Statens vegvesen

Ferry free E39 –Fjord crossings Bjørnafjorden	304624

0	15.08.2019	Issued for use	JoS			
Rev.	Publish date	Description	Made by	Checked by	Project appro.	Client appro.

Client Statens vegvesen	
Contractor Norconsult DR. TECHN. OLAV OLSEN	Contract no.: 18/91094

Document name: K12 – Design of bridge girder		
Document no.: SBJ-33-C5-OON-22-RE-017	Rev.: 0	Pages: 46

CONCEPT DEVELOPMENT FLOATING BRIDGE E39 BJØRNAFJORDEN

K12 - DESIGN OF BRIDGE GIRDER

Norconsult DR. TECHN. OLAV OLSEN

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

REPORT

Project name:

CONCEPT DEVELOPMENT FLOATING BRIDGE E39 BJØRNAFJORDEN

Document name:

K12 - DESIGN OF BRIDGE GIRDER

Project number:	12777
Document number:	SBJ-33-C5-OON-22-RE-017
Date:	15.08.2019
Revision:	0
Number of pages:	46
Prepared by:	Jon Solemsli
Controlled by:	Anette Fjeld
Approved by:	Kolbjørn Høyland

Summary

The report describes the design of the bridge girder with calculation of cross-section properties. There are two types of cross section along the bridge. Type 1 is used in the cable stayed bridge and in the lower part of the floating bridge, and type 2 is used in the high part of the floating bridge. The difference between these two types are the plate thickness in the stiffeners. All sections are reinforced with additional longitudinal bulkheads at column supports. In addition, the bridge girder ends are reinforced gradually the last about 56 meters at both abutments north and south. At these bridge ends, the reinforcement allows for full yielding with plastic hinges. This gives a robust design for overload.

In the calculations of cross-section properties, all reinforcements at bridge ends and at column/girder connections are included. In addition, shear lag effects are included both at column support and in mid spans.

Further, a complete stress calculation in ULS is performed along the whole bridge with these properties. Maximum ULS tension stress in girder is 342 MPa and maximum compression stress is -354 MPa. Due to heavy reinforcements at column supports, the stress level is significantly lower than stresses reported from the global analyses. At midspans, the stresses are higher than stresses from global analyses, due to shear lag effects.

Cross-section resistance to shear and torsion, bending and axial forces are reported. It is shown that the bridge girder has resistance to all ULS forces along the bridge. The stress level in the girder are generally well within design stresses.

It is also shown that the bridge girder has resistance to global effects of ship impact. Maximum stresses are maximum 280 MPa at abutments, and 363 MPa. The ability to develop plastic hinges at abutments gives high robustness for increased ALS loads.

Table of Content

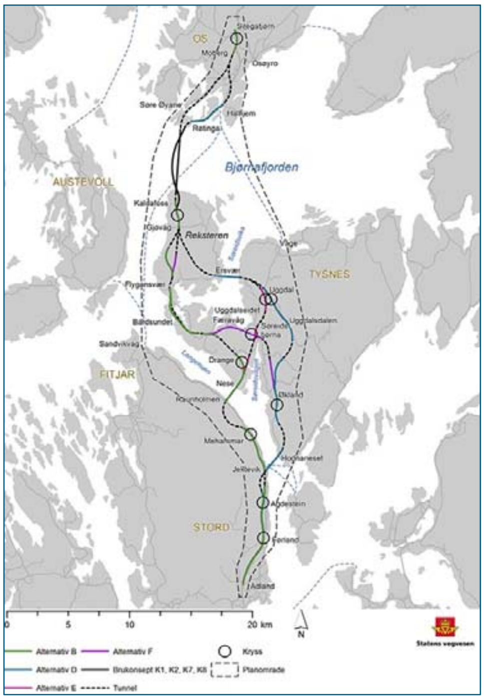
APPENDIX

SBJ-33-C5-OON-22-RE-017-B-AppA	Shear lag study
SBJ-33-C5-OON-22-RE-017-B-AppB	Forces for stress calculations
SBJ-33-C5-OON-22-RE-017-B-AppC	Optimalization study of bottom plate stiffeners

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
- Pure Logic 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 (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
- Heyerdahl Arkitekter has in the last 20 years been providing architect services to major national infrastructural projects, both for roads and rails. The company shares has been sold to Norconsult, and the companies will be merged by 2020.
- Haug og Blom-Bakke

- FORCE Technology 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 Project scope

The objective of the current project phase is to develop 4 nominated floating bridge concepts, document all 4 concepts sufficiently for ranking, and recommend the best suited alternative. The characteristics of the 4 concepts are as follows:

- K11: End-anchored floating bridge. In previous phase named K7.
- K12: End-anchored floating bridge with mooring system for increase robustness and redundancy.
- K13: Straight side-anchored bridge with expansion joint. In previous phase named K8.
- K14: Side-anchored bridge without expansion joint.

In order to make the correct recommendation all available documentation from previous phases have been thoroughly examined. Design and construction premises as well as selection criteria have been carefully considered and discussed with the Client. This form basis for the documentation of work performed and the conclusions presented. Key tasks are:

- Global analyses including sensitivity studies and validation of results
- Prediction of aerodynamic loads
- Ship impact analyses, investigation of local and global effects
- Fatigue analyses
- Design of structural elements
- Marine geotechnical evaluations
- Steel fabrication
- Bridge assembly and installation
- Architectural design
- Risk assessment

1.4 Current report

This report describes the design of the bridge girder with capacity checks.

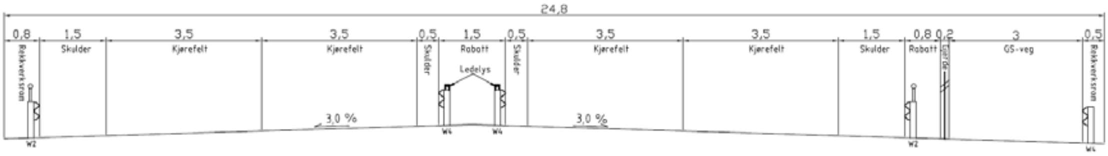
2 DESCRIPTION OF THE BRIDGE GIRDER

This chapter of the report describes the structural elements and the structural behavior of the bridge girder.

2.1 Bridge girder geometry

2.1.1 Functional criteria

The design basis [1] outlines the functional criteria for the roadway on the bridge girder. The figure below shows the division of the roadway into different traffic lanes and the required guard rails and safety zones separating the traffic.

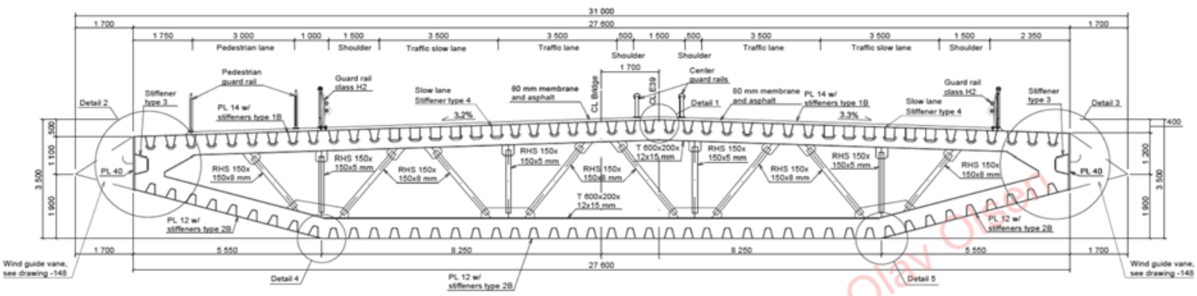


> *Figure 1 Roadway*

The total width of the roadway including safety zones is 24,8 m and the road surface should have a crossfall of about 3,0 % to each side. The apex of the roadway is centered in the roadway and there should be a pedestrian lane on one side of the bridge. The topside of the bridge girder will therefore be unsymmetrical.

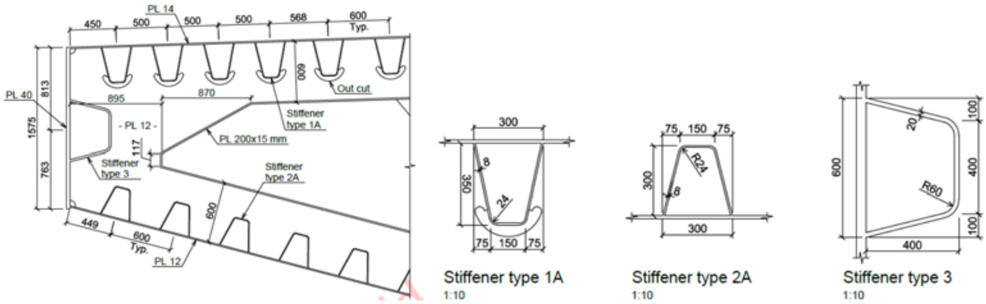
2.1.2 Cross sections

The cross section of the bridge girder is constructed as a steel box girder with stiffened plate panels welded together to form a box. There are transverse girders with 4 meters spacing. The shape of the box girder is shown in the figure below with outer dimensions.

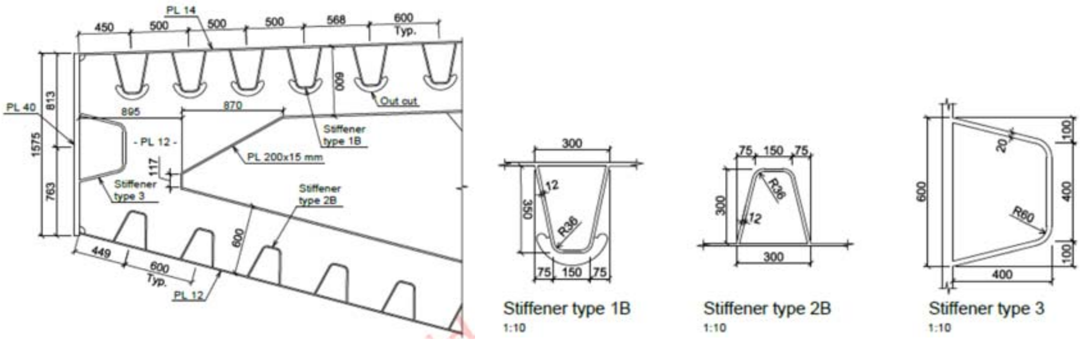


> *Figure 2 Bridge cross section*

The outer dimensions of the box girder are kept constant throughout the length of the bridge. In order to handle the varying section forces along the bridge, the thickness of the stiffeners is varied along the bridge. There are 2 types of cross-section. The figures below show the detailed geometry of the plate panels.



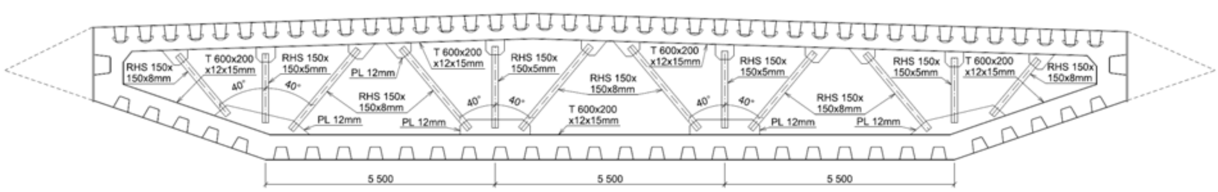
> *Figure 3* *Section 1 - standard cross section in low part of floating bridge and cable stayed bridge*



> *Figure 4* *Section 2 - reinforced cross section in high part of floating bridge*

In next phase of the project, further optimization of cross sections should be performed. In Appendix C, a study of various stiffeners is reported. It may be possible to reduce steel amount in mid spans of the girder with use of smaller stiffeners.

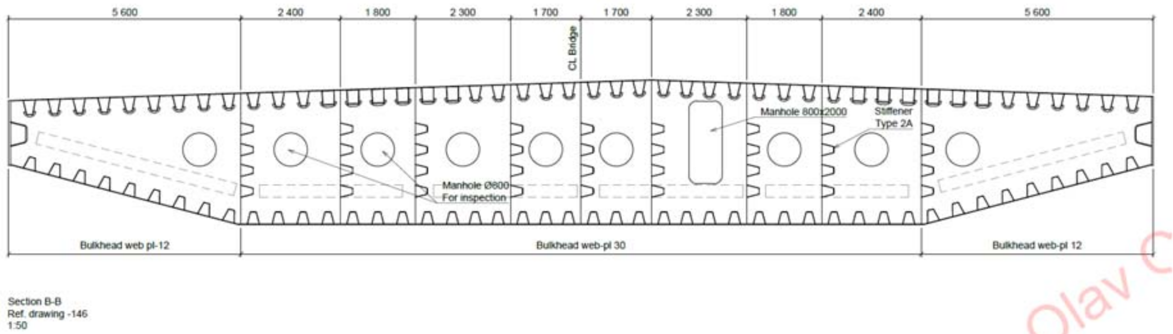
Transverse girders/frames spaced with distance 4,0 m along the bridge, except in bridge ends. They are constructed as truss structures. The geometry of the cross frames is shown in the figure below.



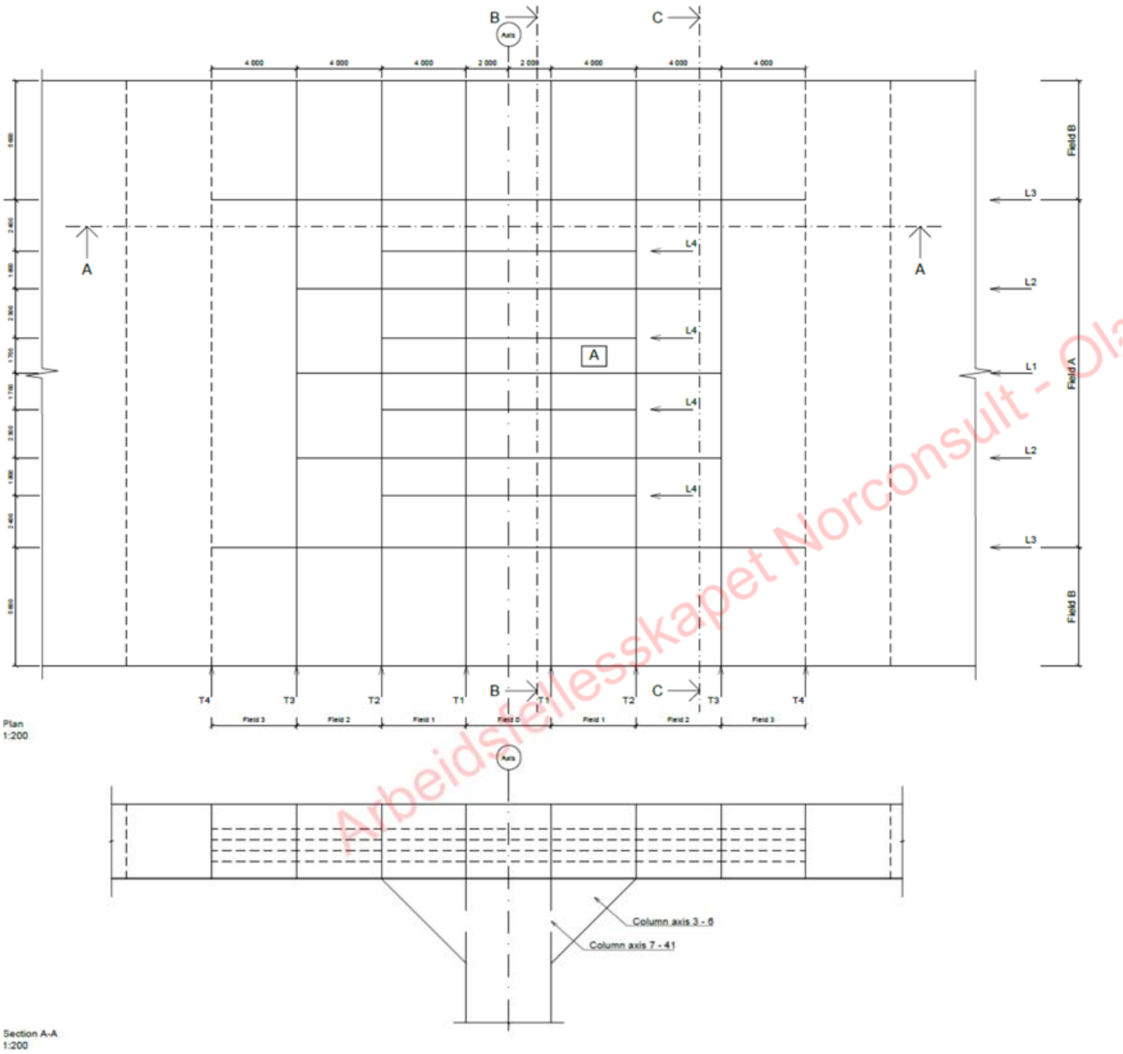
> *Figure 5* *Transverse girders in section 1 and 2*

The transverse girders consist of a T-section beam (T 600x200x12x15 mm) welded to the plates and stiffeners around the cross section and a truss structure of RHS 150x150x8 mm as diagonal struts and RHS 150x150x5 mm as vertical struts.

At column/girder connections there are heavy reinforcements with additional longitudinal bulkheads and reinforced transverse bulkheads.

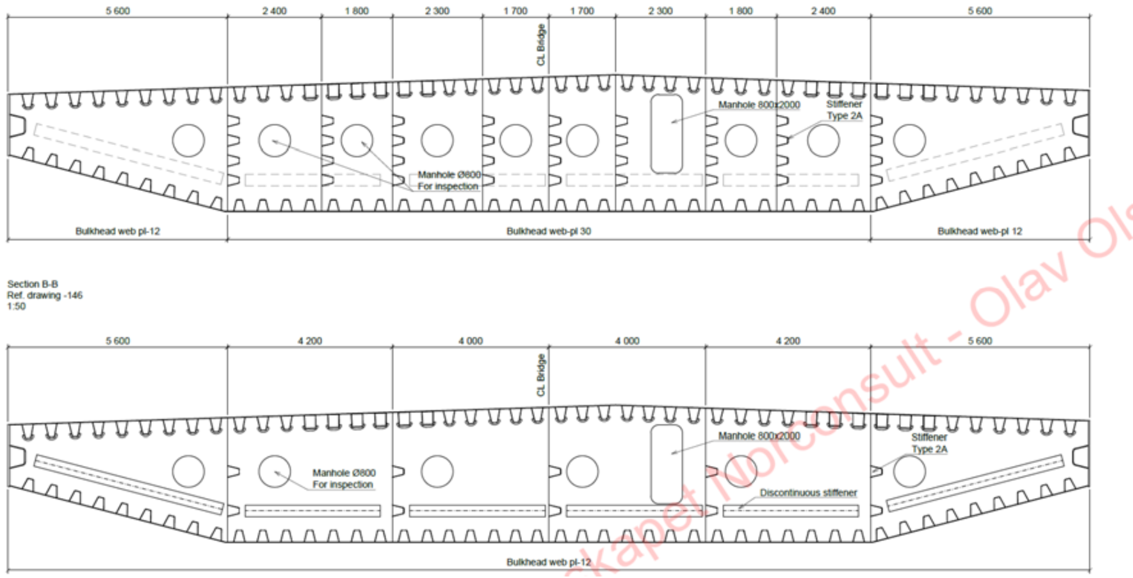


> *Figure 6* *Cross section at column connection*

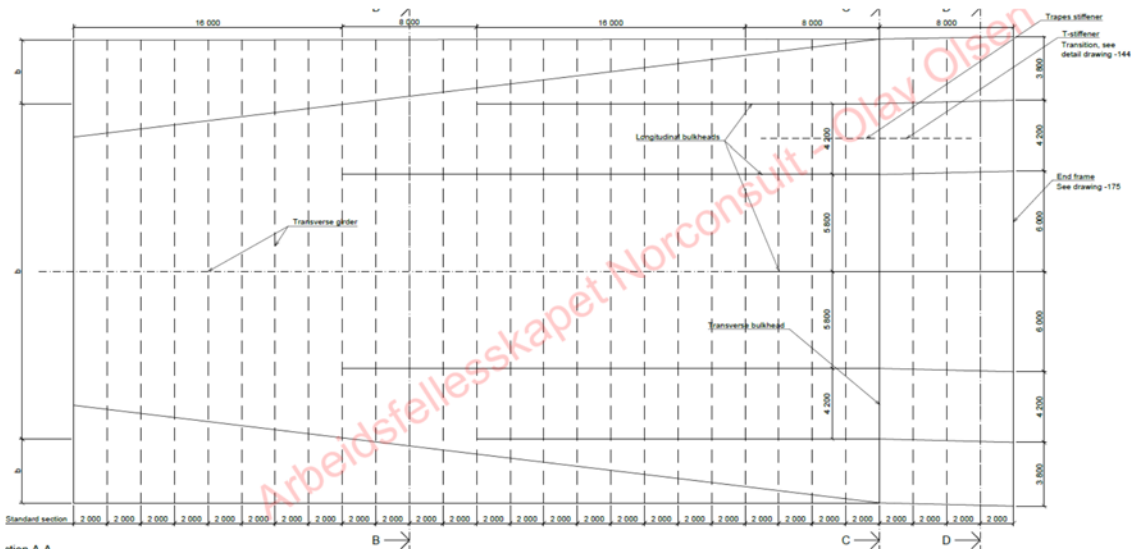


> *Figure 7* *Plan and elevation of column connection*

The last 56 meters of the girder are in both ends towards abutment connection, heavily reinforced. There are additional longitudinal bulkheads and transverse girders/bulkheads with spacing 2 meters. The cross section at the ends allows for full yielding with plastic hinges according to NS-EN 1993-1-1 sec. 5.6. This introduces a large robustness to the concept for overload.



> *Figure 8* *Typical cross sections close to abutments. Cross section allows for plastic hinges*



> *Figure 9* *Arrangement at bridge ends -plan*

2.2 Material Properties

The steel grade used in the bridge girder is steel grade S420N/NL according to NS-EN 10025-3 with the following properties:

Property	Symbol	Value	Reference
Yield strength for plate thickness t≤40mm	f_y	420 MPa	[2] table 3.1
Ultimate strength for plate thickness t≤40mm	f_u	520 MPa	[2] table 3.1
Yield strength for plate thickness 40mm<t≤80mm	f_y	390 MPa	[2] table 3.1
Ultimate strength for plate thickness 40mm<t≤80mm	f_u	520 MPa	[2] table 3.1
Young's modulus	E	210000 MPa	[2] section 3.2.6
Shear modulus	G	81000 MPa	[2] section 3.2.6
Poisson's ratio	v	0,3	[2] section 3.2.6
Coefficient of thermal expansion	α	$12*10^{-6}$ K^{-1}	[2] section 3.2.6
Safety factor for cross section resistance	γ_{m0}, γ_{m1}	1,10	[3] section NA.6.1
Safety factor for welded connections	γ_{m2}	1,25	[3] section NA.6.1

2.3 Steel weight summary

Summary of the steel weights are given in the table below.

	Mass	Length - unit	[tons]
Girder type 1 – low floating bridge	13.506 t/m	4009 m	54146
Girder type 2 – high floating bridge	15.321 t/m	665 m	10188
Girder type 1 – cable stayed bridge	13.506 t/m	654 m	8833
Girder abutment connection south	19.300 t/m	56 m	1081
Girder abutment connection north	19.300 t/m	56 m	1081
Girder column connection axis 3 – 6	164 t	4 units	656
Girder column connection axis 7 – 41	112	35 units	3920
SUM STRUCT. STEEL			**79905**

3 CROSS SECTION PROPERTIES

3.1 Properties used in global analyses

The cross-section properties used in the global static and dynamic analyses is summarized in the table below. The properties in the below table are calculated based on the gross cross section without reductions for shear lag effects and plate buckling, and without reinforcements at the column-girder connections.

	Section 1	Section 2	P1	P2	P3	P4	P5	P6
Position x from south	51 – 710 and 1375 - 5384	710 - 1375	5384 - 5395	5395 - 5405	5405 - 5415	5415 - 5425	5425 - 5435	5435 – 5440 and 0 - 51
A	1,47 m²	1,74 m²	1,59 m²	1,82 m²	2,05 m²	2,29 m²	2,52 m²	2,634 m²
COG	1,91 m	1,91 m	1,88 m	1,82 m	1,77 m	1,71 m	1,65 m	1,62 m
I_y	2,71 m⁴	3,20 m⁴	2,95 m⁴	3,41 m⁴	3,88 m⁴	4,35 m⁴	4,81 m⁴	5,049 m⁴
I_z	114,9 m⁴	132,0 m⁴	121,5 m⁴	134,8 m⁴	148,0 m⁴	161,2 m⁴	174,5 m⁴	181,1 m⁴

Stresses reported from the global analyses are calculated based on these properties. It will later be shown that these stresses give results to safe side. However, some of these stresses are above design stresses, and consequently, an updated design check based on actual cross sections must be executed.

In next chapter, cross section properties based on the actual design is calculated.

3.2 Design cross section properties

Design cross sections are based on the actual design:

- Reinforcements at column supports, as shown on drawings are included in section properties
- Reinforcements at abutment supports, as shown on drawings are included in section properties
- Shear lag effects are conservatively accounted for

Design cross sections are used to re-calculate stresses at column supports and mid spans along the bridge.

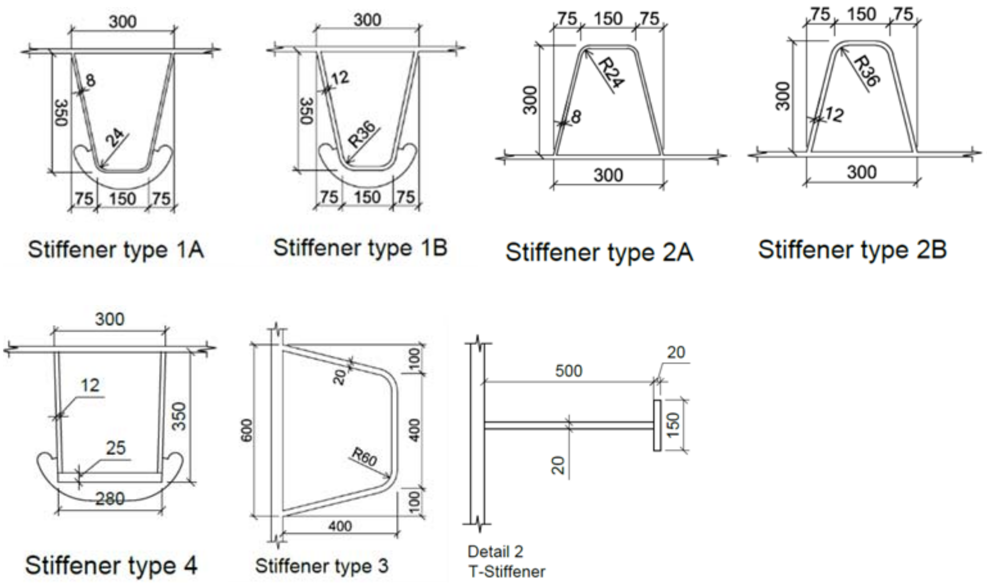
3.2.1 Cross-section member classification

All members are classified to ensure that no reduction due to local buckling must be considered in the calculations.

Cross sections members are classified according to NS-EN 1993-1-1, Table 5.2.
ε = 0,75 for S420 steel.

For compressed members class 1: $c/t < 33\varepsilon \Rightarrow c/t < 24{,}75$
For compressed members class 2: $c/t < 38\varepsilon \Rightarrow c/t < 28{,}50$
For compressed members class 3: $c/t < 42\varepsilon \Rightarrow c/t < 31{,}50$

Stiffeners are shown below:



> *Figure 10* *The different stiffener types*

> *Table 1* *Classification of plate members*

Classification of plate members in compression						
Part	Location	Max Stiffener spacing	c	t	c/t	Class
Deck plate	Whole bridge except at abutments	600	300	14	21,42857	Class 1
Deck plate	At abutments	600	580	24	24,16667	Class 1
Bottom plate	Whole bridge except supports and abutments	620	320	12	26,66667	Class 2
Bottom plate	At column supports	600	300	20	15	Class 1
Bottom plate	At abutments	600	580	24	24,16667	Class 1
Vertical webs	Whole bridge		600	40	15	Class 1
Classification of stiffener members in compression						
Stiffener	Stiffener part		c	t	c/t	Class
1A	Web		333	8	41,625	Class 3
1A	Flange		150	8	18,75	Class 1
1B	Web		322	12	26,83333	Class 2
1B	Flange		150	12	12,5	Class 1
2A	Web		283	8	35,375	Class 3
2A	Flange		150	8	18,75	Class 1
2B	Web		272	12	22,66667	Class 1
2B	Flange		150	12	12,5	Class 1
3	Web		350	20	17,5	Class 1
3	Flange		400	20	20	Class 1
4	Web		325	12	27,08333	Class 2
4	Flange		280	25	11,2	Class 1
T	Web		500	20	25	Class 2
T	Flange * Outstanding member		65	20	3,25	Class 1

As can be seen from the table, all members are classified to minimum class 3, meaning that all parts will be fully effective in compression and no reduction for local buckling is required.

3.2.2 Cross sections in mid spans

There are no reinforcements in mid spans and the cross sections are like those used in the analyses. For weak axis bending, the effect of shear lag is included. In Appendix A, the shear lag study shows that there is an effect at midspan for a typical equally distributed permanent load, but no effect for a typical forced displacement.

The total response in weak axis bending is typically consisting of 50% bending for distributed load and 50% from environmental loads which may be represented by a forced displacement as discussed in Appendix A.

A practical, and most probably conservative approach would be to use a mean between full plate width b_0 and the effective plate width b_{eff} calculated below.

Shear lag effect according to NS-EN 1993-1-5 section 3.2 is calculated. This will be valid for a typical permanent loading.

Effective length spans $L_e = 0{,}72 \cdot L_2 = 0{,}72 \cdot 120 = 86{,}4$ m

Symbols according to NS-EN 1991-1-5 Table 3.1:

A_{sl} = stiffener area
$b \cdot t$ = plate area within stiffener
$\alpha_0 = \sqrt{1 + A_{sl}/b \cdot t}$
$\kappa = \alpha_0 \cdot b_0 / L_e$
$\beta = 1/(1+6{,}4 \cdot \kappa^2)$ Sagging bending

> *Table 2 Effective width for deck and bottom plate*

	b	A_{sl}	α_0	b_0	κ	β	b_{eff}	B	Beff	B_{mean}
Deck 1A	27,6	6800	1,345	13,8	0,215	0,772	10,65	27,60	21,31	24,45
Deck 1B	27,6	10200	1,488	13,8	0,238	0,734	10,14	27,60	20,27	23,94
Bottom 2A	16,5	6000	1,354	8,25	0,129	0,903	7,45	16,50	14,91	15,70
Bottom 2B	16,5	9000	1,500	8,25	0,143	0,884	7,29	16,50	14,59	15,54

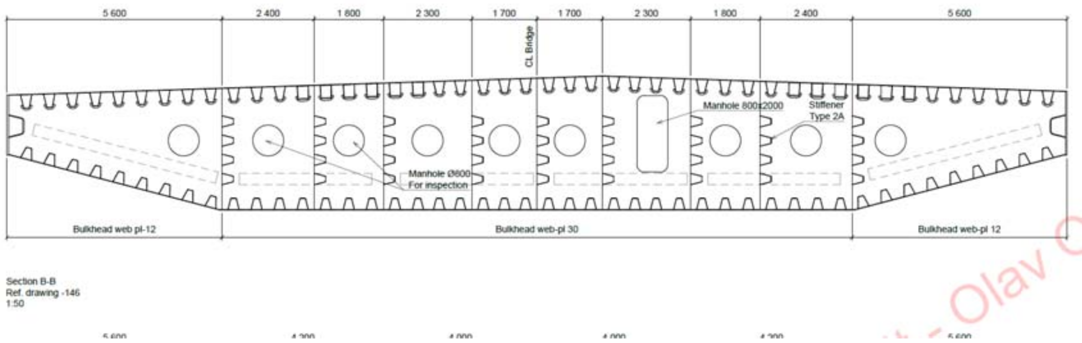
Cross section parameters are calculated with effective plate widths. Results are tabulated below compared with analyses.

> *Table 3 Cross section properties*

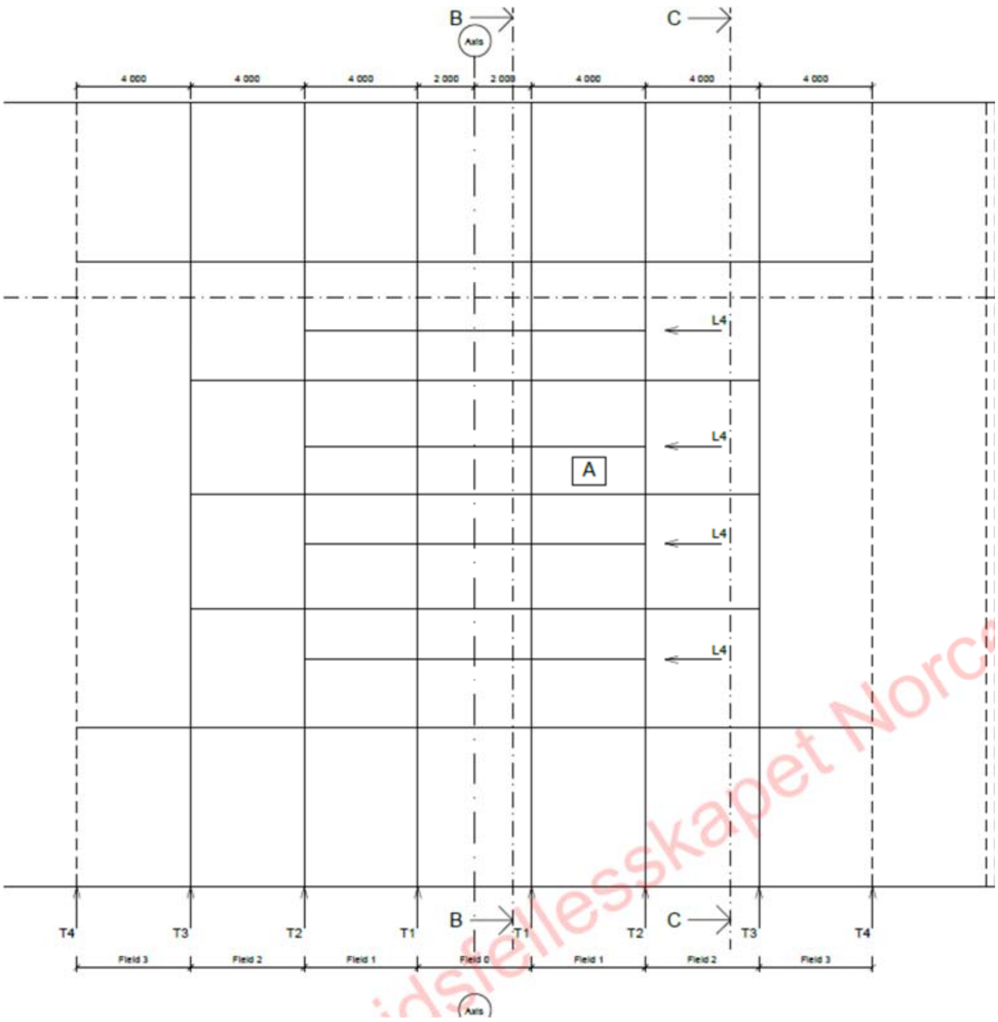
	Design		Analyses	
	Type 1	Type 2	Type 1	Type 2
A	1,47	1,74	1,47	1,74
NA	1,85	1,81	1,91	1,91
I_y	2,52	3,06	2,71	3,2
I_z	114,91	132,00	114,9	132
W_{yo}	1,362	1,688	1,419	1,675
W_{yu}	1,527	1,814	1,704	2,013

3.2.3 Cross sections at column supports

There are heavy reinforcements at supports as described. In Appendix A, the shear lag study shows that there is a small effect at support for a typical equally distributed permanent load, but no effect for a typical forced displacement. Same procedure as for mid span is used.



> *Figure 11* *Cross section at column support*



> *Figure 12* *Bulkhead arrangement at column support - plan*

Effective length span $L_e = 0{,}25\cdot(L_1+L_2) = 0{,}25\cdot(120 + 120) = 60$ m

Symbols according to NS-EN 1991-1-5 Table 3.1:

A_{sl} = stiffener area
$b\cdot t$ = plate area within stiffener
$a_0 = \sqrt{1 + A_{sl}/b\cdot t}$
$k = a_0\cdot b_0 / L_e$
$b = 1/(1+6{,}0(k-1/2500k)+1.6k^2)$ Hogging bending

> *Table 4 Effective width for deck plate*

	b	A_{sl}	a_0	b_0	k	b	b_{eff}	B	Beff	B_{mean}
Deck 1A	5,6	6800	1,345	2,8	0,063	0,727	2,03			
Deck 1A	2,4	6800	1,345	1,2	0,027	0,927	1,11			
Deck 1A	1,8	6800	1,345	0,9	0,020	0,994	0,89			
Deck 1A	2,3	6800	1,345	1,15	0,026	0,936	1,08			
Deck 1A	1,7	6800	1,345	0,85	0,019	1,008	0,86	27,60	23,90	25,75
Deck 1B	5,6	10200	1,488	2,8	0,069	0,700	1,96			
Deck 1B	2,4	10200	1,488	1,2	0,030	0,904	1,08			
Deck 1B	1,8	10200	1,488	0,9	0,022	0,970	0,87			
Deck 1B	2,3	10200	1,488	1,15	0,029	0,913	1,05			
Deck 1B	1,7	10200	1,488	0,85	0,021	0,983	0,84	27,60	23,22	25,41

> *Table 5 Effective width for bottom plate*

	b	A_{sl}	a_0	b_0	k	b	b_{eff}	B	Beff	B_{mean}
Bottom 2A	2,4	6000	1,354	1,2	0,027	0,925	1,11			
Bottom 2A	1,8	6000	1,354	0,9	0,020	0,992	0,89			
Bottom 2A	2,3	6000	1,354	1,15	0,026	0,935	1,08			
Bottom 2A	1,7	6000	1,354	0,85	0,019	1,007	0,86	16,50	15,84	16,17
Bottom 2B	2,4	9000	1,500	1,2	0,030	0,902	1,08			
Bottom 2B	1,8	9000	1,500	0,9	0,023	0,968	0,87			
Bottom 2B	2,3	9000	1,500	1,15	0,029	0,912	1,05			
Bottom 2B	1,7	9000	1,500	0,85	0,021	0,981	0,83	16,50	15,44	15,97

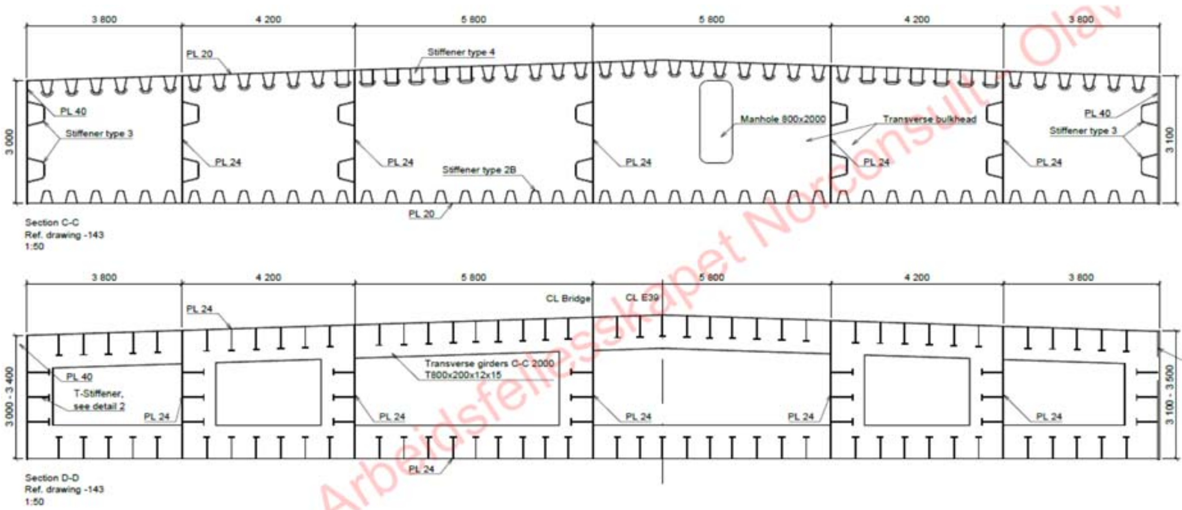
Cross section parameters are calculated with effective plate widths. Results are tabulated below compared with analyses.

> *Table 6 Cross section properties at columns*

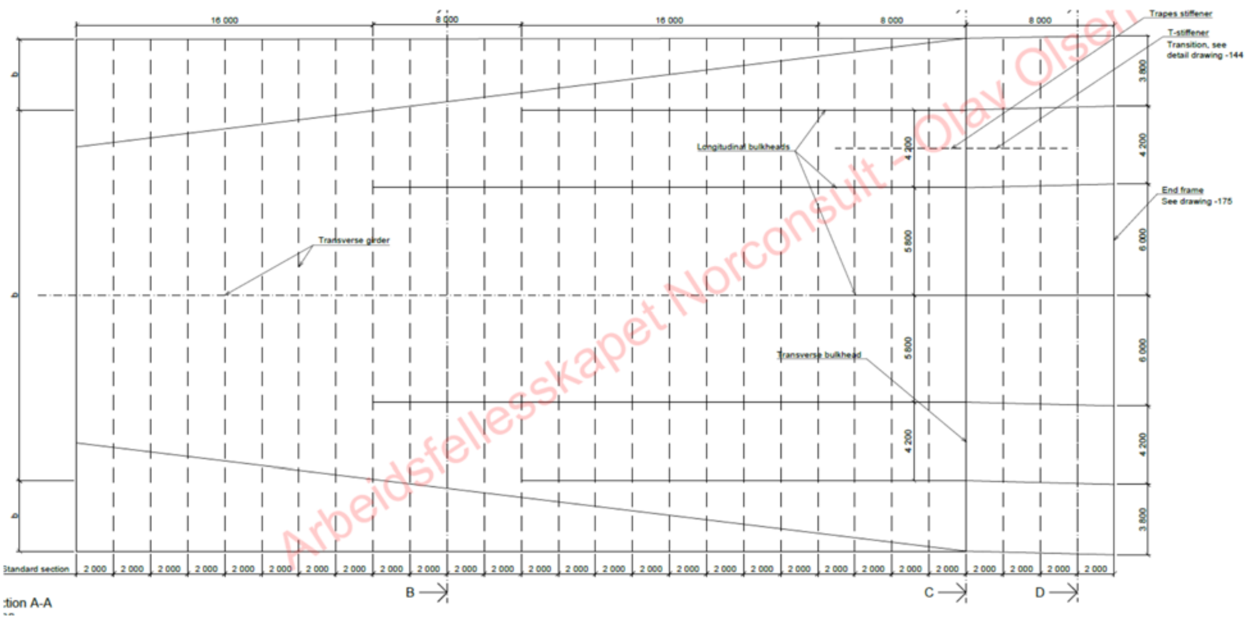
	Design		Analyses	
	Type 1	Type 2	Type 1	Type 2
A	2,25	2,52	1,47	1,74
NA	1,68	1,69	1,91	1,91
I_y	3,59	4,16	2,71	3,2
I_z	136,20	153,28	114,9	132
W_{yo}	2,132	2,469	1,419	1,675
W_{yu}	1,975	2,295	1,704	2,013

3.2.4 Cross sections at bridge ends

There are heavy reinforcements in the girder at the abutments. In Appendix A, the shear lag study shows that there is a small effect at support for a typical equally distributed permanent load, but no effect for a typical forced displacement. The total response in weak axis bending is dominated by bending from permanent loads and traffic and forced displacement from tide and waves. The same procedure as for column supports will be used for calculating effective width.



> *Figure 13 Cross sections at abutments*



> *Figure 14 Arrangement at abutments – plan*

> *Table 7 Effective width for deck plate*

	b	A_{sl}	a_0	b_0	k	b	b_{eff}	B	Beff	B_{mean}
x=0	Hogging									
Deck T	3,8	13000	1,242	1,9	0,039	0,841	1,60			
Deck T	4,2	13000	1,242	2,1	0,043	0,817	1,72			
Deck T	6	13000	1,242	3	0,062	0,729	2,19	28,00	22,01	25,00
x = 8	Hogging									
Deck 1B	3,6	10200	1,360	1,8	0,041	0,832	1,50			
Deck 1B	4,2	10200	1,360	2,1	0,048	0,796	1,67			
Deck 1B	6	10200	1,360	3	0,068	0,706	2,12	27,60	21,14	24,37
x = 16	Hogging									
Deck 1B	3,6	10200	1,360	1,8	0,041	0,832	1,50			
Deck 1B	4,2	10200	1,360	2,1	0,048	0,796	1,67			
Deck 1B	12	10200	1,360	6	0,136	0,506	3,04	27,60	18,75	23,17
x = 32	Sagging									
Deck 1B	7,8	10200	1,488	3,9	0,097	0,944	3,68			
Deck 1B	12	10200	1,488	6	0,149	0,876	5,26	27,60	25,23	26,41
x = 56	Sagging									
Deck 1B	27,6	10200	1,488	13,8	0,238	0,734	10,14	27,60	20,27	23,94

> *Table 8 Effective width for bottom plate*

	b	A_{sl}	a_0	b_0	k	b	b_{eff}	B	Beff	B_{mean}
x=0	Hogging									
Bottom T	3,8	13000	1,242	1,9	0,039	0,841	1,60			
Bottom T	4,2	13000	1,242	2,1	0,043	0,817	1,72			
Bottom T	6	13000	1,242	3	0,062	0,729	2,19	28,00	22,01	25,00
x = 8	Hogging									
Bottom 2B	3,6	9000	1,323	1,8	0,040	0,838	1,51			
Bottom 2B	4,2	9000	1,323	2,1	0,046	0,802	1,68			
Bottom 2B	6	9000	1,323	3	0,066	0,713	2,14	27,60	21,33	24,47
x = 16	Hogging									
Bottom 2B	2,675	9000	1,323	1,3375	0,029	0,906	1,21			
Bottom 2B	4,2	9000	1,323	2,1	0,046	0,802	1,68			
Bottom 2B	12	9000	1,323	6	0,132	0,515	3,09	25,75	17,76	21,75
x = 32	Sagging									
Bottom 2B	5,025	9000	1,500	2,5125	0,063	0,975	2,45			
Bottom 2B	12	9000	1,500	6	0,150	0,874	5,24	22,05	20,29	21,17
x = 56	Sagging									
Bottom 2B	16,5	9000	1,500	8,25	0,143	0,884	7,29	16,50	14,59	15,54

Cross section parameters are calculated with effective plate widths. Results are tabulated below compared with analyses.

> *Table 9 Cross section properties at bridge ends*

Pos x	0		8		16	
	Design	Analyses	Design	Analyses	Design	Analyses
A	4,49	2,634	3,51	2,52	2,77	2,29
NA	1,60	1,62	1,61	1,65	1,71	1,71
I_y	8,95	5,049	6,58	4,81	5,23	4,35
I_z	329,4	181,1	254,5	174,5	218,5	161,2
W_{yu}	5,580	3,117	4,077	2,915	3,057	2,544
W_{yo}	3,897	2,214	3,488	2,600	2,918	2,430

Pos x	32		48	
	Design	Analyses	Design	Analyses
A	2,11	2,05	2,03	1,47
NA	1,78	1,77	1,78	1,91
I_y	4,22	3,88	3,32	2,71
I_z	158,2	148	142,4	114,9
W_{yu}	2,365	2,192	1,863	1,419
W_{yo}	2,460	2,243	1,932	1,704

3.2.5 Shear areas

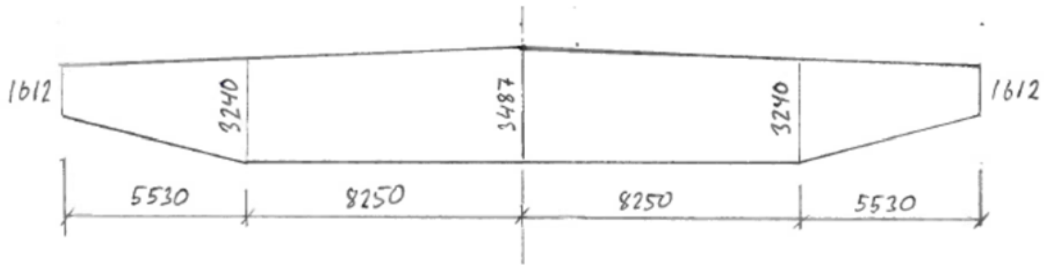
Shear areas in y and z direction are calculated below. For simplification, and to safe side, only vertical webs are considered to contribute to A_{Vz}. Deck, bottom plate and inclined plates are considered to contribute to A_{Vy}.

In mid span: $A_{Vz} = (1{,}575 + 1{,}675)\cdot 0{,}04 = 0{,}13 \text{ m}^2$
$A_{Vy} = 27{,}6\cdot(0{,}014 + 0{,}012) = 0{,}72 \text{ m}^2$

At columns: 9 additional webs with a total thickness 198 mm
$A_{Vz} = 0{,}13 + 3{,}25\cdot 0{,}198 = 0{,}77 \text{ m}^2$
$A_{Vy} = 0{,}72 \text{ m}^2$

3.2.6 Torsion area

Torsion area of outer plates is calculated from idealized figure below, in the center of outer plates.



> Figure 15 Measures used for calculating torsion area

$A_T = (1{,}612 + 3{,}240) \cdot 5{,}530 + (3{,}24 + 3{,}487) \cdot 8{,}25 = 82{,}3 \text{ m}^2$

4 ULS CROSS-SECTION CAPACITY CHECK

4.1 Method

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

1. Calculation of shear resistance according to EN 1993-1-1, section 6.2.6
2. Interaction between shear and bending are neglected when utilization in shear is less than 50% according to EN 1993-1-1, section 6.2.8
3. Tension stress in all members are calculated. Capacity is limited to yield-stress
4. Compression stress in plates are calculated. Since local buckling will not occur, capacity is limited to yield-stress. Stresses are calculated in outermost fiber.
5. Compression stress in stiffeners is calculated and stiffener buckling is checked. Stresses are calculated in COG of the stiffener.
    - Column type buckling behavior will be dominant for all stiffened plates
    - The buckling load resistance gives the maximum allowable compression stress in COG of the stiffener/plate column.

4.2 Plate buckling due to shear stresses t_{xy}

Shear buckling resistance is calculated for the stiffened plates and summarized below. All plate panels can be utilized to full yield.

Resistance to shear of plates with trapezoidal longitudinal stiffeners NS-EN 1993-1-5 sec 5

	Type 1 Deck		Type 1 Deck		Type 1 Bottom		Type 2 Bottom		Side panel		Side panel	
hw	27600	mm	27600	mm	16500	mm	27600	mm	1700	mm	1700	mm
Isl	2,47E+08	mm^4	3,19E+08	mm^4	1,63E+08	mm^4	2,09E+08	mm^4	1,71E+09	mm^4	5,81E+08	mm^4
k_{tsl}	18348		22234		6786		22927		14		132	
k_t	18606		22492		6881		23185		19		137	
s_E	0,0489	Mpa	0,0489	Mpa	0,1005	Mpa	0,0359	Mpa	105,1903	Mpa	12,8858	Mpa
t_{cr}	910	MPa	1100	MPa	692	MPa	833	MPa	1953	MPa	1770	MPa
l_w	0,52		0,47		0,59		0,54		0,35		0,37	
c_w	1,20		1,20		1,20		1,20		1,20		1,20	
V_{bwRd}	102215	kN	102215	kN	52377	kN	87613	kN	17988	kN	6296	kN
t_{bwRd}	265	Mpa	265	Mpa	265	Mpa	265	Mpa	265	Mpa	265	Mpa

4.3 Shear resistance

Shear and torsion resistance are calculated as follows:

Mid span

$$V_{c,Rdz} = A_{Vz} \cdot t_d = 0{,}13 \cdot 220 = 28{,}6 \text{ MN}$$
$$V_{c,Rdy} = A_{Vy} \cdot t_d = 0{,}72 \cdot 220 = 158{,}4 \text{ MN}$$
$$T_{Rd} = 2 \cdot A_T \cdot t_{min} \cdot t_d = 2 \cdot 82{,}3 \cdot 0{,}012 \cdot 220 = 435 \text{ MNm}$$

At column

$$V_{c,Rdz} = A_{Vz} \cdot t_d = 0{,}77 \cdot 220 = 169{,}4 \text{ MN}$$
$$V_{c,Rdy} = A_{Vy} \cdot t_d = 0{,}72 \cdot 220 = 158{,}4 \text{ MN}$$
$$T_{Rd} = 2 \cdot A_T \cdot t_{min} \cdot t_d = 2 \cdot 82{,}3 \cdot 0{,}012 \cdot 220 = 435 \text{ MNm}$$

Shearforce and torsion moment envelopes are given from analyses.

> *Figure 16 Shear and torsion envelopes*

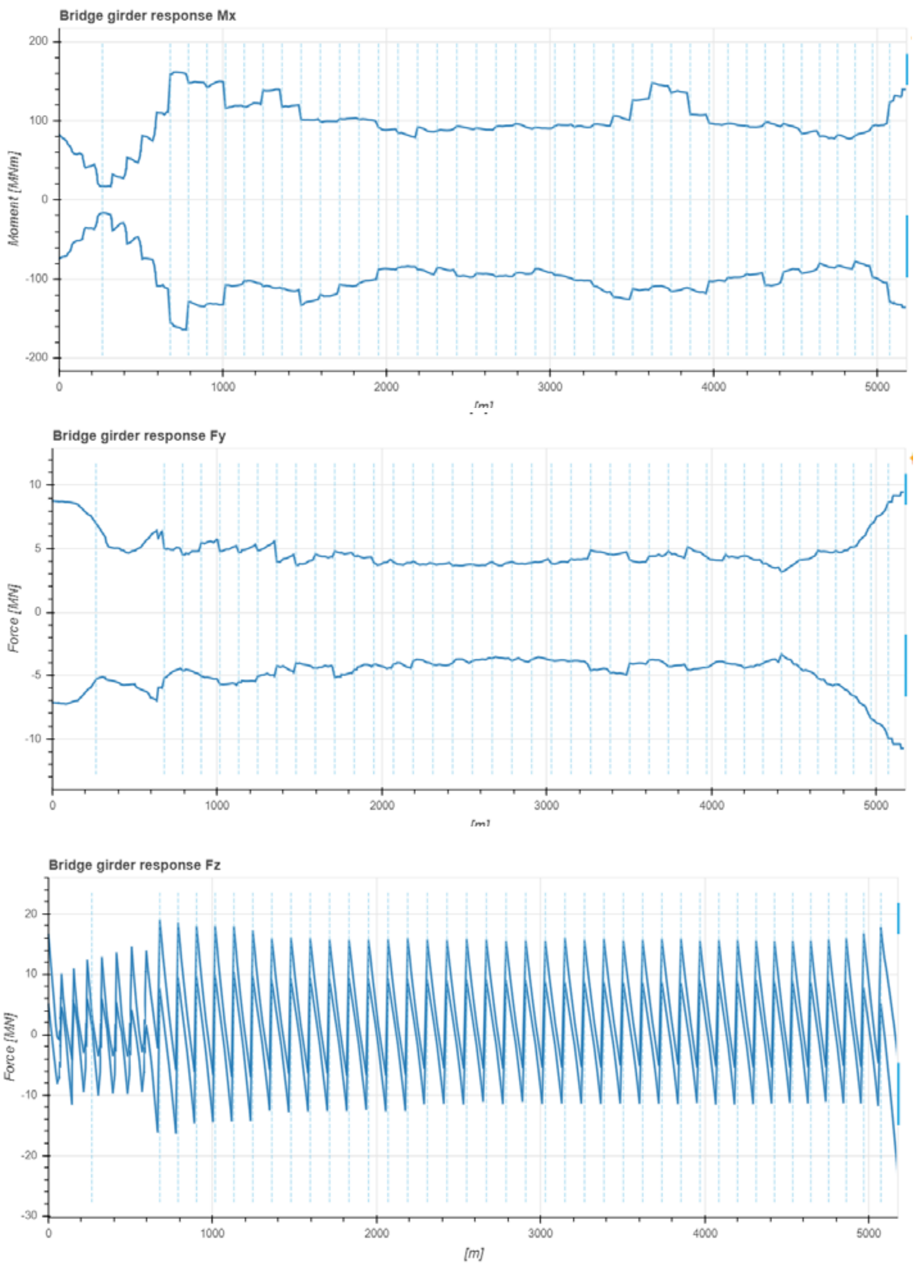


Figure 17 ULS Torsion moment and shear forces along the bridge

Maximum values and utilization:

T = 162 MNm at cable stayed bridge   Utilization 162/435 = 0,37

Vy = 10 MN at abutment   Utilization 10/158 = 0,06

Vz = 19 MN at column Utilization 19/169 = 0,11

Vz = 3,5 MN at mid span Utilization 3,5/28,6 = 0,12

Combined shear and torsion utilization is maximum 0,49 in mid span. This is a conservative assumption since maximum values are combined. Interaction with bending can therefore be neglected.

4.4 Plate buckling due to direct stress

Plate buckling resistance is checked according to NS-EN 1993-1-5 sec. 4.5. Column behavior buckling is governing.

Maximum compression stresses are calculated for stiffener type 1A, 2A, 1B, 2B and 3 in ordinary cross section 1 and 2 below. The stresses are in the centroid of the effective column.

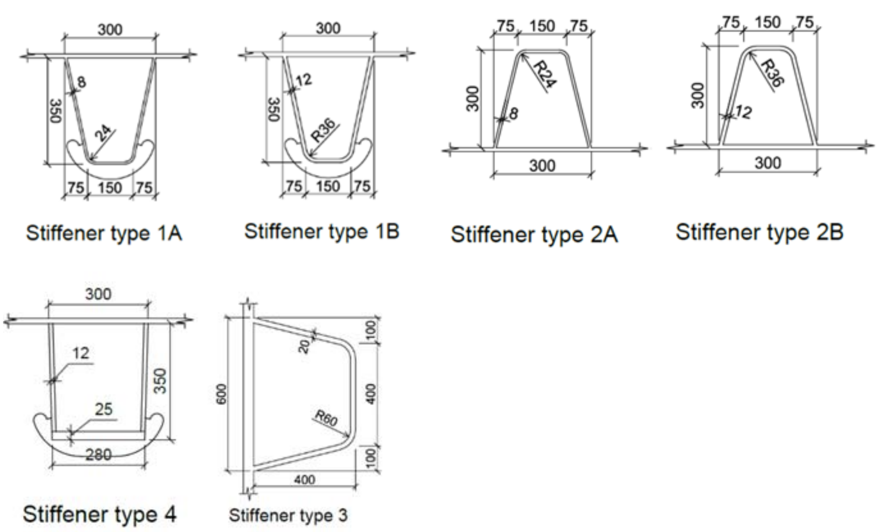


Figure 18 Stiffener types

Buckling resistance of plates with trapezoidal longitudinal stiffeners NS-EN 1993-1-5 sec 4.5

Material factor	1,10											
	Stiffener type 1A		Stiffener type 2A		Stiffener type 2A		Stiffener type 2B		Stiffener type 3		Stiffener type 4	
	Deck		Deck		Bottom plate		Bottom plate		Web		Deck slowlane	
Effective plate width	600	mm	600	mm	600	mm	600	mm	1575	mm	600	mm
Plate thickness	14	mm	14	mm	12	mm	12	mm	40	mm	14	mm
With stiffener top	300	mm	300	mm	300	mm	300	mm	600	mm	300	mm
Heigth stiffener web	342	mm	338	mm	292	mm	288	mm	380	mm	325	mm
With stiffener bottom	150	mm	150	mm	150	mm	150	mm	400	mm	280	mm
Thickness stiffener web	8	mm	12	mm	8	mm	12	mm	20	mm	24	mm
Gross section properties column (stiffener and plate)												
Gross weight column	118	kg	144	kg	103	kg	125	kg	677	kg	241	kg
Area	1,51E+04	mm^2	1,83E+04	mm^2	1,31E+04	mm^2	1,59E+04	mm^2	8,62E+04	mm^2	3,07E+04	mm^2
Moment of inertia	2,47E+08	mm^4	3,19E+08	mm^4	1,63E+08	mm^4	2,09E+08	mm^4	1,71E+09	mm^4	5,81E+08	mm^4
Equivalent plate thickness	25,12	mm	30,52	mm	21,79	mm	26,52	mm	54,73	mm	51,20	mm
COG from top plate	100	mm	119	mm	88	mm	105	mm	95	mm	168	mm
Yield stress	420	Mpa	420	Mpa	420	Mpa	420	Mpa	420	Mpa	420	Mpa
Buckling length = Distace between transverse girders	4000	mm	4000	mm	4000	mm	4000	mm	4000	mm	4000	mm
Critical buckling load	31947671	N	40390507	N	21059355	N	26562812	N	211932071	N	75149789	N
Critical column buckling stress $s_{cr,sl}$	2120	Mpa	2254	Mpa	1611	Mpa	1702	Mpa	2573	Mpa	2451	Mpa
Relative column slenderness l_c	0,43		0,43		0,50		0,50		0,40		0,41	
Buckling curve a-factor	0,42		0,42		0,42		0,42		0,47		0,44	
F	0,64		0,64		0,69		0,69		0,63		0,63	
C	0,90		0,90		0,86		0,86		0,90		0,90	

Buckling load resistance $N_{b,Rd}$	**4827**	**kN**	**6264**	**kN**	**4170**	**kN**	**5247**	**kN**	**29568**	**kN**	**10411**	**kN**
Tension load capacity $N_{t,Rd}$	**5755**	**kN**	**6992**	**kN**	**4991**	**kN**	**6075**	**kN**	**32913**	**kN**	**11729**	**kN**
Equivalent buckling stress s_{ekv}	**342**	**Mpa**	**342**	**Mpa**	**328**	**Mpa**	**330**	**Mpa**	**343**	**Mpa**	**344**	**Mpa**

Dimensioning compression stress is:

Bridge deck and web:	ULS	342 MPa	Bottom plates:	ULS	330 MPa
	ALS	376 MPa		ALS	363 MPa

The stresses should be calculated in centroid of Plate/stiffener column

Maximum compression stresses are calculated for T-stiffener used at the end of the bridge at the abutments. The stresses are in the centroid of the effective column. Note that spacing between transverse beams are 2 meters in this area. This ensures that buckling will not occur even at yield stress.

Buckling resistance of plates with T-longitudinal stiffeners NS-EN 1993-1-5 sec 4.5

Material factor 1,10

	Outer web		Inner web		Deck		Bottom	
Effective plate width	600	mm	600	mm	600	mm	600	mm
Plate thickness	40	mm	24	mm	24	mm	24	mm
Thickness stiffener web	20	mm	20	mm	20	mm	20	mm
Heigth stiffener web	500	mm	500	mm	500	mm	500	mm
Width stiffener flange	150	mm	150	mm	150	mm	150	mm
Thickness stiffener flange	20	mm	20	mm	20	mm	20	mm
Gross section properties column (stiffener and plate)								
Gross weight column	290	kg	215	kg	215	kg	215	kg
Area	3,70E+04	mm^2	2,74E+04	mm^2	2,74E+04	mm^2	2,74E+04	mm^2
Moment of inertia	1,28E+09	mm^4	1,07E+09	mm^4	1,07E+09	mm^4	1,07E+09	mm^4
Equivalent plate thickness	61,67	mm	45,67	mm	45,67	mm	45,67	mm
COG from top plate	136	mm	165	mm	165	mm	165	mm
Yield stress	420	Mpa	420	Mpa	420	Mpa	420	Mpa
Buckling length = Distace between transverse girders	2000	mm	2000	mm	2000	mm	2000	mm
Critical buckling load	6,65E+08	N	5,56E+08	N	5,56E+08	N	5,56E+08	N
Critical column buckling stress $s_{cr,sl}$	17963	Mpa	20287	Mpa	20287	Mpa	20287	Mpa
Relative column slenderness l_c	0,15		0,14		0,14		0,14	
Buckling curve a-factor	0,34		0,34		0,34		0,34	
F	0,50		0,50		0,50		0,50	
C	1,02		1,02		1,02		1,02	
Buckling load resistance $N_{b,Rd}$	**14127**	**kN**	**10462**	**kN**	**10462**	**kN**	**10462**	**kN**
Tension load capacity $N_{t,Rd}$	**14127**	**kN**	**10462**	**kN**	**10462**	**kN**	**10462**	**kN**
Equivalent buckling stress s_{ekv}	**382**	**Mpa**	**382**	**Mpa**	**382**	**Mpa**	**382**	**Mpa**

Dimensioning compression stress is:

Bridge deck and web:	ULS	382 MPa	Bottom plates:	ULS	382 MPa
	ALS	420 MPa		ALS	420 MPa

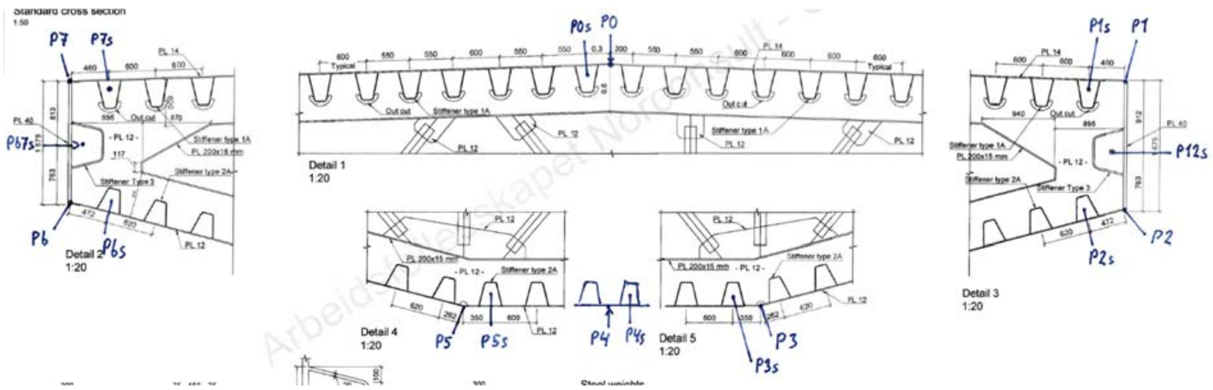
4.5 Stress calculations in bridge girder

4.5.1 Design load action

From global analyses max and min stresses in each stress point is given, along with the combination of FX, FY, FZ, MX, MY and MZ which gives the max/min stress. It's therefor easy to recalculate stresses with design cross section properties. Both ULS comb 33 with traffic and ULS comb 34 without traffic (100 years RTP) are recalculated.

4.5.2 Section modulus for stress control

Stress control is executed in 7 points in plates, and 7 points in stiffeners along the cross section as shown in the figure below:



> *Figure 19* *Points for stress calculations*

In the following, section modulus Wy and Wz is tabulated for cross sections along the bridge in each stress point.

> *Table 10 Section modulus cross sections 1 and 2*

	Mid span		At column support	
	Type 1	Type 2	Type 1	Type 2
A	1,47	1,74	2,25	2,52
NA	1,85	1,81	1,68	1,69
I_z	114,9	132,0	136,2	153,3
I_y	2,52	3,06	3,59	4,16
y - vertical action plate check				
W_{y0}	-1,53	-1,81	-1,97	-2,30
W_{y1}	-2,02	-2,38	-2,53	-2,94
W_{y2}	6,00	7,99	14,17	16,24
W_{y3}	1,36	1,69	2,13	2,47
W_{y4}	1,36	1,69	2,13	2,47
W_{y5}	1,36	1,69	2,13	2,47
W_{y6}	6,00	7,99	14,17	16,24
W_{y7}	-2,19	-2,58	-2,72	-3,17
z - horizontal action plate check				
W_{z0}	67,60	77,65	80,12	90,16
W_{z1}	8,33	9,57	9,87	11,11
W_{z2}	8,33	9,57	9,87	11,11
W_{z3}	13,93	16,00	16,51	18,58
W_{z4}				
W_{z5}	-13,93	-16,00	-16,51	-18,58
W_{z6}	-8,33	-9,57	-9,87	-11,11
W_{z7}	-8,33	-9,57	-9,87	-11,11
y - vertical action plate check				
$W_{y0}s$	-1,63	-1,95	-2,09	-2,46
$W_{y1}s$	-2,19	-2,62	-2,72	-3,22
$W_{y12}s$	-6,07	-6,77	-6,17	-7,19
$W_{y2}s$	5,60	6,24	12,67	11,44
$W_{y3}s$	1,43	1,70	2,25	2,48
$W_{y4}s$	1,43	1,70	2,25	2,48
$W_{y5}s$	1,43	1,70	2,25	2,48
$W_{y6}s$	5,60	6,24	12,67	11,44
$W_{y67}s$	-6,07	-6,77	-6,17	-7,19
$W_{y7}s$	-2,40	-2,86	-2,95	-3,48
z - horizontal action plate check				
$W_{z0}s$	82,08	94,28	97,28	109,48
$W_{z1}s$	8,61	9,89	10,21	11,49
$W_{z12}s$	8,38	9,63	9,94	11,18
$W_{z2}s$	8,61	9,89	10,21	11,49
$W_{z3}s$	14,55	16,71	17,24	19,40
$W_{z4}s$				
$W_{z5}s$	-14,55	-16,71	-17,24	-19,40
$W_{z6}s$	-8,61	-9,89	-10,21	-11,49
$W_{z67}s$	-8,38	-9,63	-9,94	-11,18
$W_{z7}s$	-8,61	-9,89	-10,21	-11,49

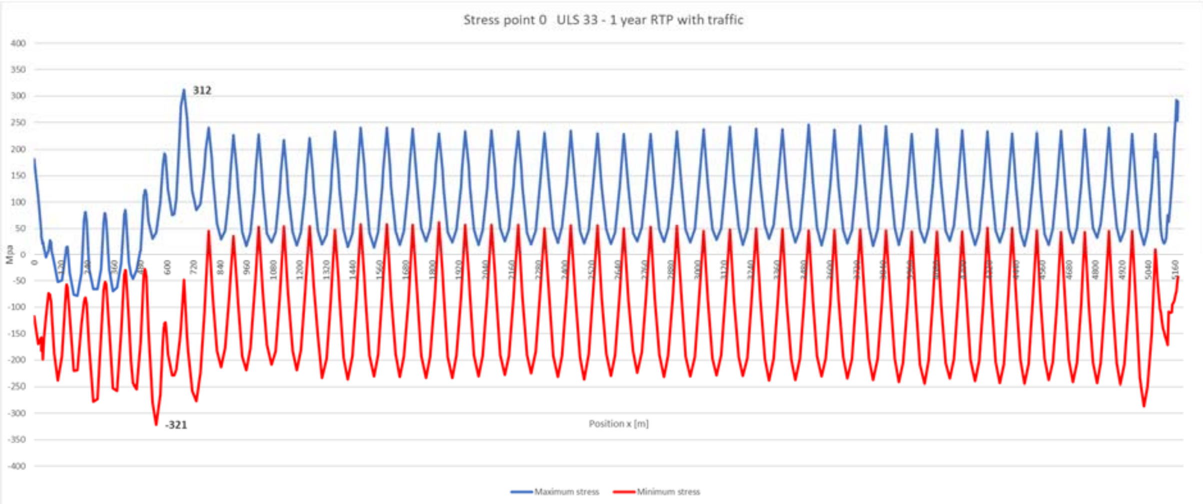
> *Table 11 Section modulus reinforced cross sections at bridge ends*

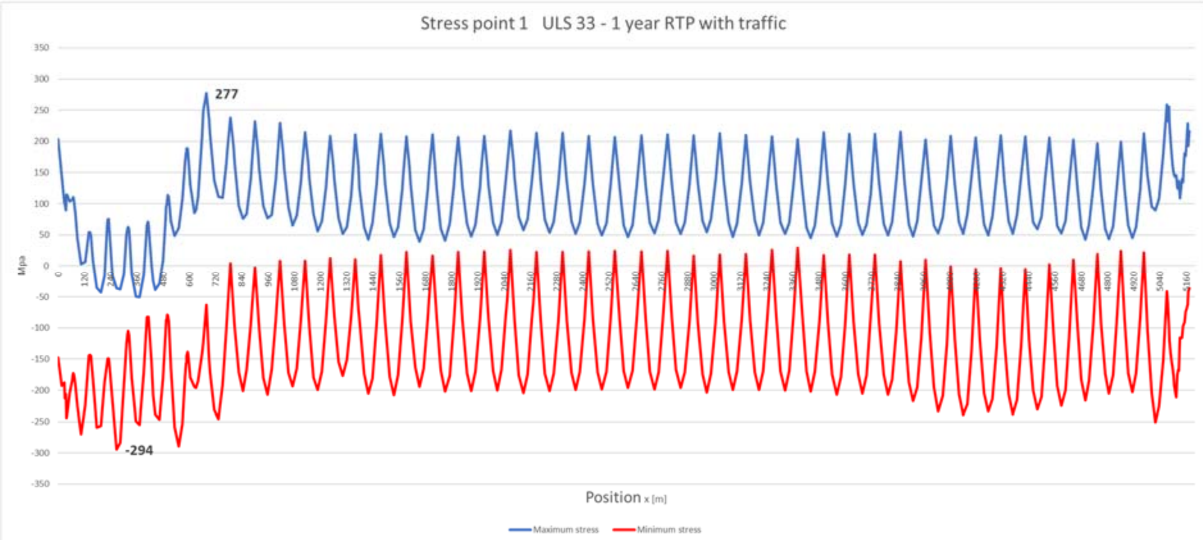
Pos x	0	8	16	32	48
	Design	Design	Design	Design	Design
A	4,49	2,96	2,78	2,11	2,03
NA	1,60	1,64	1,65	1,78	1,78
I_y	8,95	5,43	5,11	4,22	3,32
I_z	329,4	219,4	215,0	158,2	142,4
W_{y0}	-3,897	-2,923	-2,758	-2,460	-1,932
W_{y1}	-4,981	-4,000	-3,776	-3,471	-2,725
W_{y2}	5,580	3,305	3,630	3,935	5,580
W_{y3}	5,580	3,305	3,107	2,365	1,863
W_{y4}	5,580	3,305	3,107	2,365	1,863
W_{y5}	5,580	3,305	3,107	2,365	1,863
W_{y6}	5,580	3,305	3,630	3,935	5,580
W_{y7}	-4,718	-3,726	-3,517	-3,207	-2,519
W_{z0}	106,529	102,647	94,824	87,059	67,588
W_{z1}	12,936	12,645	11,681	10,725	8,326
W_{z2}	12,936	12,645	11,681	10,725	8,326
W_{z3}	12,936	12,645	12,520	13,424	12,523
W_{z4}	0,000	0,000	0,000	0,000	0,000
W_{z5}	-12,936	-12,645	-12,520	-13,424	-12,523
W_{z6}	-12,936	-12,645	-11,681	-10,725	-8,326
W_{z7}	-12,936	-12,645	-11,681	-10,725	-8,326

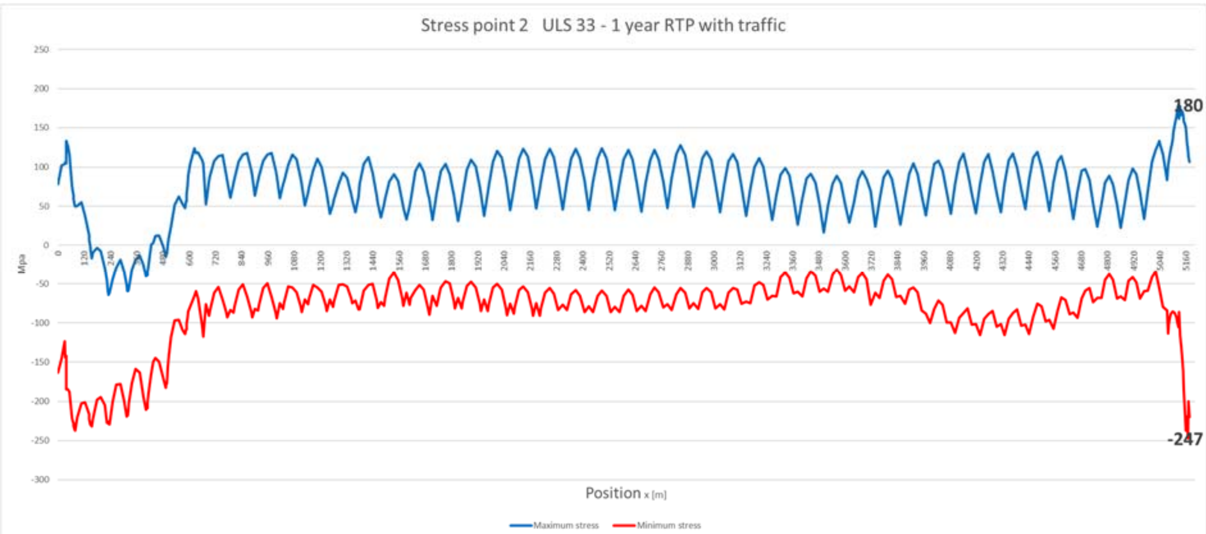
At bridge ends, no stress control in stiffeners is required since the transverse cross girder distance is 2,0 meters and the stiffeners may be utilized to full yield.

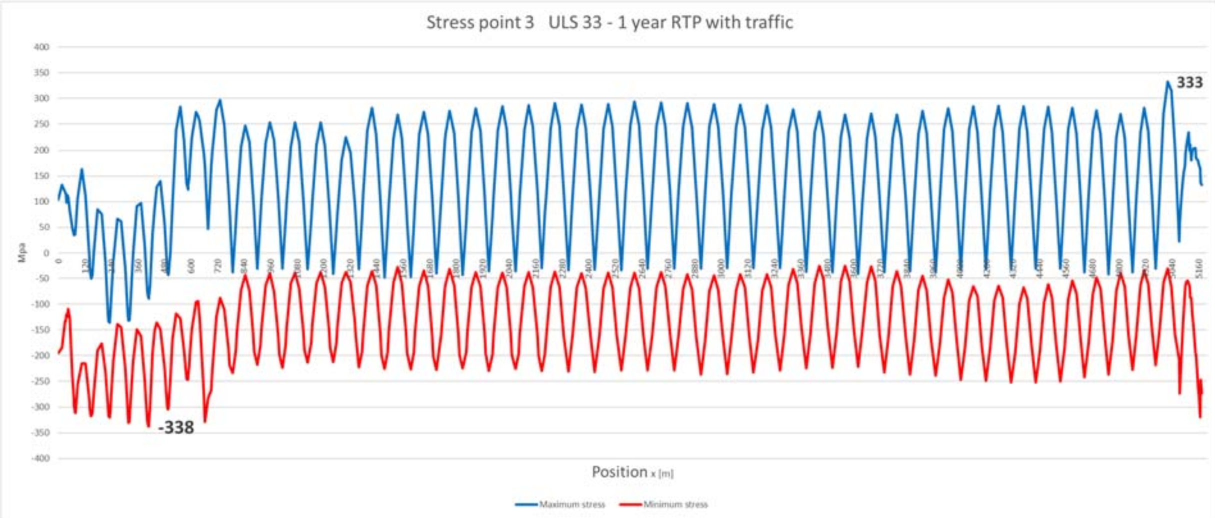
4.5.3 Stresses ULS 33 1-year return period with traffic

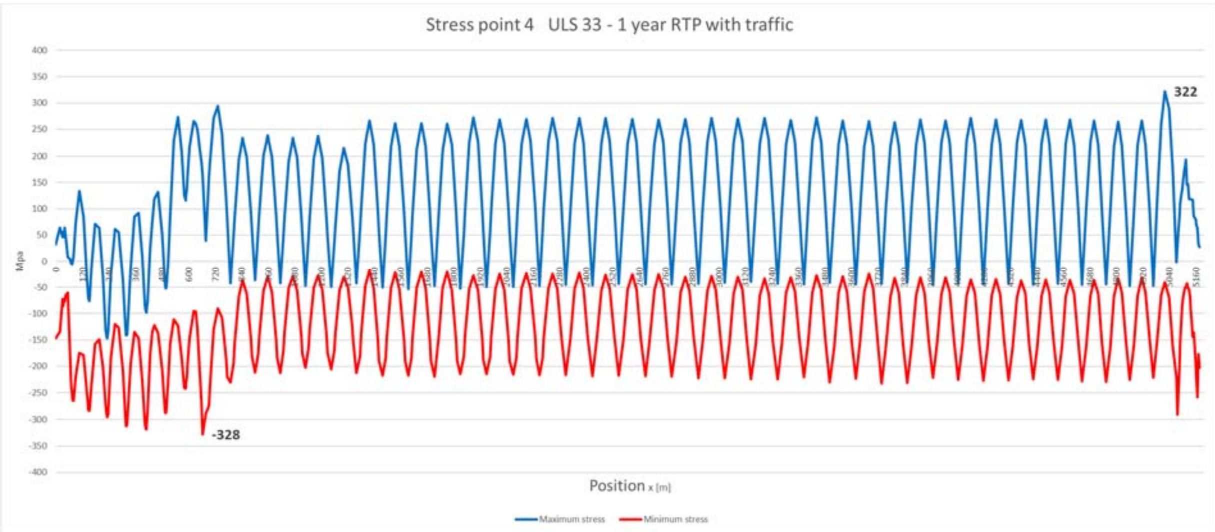
Results for plate side points 1 – 7 are presented in the following figures:

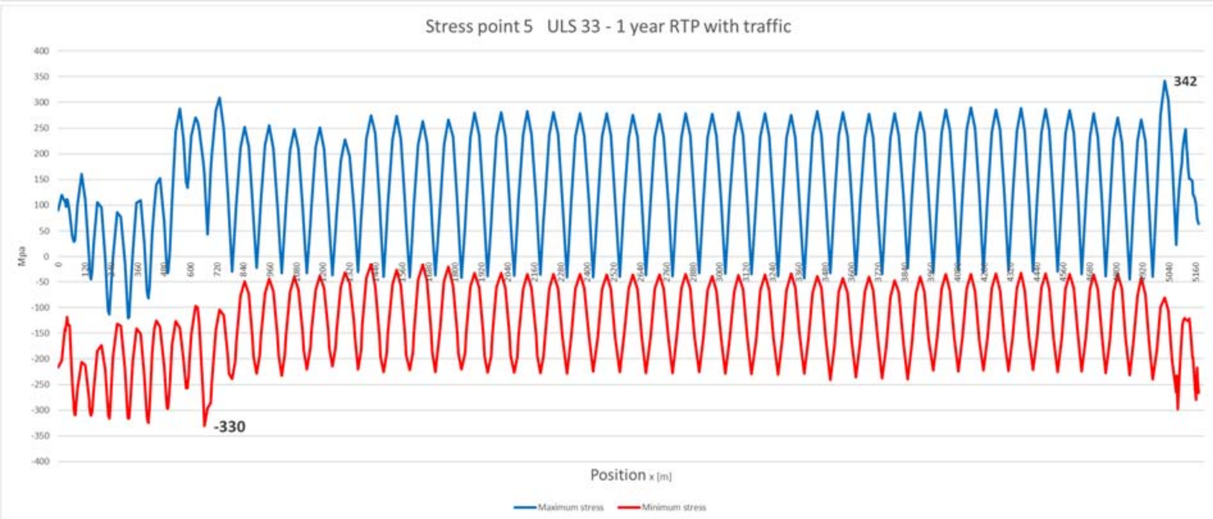


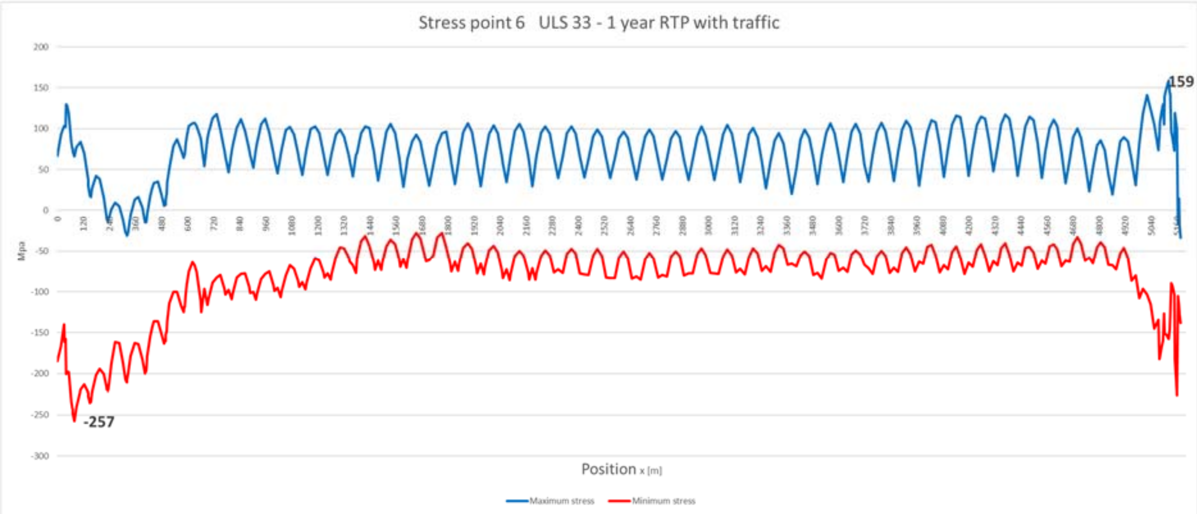


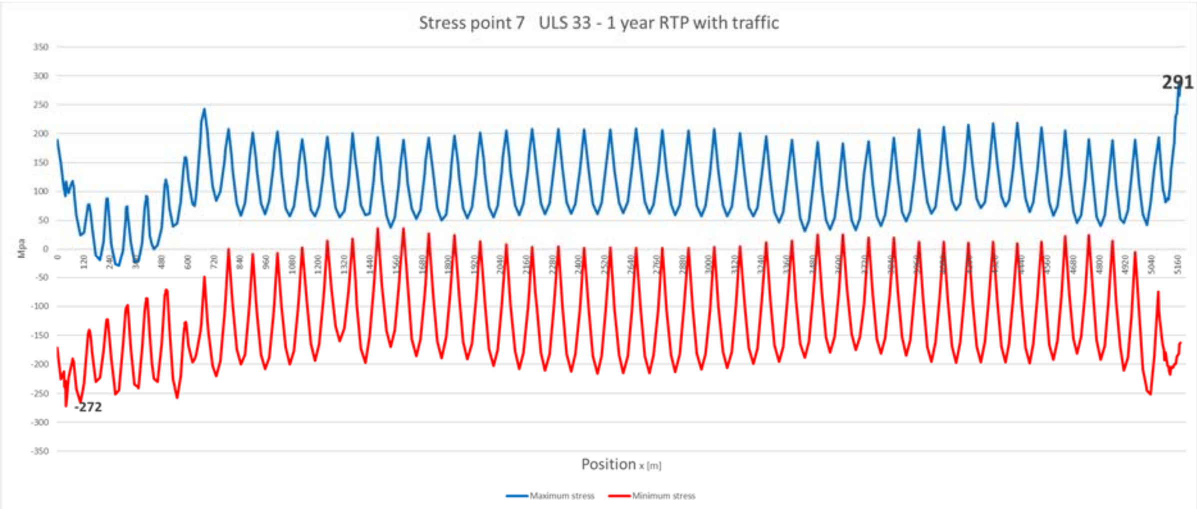



Stress point 3 ULS 33 - 1 year RTP with traffic
Mpa
333
-338
Position x [m]
Maximum stress
Minimum stress


Stress point 4 ULS 33 - 1 year RTP with traffic
Mpa
322
-328
Position x [m]
Maximum stress
Minimum stress


Stress point 5 ULS 33 - 1 year RTP with traffic
Mpa
342
-330
Position x [m]
Maximum stress
Minimum stress



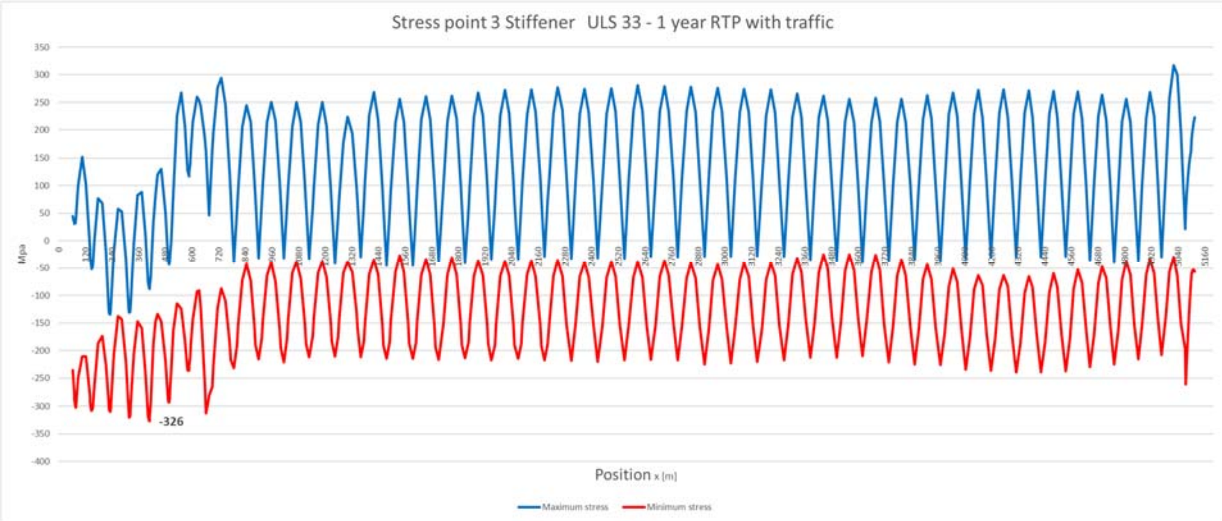


From the figures, we observe that maximum and minimum normal stresses are:

s_{max} = 342 MPa	Stress point 5	x-pos 5021	End span between axis 41 and 42
s_{min} = -338 MPa	Stress point 3	x-pos 410	In cable stayed bridge

All stresses in plates < s_d =382 MPa OK

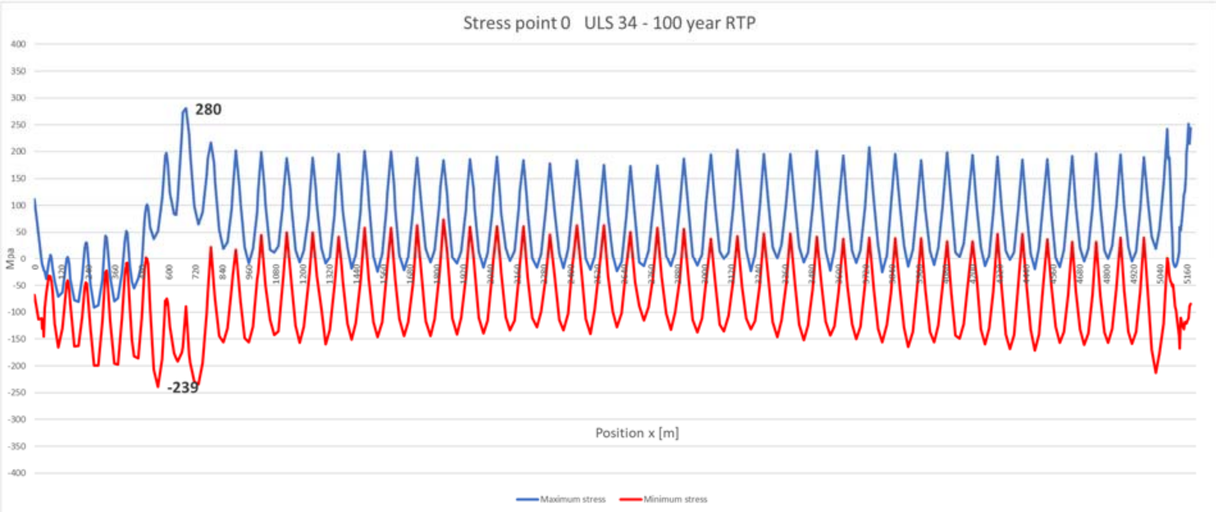
According to 4.4 max compression stress in stiffener at point 3 is -330 MPa. We need to do a stress control in the stiffener. This is presented in the following figure:

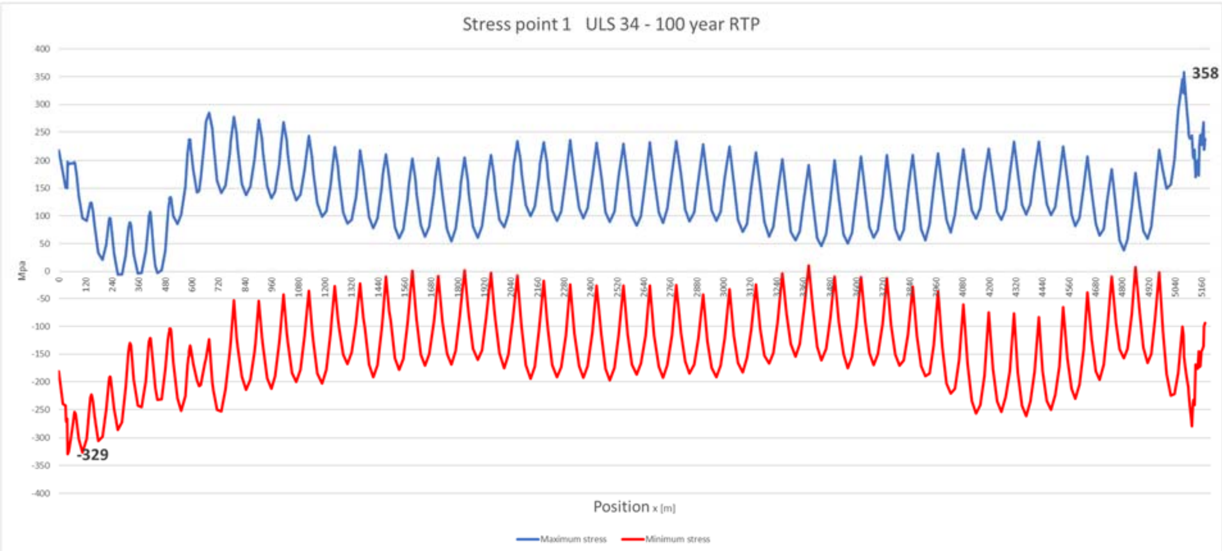


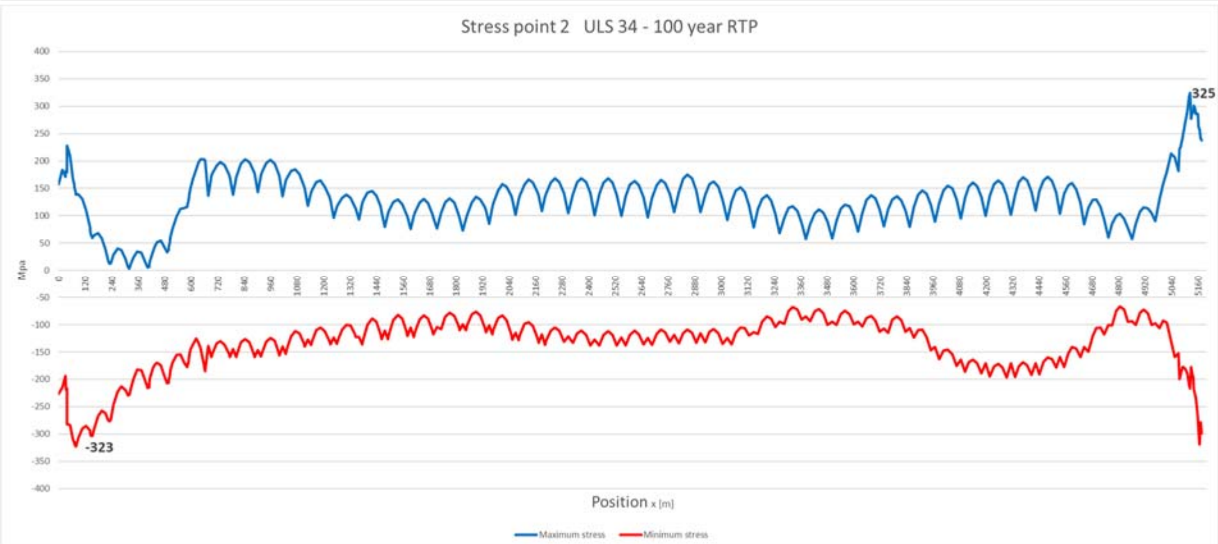
From the figure, we see that maximum compression is -326 MPa < -330 MPa OK

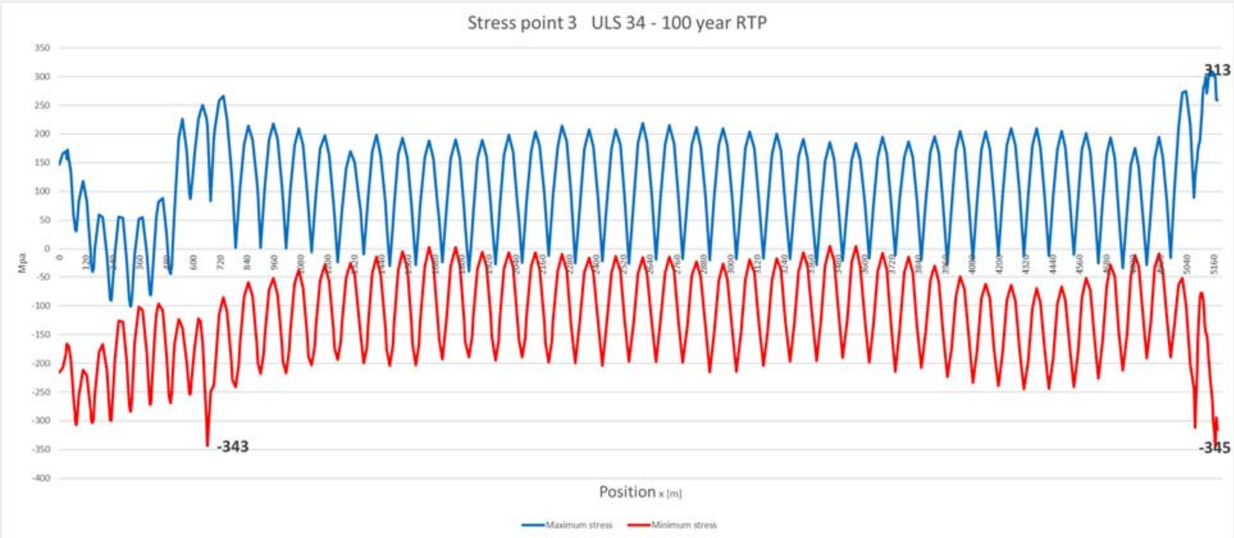
4.5.4 Stresses ULS 34 100-year return period

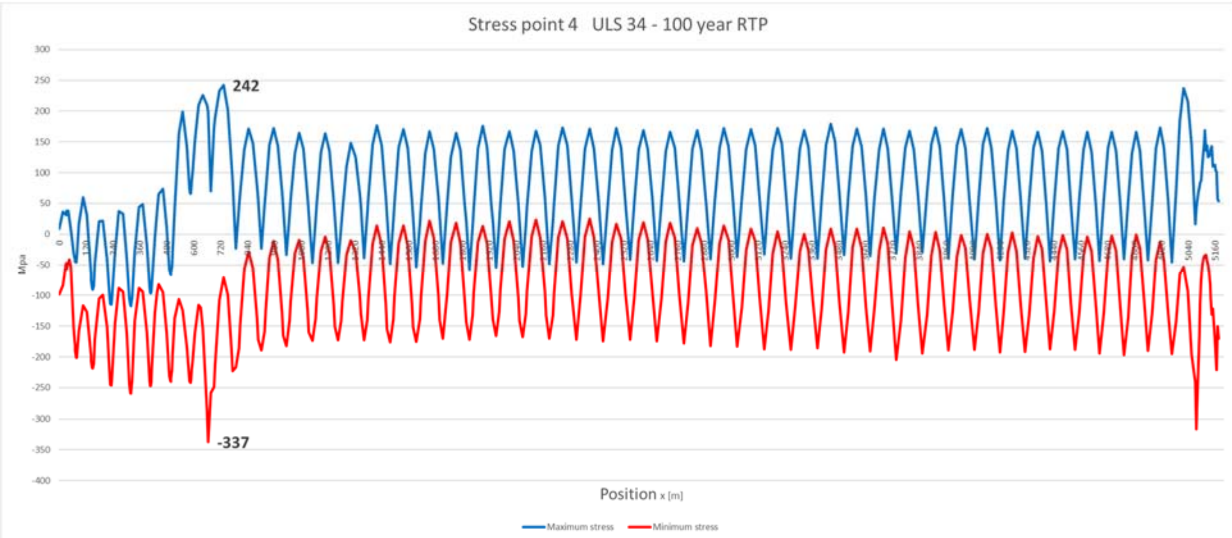
Results for plate side points 1 – 7 are presented in the following figures:

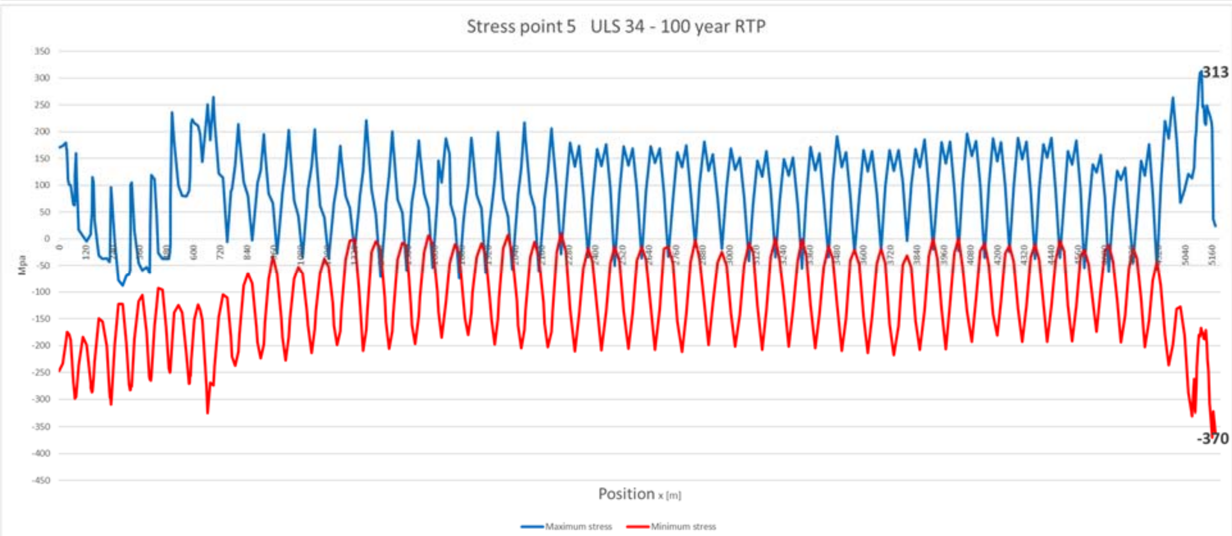


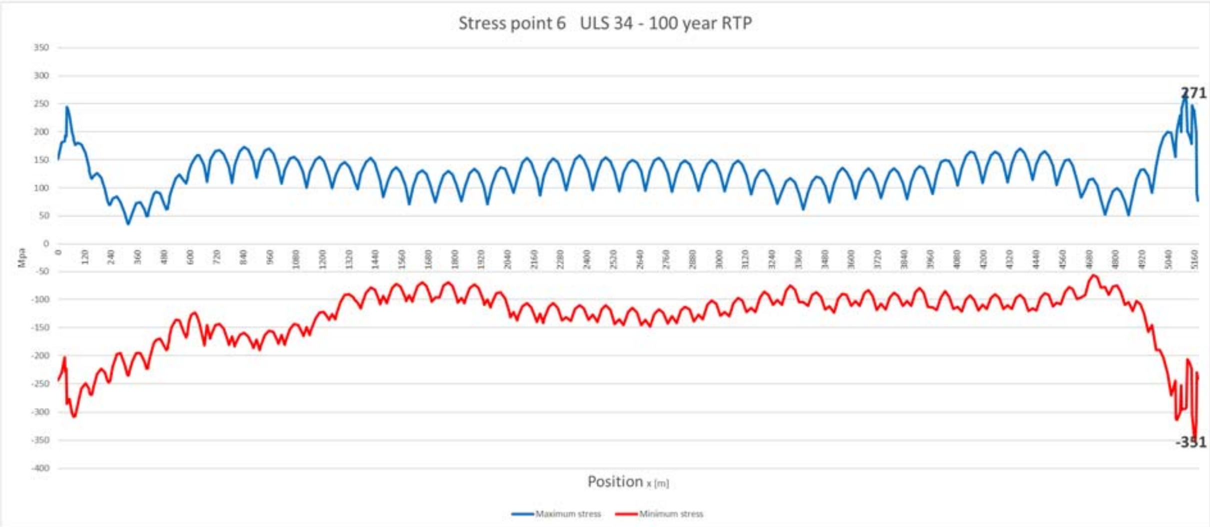


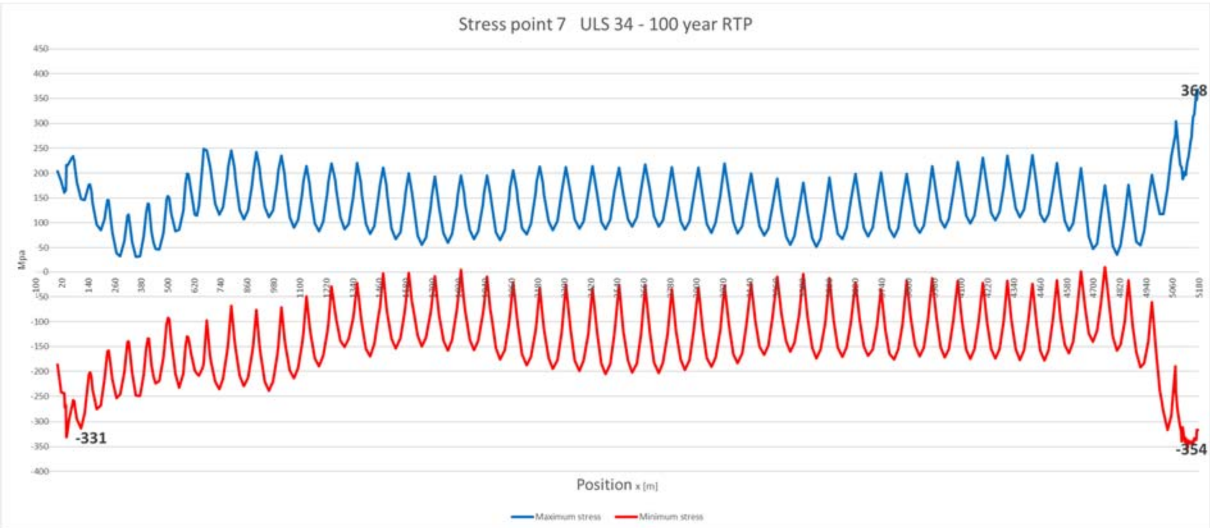



Stress point 3 ULS 34 - 100 year RTP
313
-343
-345
Mpa
Position x [m]
Maximum stress
Minimum stress


Stress point 4 ULS 34 - 100 year RTP
242
-337
Mpa
Position x [m]
Maximum stress
Minimum stress


Stress point 5 ULS 34 - 100 year RTP
313
-370
Mpa
Position x [m]
Maximum stress
Minimum stress





From the figures, we observe that maximum and minimum normal stresses are:

s_{max} = 368 MPa	Stress point 7	x-pos 5176	Reinforced end span
s_{min} = -370 MPa	Stress point 5	x-pos 5163	Reinforced end span

All stresses in plates < s_d =382 MPa OK
Compression stress -370 MPa is OK in reinforced end span

We also observe that maximum compression stress in ordinary cross sections are:

s_{min} = -343 MPa	Stress point 3	x-pos 664	Closes to axis 3

According to 4.4 max compression stress in stiffener at point 3 is -342 MPa. However, the stress in stiffener is lower than 343 MPa and stiffener check is OK.

4.6 ULS check – conclusion

ULS check shows that the girder has capacity to resist ultimate limit state load actions. If we compare to stress calculations from global analyses, we will see that design stresses are lower at column supports due to reinforcements even if shear lag effects are included. In mid span, design stresses are higher due to shear lag effects.

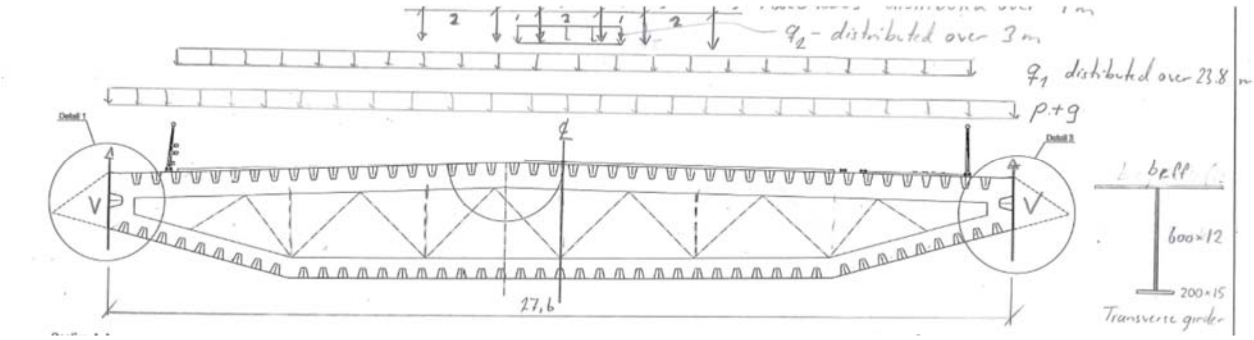
4.7 Transverse girders (bulkheads)

4.7.1 Calculation model

Purpose of transverse girder:

- Support stiffeners. Calculated according to NS-EN 1993-1-5, section 9.2,1
- Transfer support loads from stiffeners to the web plates in the steel girder

Model is shown below. A conservative assumption for the transverse girder span is 27,6 meters, which is the width of the steel box. Typical spacing between transverse girders 4,0 m.



Loads used in calculation. Refer to figure.

- Eigen weight

  180 kN/m distributed over 27,6 meters multiplied with 4 m spacing
  ⇒ g = 26 kN/m

- Equally distributed traffic 2,5 kN/m^2 between outer parapets multiplied with 4 m spacing
  ⇒ q_1 = 10 kN/m

- Equally distributed traffic 5,4 -2,5 kN/m^2 in most loaded traffic lane multiplied with 4 m spacing
  ⇒ q_2 = 11,6 kN/m distributed over 3 meters

- Traffic axle loads in 3 lanes, 2*300 + 2*200 + 2*100 = 1200 kN assumed as equally distributed over 9 meters
  ⇒ q_{eqv} = 133,3 kN/m distributed over 9 meters

- Transverse load from direct stresses in stiffened plates
  Assume maximum load in stiffened deck plates N = 4806 kN over 0,6 meters = 8010 kN/m. Refer to 4.4.

  According to EN 1993-1-5, section 9.2.1the transverse girder shall be considered as a simply supported beam with lateral loading as follows:

Imperfection w_0 = min(a_1, a_2, b)/ 300 here $a_1 = a_2$ = 4 meters , b = 27,6 meters
⇒ w_0 = 4000/300 = 13,3 mm

⇒ p = 8010*13,3/4000 = 8010/300 = 26,7 kN/m at deck

Similar, assume maximum load in stiffened bottom plates N = 3942 kN over 0,6 meters = 6570 kN/m. Refer to 4.4.

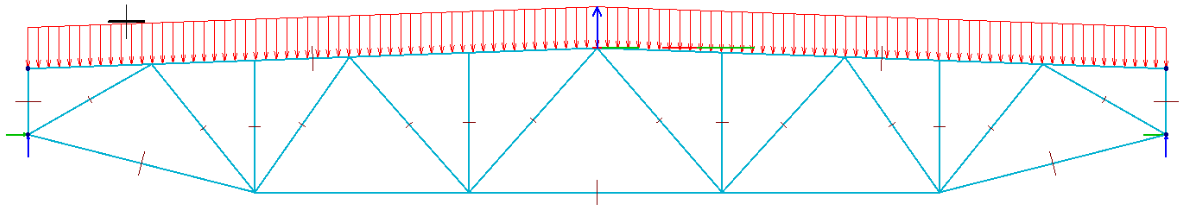
⇒ p = 6570/300 = 21,8 kN/m at bottom plates

Two different load cases are considered:

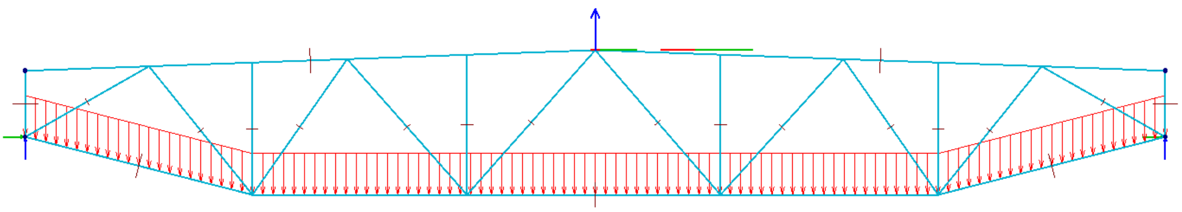
1. Compression in deck and tension in bottom plates
   Applied weight, traffic and lateral load downwards on deck

2. Compression in bottom plates and tension in deck plates
   Applied weight at deck and lateral load downwards at bottom plate

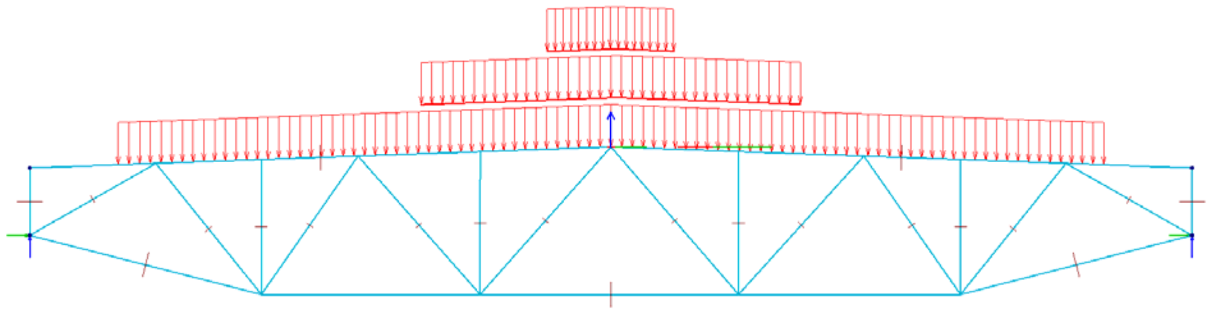
The transverse girder is analyzed in FEM-Design. Model I shown below with the different loads applied. Girder is pin joint supported at the outermost parts of the model.



Weight and lateral load from stiffeners at deck



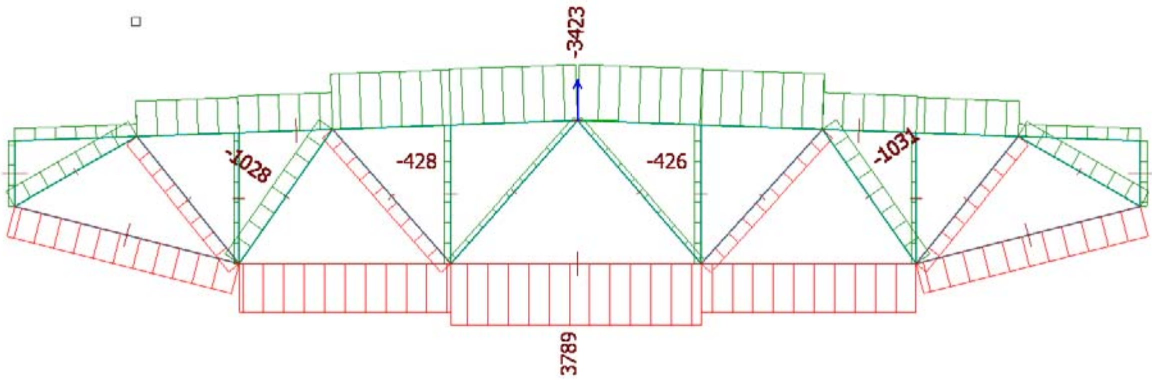
Lateral load from stiffeners at bottom plates



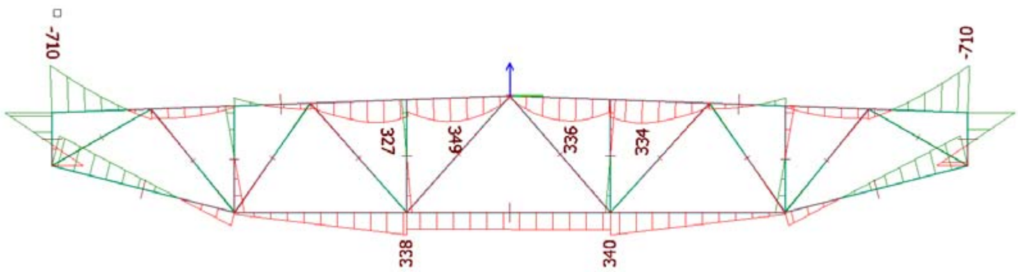
Traffic load q_1 = 10 kN/m between parapets q_2 = 11,6 kN/m in most loaded lane and axle loads as equivalent line loads q = 133,3 kN/m in 3 most loaded lanes

4.7.2 Results

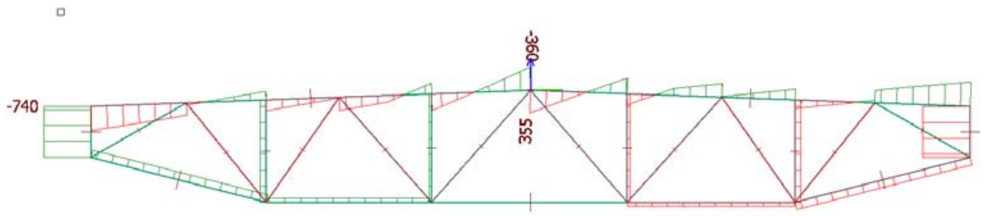
Load case 1



Axial forces (kN) – green compression

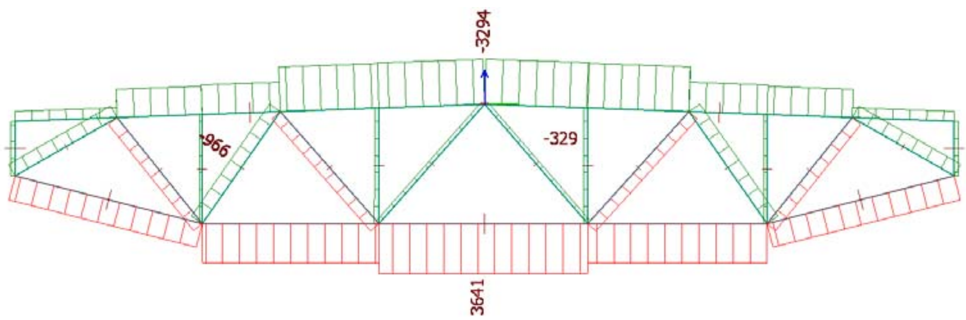


Bending moment (kNm)

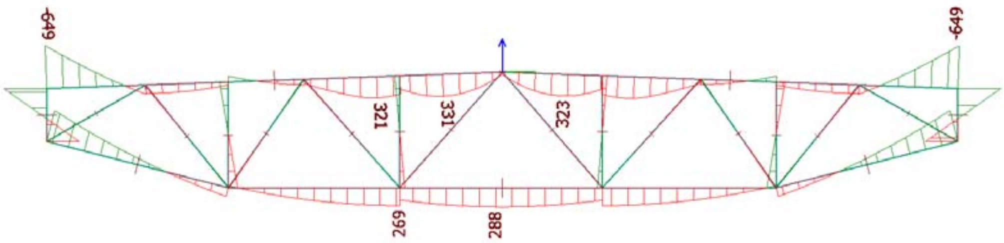


Shear forces (kN)

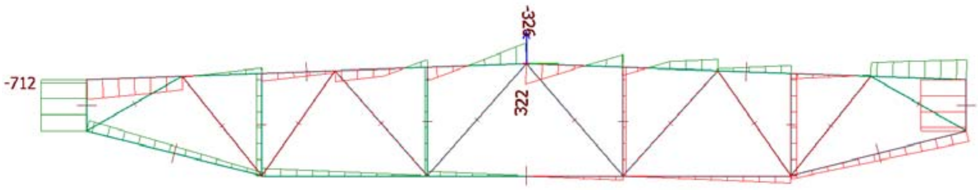
Load case 2



Axial forces (kN) – green compression



Bending moment (kNm)



Shear forces (kN)

4.7.3 Resistance verification

Verification of resistance performed for upper and lower girder. Effective plate width calculated according to NS-EN 1993-1-5 section 3.2.1.

Span L = 27.6 m Plate width b = 4.0 m

L_e = 27.6 m b_0 = 2.0 m k = 2/27,6 = 0.072 b = $1/(1+6.4k^2)$ = 0.967 b_e = 3.87 m

Stress control upper girder

Transverse girder stress control

Maximum sagging

Effective width of deck plate	3870	(mm)
Deck plate thickness	14	(mm)
Web height	600	(mm)
Web thickness	12	(mm)
Flange width	200	(mm)
Flange thickness	15	(mm)

Bending moment kNm	349
Shear Force kN	0
Axial Force kN	-3423

Stresses (MPa) :	s	t	sj
Plate	-66	0	66
Web top	-64	0	64
Web bottom	54	0	54
Flange	55	0	55

COG from top plate	70	(mm)
Cross section area	6,44E+04	(mm^2)
Moment of inertia	1,77E+09	(mm^4)
Section modulus plate	2,81E+07	(mm^3)
Section modulus top web	3,17E+07	(mm^3)
Section modulus bottom web	3,26E+06	(mm^3)
Section modulus flange	3,21E+06	(mm^3)

Stress control lower girder

Transverse girder stress control

Maximum sagging

Effective width of deck plate	3870	(mm)	Bending moment kNm	-340	*Negative bending moment*		
Deck plate thickness	12	(mm)	Shear Force kN	0	*to ensure tension in plate*		
Web height	600	(mm)	Axial Force kN	3789			
Web thickness	12	(mm)					
Flange width	200	(mm)					
Flange thickness	15	(mm)					
			Stresses (MPa) :	s		t	sj
			Plate	81		0	81
			Web top	80		0	80
			Web bottom	-38		0	38
			Flange	-40		0	40

COG from top plate	77	(mm)
Cross section area	5,66E+04	(mm^2)
Moment of inertia	1,73E+09	(mm^4)
Section modulus plate	2,42E+07	(mm^3)
Section modulus top web	2,65E+07	(mm^3)
Section modulus bottom web	3,24E+06	(mm^3)
Section modulus flange	3,19E+06	(mm^3)

Shear capacity girders at cut outs:

Web height 600 mm. Maximum cut out 350 mm at upper girder.

A_{weff} = (600-350)*12 = 3000 mm^2 $V_{pl,Rd}$ = 3000*220 ⇒ $V_{pl,Rd}$ = 661 kN > V_{Ed} = 360 kN OK

Buckling control inclined truss – HUP 150x8:

Column buckling control - RHS profile

Column width	150	mm
Thickness	8	mm
Axial Force kN	-1031	kN
Yield stress	355	Mpa
Column length	3000	mm
Buckling factor	1,00	
Buckling length	3200	mm
Buckling curve a-factor	0,21	
Cross section area	4,54E+03	mm^2
Moment of inertia	1,53E+07	mm^4
Section modulus	2,04E+05	mm^3
Critical buckling load	3098198,192	N
Reduced slenderness	0,72157045	
F	0,815096855	
C	0,837380883	
$N_{b,Rd}$	1228	kN
Nd	1466	kN
Utilization	0,84	OK

Buckling control vertical truss - HUP 150x5:

Column buckling control - RHS profile		
Column width	150	mm
Thickness	5	mm
Axial Force kN	-428	kN
Yield stress	355	Mpa
Column length	2300	mm
Buckling factor	1,00	
Buckling length	2300	mm
Buckling curve a-factor	0,21	
Cross section area	2,90E+03	mm^2
Moment of inertia	1,02E+07	mm^4
Section modulus	1,36E+05	mm^3
Critical buckling load	3984850,933	N
Reduced slenderness	0,508284815	
F	0,661546632	
C	0,921681316	
$N_{b,Rd}$	863	kN
Nd	936	kN
Utilization	0,50	OK

4.8 Column/girder connection

The design of column/girder connections are governed by ship impact. Reference is made to 6.4.

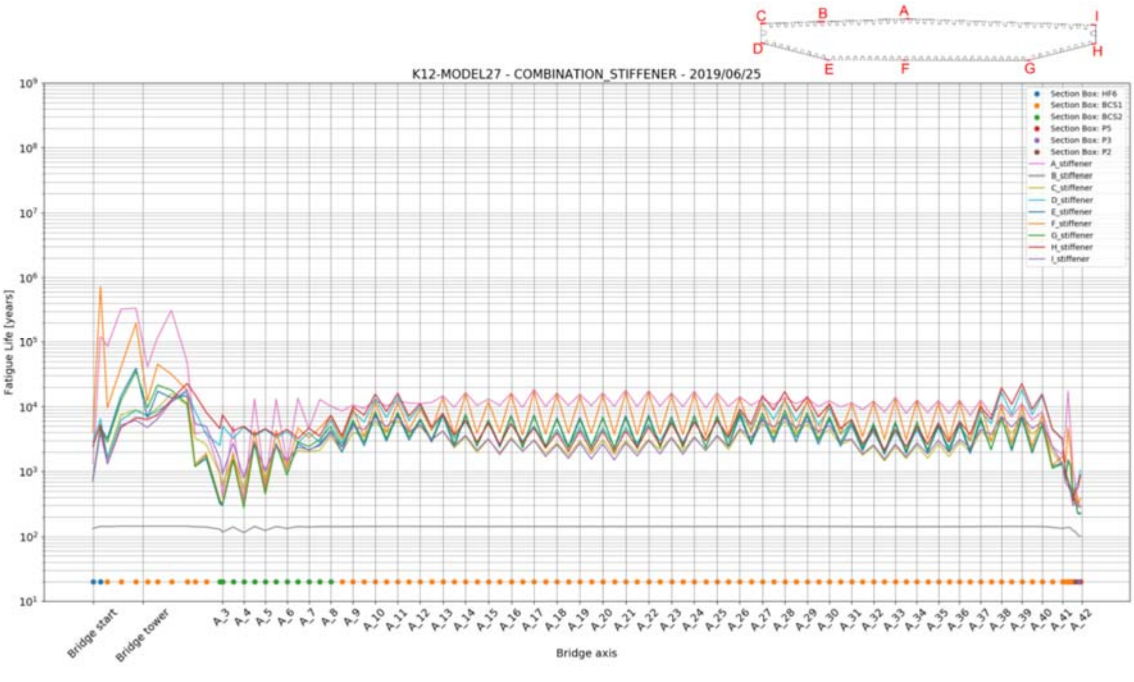
5 FATIGUE LIMIT STATE (FLS)

5.1 General

Fatigue limit state is documented in separate reports. Reference is made to SBJ-33-C5-OON-22-RE-016. This chapter summarizes the fatigue assessment.

5.2 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 20 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.

6 ACCIDENTAL LIMIT STATE (ALS)

6.1 General

Accidental limit state is documented in separate reports. Reference is made to SBJ-33-C5-OON-22-RE-013 - 015. This chapter summarizes the assessment.

6.2 Results

The global response of the bridge due to ship impact has been studied. The main focus is impacts between ship deckhouse and girder and between ship bulb and pontoon. Girder impacts all along the bridge length have been considered, orthogonal to the bridge girder from both directions. Pontoon impacts have been considered on all three pontoon types, at selected characteristic locations along the bridge. Three impacts are considered; head on (0-degrees) and centric and eccentric side impacts (90-degrees). Pontoon impact from a sideway drifting ship and submarine impact has also been discussed. Post impact the bridge must withstand 100-years environmental conditions.

The global ship impact analysis shows that the bridge will survive both a ship impact as given in the design basis and the following 100-years conditions.

A performed screening of girder impacts gives a maximum girder strong axis bending moment of almost 3000 MNm in the bridge "span", while it is 3750 MNm at the south end (near the cable stayed bridge) and 6600 MNm in the north end.

The ship impact energy is expected to be reduced in the next phase. This will give lower global response and lower damage/indentation of pontoons and girder. It might still give large forces in the column and girder, so these are details that still needs to be addressed.

6.3 Resistance of cross section due to ship impact

The maximum estimated strong axis bending moment in the girder outside reinforced end spans are 3000 MNm.

From 3.2.2 we find that I = 114 m^3 in mid spans and the stress in outermost fiber is:

$$s = 3000 \cdot 13{,}8/114 = 363 \text{ MPa}$$

From 4.4 we find that design compression stress in ALS is 376 Mpa > 363 MPa OK

Maximum estimated strong axis bending moment at abutments are 6600 MNm.

From 3.2.4 we find that I = 329 m^3 in mid spans and the stress in outermost fiber is:

$$s = 6600 \cdot 14/329 = 280 \text{ MPa}$$

From 4.4 we find that design compression stress in ALS is 420 Mpa > 280 MPa OK

6.4 Resistance of column/girder connection due to ship impact

The connection between girder and columns have been described in earlier sections. The design is governed by ship impact, and reference is made to report SBJ-33-C5-OON-22-RE-015-A K12 - Ship impact, Bridge girder.

7 REFERENCES

[1] SBJ-32-C4-SVV-90-BA-001, «Design Basis Bjørnafjorden floating bridges,» Statens Vegvesen, 2018.

[2] NS-EN 1993-1-1:2005+A1:2014+NA:2015, «Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings,» Standard Norge, 2005.

[3] NS-EN 1993-2:2006+NA:2009, «Eurocode 3: Design of steel structures - Part 2: Bridges,» Standard Norge, 2006.

[4] Håndbok N400 , «Bruprosjektering,» Statens vegvesen Vegdirektoratet, 2015.