirms that the frequency domain method gives results to the safe side compared to coupled time-domain.

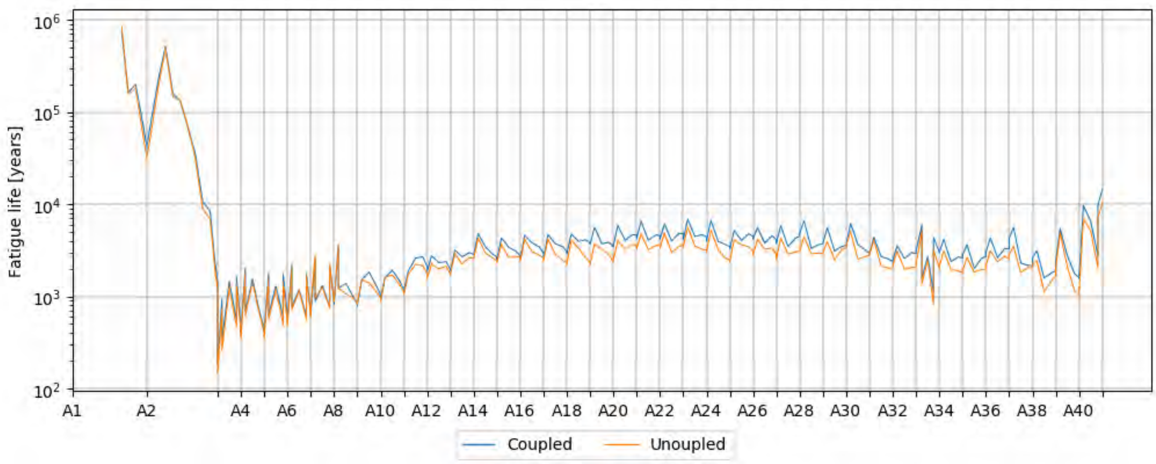


Figure 5-28 Comparison of fatigue life due to environmental loads for time-domain analysis (coupled) vs frequency domain analysis (uncoupled) at stress point A (bottom flange) in the bridge girder

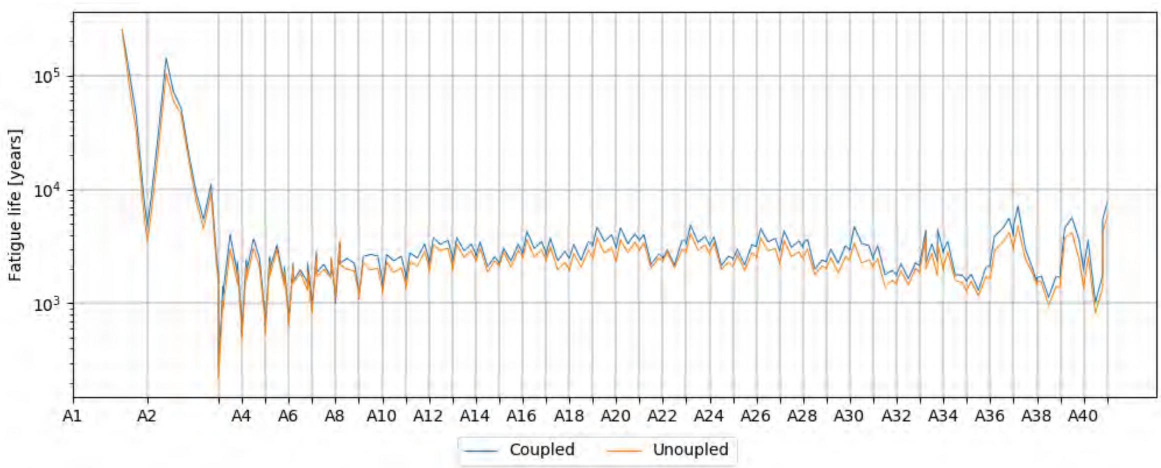


Figure 5-29 Comparison of fatigue life due to environmental loads for time-domain analysis (coupled) vs frequency domain analysis (uncoupled) at stress point B (top flange) in the bridge girder

6 Discussion and recommendations

Fatigue calculations have been performed for selected details in the bridge girder, the connection between columns and bridge girder/pontoons, mooring chains and stay cables. Large parts of the assessed structures have an acceptable fatigue utilization. However, some areas are found to have insufficient fatigue life. These areas are the parts of the bridge girder deck which are susceptible to direct wheel loads and the connection between bridge girder and columns in the upper part of the floating bridge. In the following, essential findings from the fatigue assessment are discussed and recommendations for future fatigue work is presented. Finally, some uncertainties with respect to fatigue loading is mentioned.

6.1 Bridge girder

Several structural details in the bridge girder which are subjected to local traffic loads are found to have insufficient fatigue life with the applied traffic load model (FLM4).

16mm plate thickness is used in the top plate along the entire bridge to increase fatigue robustness for traffic loads. This has a direct effect on detail 2a and 2b (transverse plate splice). This measure is also believed to have an indirect positive effect on other bridge girder details.

Further measures are needed to arrive at acceptable fatigue lives for details governed by local traffic. The critical areas are limited to the parts directly loaded by wheel loads, especially in the slow lanes. Critical areas are indicated in Figure 6-1.

The following measures are recommended for future fatigue work to arrive at acceptable fatigue lives in the bridge girder deck:

- A fatigue load model should be developed based historical/forecasted traffic data for the actual bridge location (FLM5). This is expected to give a less conservative load model which will improve fatigue life. The load model sensitivity study (Section 5.5) show that to achieve acceptable fatigue life for some of the most common details the *medium range* traffic distribution should be applied along with reduced axle loads to 75% of the full FLM4 axle loads. This measure alone may be enough to achieve sufficient fatigue life for detail types 2b, 3a, 4 and 5.
- A fatigue friendly design should be further developed. This involves finding optimal structural dimensions (plate thicknesses, stiffener and cross-beam heights, stiffener spacing etc.) and shape for the cutout in cross-beams around trapezoidal stiffeners. In literature and design recommendations the importance of careful design of the stiffener to cross-beam connection with respect to fatigue is emphasized. Especially the shape of the cutout in transverse frames around trapezoidal stiffeners is important. In addition to a reduced fatigue load model, design optimization will be required at least for the cut-out detail around trapezoidal stiffeners.
- Apply a more refined local FE-analysis (preferably with use of solid elements) where the composite stiffness effect of the asphalt layer is included. It is shown in [15] that the wheel load is *not* uniformly distributed on the deck plate, but the load is concentrated to the trapezoidal webs. This is expected to give less bending stress in both the deck plate and trapezoidal web compared to a uniform load distribution. The asphalt stiffness vary strongly with temperature and a distribution or equivalent value must be established for the asphalt stiffness. During winter it is expected that the asphalt stiffness effect will be significant.

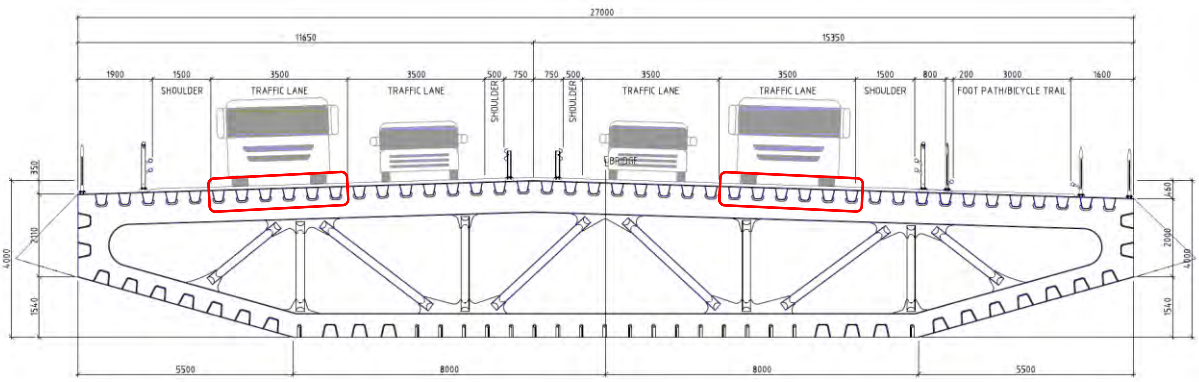


Figure 6-1 Areas of bridge girder deck with high fatigue utilization due to local traffic loads

Standard structural details in the bridge girder subjected to *global load effects* have been shown to generally have sufficient fatigue life. For the current assessment, detail type 1 has been assessed (transverse plate splice in the bridge girder susceptible to longitudinal stresses), which is considered as the worst detail with respect to global load effects. Both splices that will be welded during controlled conditions at the fabrication yard and splices that will be welded on site have been checked. The lowest calculated fatigue life in the two cases are 157 and 97 years respectively.

Larger misalignment tolerances may be required for the on-site splices and will hence give higher stress concentrations. If needed, the recommended strategy to increase fatigue life for the on-site splices is to locally increase the plate thickness in the area of the splice to reduce both SCF's and nominal stresses. The transition of plate thickness may need to be performed in several steps.

Other fatigue related work that has been identified and recommended as future fatigue work for the bridge girder is summarized here:

- For global loads, an additional check point on the cross section should be introduced at the topmost point on bridge deck as weak axis bending is governing for fatigue in certain parts of the bridge.
- For combined global and local loads, the inner wheel position may be worse for the same reason as the above point.
- For local loads and combined global and local loads, effect of tide has not been included. This may give significant contribution near the ends of the bridge and should be included in future fatigue work.
- Establish final misalignment tolerances for the bridge girder plate splices, both for shop and on-site welds.
- For detail type 3 only local load effects have been considered. It is expected that global load effects also will contribute to fatigue at some of the hotspots and global load effects should therefore be included in future fatigue work.

6.2 Mooring lines

Mooring lines are found to have sufficient fatigue life. In general, it is seen that short lines gets more fatigue damage. This is because the shorter lines have more dynamic response in the wind sea conditions.

Experience for the previous phase and the independent analysis performed by DNVGL [8] also indicate that the mooring lines in general have sufficient fatigue life. However, the effect of marine growth has been shown to have a negative effect on the fatigue life and needs to be accounted for in the analyses in further fatigue work.

The mooring line top chains have been assessed by both tension fatigue method and OPB/IPB method. The two methods give similar results.

6.3 Bridge girder / column / pontoon connections

The most highly stressed areas in the column connections are in the vicinity of the column corners towards the bridge girder and towards the pontoon.

Cast pieces are proposed for these locations and the fatigue checks performed in this assessment are for the butt welds between the bridge girder/column plating and the cast piece.

All column top locations along the bridge has been checked and the worst location is at Axis 3 where the calculated fatigue life is 44 years and 47 years for the girder side and column side respectively. To achieve a fatigue life of 100 years on the girder side the stresses must be reduced to the equivalent of an SCF of 1.50 in the current calculation. The column top at Axis 4 has a calculated fatigue life of 89 years (column side). All other column top locations are found to have sufficient fatigue lives. It is expected that sufficient fatigue life can be achieved at Axes 3 and 4 by increasing structural dimensions locally.

At the column bottom towards the pontoon the dynamic stresses are significantly lower and fatigue is not expected to be a problem. The lowest fatigue life is found to be around 9000 years, which indicate that cast pieces may not be necessary at the column to pontoon connection from a fatigue point of view.

It is noted that the approach taken in this assessment is quite simplified and a more refined method should be adopted in future fatigue work. E.g. a more direct coupling between global and local models should be applied.

The cast piece itself will also need to be assessed for fatigue.

6.4 Uncertainties

In Appendix H [16] it is found that wave interaction effects and the effect of inhomogeneous sea states may give increased weak axis bending moments in the bridge girder. These effects have not been addressed for the current fatigue assessment, but as the weak axis bending moment in wind sea already give significant contribution to fatigue, these effects should be addressed in future fatigue work.

7 References

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[2] AMC, "10205546-13-NOT-084 Shear lag and buckling effects of Bridge Girder," 24/05/2019.

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[4] Statens vegvesen, "Design basis Bjørnafjorden floating bridges, Rev 0," 2018.

[5] Statens vegvesen, "Metocean design basis, Rev 1," 2018.

[6] DNVGL, "Bjørnafjord side anchored floating bridge - independent local analyses, Rev 0," 2018.

[7] Eurocode 1, "NS-EN 1991-2-2: Traffic loads on bridges," 2003.

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[9] AMC, "SBJ-33-C5-AMC-28-RE-114 Appendix N: Construction and Marine Operations - K12," 15/08/2019.

[10] DNVGL, "DNVGL-OS-E301 Position mooring," 2018.

[11] Bureau Veritas, "NI 604 DT R00 E "Fatigue of Top Chain of Mooring Lines due to In-plane and Out-of-plane Bendings"," October 2014.

[12] AMC, "SBJ-33-C5-AMC-26-RE-113 Appendix M: Anchor systems - K12," 15/08/2019.

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[15] M. Li, "Fatigue Evaluation of Rib-to-Deck Joint in Orthotropic Steel Bridge Decks," Department of Civil and Earth Resources Engineering, Kyoto University, 2014.

[16] AMC, "SBJ-33-C5-AMC-21-RE-108 Appendix H: Global Analyses - Special studies - K12," 15/08/2019.

8 Enclosures

Enclosure 1	K12_05 Fatigue – Bridge girder screening
Enclosure 2	K12_07 Fatigue – Bridge girder selected points
Enclosure 3	Environmental load cases
Enclosure 4	Environmental load cases – Subset for coupled analyses
Enclosure 5	Local stress series from traffic