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CONCEPT DEVELOPMENT, FLOATING BRIDGE E39 BJØRNAFJORDEN

Preferred solution, K12

Appendix K – Design of Floating Bridge Part

CLIENT

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REPORT

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		RESPONSIBLE UNIT	AMC

SUMMARY

This appendix outlines the design of the floating bridge part for the concept K12.

The floating bridge part consists of the bridge girder, columns and pontoon.

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REV.	DATE	DESCRIPTION	PREPARED BY	CHECKED BY	APPROVED BY

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1 Design of Floating Bridge Part

1.1 General for Floating Bridge Part

The floating bridge part consists of the bridge girder, columns and pontoons. The typical floating bridge span is 125 meters. The floating bridge is divided in high part, valid for axis 3 – 8, and low part, valid for axis 9 - 40.

The focus has been to optimize the design of the floating bridge with respect to reduce the fabrication costs and increase the amount of automatic welding, and this in combination with a robust design.

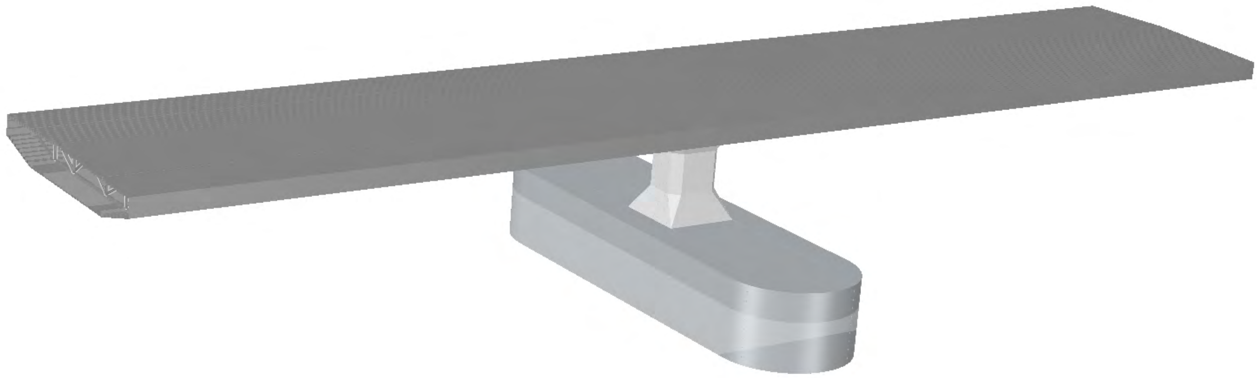


Figure 1-1: Typical floating bridge at lower part showing bridge girder, column and pontoon

Table 1-1: Summary for floating bridge part

	K12
Length of floating bridge part (m)	4770
Number of columns at high part	6
Number of columns at low part	32
Number of pontoons without mooring lines	35
Number of pontoons with mooring lines	3
Total sum steel for floating bridge part (tonnes) Note 1	103 420

Note 1: The increased weight of 15 % for pontoons is not included in the table above. However, these increases are included in the final material quantities.

1.2 Pontoons

There are two different pontoon designs; a pontoon without mooring lines and with supports for mooring lines. The concept K12 has total 38 pontoons where 3 of the pontoons have supports for mooring lines.

The pontoons have a “circtangel” shape i.e. a rectangle with half cylinders at each end in the transverse bridge girder direction and with flat bottom plate. The top plate will have an inclination in transverse direction to let the water run off. This inclination is not implemented in the drawings. The outer shell plates, inner transverse- and longitudinal bulkheads are reinforced with bulb stiffeners. Additional structural strength is provided by web-frames in the bridge girder longitudinal direction.

The” circtangel” shape was chosen over a “kayak” design. A pontoon with a kayak design is likely the pontoon with the lowest loads. In addition, the shape of this pontoon is in correlation with the moment distribution. Small moments act at the ends where the cross-section is small and a larger moment at the center where the cross-section is larger. Looking purely at the load scenario, the kayak would be the best choice. But the fabrication of the circtangel is easier, and has the following advantages:

1. Center region of the circtangel is homogeneous with continuous stiffeners and frames normal to the plate. There will be many stiffener terminations in the kayak pontoon which is not ideal. The frames will also have to be welded to the outer plate with an angle.
2. Frames in the center part of the pontoon will be identical for the circtangel making fabrication easy. For the kayak pontoon, all the frames will be different.
3. There is no curved plate in the center region for the circtangel. No need for bending of plate and stiffeners. For the kayak pontoon, the curved portion of the pontoon will have to be built in some kind of support rather than on a flat floor which will complicate the fabrication.
4. The end of both pontoons consists of a curved plate. The complexity of the fabrication of the ends of both pontoons is considered to be similar. The kayak has a smaller radius with requires more bending and the circtangel has a larger radius which requires more curved material.

The choice of pontoon design is based on engineering judgment. Our recommendation is using the “circtangel” mainly due to the simple fabrication compared to the kayak pontoon.

The proposed design with calculations is given in the attached memo 10205546-13-NOT-087. Note that the design calculations are based upon steel quality S355 but is changed to S420; the same steel quality as the bridge girder and columns.

The pontoons are dimensioned for operating conditions (ULS) and for accidental filling of pontoon compartments (ALS). A conservative load approach is used where external sea pressure is put at the top of pontoons with relevant load factors for ULS and ALS limit states.

To avoid corrosion, the outer surfaces in the splash zone are made of steel material grade 25CR super duplex. The splash zone with a vertical extent of 6.5 m, is based on vertical movement from environmental loads with a return period of 100 years. This is conservative considering that according to DNVGL-OS-C101, the 100-year wave height shall be divided by three.

The structural net scantling weight for the “base case” pontoon without mooring lines is 705 Ton for a displacement of 3710 m^3. The structural net scantling weight for the pontoon with mooring lines is 934 Ton for a displacement of 5565 m^3.

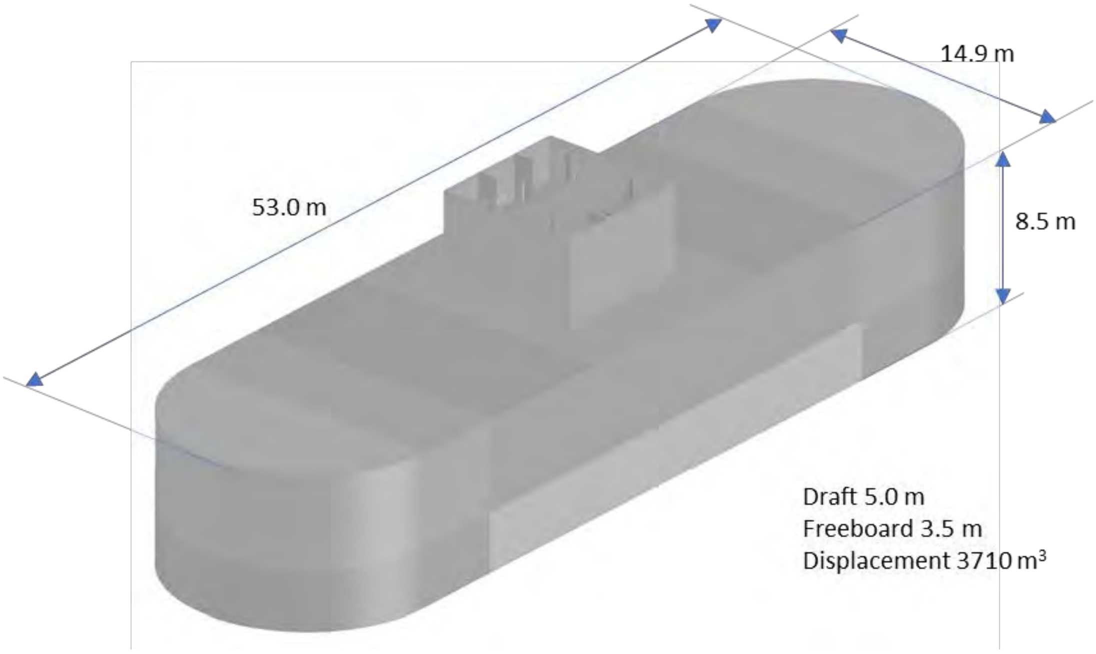


Figure 1-2: The geometric shape of pontoon without mooring lines. The inclination of top plate is not shown. The interface towards column is shown for illustration only.

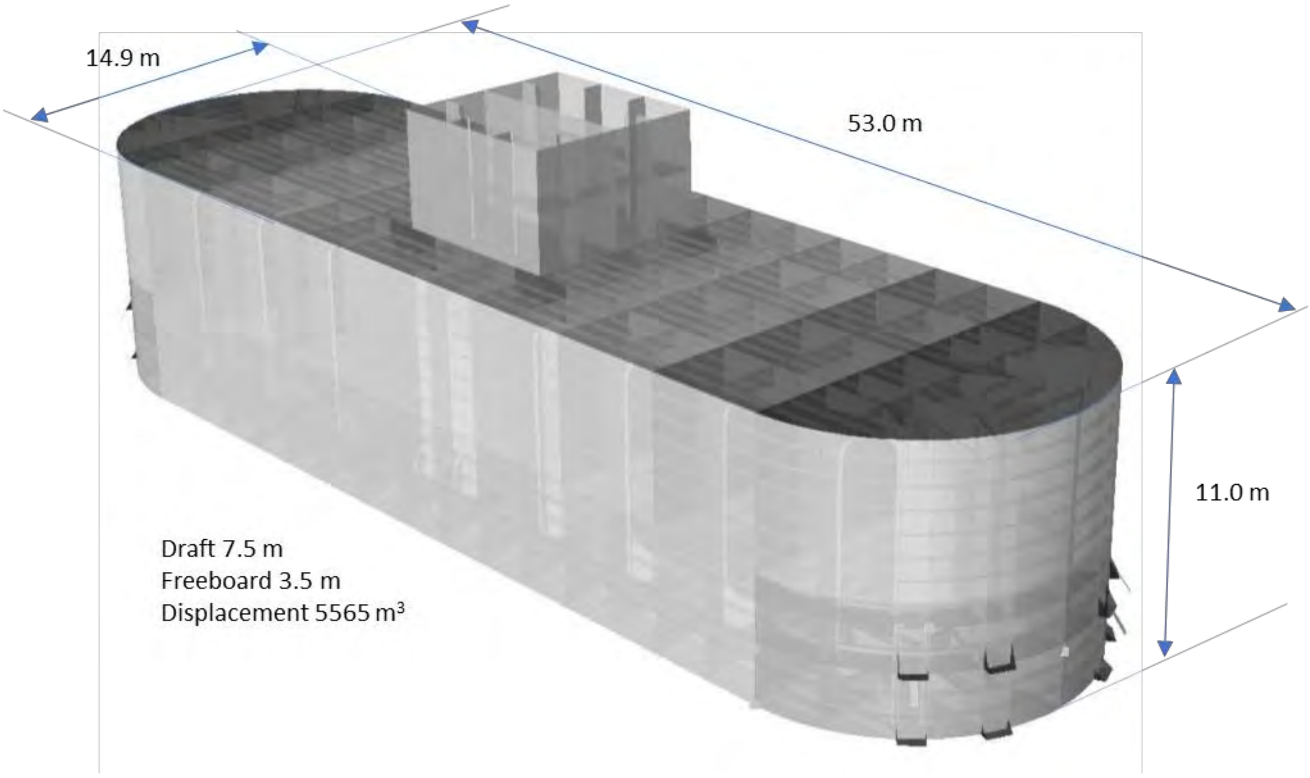


Figure 1-3: The geometric shape of pontoon with mooring lines. The inclination of top plate is not shown. The interface towards column is shown for illustration only.

1.3 Columns

There are two different column designs; one for axis 3-8 the higher bridge part, and another for axis 9-40 the lower bridge part.

The proposed design with calculations is given in the attached memo 10205546-13-NOT-086 and 10205546-13-NOT-099.

The columns are designed as rectangular sections towards the bridge girder and pontoons. The middle part of the columns has chamfered corners and is narrower than the top and bottom. This is done to improve wind drag, and to give the columns a more aesthetic appearance.

The main dimensions for the columns at axis 3-8 are 7.6m x 7.6m at the middle part, 9.6m x 9.6 m towards the bridge girder and 8m x 8m towards the pontoon, see also figure 1-4. The height of the columns differs between 45.566m and 26.855m.

The main dimensions for the columns at axis 9-40 are 5.2m x 6m at the middle part, 7.2m x 8 m towards the bridge girder and 8m x 8m towards the pontoon, see also figure 1.5. The height of the columns differs between 23.105m and 10.500m.

At the four corners between the column and the bridge girder/pontoon, there is casted steel details to simplify the welding and improve the fatigue life.

The columns with a skin plate thickness of 25 mm have capacity to withstand ULS combinations based on elastic capacity. However, the columns have insufficient capacity to withstand ALS combinations with ship impact. With an increase in plate thickness from 25 mm to 40 mm, the columns can absorb approximately 50 % of the currently defined energy during an impact. Another alternative is to increase the size of the narrow middle part of the columns.

Even though the current design is a single vertical column, there was performed a comparison with an A-shaped column as proposed by the architects. The attached memo 10205546-13-NOT-020 compares the required steel weight required to carry the loads from an eccentric ship impact between a rectangular and an A-shaped column design.

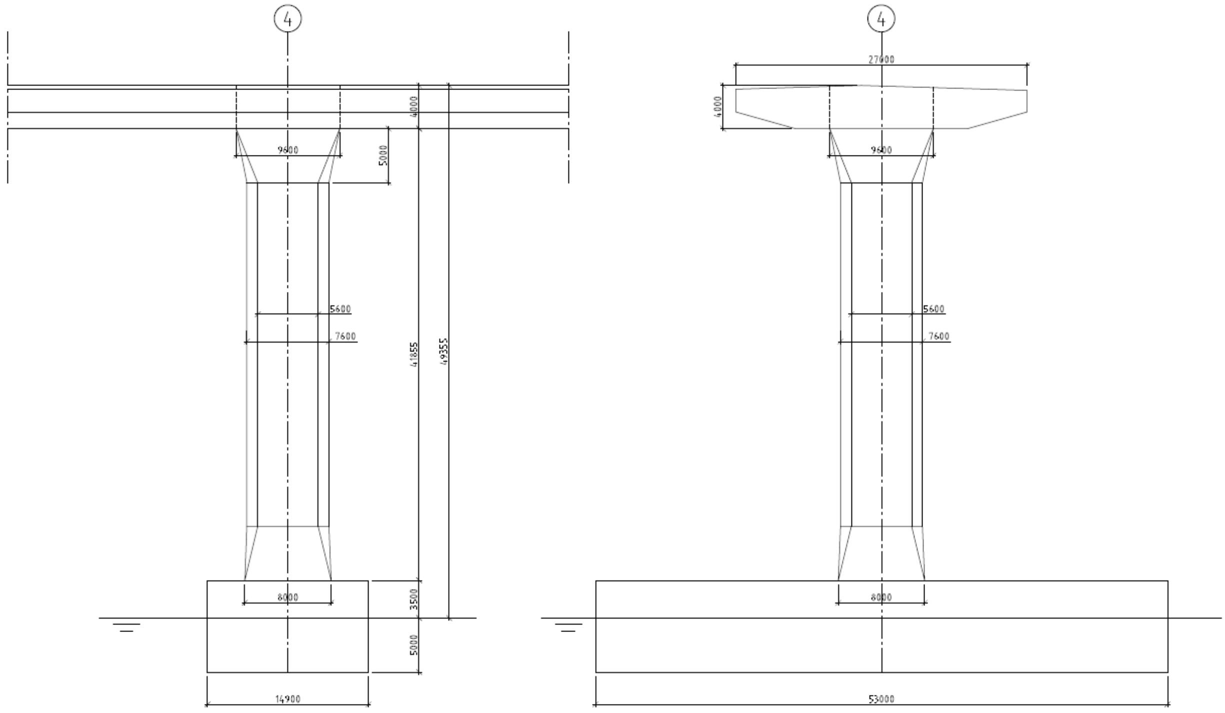


Figure 1-4: The geometric shape of column for high floating bridge part

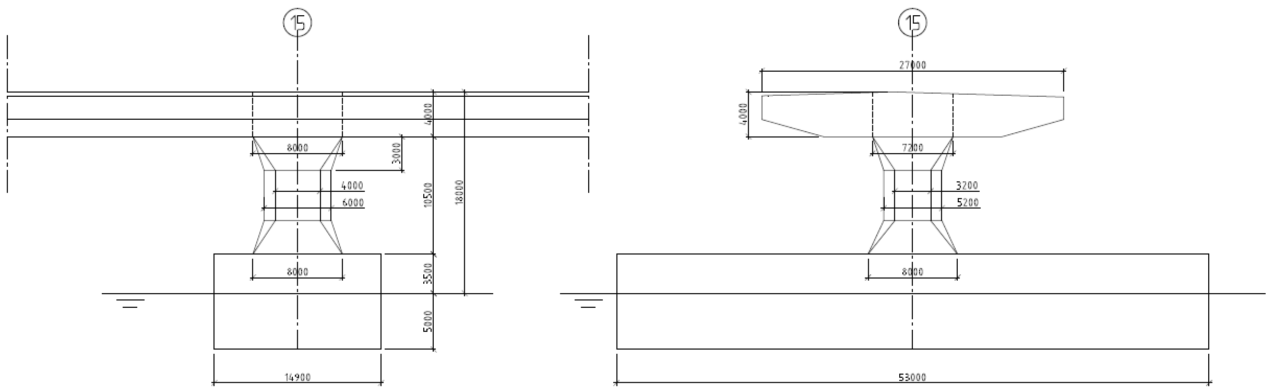


Figure 1-5: The geometric shape of column for low floating bridge part

1.4 Bridge Girder

In previous phase of the project, the bridge girder was designed as a standard suspension bridge box girder with optimal aerodynamic inclined outer webs. The design was optimized for a suspension bridge and not for a floating bridge where the structural response differs. The fabrication especially at the outer edges is complex and therefore relatively costly.

To simplify the design of the box girder and thereby reduce the fabrication costs and increase the amount of automatic welding, the recommended design is a rectangular shape where the lower flange is partly shaped towards the outer webs that are approximately half the height of the box.

The bridge girder has separate wind fairings, to be designed for optimized aerodynamic shape. These fairings are not included in the overall structural strength of the box girder and can therefore be of light weight.

The total width of the box girder without wind fairings is 27 m, and the total height is 4 m.

The design of the longitudinal stiffeners is based on panel buckling, fabrication costs and interface between bridge girder and column. Below the top deck plate, trapes stiffeners must be applied. In the interface between the bridge girder and column, bulb stiffeners are preferable. To simplify the design, the longitudinal bulb stiffeners active in the interface between bridge girder and column was kept continuous. Elsewhere, the longitudinal trapes stiffeners were used. The trapes stiffeners below the heavy lanes are additional strengthened due to local fatigue damage.

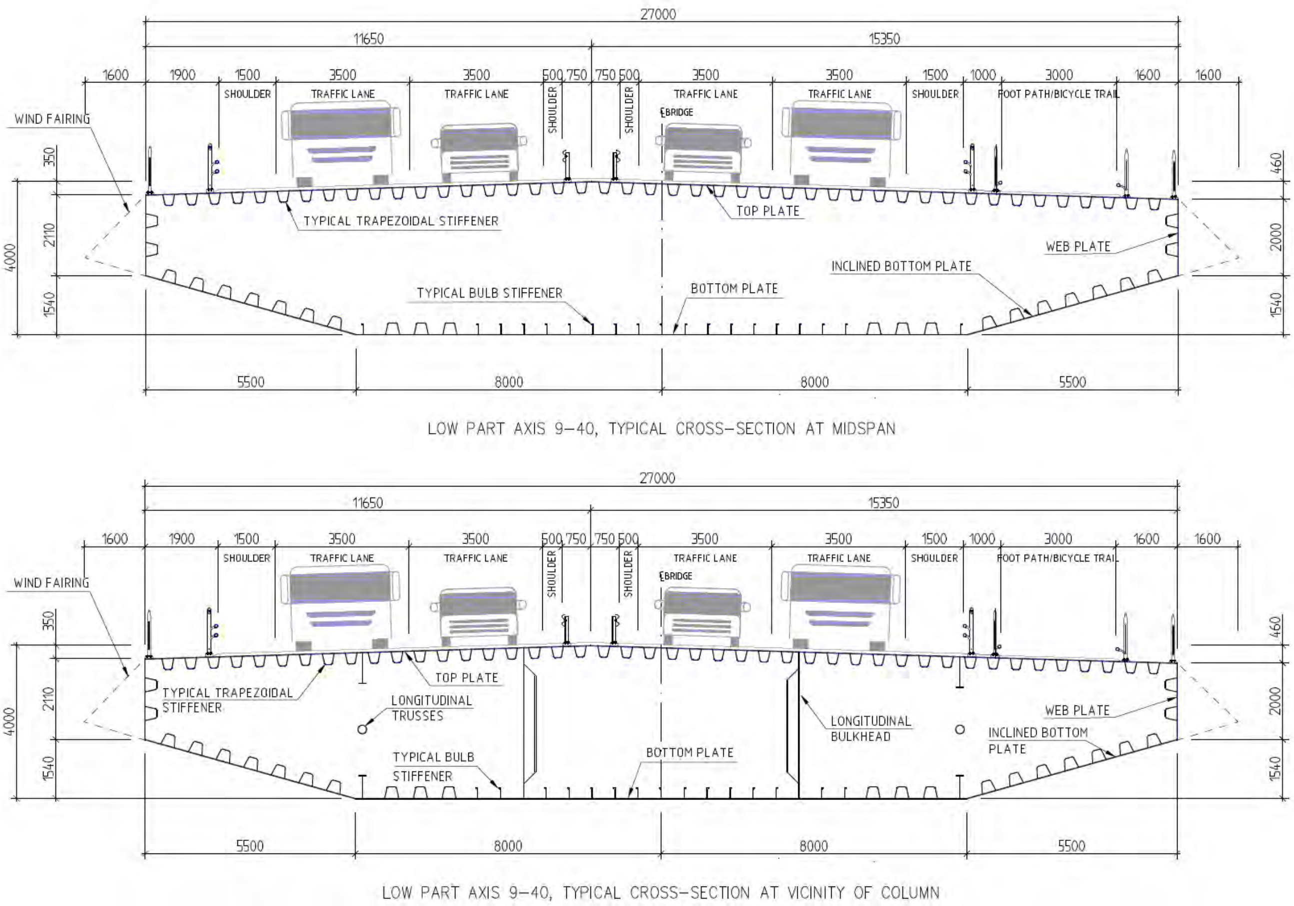
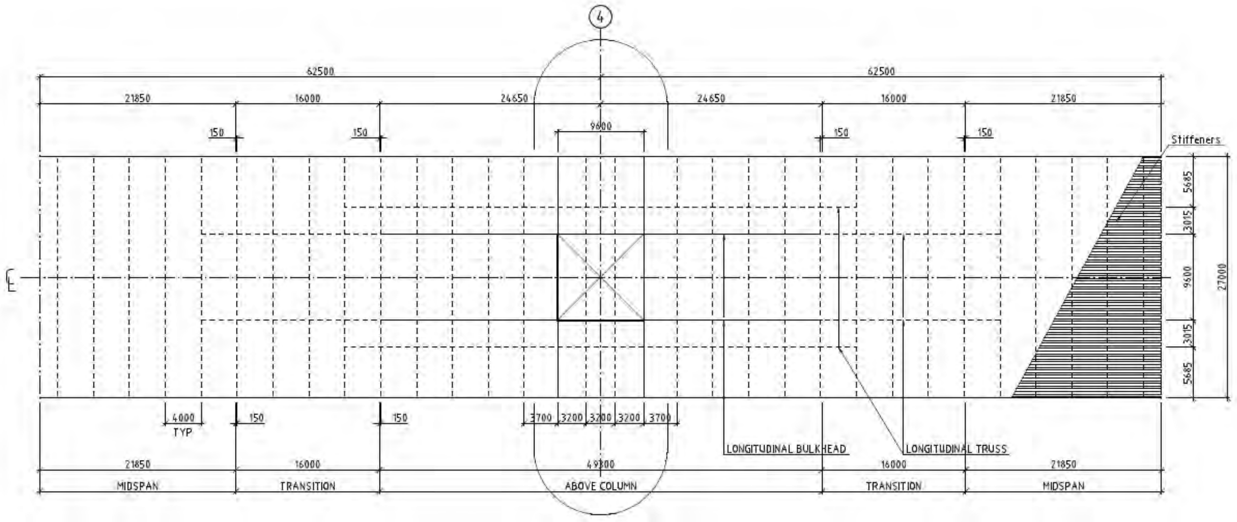


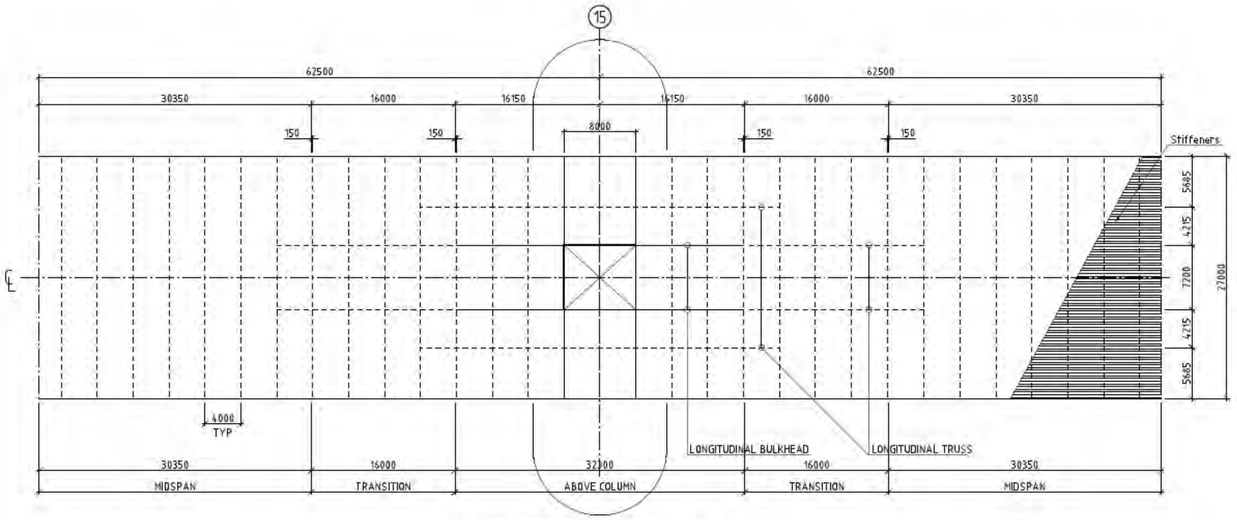
Figure 1-6: Cross section of bridge girder at midspan and in vicinity of the column

The bridge girders are designed with transverse trusses and bulkheads with cc 4.0 m. The transverse trusses and bulkheads help to carry dead loads and traffic loads from the orthotropic deck plate out to the outer vertical web-plates in the box girder. The trusses and bulkheads also give a rigid support to the longitudinal stiffened plates and helps to maintain the shape of the steel box. The design with calculations for the transverse trusses in the bridge girder is given the attached memo 10205546-13-NOT-083.

Longitudinal walls as trusses and bulkheads are introduced to reduce the shear lag effect in ultimate limit state and fatigue limit state. The walls are placed as shown in the figure below. Note that no longitudinal walls are placed in the midspan.The effective bending stiffness is reduced by approximately 75 % in midspan and 80 % at support for fatigue calculations.



FLOATING BRIDGE, HIGH PART AXIS 3–8



FLOATING BRIDGE, LOW PART AXIS 9–40

Figure 1-7: Plan view of bridge girder

The concept K12 is designed with three different types of sections along the bridge girder; one slimmer at midspan, another strengthened above the column and a transition section in between. In addition, the sections at midspan and above the column for axis 3 – 8 are additional strengthened.

Based on the results from the global analyses, the plate thicknesses and stiffeners were optimized. The plate thicknesses for the bridge girder skin plates are given in Table 1-2.

Table 1-2: The plate thicknesses for the bridge girder skin for K12

Concept K12	**Axis 3 - 8**			**Axis 9 - 40**		
	Midspan	Transition	Above column	Midspan	Transition	Above column
Top plate [mm]	16	16	16	16	16	16
Web plate [mm]	14	14	20	12	14	14
Inclined bottom plate [mm]	14	16	22	12	16	20
Bottom plate [mm]	14	16	22	12	16	20
Length within span of 125 m [m]	43.7	32.0	49.3	60.7	32.0	32.3
Length applied in global analysis	3/8 L	2/8 L	3/8 L	4/8 L	2/8 L	2/8 L

1.5 Connections between Columns and Bridge Girder/Column

The transition between the column and bridge girder is designed for continuous pass of forces between the elements. The junctions are reinforced with fatigue friendly details as shown in the figure below.

The four corners are made of cast steel to simplify the welding and improve the fatigue life.

The connections between the columns and pontoons are designed in a similar matter.

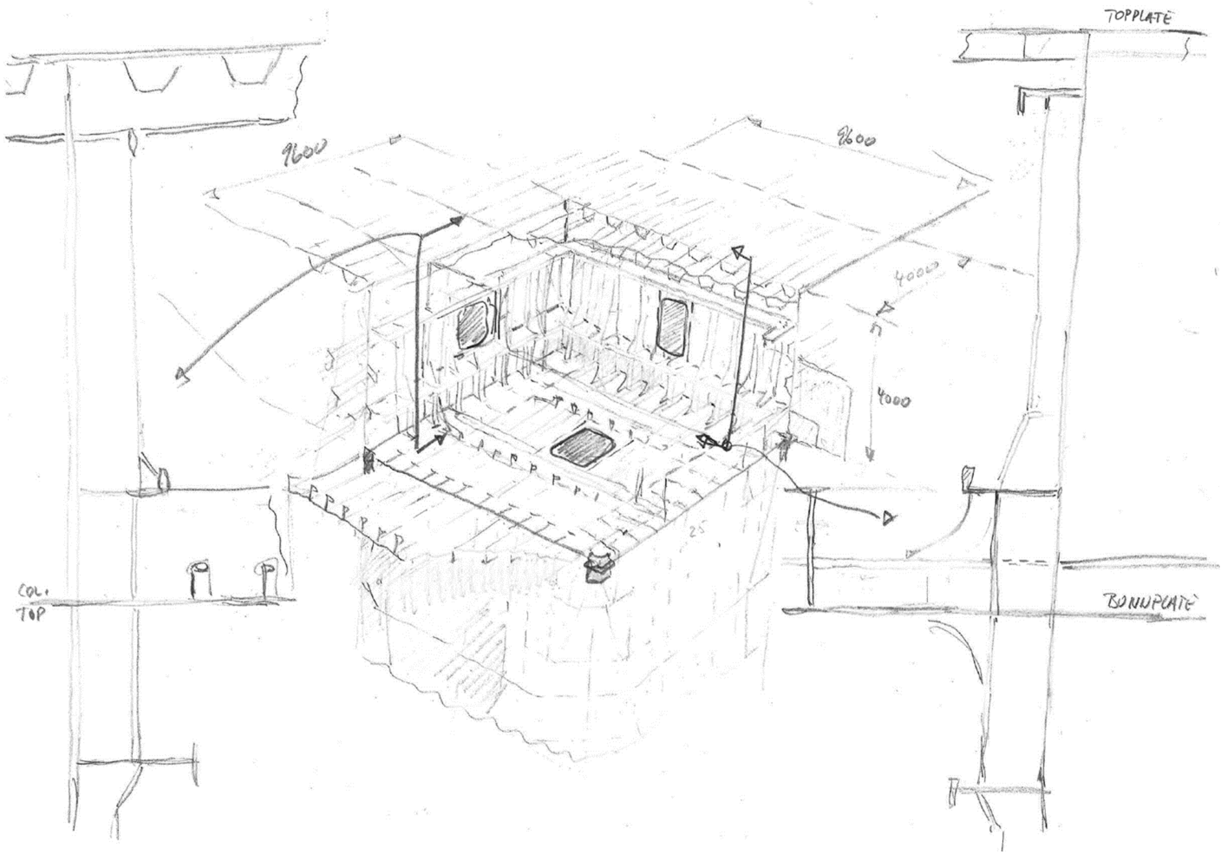


Figure 1-8: Connection between column and bridge girder

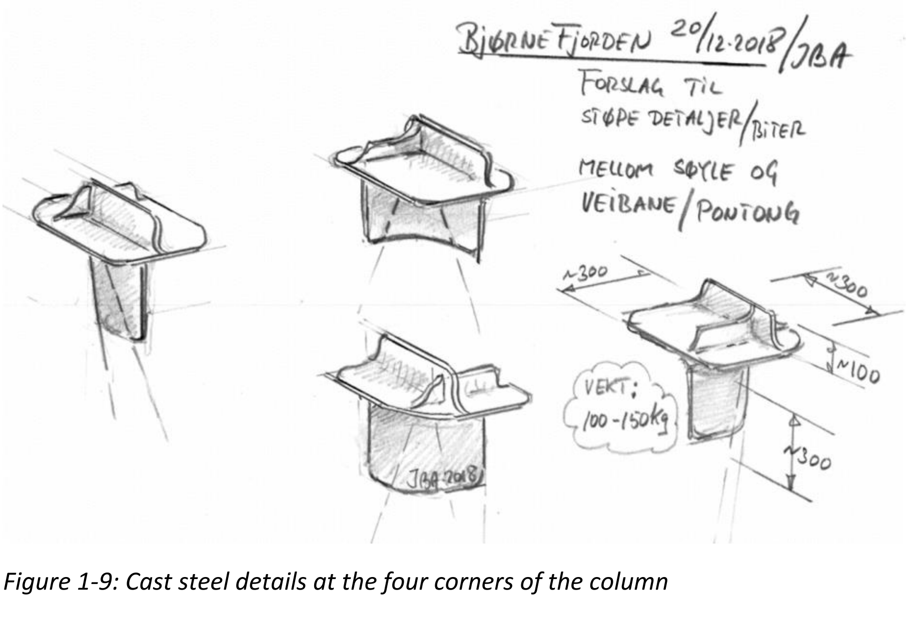


Figure 1-9: Cast steel details at the four corners of the column

1.6 End of Bridge Girder at Abutment North

The bridge girder is fixed to the abutment in North. The sectional forces in the bridge girder, in this end part of the bridge, have the largest sectional forces in the floating part of the bridge. The end part of the bridge girder towards the North abutment is therefore reinforced compared to the typical section of the bridge girder.

The performed design calculations are given in the attached memo 10205546-13-NOT-085. The proposed design results in an additional reinforcement of 267 ton at the last 52 meters of the bridge girder towards the North abutment.

1.7 Parapets and Railings

The attached memo 10205546-13-NOT-082 summarizes the requirements for traffic- and foot path/bicycle trail railings and defines which railings may be relevant to use in the design.

We have also looked at safety barriers, mainly for input to aerodynamic calculations. For this wide cross section of 27 m, we have uncovered the need for an extra railing at the outer edge of box girder to prevent climbing on the outside of the parapets. This according to SVV hb. N101, paragraph 3.4.3.

1.8 Access

The design of the bridge girder includes space for a continuous access way within the bridge girder along the floating bridge. This is marked in the design drawings of the bridge girder.

There are also considered hatches at every 125 m for vertical access into the bridge girder. Access to pontoons and columns needs to be developed in the later stage of the project.

2 Design Checks

2.1 ULS Load Combinations

The bridge is designed for ULS using the partial factor method described in Eurocode NS-EN 1990:2002/A1+NA:2016.

The following two load combinations are governing for the design of bridge girder in ULS:

- ULS2: Dominating traffic combined with 1-year environmental load.
- ULS3: 100-years environmental load with no traffic. The bridge is closed for traffic.

The two load combinations with factors are shown in the next two tables. The load factor γ is according to table NA.A2.4(B) in NS-EN 1990:2002/A1+NA:2016. The combination factor Ψ is according to table NA.A2.1 in NS-EN 1990:2002/A1 +NA:2016.

Table 2-1: ULS 2- Dominating traffic load

	Load Factor γ	**Comb. Factor ψ0**	**Total Factor γ * ψ0**	**Environmental Return Period**
Permanent	1.20	1.0	1.20	
Temperature	1.20	0.7	0.84	
Traffic	1.35	1.0	1.35	
Tide	1.60	0.7	1.12	100
Dynamic Wind	1.60	0.7	1.12	1
Static Wind	1.60	0.7	1.12	1
Wave	1.60	0.7	1.12	1
Swell	1.60	0.7	1.12	1
Current	1.60	0.7	1.12	100

Table 2-2: ULS 3 - Dominating environmental loads

	Load Factor γ	**Comb. Factor ψ0**	**Total Factor γ * ψ0**	**Environmental Return Period**
Permanent	1.20	1.0	1.20	
Temperature	1.20	0.7	0.84	
Traffic	1.35	0.0	0.00	
Tide	1.60	1.0	1.60	
Dynamic Wind	1.60	1.0	1.60	100
Static Wind	1.60	1.0	1.60	100
Wave	1.60	1.0	1.60	100
Swell	1.60	1.0	1.60	100
Current	1.60	1.0	1.60	100

Two different methods are used for combinations of basic loads; a direct method based on combination of time series of the individual loads and a factorized method in which design forces are established individually and then combined. The former method can maximize the stress in each selected design point and is used for all design evaluations, while the latter gives an easier overview of the contributions of the individual load components. For the presented plot in this chapter, the direct method is used.

2.2 Materials

The floating bridge part is made of steel plates and profiles with material quality S420 M/N except for the pontoon plates in the splash zone which is of material quality superduplex 25CR. For the S420 M/N, the yield strength is 420 MPa and the ultimate strength is 500 MPa, according to NS-EN 1993-1-1:2005/A1+NA:2015. For the super duplex 25CR, the yield strength is 550 MPa and the ultimate strength is 800 MPa.

The material factor for steel design in ULS is 1.1, according to NS-EN 1993-2:2006+NA:2009.

2.3 Design Checks of Bridge Girder

The maximum von Mises stresses are calculated at seven different extremity locations within the bridge girder. The seven check points are marked in the figure below.

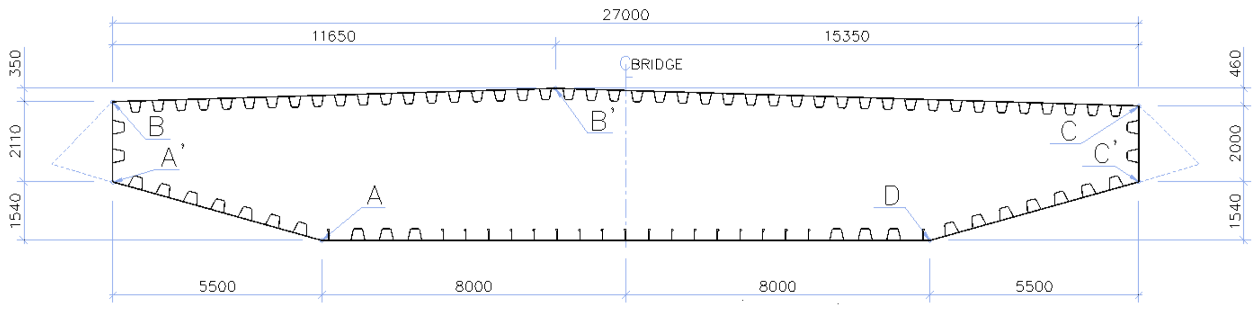


Figure 2-1: Stress check points at bridge girder

In the design of the bridge girder, the effects of shear lag and plate buckling are included in the ultimate, serviceability and fatigue limit state. Both the stiffness and the capacity of the bridge girder is reduced due to shear lag.

The attached memo 10205546-13-NOT-194 presents the design requirements and the applied design approach accounting for these effects.

The capacity of the bridge girder subject to compression and biaxial bending is verified based on equation (4.15) in NS-EN 1993-1-5:2006 + NA 2009. Equation (4.15) is a linear summation of the utilization that each force component utilizes the capacity corresponding to the respective type of force. Due to the bridge girders shape with an inclined bottom plate, the capacity check will give conservative utilization results for biaxial bending when the utilization about each axis is large at the same time. This capacity check is further referred to as Method 1.

Since the Eurocode does not account for conservative utilizations due to geometric shapes, a second method of performing the capacity check has been introduced. In the second method, the geometric shape is considered in the capacity check by calculating the utilization at all the 7 extremity points of the girder based on the effective elastic section modulus for the specific point. This capacity check is further referred to as Method 2.

3 Analyses Results

3.1 Limitations to Static Motion of Bridge Girder

According to Design Basis, the following static motion limitation shall be satisfied:

Motion limitation	Load scenario	Maximum motion
Vertical deformation from traffic loads	0.7 x traffic	$u_y \leq 1.5m$
Rotation about bridge axis from eccentric traffic loading	0.7 x traffic	$\theta_x \leq 1.0deg$
Rotation about bridge axis from static wind load	1-year static wind	$\theta_x \leq 0.5deg$

The maximum vertical deflection due to traffic with a load factor of 0.7 is 0.7 x 1.34m = 0.94 m which is below 1.5 m, ref. memo 10205546-11-NOT-088. The vertical deflection from traffic will be similar for all concepts.

Roll due to traffic shall be limited to 1 degree for 70 % of characteristic traffic loading. With traffic placed in the middle of the actual traffic lanes, the resulting roll will be 1.05 degree which is close to the criterion. However, if traffic is placed on the shoulder of the roadway, a roll of 0.7 x 1.9 = 1.33 degree is obtained around axis 6 which is above the given criterion, ref. memo 10205546-11-NOT-088 chapter 3.2.5.

Roll due to static 1-year wind is less than 0.1 m.

3.2 Von Mises Stresses

The results of maximum von Mises stresses, include the effect from all sectional forces, along the bridge based upon the result of the global analyses are shown in the figures below.

The following figures are extracted from the following enclosures in appendix G:

- Enclosure 2: K12_07 Load combination direct method

ULS2 is traffic loads with reduced environmental loads and ULS3 is 100-years environmental loads without traffic.

ULS2 is governing except for the strengthened bridge girder towards the North abutment. The utilization for the floating bridge girder between axis 3 and 40 is 1.00, occurring at midspan between axis 3 and 4.

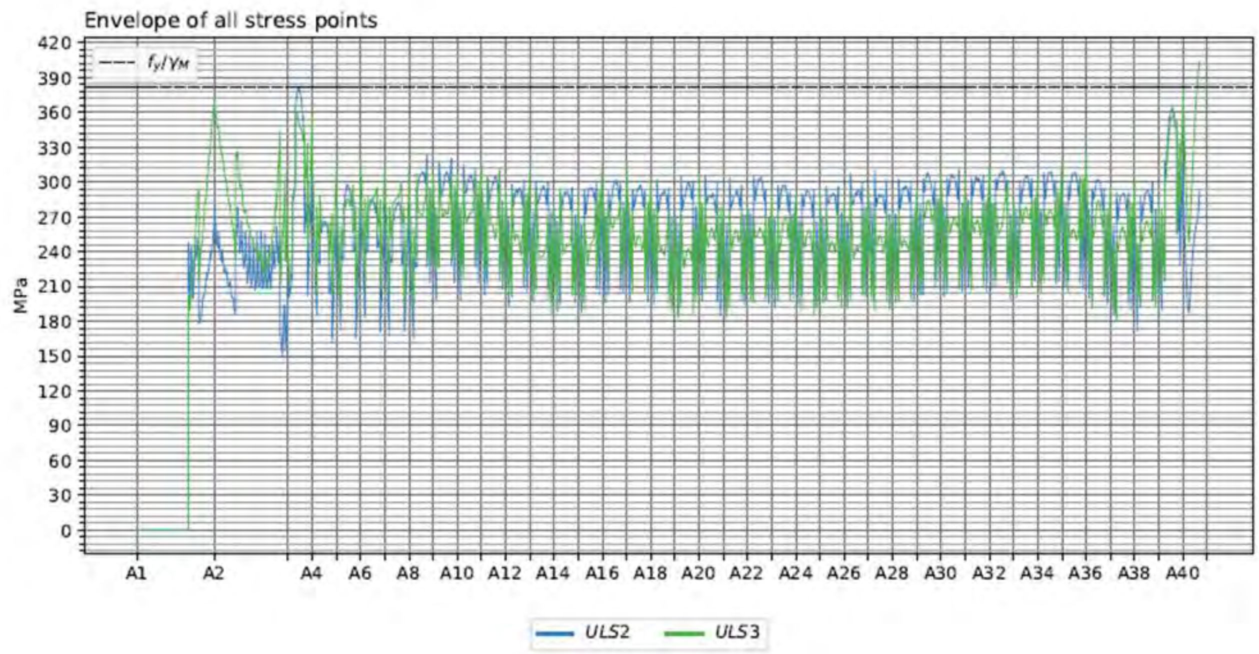


Figure 3-1: Maximum von Mises stresses along the bridge for K12

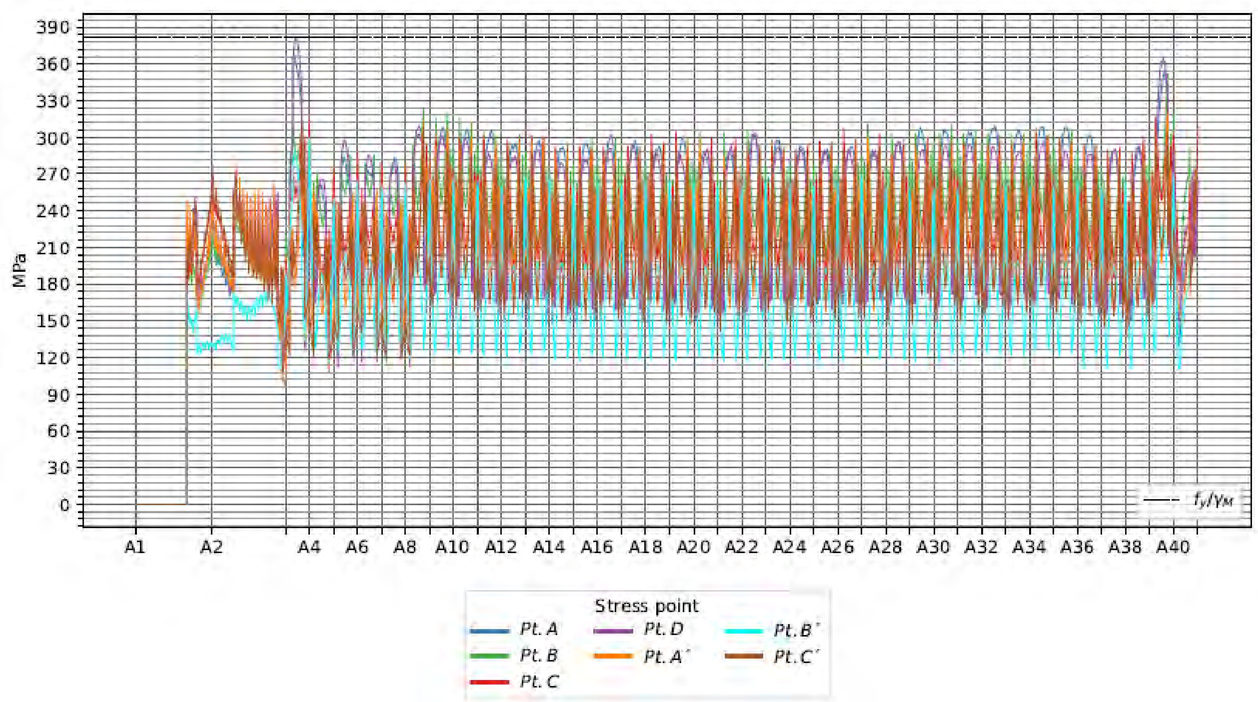


Figure 3-2: von Mises stresses along the bridge for K12 for ULS 2

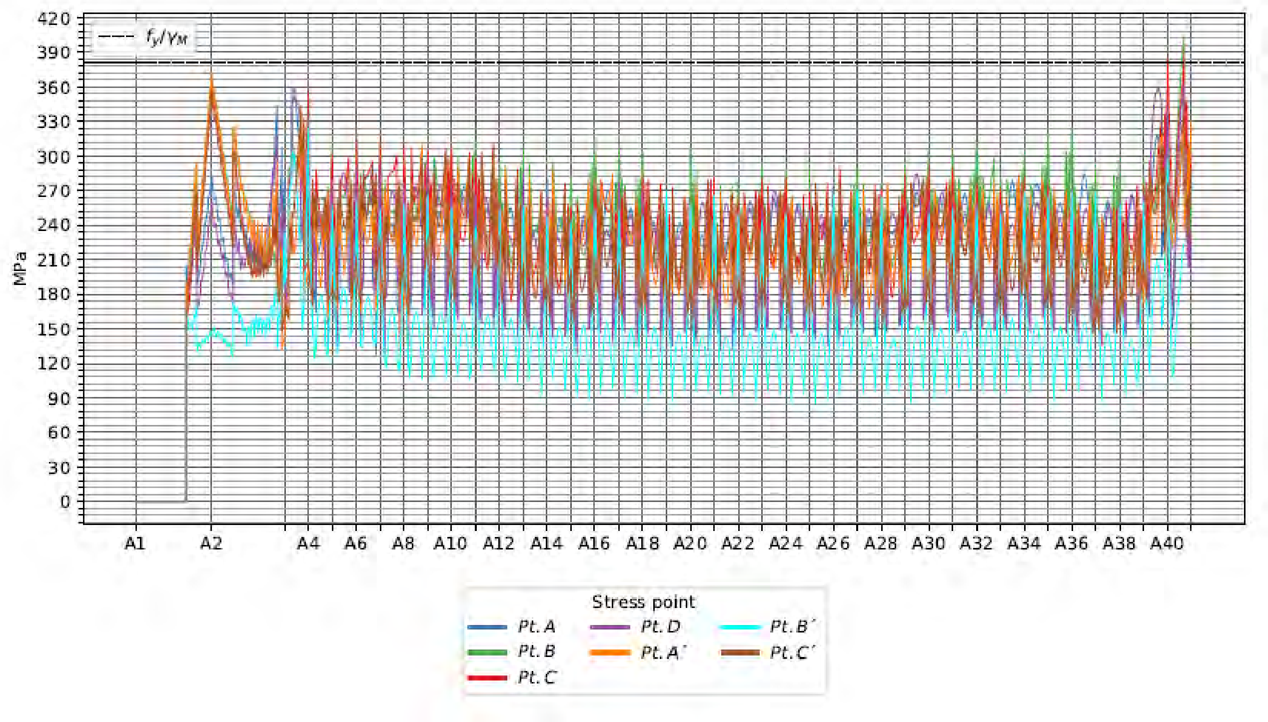


Figure 3-3: von Mises stresses along the bridge for K12 for ULS 3

3.3 Plate Buckling Capacity

The utilizations of capacity check along the bridge are shown in the figures below.

The following figures are extracted from the following enclosures in appendix G:

- Enclosure 2: K12_07 Load combination direct method

ULS2 is traffic loads with reduced environmental loads and ULS3 is 100-years environmental loads without traffic.

The capacity check is performed with the two methods described in attached memo 10205546-13-NOT-194 and defined in section 2.3.

Except for the ends of the floating bridge girder, the utilizations for plate buckling capacity are at acceptable values. The maximum utilization ratio between axes 3 and 4 is 1.09 with method 1 and 1.03 with method 2. The maximum utilization ratio at axis 40 is 1.21 indicating need of local reinforcement.

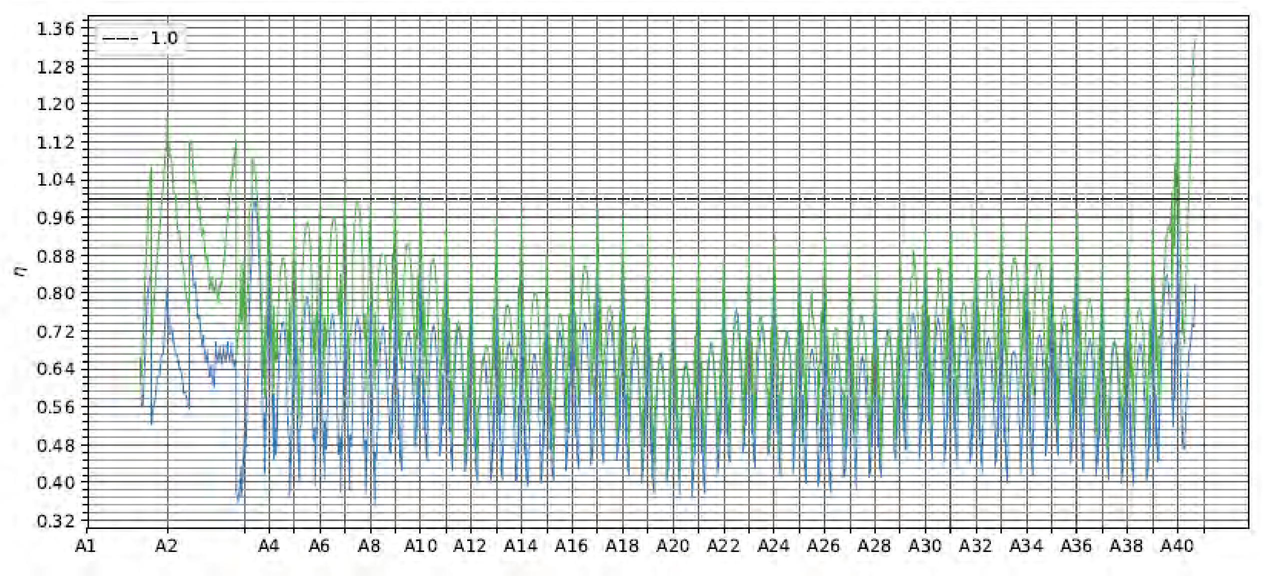


Figure 3-4: The utilization of capacity check along the bridge for K12 with method 1

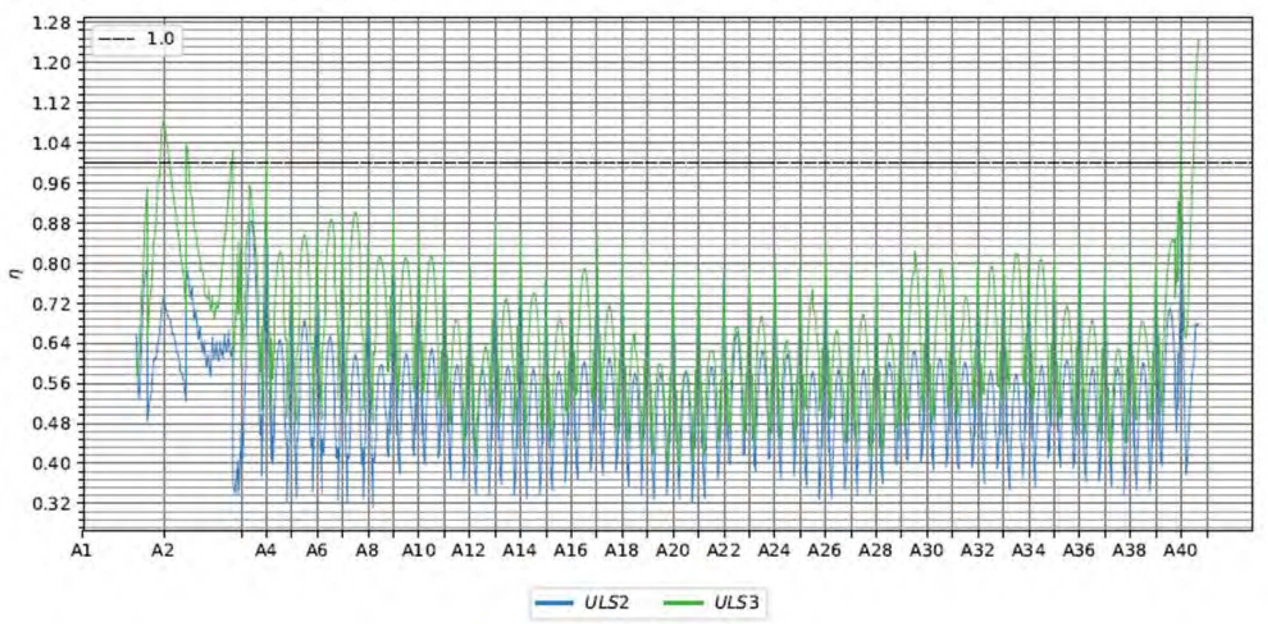


Figure 3-5: The utilization of capacity check along the bridge for K12 with method 2

3.4 Fatigue

The results and evaluation of the fatigue analyses are given in appendix I.

The following details of the bridge girder and the connection between bridge girder and column have been subjected to fatigue checks:

- Transverse plate splice, both outside and inside traffic lanes.
- Trapes stiffener with respect to cut-outs around stiffener, longitudinal weld and transverse splice at infilled section.
- Weld between cast piece and bridge girder, column top and bottom

To improve the fatigue life, the thickness of the top deck plate is increased to 16mm and the thickness of the trapes stiffeners below the heavy lanes is increased from 8mm to 10mm.

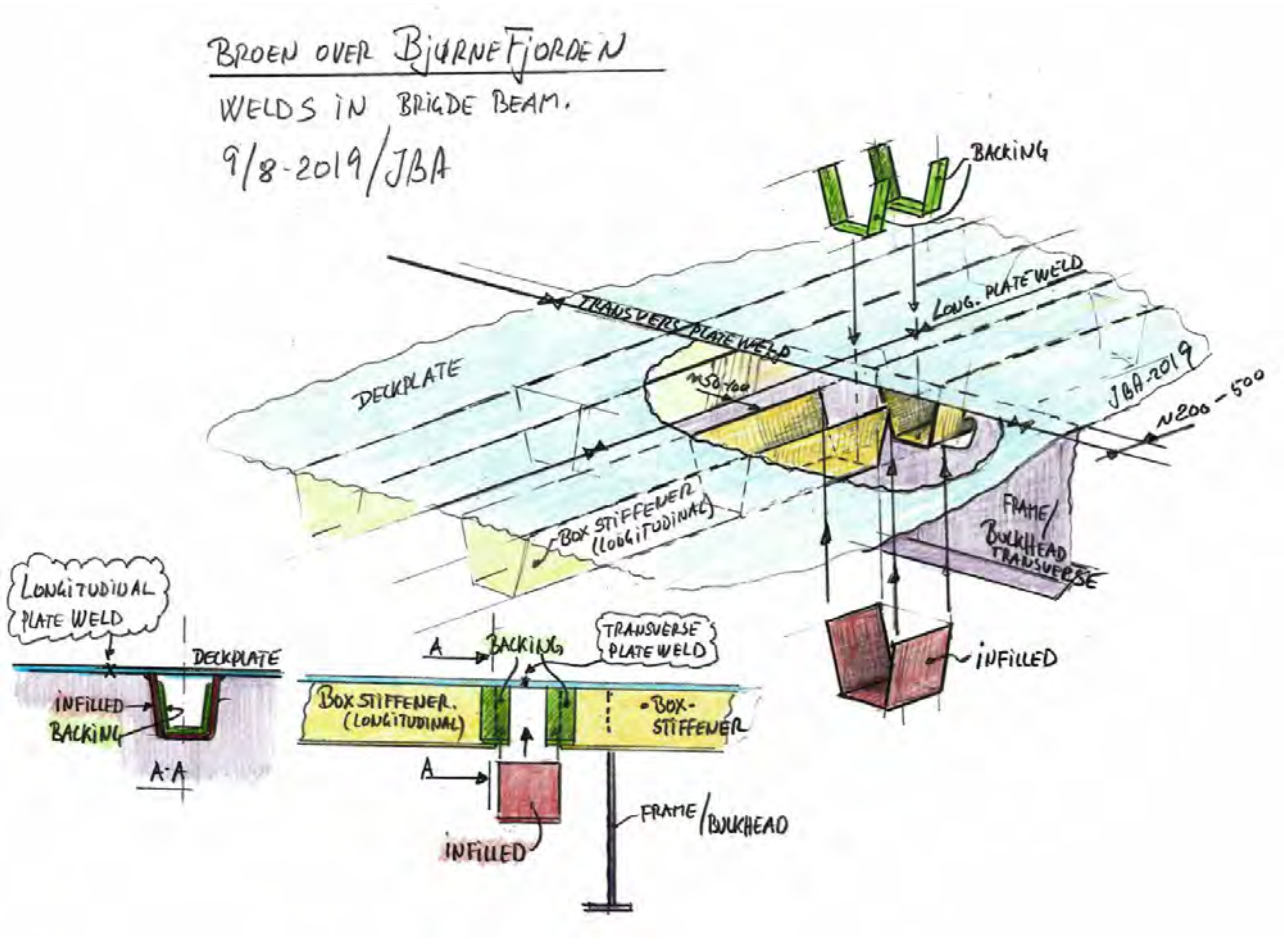


Figure 3-6: Details at top deck subjected to fatigue calculations.

3.5 Ship Collision

The results and evaluation of the analyses due to ship impacts are given in appendix J and in memo 10205546-13-NOT-099. See also section 0.

The severe damage to the bridge occurs in the pontoons and columns. Pontoon damage is acceptable in the sense of flooded volume whereas column damage is more challenging. Deckhouse collisions to the bridge girder causes limited damage to the bridge girder itself.

3.6 Local Traffic Load at Bridge Girder

In the global ULS analysis, the concentrated traffic loads from wheels are not included. Therefore, a local analysis of the bridge girder with these wheel loads from traffic is performed. Longitudinal stresses in longitudinal stiffener due to wheel load are added to the longitudinal stresses from global girder effects. A combination factor of 0.7 is used.

The double-axle concentrated loads for load model 1 are applied in accordance with NS-EN 1991-2:2003+NA:2010. Three tandem axel loads of 300 KN, 200 KN and 100 kN are placed such that the first wheel load is 0.5 m from the guard rail. The next wheel loads are following at distances 2.0 m, 0.5m, 2.0m, 0.5m and 2m. The guard rail is positioned 1.9 m from girder edge. The foot print from the wheel of 400mm x 400 mm is increased through the 80mm thick asphalt to a foot print of 560mm x 560mm.

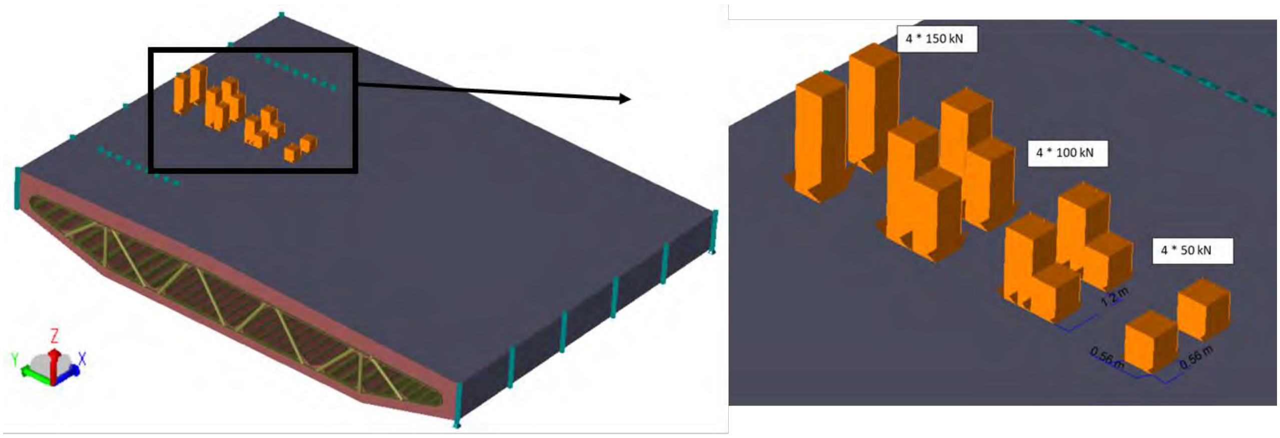


Figure 3-7: Applied concentrated traffic load in local analysis of bridge girder

The highest utilized stiffener is 4.8 m from edge of girder, where the longitudinal membrane stress is 131 MPa in the bottom of trapes stiffener at mid span between the transverse stiffener, see Figure 3-8. Under the wheel load near the guard rail at 1.9 m from girder edge, the stress in the stiffener is 110 MPa.

The maximum longitudinal membrane stress in the top plate is 37 MPa and will occur at transverse stiffener, see Figure 3-9.

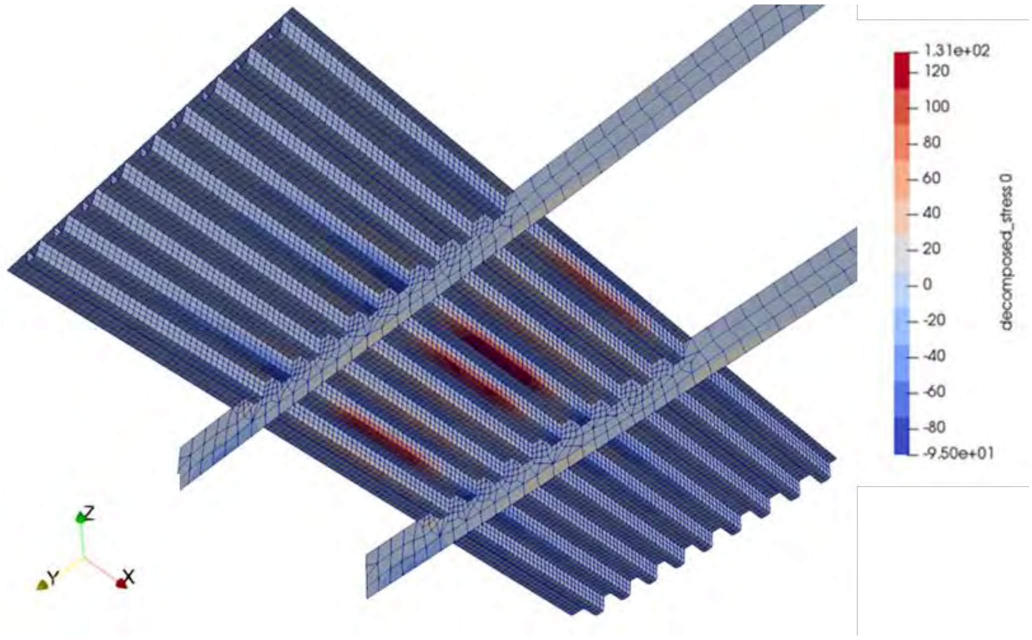


Figure 3-8: Longitudinal membrane stresses in trapes stiffeners below deck plate

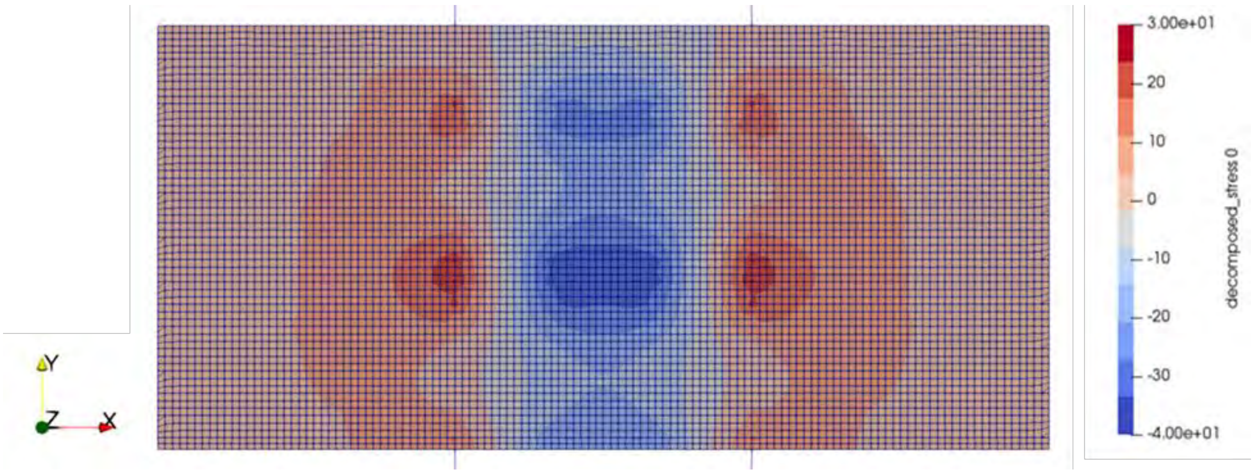


Figure 3-9: Longitudinal membrane stresses in deck plate

These local stresses are combined with the stresses from global stresses using a combination factor of 0.7 and relevant load factor. In ULS 2, the traffic is dominating, and load factor is 1.35. The overall factor to be used for the local stress is then 0.7 x 1.25 = 0.95.

The largest stress from local loads is at bottom of the stiffener. These stiffeners will only be subjected to longitudinal stresses from global axial load and biaxial bending moments. Shear stress from torsion and global shear forces will load only the skin and not the stiffener. The highest local stress is conservatively combined with the maximum normal stress in the bridge girder from ULS2 global forces. In a typical span, the normal stress is approximately 240 MPa. Adding 131 x 0.95 = 124 MPa gives a total combined stress of 364 MPa which is below 420/1.1 = 382 MPa.

3.7 Local Analyses of Bridge Girder and Column

A local finite element model has been made of a 125 m long bridge girder with column at the lower part of the floating bridge. Several analyses have been performed and are documented in the attached memo 10205546-13-NOT-099.

The major finding are as follows:

- ULS3 loads from the global analysis have been applied to the column to investigate the interface between bridge girder and column. The stresses in the column and bridge girder close to the column are acceptable. Hence, the structure has capacity to carry the forces applied.
- SCF factors have been found by applying unit forces to the beam ends. Focus has been devoted to the interface between column and bridge girder.
- Shear lag found in the FEM have been compared to the shear lag calculated with Eurocode rules. The results show that the shear lag calculated with Eurocode rules is slightly more conservative than the shear lag found with the FEM.
- Transverse frames have been checked for traffic loads. Findings are that the transverse frames have low utilization, and that the trapezoidal stiffeners carry shear forces and distribute local loads in a very effective manner.
- Torsion from an eccentric ship impact has been applied to three different column variations. Two columns with a narrow middle part, 25 mm and 40 mm skin plate thickness has been checked. One straight column with 25 mm skin plate has been checked. Results show that increasing the skin plate thickness will significantly increase the column torsional capacity with a moderate weight increase. Removing the narrow middle part of the columns so that the column is straight will increase the column torsional capacity even more with less added weight.
- Torsion from an eccentric ship impact has been applied to the column and bridge girder. Stress in the bridge girder is overall acceptable. The column is the weak link between pontoon and bridge girder.

4 Weight Estimates

An estimate of the total steel weight for the floating bridge is given in the table below. The listed weight corresponds to the design drawings.

Table 4-1: Estimates of total steel weight for the floating bridge part for K12

	Steel Weight Estimate (Ton)
Pontoon without moorings	24 899
Pontoon with moorings	2 803
Columns (incl 40mm plate)	6 681
Bridge girder	69 037
Total sum steel for floating bridge part	**103 420**

The weight of the columns is based on 40 mm skin plate thickness.

The increased weight of 15 % for pontoons is not included in the table above. However, these increases are included in the final material quantities.

Table 4-2: Weight estimates of the bridge girder for K12

Total per 125 meter for axis 3 -8	**Steel Weight Estimate (Ton)**
bridge girder skin with longitudinal stiffeners at midspan	465
bridge girder skin with longitudinal stiffeners in between	370
bridge girder skin with longitudinal stiffeners close to column	647
transverse frame of trusses (31 pcs)	181
transverse frame of bulkhead (2 pcs)	39
longitudinal trusses (184 m)	76
longitudinal bulkheads outside column (78.8 m)	87
longitudinal bulkheads within columns (19.2 m)	48
Sum for span of 125 meter	1 912
Sum for span per meter	15.3
Total per 125 meter from axis 9 -	
bridge girder skin with longitudinal stiffeners at midspan	615
bridge girder skin with longitudinal stiffeners in between	370
bridge girder skin with longitudinal stiffeners close to column	398
transverse frame of trusses (30 pcs)	175
transverse frame of bulkhead (2 pcs)	39
longitudinal trusses (128 m)	59
longitudinal bulkheads outside column (48 m)	53
longitudinal bulkheads within columns (16 m)	40
Sum for span of 125 meter	1 749
Sum for span per meter	14.0
Total	
steel weight for axis 3 - 8	10 823
steel weight for axis 9 - 41	56 150
additional reinforcement towards abutment in North	267
wind profile nose, 6mm steel plate	1 797
Total sum	**69 037**
Length of floating bridge part (m)	4 770

5 Reference List

5.1 Enclosed memos

/1/ 10205546-13-NOT-020 AMC status 2 - Weight comparison of rectangular and A-shaped columns, rev. 0

/2/ 10205546-13-NOT-082 AMC status 2 - Railings on bridge girder, rev. 0

/3/ 10205546-13-NOT-083 Transverse trusses in bridge girder, rev. 1

/4/ 10205546-13-NOT-085 End of bridge girder at abutment north, rev. 1

/5/ 10205546-13-NOT-086 Column design, rev. 1

/6/ 10205546-13-NOT-087 Design of pontoons, rev. 1

/7/ 10205546-13-NOT-099 FEM analysis of bridge girder and column, rev. 0

/8/ 10205546-13-NOT-194 Shear lag and buckling effects of bridge girder concept 12, rev. 0

5.2 Design Drawings

5.2.1 Pontoons

/9/ SBJ-33-C5-AMC-22-DR-300 Floating bridge pontoon K12, general arrangement, dimensions, rev.1

/10/ SBJ-33-C5-AMC-22-DR-301 Floating bridge pontoon K12, arrangement, tank plan, rev.0

/11/ SBJ-33-C5-AMC-22-DR-302 Floating bridge pontoon K12, bottom plate, dimension plate and stiffeners, rev.0

/12/ SBJ-33-C5-AMC-22-DR-303 Floating bridge pontoon K12, top plate, dimension plate and stiffeners, rev.0

/13/ SBJ-33-C5-AMC-22-DR-304 Floating bridge pontoon K12, internal plate, longitudinal structure 4000 mm from CL, rev.0

/14/ SBJ-33-C5-AMC-22-DR-305 Floating bridge pontoon K12, internal plate, longitudinal structure in CL, rev.0

/15/ SBJ-33-C5-AMC-22-DR-306 Floating bridge pontoon K12, side, longitudinal structure 7450 mm from CL, rev.0

/16/ SBJ-33-C5-AMC-22-DR-307 Floating bridge pontoon K12, internal structure, transverse frame no. 02 (no. 19), rev.0

/17/ SBJ-33-C5-AMC-22-DR-308 Floating bridge pontoon K12, internal structure, transverse frame no. 07 (no. 14), rev.0

/18/ SBJ-33-C5-AMC-22-DR-309 Floating bridge pontoon K12, internal structure, transverse frame no. 08 (no. 13), rev.0

/19/ SBJ-33-C5-AMC-22-DR-310 Floating bridge pontoon K12, internal structure, transverse frame no. 09, rev.0

/20/ SBJ-33-C5-AMC-22-DR-351 Floating bridge pontoon K12, plan bottom deck, fairlead reinforcement, rev.0

/21/ SBJ-33-C5-AMC-22-DR-352 Floating bridge pontoon K12, plan pontoon deck 11000 ab. base line, fairlead reinforcement, rev.0

/22/ SBJ-33-C5-AMC-22-DR-353 Floating bridge pontoon K12, longitudinal structure in CL, fairlead reinforcement, rev.0

/23/ SBJ-33-C5-AMC-22-DR-354 Floating bridge pontoon K12, longitudinal structure 4000 mm from CL, fairlead reinforcement, rev.0

/24/ SBJ-33-C5-AMC-22-DR-355 Floating bridge pontoon K12, longitudinal structure 7450 from CL, fairlead reinforcement, rev.0

/25/ SBJ-33-C5-AMC-22-DR-356 Floating bridge pontoon K12, curved structure bow and stern, fairlead reinforcement, rev.0

5.2.2 Bridge Girder

/26/ SBJ-33-C5-AMC-22-DR-401 Floating bridge girder K12, high part axis 3 - 8, typical plan, rev.0

/27/ SBJ-33-C5-AMC-22-DR-402 Floating bridge girder K12, high part axis 3 - 8, typical cross-section at midspan, rev.1

/28/ SBJ-33-C5-AMC-22-DR-403 Floating bridge girder K12, high part axis 3 - 8, typical cross-section at transition, rev.0

/29/ SBJ-33-C5-AMC-22-DR-404 Floating bridge girder K12, high part axis 3 - 8, typical cross-section above column, rev.0

/30/ SBJ-33-C5-AMC-22-DR-405 Floating bridge girder K12, high part axis 3 - 8, typical transverse bulkhead above column, rev.0

/31/ SBJ-33-C5-AMC-22-DR-406 Floating bridge girder K12, high part axis 3 - 8, typical longitudinal truss and bulkhead, rev.0

/32/ SBJ-33-C5-AMC-22-DR-407 Floating bridge girder K12, high part axis 3 - 8, typical longitudinal detail above column, rev.0

/33/ SBJ-33-C5-AMC-22-DR-431 Floating bridge girder K12, low part axis 9 - 40, typical plan, rev.0

/34/ SBJ-33-C5-AMC-22-DR-432 Floating bridge girder K12, low part axis 9 - 40, typical cross-section at midspan, rev.1

/35/ SBJ-33-C5-AMC-22-DR-433 Floating bridge girder K12, low part axis 9 - 40, typical cross-section at transition, rev.0

/36/ SBJ-33-C5-AMC-22-DR-434 Floating bridge girder K12, low part axis 9 - 40, typical cross-section above column, rev.0

/37/ SBJ-33-C5-AMC-22-DR-435 Floating bridge girder K12, low part axis 9 - 40, typical transverse bulkhead above column, rev.0

/38/ SBJ-33-C5-AMC-22-DR-436 Floating bridge girder K12, low part axis 9 - 40, typical longitudinal truss and bulkhead, rev.0

/39/ SBJ-33-C5-AMC-22-DR-437 Floating bridge girder K12, low part axis 9 - 40, typical longitudinal detail above column, rev.0

/40/ SBJ-33-C5-AMC-22-DR-451 Floating bridge girder K12, stiffener details, rev.1

/41/ SBJ-33-C5-AMC-22-DR-461 Floating bridge girder K12, low part, bridge girder K11-K14, end of bridge girder North abutment, plan and elevation, rev.0

/42/ SBJ-33-C5-AMC-22-DR-462 Floating bridge girder K12, low part, bridge girder K11-K14, end of girder at North abutment and section, rev.0

5.2.3 Column

/43/ SBJ-33-C5-AMC-22-DR-471 Floating bridge column K12, high part axis 3 - 8, structural arrangement and dimensions, rev.0

/44/ SBJ-33-C5-AMC-22-DR-481 Floating bridge column K12, low part axis 9 -40, structural arrangement and dimensions, rev.0

5.2.4 Floating Bridge Arrangement

/45/ SBJ-33-C5-AMC-22-DR-491 Floating bridge K12, high part axis 3 - 8, typical structural arrangement, rev.0

/46/ SBJ-33-C5-AMC-22-DR-492 Floating bridge K12, low part axis 9 - 40, typical structural arrangement, rev.0

Concept development, floating bridge
E39 Bjørnafjorden

Appendix K – Enclosure 1

10205546-13-NOT-020
Weight Comparison of rectangular and A-shaped columns

MEMO

PROJECT	**Concept development, floating bridge E39 Bjørnafjorden**	DOCUMENT CODE	10205546-13-NOT-020
CLIENT	Statens vegvesen	ACCESSIBILITY	Restricted
SUBJECT	AMC status 2 – Weight Comparison of rectangular and A-shaped columns	PROJECT MANAGER	Svein Erik Jakobsen
TO	Statens vegvesen	PREPARED BY	Bjørn William Strand
COPY TO		RESPONSIBLE UNIT	AMC

SUMMARY

This memo was created to compare the required steel weight to carry the loads from an eccentric ship impact between a rectangular, and an A-shaped column design.

The main findings were that the required steel weight in the A-shaped column was about 50% more than for the rectangular column and the rectangular column was considered as the preferable design.

One of the reasons for this difference in weight was because a column split in two, with the same base area as a single column, requires two extra skin plane panels. Another reason was that the A-shaped column geometry is less favourable than the rectangular column when acted upon by the forces from an eccentric ship impact.

When considering fabrication, it was also concluded that the A-shaped column requires more welding and two additional longitudinal bulkheads over each column.

0	29.03.2019	Status 2 issue	B.W.Strand	P.N.Larsen	S.E.Jakobsen
REV.	DATE	DESCRIPTION	PREPARED BY	CHECKED BY	APPROVED BY

1 Introduction

The purpose of this memo is to investigate the difference in the resulting steel weight between an A-shaped and a rectangular column when subjected to an eccentric ship impact. In addition, some remarks regarding fabrication is given.

To achieve this, two Finite Element (FE) models will be investigated. One, with the A-shaped column and one with the rectangular column.

Section forces resulting from the ship impact in the top and bottom of the column, obtained from the global analysis are applied to the column. The goal is to adjust the skin-plate thickness in the two columns such that the Von-Mises stress along the two columns have similar values. To obtain comparable models, the base area of the two column types is equal.

2 FE-Models

A 37 m long section of the bridge is modelled with shell elements. The element size is approximately 250x250 mm. The column height is according to Axis 3, 48.03m. The modelled section of the bridge girder is included to obtain a more realistic stiffness at the column top.

The plate thickness of the columns is adjusted such that the two columns have comparable von-Mises stress values along the height. The columns in the model are modelled without internal stiffeners. The resulting plate thickness can thus be considered as an equivalent plate thickness, consisting of the skin-plate, plus a smeared representation of the stiffeners. At the mid-section of the columns, a plate thickness of about 40mm is applied. A section near the top of the columns is removed from the analysis, due to high concentrated stresses due to a lack of refinement in the model. A small distance from the top, the plate thickness is set to 50mm for the rectangular column, and 55 mm for the A-shaped column.

The geometry used in the FE models for the two solutions are shown in Section 2.2 and 2.3.

Table 1: Plate thickness and weight.

Column Type	Weight [ton]	Equivalent Plate Thickness [mm]
Rectangular	659	40 – 50 – 80
A-Shape	967	40 – 50 - 55 – 80

2.1 Coordinate System

The global coordinate system is defined according to Table 2.

Table 2: Global Coordinate system.

Axis	Direction
X	North
Y	West
Z	Up

2.2 A-shaped columns

The investigated A-shaped column is shown in Figure 1. Each column leg has a width of 4.5m and a depth in the bridge direction of 8m. In the column-girder connection, the columns are separated by 1m.

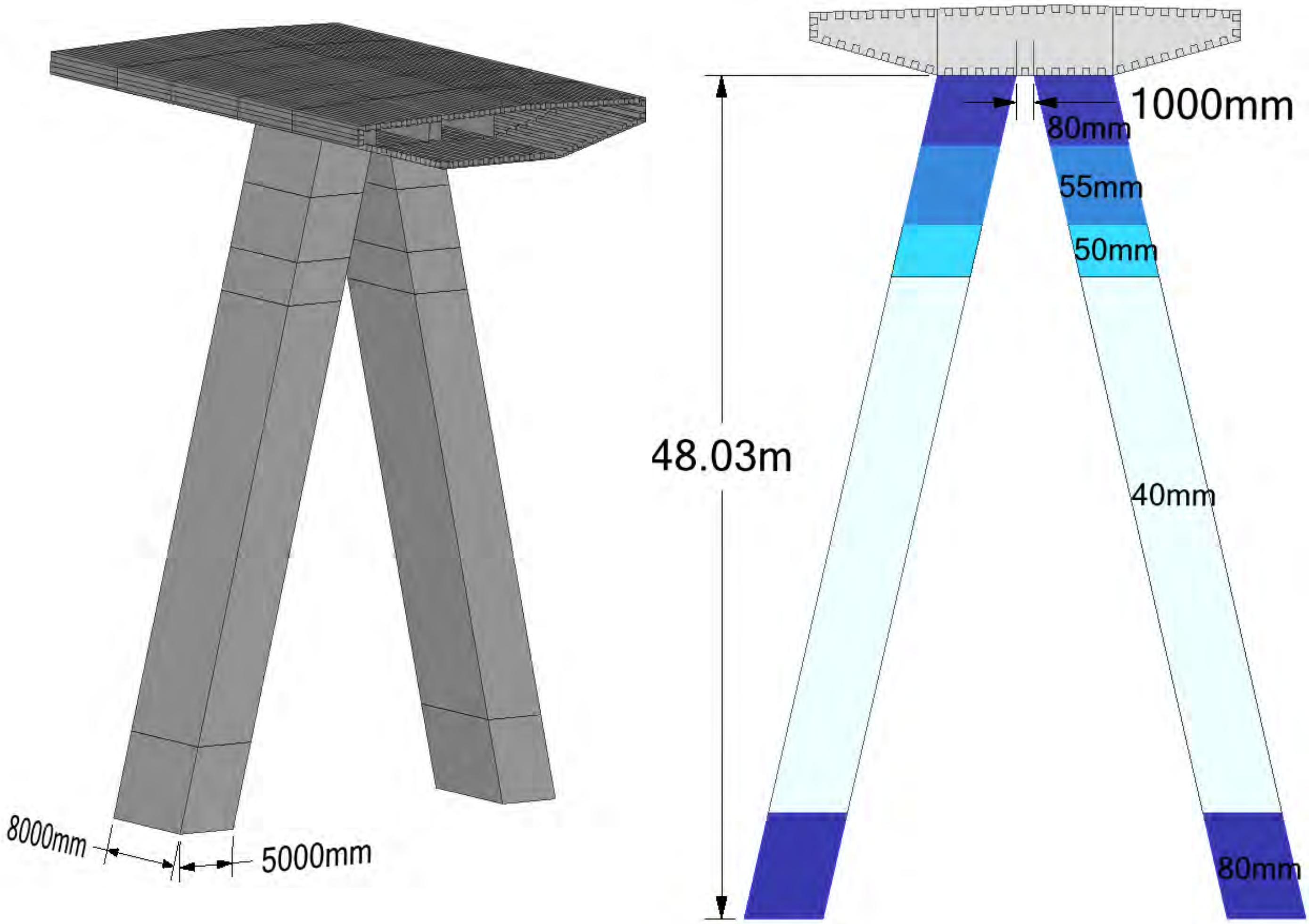


Figure 1: A-Shaped column

2.3 Rectangular column

The investigated rectangular column can be seen in Figure 2. The column has a width of 9m, and a depth along the bridge direction of 8m. The dimensions are chosen such that the total base area is equal to that of the A-shaped column.

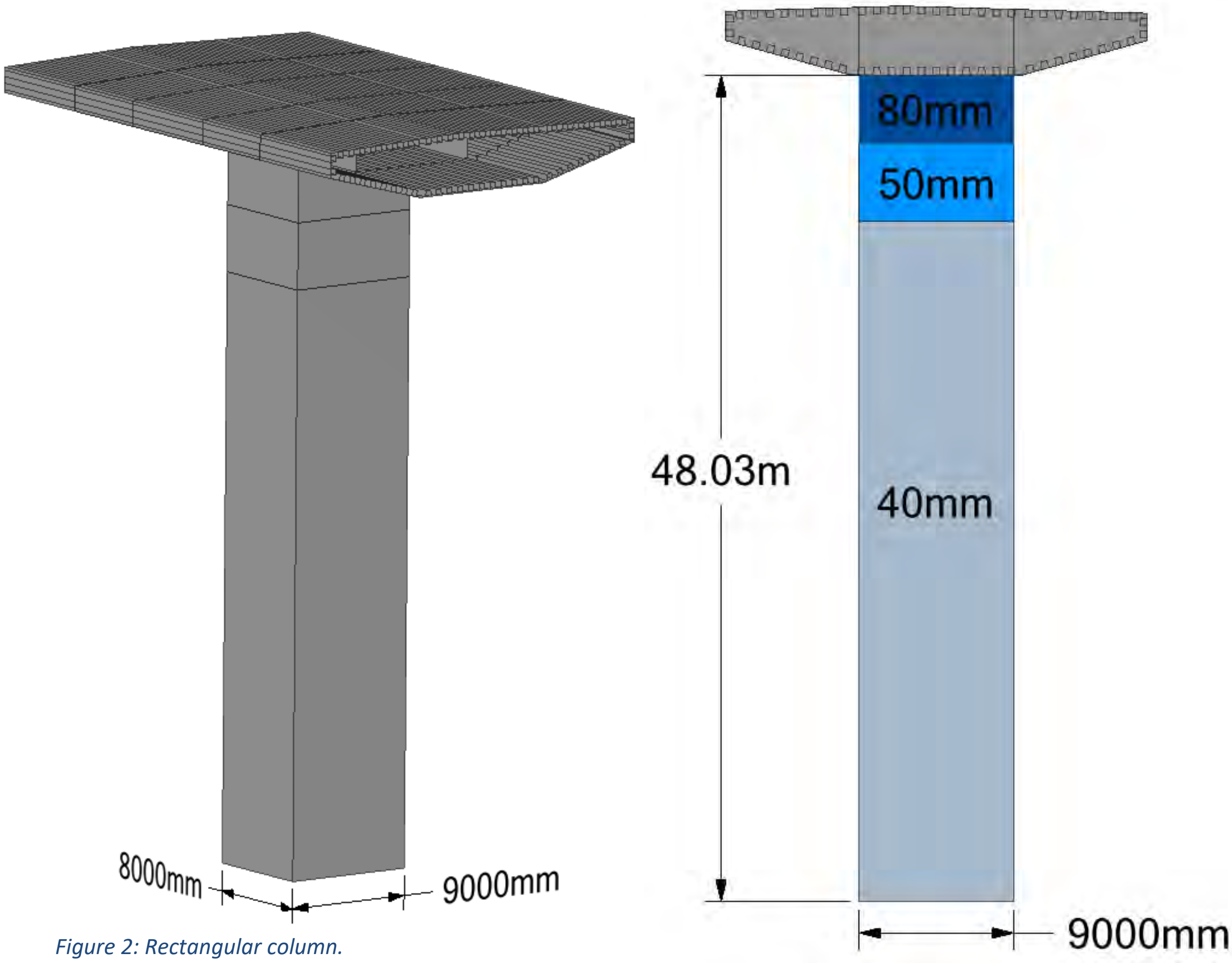


Figure 2: Rectangular column.

2.4 Boundary Conditions

Fixed boundary conditions are applied to the two ends of the bridge girder as described in Table 3.

Table 3: Boundary Conditions

	Translation			**Rotation**		
Girder end	X	Y	Z	X	Y	Z
North	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
South	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed

2.5 ALS Load

Loads from an eccentric ship impact from phase 3 of the project is applied to the bottom of the columns.

The resulting moments in the column top and bottom due to the ship impact is shown in Table 4.

Table 4: Moments from ship impact.

Location	**M_x [MNm]**	**M_y [MNm]**	**M_z [MNm]**
Bottom	-366	1455	600
Top	525	179	600

The loads applied to the bottom of the column to induce the column top and bottom moments are presented in Table 5. In addition, a vertical load is added (Z-direction). This is the self-weight calculated in the previous project phase.

Table 5: Applied Loads.

F_x [MN]	**F_y [MN]**	**F_z [MN]**	**M_x [MNm]**	**M_y [MNm]**	**M_z [MNm]**
26.57	18.55	21.30	-366	179	600

The forces and moments shown in Table 4, are applied to the column bottom with a rigid multi point constraint. The force components are applied according to the local coordinate system shown in Figure 4 and as denoted in Table 4.

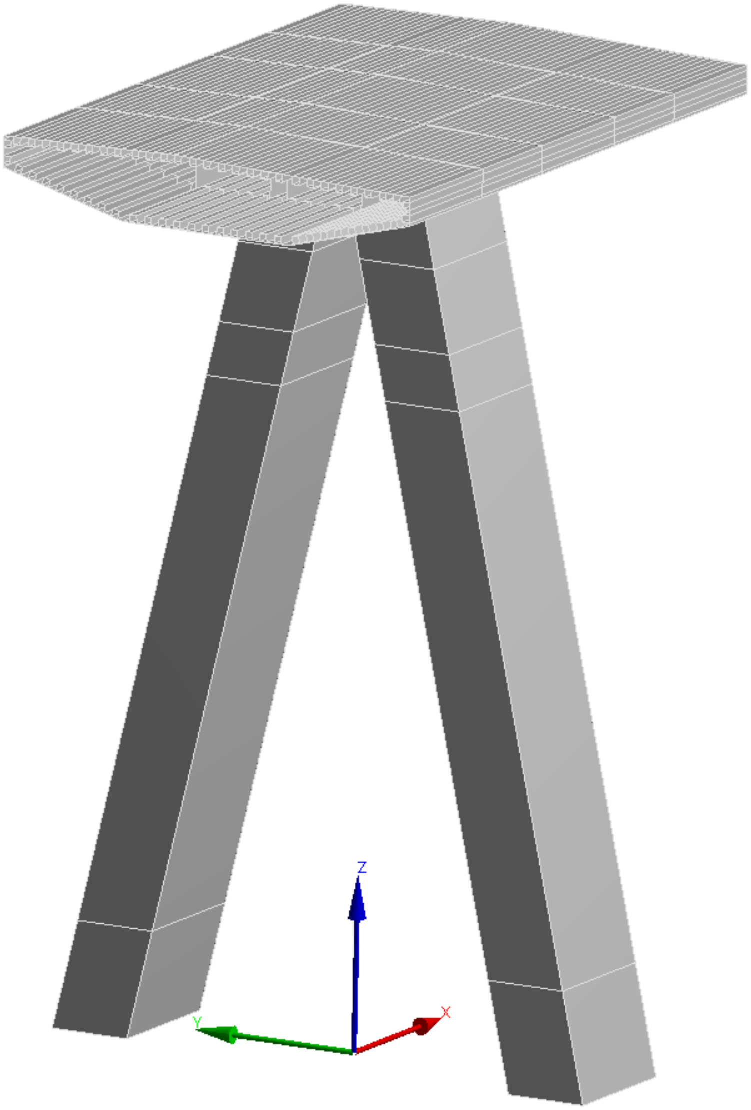


Figure 4: Local coordinate system for load application.

3 Results

3.1 Equivalent stress

In order to compare the resulting steel weight in the columns, the plate thicknesses along the columns were adjusted such that the equivalent stress along the two column types were similar in magnitude.

The colour scale in Figure and Figure is adjusted such that if the Von-Mises stress is above 400MPa, the colour becomes red. Apart from local concentrated stresses in the column tops, resulting from simplification of the model, it can be seen that the Von-Mises stress along the columns are adjusted such that they do not exceed 400MPa. The plate thickness along the columns was adjusted, such that the stress level in general is similar for the two models.

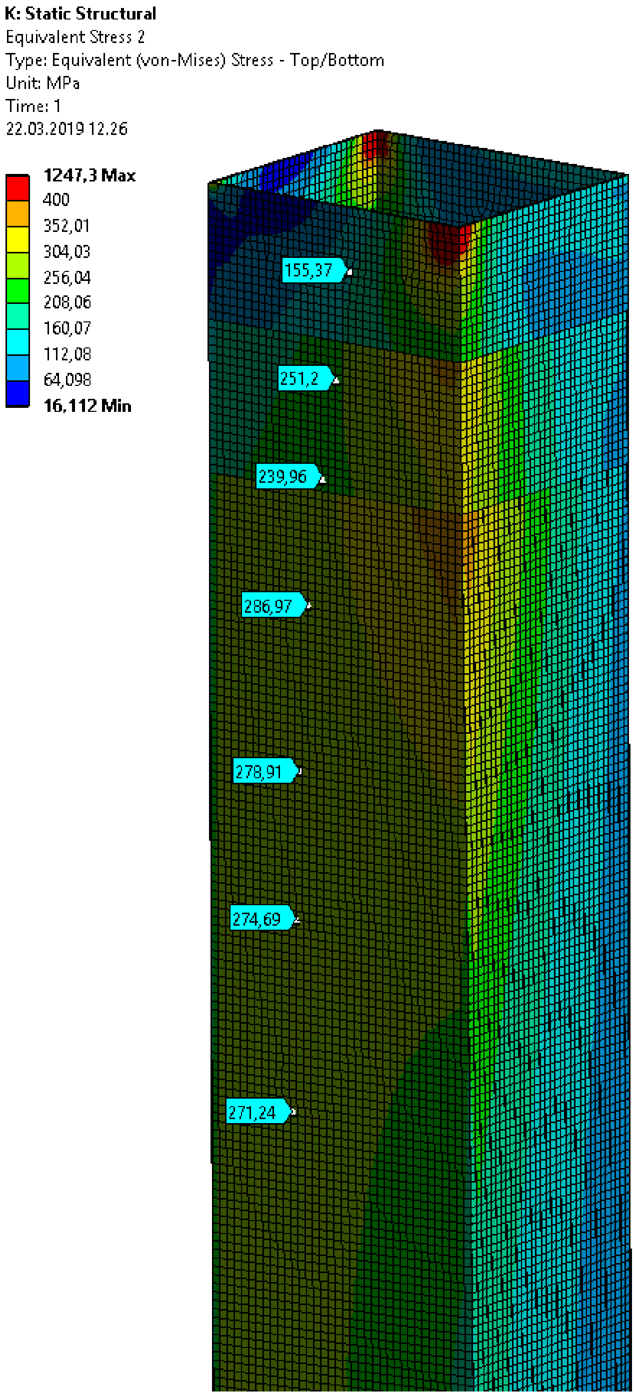


Figure 5: Equivalent stress, rectangular column.

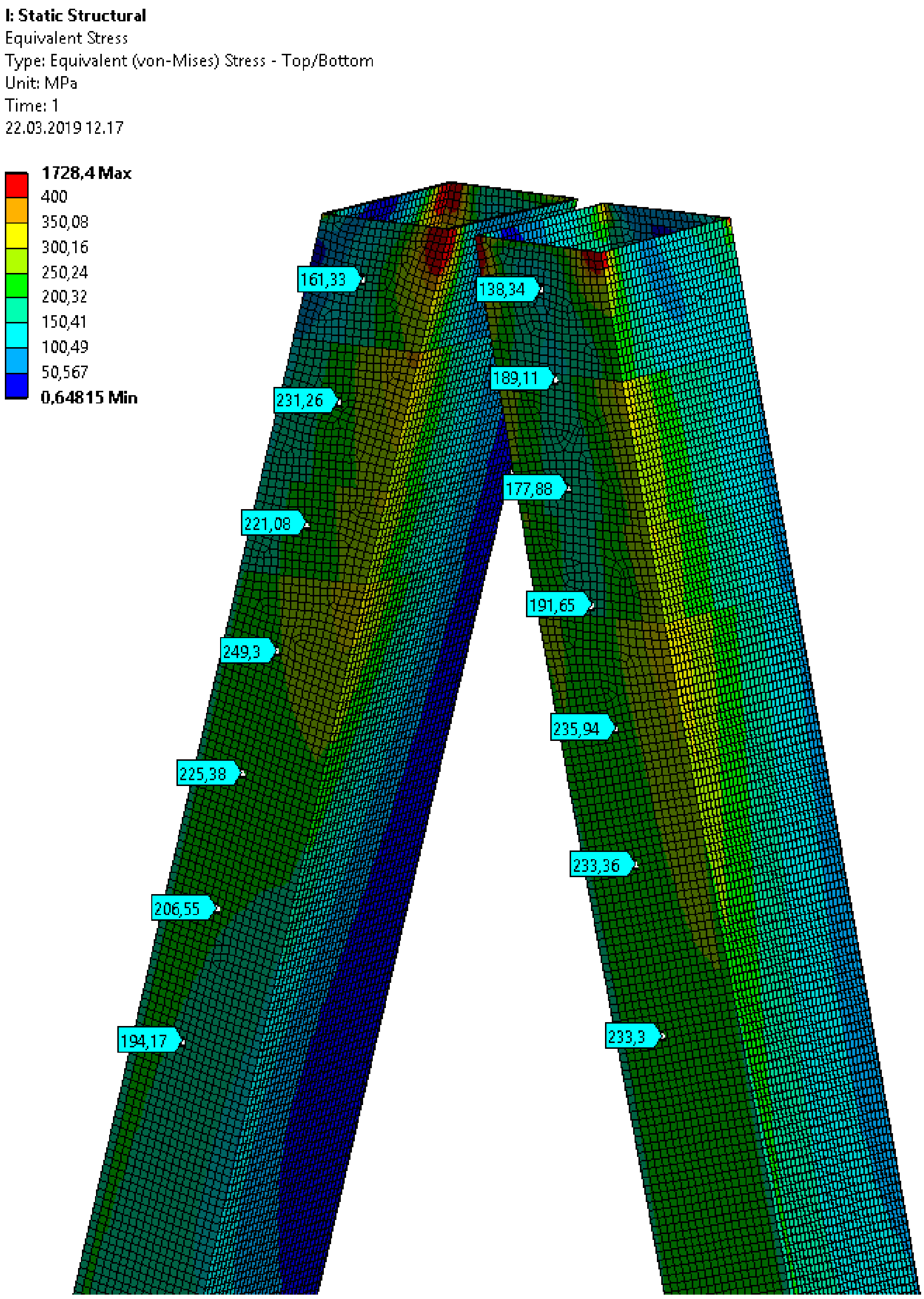


Figure 6: Equivalent stress, A-shaped column.

3.2 Equivalent plate thickness and weight

Table 1 shows that that the A-shaped column has a 47% higher weight than the rectangular column to obtain similar Von-Mises stress values. Although the model created for this investigation is a rather simplified one, the result should be indicative of the real situation.

4 Buckling Check

A buckling check was performed in STIPLA DNV-RP-C201 Version 2.2.1. The simplified buckling check is performed to show that buckling is something that can be handled in both designs.

4.1 Input

The membrane stress used as input is extracted from the Ansys results. First, the normal stress in the column is plotted along a path as illustrated in Figure 7. Then, at the point of maximum compressive stress, a path along the column width as seen in Figure 8 is created, and the normal stress along the plate width is plotted and used as input in the buckling analysis. As this is a peak value along the column, this should be highly conservative, as the compressive stress at other points along the column is either considerably lower or are even in tension.

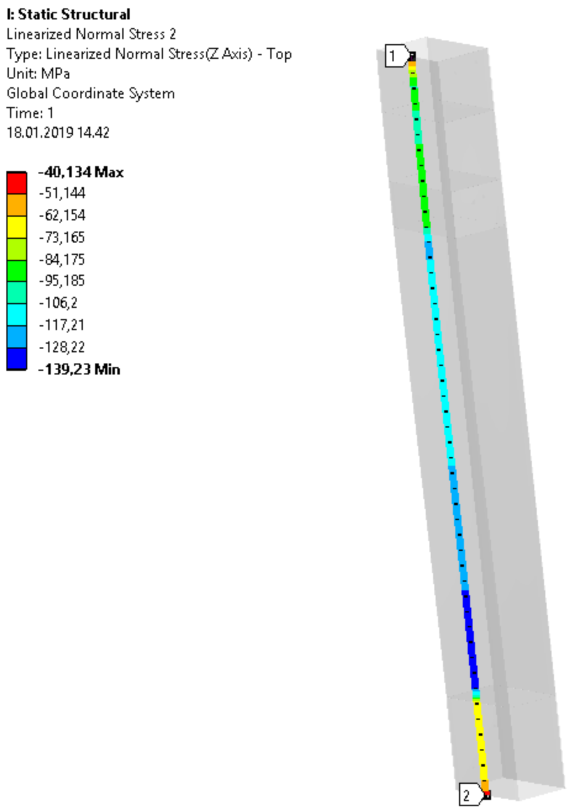


Figure 7: Normal stress along column.

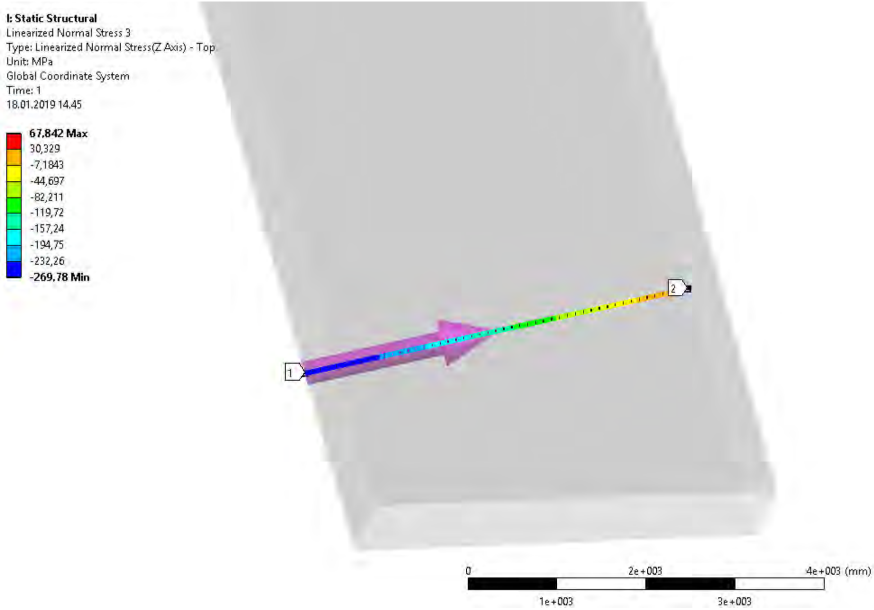


Figure 8: Normal stress along plate width.

The resulting normal stress along the plate width is illustrated in Figure 9 for one of the panels in the A-shaped column. The total normal stress used as input, is a superposition of the membrane stress and the bending stress along the plate section.

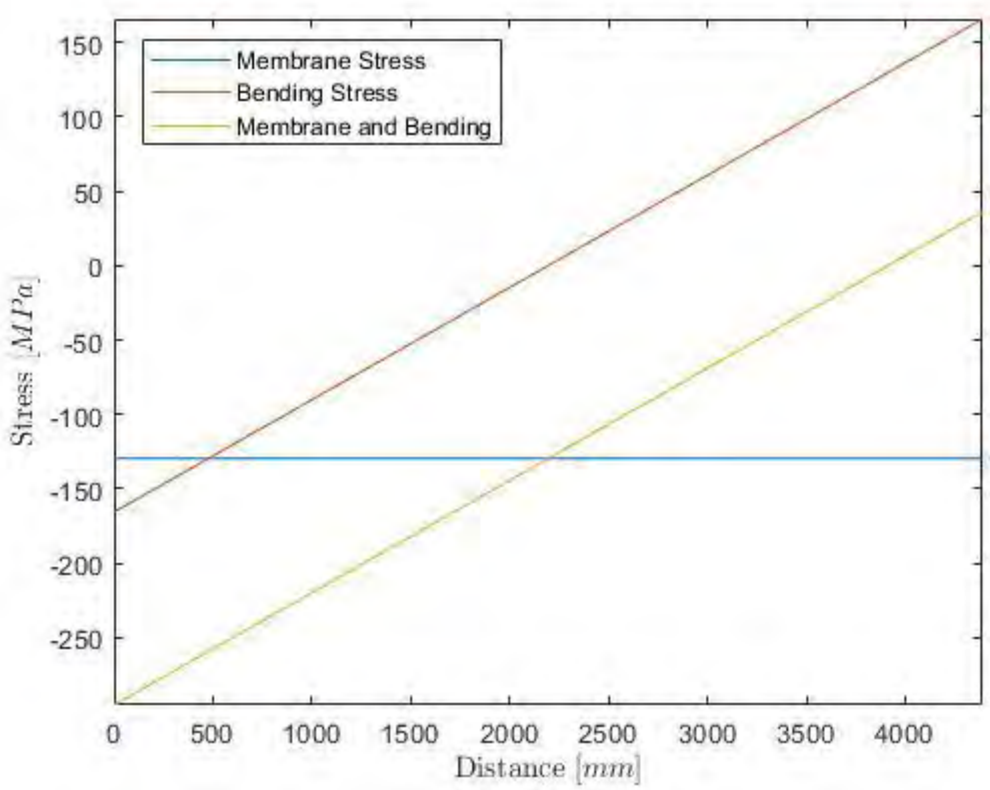


Figure 9: Normal stress along the plate width.

4.2 Results

Results of the buckling checks for the panels subjected to highest compressive stresses are seen below. As can be seen from the utilization ratios, this is not an optimized solution. However, it can be concluded from the check that buckling of the columns is a phenomenon that can be handled, and the differences between the concepts is not large.

4.2.1 Rectangular Column

D N V R P G Girder check based on DNV-RP-C201 Version 2.2.1 Copyright (C) 2004-2015 StruProg AB	Project: Bjørnafjorden Identification: Søyle 8m x 9m - 8m platefelt sjekket her - Ship impact eccentric - axis 3	Page: 1/1	Date: 18.01.2019 Time: 17:21
File: c:\01_prosjekt\04_bjørnafjorden\facing_ns_mono.drpg			

Material/Safety Format:

Plate/girder:			NV-420/NV-420
Yield stress	fyp/fyg	=	420/420 MPa
Youngs modulus	E	=	2,10E+5 MPa
Material Factor:	gm	=	1,00
Allowable Usage Factor:	UF	=	1,00

General:

Buckling length	Lk	=	8000 mm
Mom fact - Field	km2	=	24,0
- Support	km1	=	12,0

Continuous girder

Geometry:

Girder spacing	L1	=	4000 mm
	L2	=	4000 mm
Girder span	Lg	=	8000 mm
Length of panel	Lp	=	48030 mm
Lat tors buckl length	Lt	=	4000 mm
Stiffener spacing:	s	=	600 mm
Plate thickness	t	=	35,0 mm

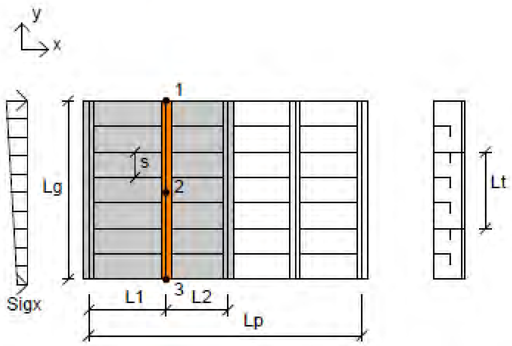
Stiffener: BF 200x10,0

Stiffener continuos through girder (Eq 8.4)

Stress/Force:

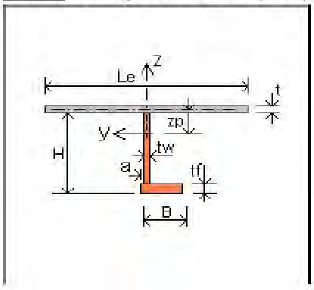
Sigx1	=	-378,0 MPa
Sigx3	=	-179,0 MPa
Sigy	=	0,0 MPa
Tau	=	0,0 MPa

Stiffened plate:



Stiffened plate effective against Sigy-stress (Method 1 ch 8.4.2)

Girder: Built-up: L 550x300x16,0x22,0



Local buckling of web taken into account according to Eurocode 3/NS3472

Girder property:

H	=	550 mm
B	=	300 mm
tw	=	16,0 mm
tf	=	22,0 mm
A	=	15048 mm2
g	=	118,1 kg/m
Iy	=	4,767E+8 mm4
Iz	=	1,242E+8 mm4

Girder incl. eff. plate:

zp	=	59,1 mm (elastic)
zp	=	3,0 mm (plastic)
Ae	=	1,017E+5 mm2
Ie	=	2,561E+9 mm4
Wep	=	4,172E+7 mm3
Weg	=	5,033E+6 mm3

Webclass: 1 M - Pl in compr.
Webclass: 3 M - Pl in tens.
Webclass: 4 N - Axial force
Flangeclass: 4

GIRDER BUCKLING CONTROL: (1 = Support, 2 = field g = girder, p = plate)

Le = 2495,4 / 2400,0 mm (buckling/bending, ref ch 8.4.2) Sigxsd = -328,3 MPa p0 = 0,084 MPa z* = -507,0 mm

UF1g=Nsd/NksRd-2*NSd/NRd+(M1Sd+NSd*z)/(Mst1Rd*(1-Nsd/Ne)) =
0,0/28904,3-2*0,0/42728,4+(1802,1+0,0*-0,507)/(2113,9*(1-0,0/82944,8)) = 0,85 < 1,00 (Eq 7.54)

UF1p=Nsd/NkpRd+(M1Sd+NSd*z)/(Mp1Rd*(1-Nsd/Ne))= 0,0/32438,7+(1802,1+0,0*-0,507)/(17523,1*(1-0,0/82944,8)) = 0,10 < 1,00 (Eq 7.55)

UF2g=Nsd/NksRd+(M2Sd-NSd*z)/(Ms2Rd*(1-Nsd/Ne)) = 0,0/28904,3+(901,0-0,0*-0,507)/(2113,9*(1-0,0/82944,8)) = 0,43 < 1,00 (Eq 7.56)

UF2p=Nsd/NkpRd-2*NSd/NRd+(M2Sd-NSd*z)/(Mp2Rd*(1-Nsd/Ne)) =
0,0/32438,7-2*0,0/42728,4+(901,0-0,0*-0,507)/(17523,1*(1-0,0/82944,8)) = 0,05 < 1,00 (Eq 7.57)

Recommended maximum distance between tripping brackets to avoid lateral torsional buckling = 6178 mm (Eq 8.31)

4.2.2 A-Shaped column

D N V R P G	Project: Bjørnafjorden	Page: 1/1
Girder check based on DNV-RP-C201 Version 2.2.1 Copyright (C) 2004-2015 StruProg AB	Identification: Søyle 8m x 4.5m - 8m platefelt sjekket her - Ship impact eccentric - axis 3	Date: 18.01.2019 Time: 17:20
File: c:\01_prosjekt\04_bjørnafjorden\facing_ns_a.drpg		

Material/Safety Format:

Plate/girder:			NV-420/NV-420
Yield stress	fyp/fyg	=	420/420 MPa
Youngs modulus	E	=	2,10E+5 MPa
Material Factor:	gm	=	1,00
Allowable Usage Factor:	UF	=	1,00

General:

Buckling length	Lk	=	8000 mm
Mom fact - Field	km2	=	24,0
- Support	km1	=	12,0

Continuous girder

Geometry:

Girder spacing	L1	=	4000 mm
	L2	=	4000 mm
Girder span	Lg	=	8000 mm
Length of panel	Lp	=	48030 mm
Lat tors buckl length	Lt	=	4000 mm
Stiffener spacing:	s	=	600 mm
Plate thickness	t	=	35,0 mm

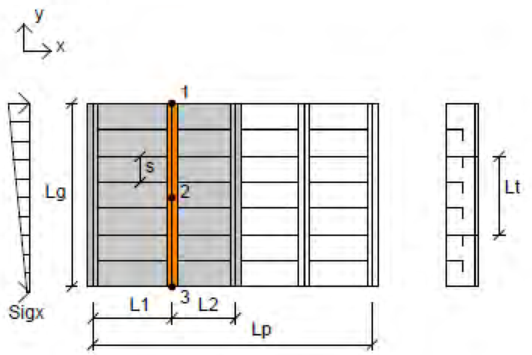
Stiffener: BF 200x10,0

Stiffener continuos through girder (Eq 8.4)

Stress/Force:

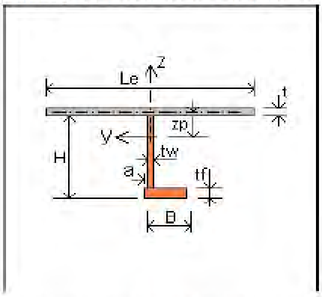
Sigx1	=	-279,0 MPa
Sigx3	=	-31,0 MPa
Sigy	=	0,0 MPa
Tau	=	0,0 MPa

Stiffened plate:



Stiffened plate effective against Sigy-stress (Method 1 ch 8.4.2)

Girder: Built-up: L 550x300x16,0x22,0



Local buckling of web taken into account according to Eurocode 3/NS3472

Girder property:

H	=	550 mm
B	=	300 mm
tw	=	16,0 mm
tf	=	22,0 mm
A	=	15048 mm2
g	=	118,1 kg/m
Iy	=	4,767E+8 mm4
Iz	=	1,242E+8 mm4

Girder incl. eff. plate:

zp	=	44,9 mm (elastic)
zp	=	2,2 mm (plastic)
Ae	=	1,343E+5 mm2
Ie	=	2,651E+9 mm4
Wep	=	4,172E+7 mm3
Weg	=	5,033E+6 mm3

Webclass: 1 M - Pl in compr.
Webclass: 3 M - Pl in tens.
Webclass: 4 N - Axial force
Flangeclass: 4

GIRDER BUCKLING CONTROL: (1 = Support, 2 = field g = girder, p = plate)

Le = 3424,7 / 2400,0 mm (buckling/bending, ref ch 8.4.2) Sigxsd = -217,0 MPa p0 = 0,056 MPa z* = -521,0 mm

UF1g=Nsd/NksRd-2*NSd/NRd+(M1Sd+NSd*z)/(Mst1Rd*(1-Nsd/Ne)) =
0,0/34149,8-2*0,0/56390,1+(1191,3+0,0*-0,521)/(2113,9*(1-0,0/85844,6)) = 0,56 < 1,00 (Eq 7.54)

UF1p=Nsd/NkpRd+(M1Sd+NSd*z)/(Mp1Rd*(1-Nsd/Ne))= 0,0/39746,9+(1191,3+0,0*-0,521)/(17523,1*(1-0,0/85844,6)) = 0,07 < 1,00 (Eq 7.55)

UF2g=Nsd/NksRd+(M2Sd-NSd*z)/(Ms2Rd*(1-Nsd/Ne)) = 0,0/34149,8+(595,7-0,0*-0,521)/(2113,9*(1-0,0/85844,6)) = 0,28 < 1,00 (Eq 7.56)

UF2p=Nsd/NkpRd-2*NSd/NRd+(M2Sd-NSd*z)/(Mp2Rd*(1-Nsd/Ne)) =
0,0/39746,9-2*0,0/56390,1+(595,7-0,0*-0,521)/(17523,1*(1-0,0/85844,6)) = 0,03 < 1,00 (Eq 7.57)

Recommended maximum distance between tripping brackets to avoid lateral torsional buckling = 6178 mm (Eq 8.31)

5 Conclusion

In this memo, FE-models of two different column designs was made. One, with a rectangular column, and one with an A-shaped column. The goal was to compare the difference in required steel mass to carry the load from an eccentric ship impact.

It is concluded that both column types can handle the ship impact, although with different levels of effectiveness.

It is concluded that the A-shaped column requires a significant increase in steel weight compared to the rectangular column. In this memo, the relative weight increase compared to the rectangular column was 47%. In addition, the A-shaped column leads to more complicated fabrication. Due to these observations, it is concluded in this memo that the rectangular column is clearly a preferable solution.

The increased weight for the A-shaped column arises from several reasons.

One reason is the inherent attribute of the geometry. Obtaining the same base area with two rectangular columns rather than one, requires two extra skin-plate panels, resulting in a weight increase.

Another reason is that the stresses induced in the columns from the ship impact is observed to not be distributed equally to the two legs of the A-shaped column. This means that splitting the 8m x 9m column into two separate 8m x 4.5m sections does in practice not yield the same sectional resistance. It is considered that this may lead to a less effective sectional resistance at the column top for the A-shaped column, considering the combined force components from the ship impact.

It is believed that the A-shaped column will complicate the fabrication and increase the fabrication cost. Connecting two separate column legs to the bridge girder and pontoon rather than one single column will lead to more welding. In addition, transferring the forces between the column and bridge girder through two connection points rather than one will require four longitudinal bulkheads rather than two as for the rectangular column, increasing the steel weight in the bridge girder for all column/support sections along the bridge.

Concept development, floating bridge
E39 Bjørnafjorden

Appendix K – Enclosure 2

10205546-13-NOT-082
Railings on Bridge Girder

MEMO

PROJECT	**Concept development, floating bridge E39 Bjørnafjorden**	DOCUMENT CODE	10205546-13-NOT-082
CLIENT	Statens vegvesen	ACCESSIBILITY	Restricted
SUBJECT	**AMC status 2 - Railings on Bridge Girder**	PROJECT MANAGER	Svein Erik Jakobsen
TO	Statens vegvesen	PREPARED BY	Dag Ivar Ytreberg
COPY TO		RESPONSIBLE UNIT	AMC

SUMMARY

The note summarizes requirements for traffic- and foot path/bicycle trail railings - and choose which railings may be relevant to use in the design. The following types of railings are relevant for the Bridge Girder:

- H4 traffic railing with W-class W2 or better (only on the High Bridge)
- H2 traffic railing med W-class W2 or better (only on the Low Bridge)
- H1 central traffic railing with W-class W4 or better
- Foot path/bicycle trail railing

0	29.03.2019	Status 2 issue	D.I. Ytreberg	P.N. Larsen	S.E. Jakobsen
REV.	DATE	DESCRIPTION	PREPARED BY	CHECKED BY	APPROVED BY

1 Introduction

The Bridge Girder needs to be equipped with different kinds of traffic- and foot path/bicycle trail railings according to specific rules in the Design Basis and in SVV handbook N101 - over the entire bridge length.

A typical section in the bridge Girder are shown below.

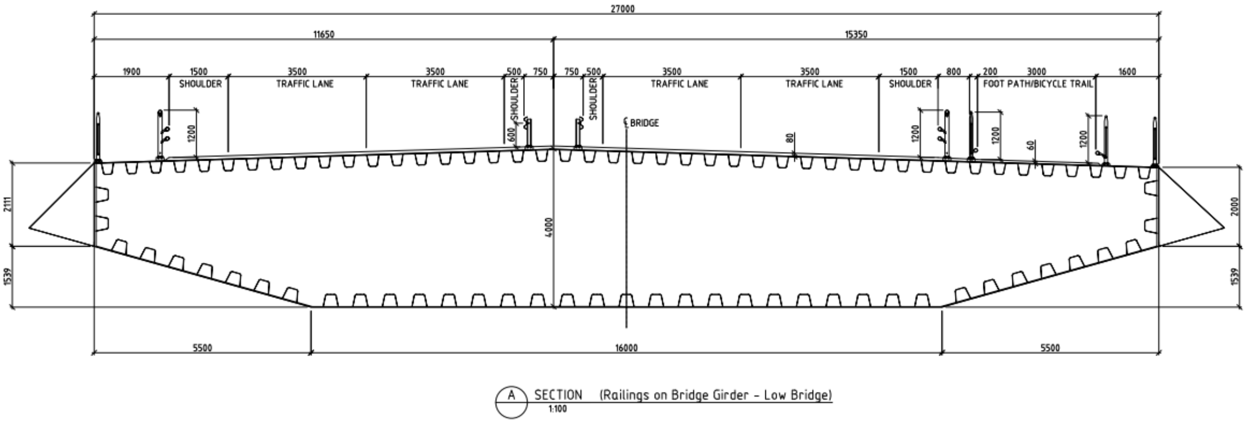


Figure 1-1Typical section of Bridge Girder with traffic lines, other parts of the bridge deck and location of the Railings

2 Railing requirements

The requirements are given in Bjørnafjorden Design Basis and in SVV hand- book N101.

2.1 Design Basis

4 FUNCTIONAL CRITERIA

4.1 Roadway

The partitioning of bridge deck due to traffic related functions is shown in Figure 5-1.

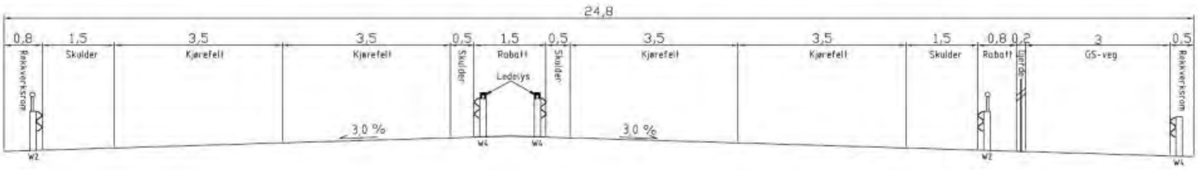


Figure 4-1 Bridge deck sectioning principles

4.2 Guard rails

Guard rails are type W2 and W4 s shown in Figure 5-1, ref. [2].

Figure 2-1 Extract from Design Basis

The railings shall have working width W2 or W4 as shown in figure above.

W-klasse	W1	W2	W3	W4
Arbeidsbredde (m)	≤ 0,6	≤ 0,8	≤ 1,0	≤ 1,3
VI-klasse	VI1	VI2	VI3	VI4
Inntrengning (m)	≤ 0,6	≤ 0,8	≤ 1,0	≤ 1,3

Figure 2-2Extract from SVV handbook N101

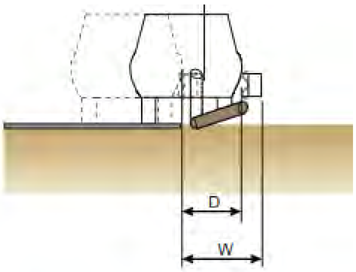


Figure 2-3 Extract from SVV handbook N101

2.2 Public Road Administration handbook N101:

All railings against the traffic lanes:

The railings shall be in damage class A or B where A is the best class with respect to injury of passengers.

Outer traffic railing:

The height of the railing shall be 1200 mm above asphalt top surface in accordance with section 3.4.3. Free openings in the railing must be max. 400 mm when road line is prohibited for pedestrians.

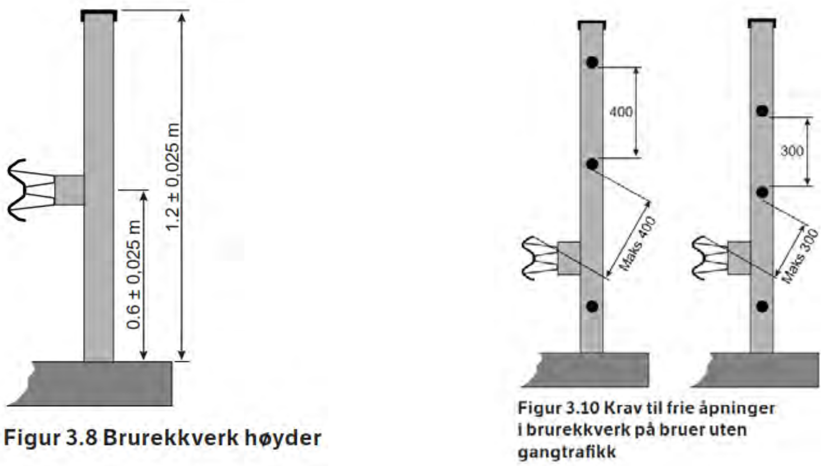


Figure 2-4 Extract from SVV handbook N101

The railing shall have a strength class H2/L2 on the floating bridge and H4/L4 on the cable-stayed bridge. This according to Table 3.1 in section 3.2.2. The cable-stayed bridge requires an increased strength class on the railing because collisions that hit the cables can cause serious damage to load bearing constructions. This is not the case on the floating bridge.

Central traffic railings between traffic lines:

The height of the railings should be such that the guard rails can be placed at a height of 600 mm above asphalt top surface. The railing shall have strength class H1 for the entire bridge according to Table 3.1 in section 3.2.2 as it is assumed that the proportion of heavy vehicles (> 10 tons) is below 20%.

Inner traffic railing - between traffic lines and guide fence for foot path/bicycle trail:

The height of the railing must be such that the guard rails can be placed at a height of 600 mm (measured at the center of the rail) above asphalt top surface.

The railing shall have a strength class H2/L2 on the floating part and H4/L4 on the cable-stayed bridge. We chose to use the same railing as for the outer traffic railing. This is because there are only concrete railings approved in Public Road Administration list for approved traffic railings with the appropriate W-class and height.

Guide fence between inner traffic railing and foot path/bicycle trail:

There is no special requirement for such a railing in the handbook N101, but we choose to use the same railing as for the outer railing for the foot path/bicycle trail specified below. The railing should also have a guide rail for snow shoveling.

Outer railing for the foot path/bicycle trail:

Normal foot path railing with height 1.2 m above asphalt top surface.

The railing should also have a guide rail for snow shoveling.

Extra outer railing on both edges of the bridge girder:

As stated in chapter 3.4.3, the distance from the outer side of the railing to the outer edge of the bridge must be max. 200 mm to reduce the climbing opportunity on the outside of the railing.

Where the width of the bridge girder is greater than 200 mm the requirement for climbing ability is not satisfied and there must be extra railings on both edges of the bridge girder. There are no special requirements for this railing in the handbooks. Therefore, we choose to use the same railing as for outer railings on the foot path/bicycle trail as indicated above, but without guide rail for snow shoveling.

Railings that are approved by the Public Road Administration (SVV) and may be relevant to use in the design:

- As an H4 traffic railing with W-class W2 or better, there is only one approved railing in Public Road Administration list for approved traffic railings. This is “PASS+CO” H4B-W1. https://www.vegvesen.no/fag/teknologi/Rekkverk+og+master/Sok+etter+godkjent+produkt/Vegutstyr?key=1712798&method=avansert&produkttype=12621

- As an H2 traffic railing med W-class W2 or better we choose the “Safeline Parapet” from VikØrsta H2-W2.
  https://www.vegvesen.no/fag/teknologi/Rekkverk+og+master/Sok+etter+godkjent+produkt/Vegutstyr?key=508598&method=alle&produkttype=12621

- As a H1 central traffic railing with W-class W4 or better we choose “Vik-EP” from VikØrsta H1-W3.
  https://www.vegvesen.no/fag/teknologi/Rekkverk+og+master/Sok+etter+godkjent+produkt/Vegutstyr?key=2135775&method=avansert&produkttype=12621

- As a foot path/bicycle trail railing we choose “Vikafjell gang- og sykkelrekkverk” from VikØrsta.
  https://www.vikorsta.no/globalassets/vik-orsta/trafikk/gs/vikafjell_gang_og_sykkelrekkverk_hq.pdf

Concept development, floating bridge
E39 Bjørnafjorden

Appendix K – Enclosure 3

10205546-13-NOT-083
Transverse trusses in bridge girder

MEMO

PROJECT	Concept development, floating bridge E39 Bjørnafjorden	DOCUMENT CODE	10205546-13-NOT-083
CLIENT	Statens vegvesen	ACCESSIBILITY	Restricted
SUBJECT	Transverse trusses in bridge girder	PROJECT MANAGER	Svein Erik Jakobsen
TO	Statens vegvesen	PREPARED BY	Dag Ivar Ytreberg / Anne Kristine Lunke / Emilie Marley
COPY TO		RESPONSIBLE UNIT	AMC

SUMMARY

This memo summarizes the design of the transverse trusses in the bridge girder. Two different types of trusses are used in the design, and the type of truss is related to the width of the bridge columns.

The transverse trusses also provide a rigid support for the longitudinal stiffened plates and help to maintain the shape of the steel box. It is shown that the transverse trusses satisfy the design criteria given in section 9 of NS-EN 1993-1-5:2006 + NA:2009.

1	24.05.2019	Final issue	A.K. Lunke/E. Marley	P. N. Larsen	S. E. Jakobsen
0	29.03.2019	AMC status 2 issue	D. I. Ytreberg	P. N. Larsen	S. E. Jakobsen
REV.	DATE	DESCRIPTION	PREPARED BY	CHECKED BY	APPROVED BY

1 Introduction

The bridge girder has transverse trusses or transverse bulkheads with of approximately 4.0 m. In this memo, the design of the transverse trusses is explained.

It is shown that the transverse trusses and bulkheads can carry the dead loads and traffic loads from the orthotropic deck plate out to the webs in the box girder.

The transverse trusses also provide a rigid support for the longitudinal stiffened plates and help to maintain the shape of the steel box. It is shown that the transverse trusses satisfy the design criteria given in section 9 of NS-EN 1993-1-5:2006 + NA:2009.

Two different types of trusses are used in the design, and the type of truss is related to the width of the bridge columns. On the lower part of the bridge girder, from axis 9, the bridge columns are 7200 mm wide at the connection towards the bridge girder. The design of these trusses is as shown in Figure 1-1. For the higher part of the bridge girder, from axis 3 to 8, the bridge columns are 9600 mm wide and the design of the truss is as shown in Figure 1-2.

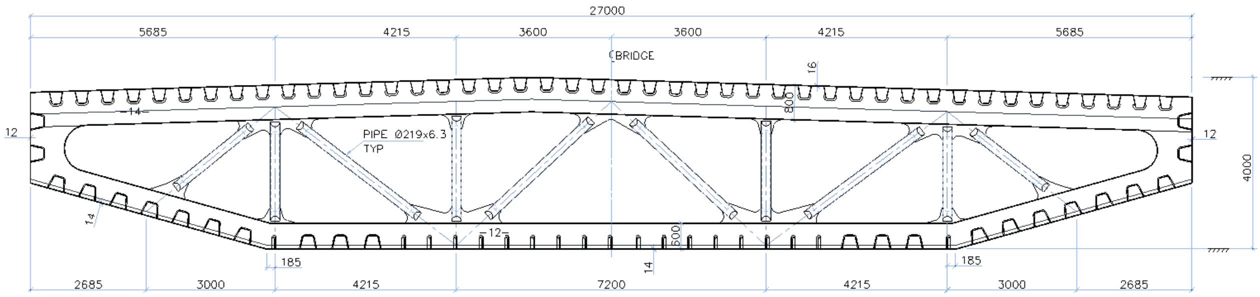


Figure 1-1 Typical transverse truss from axis 9, column width of 7200 mm

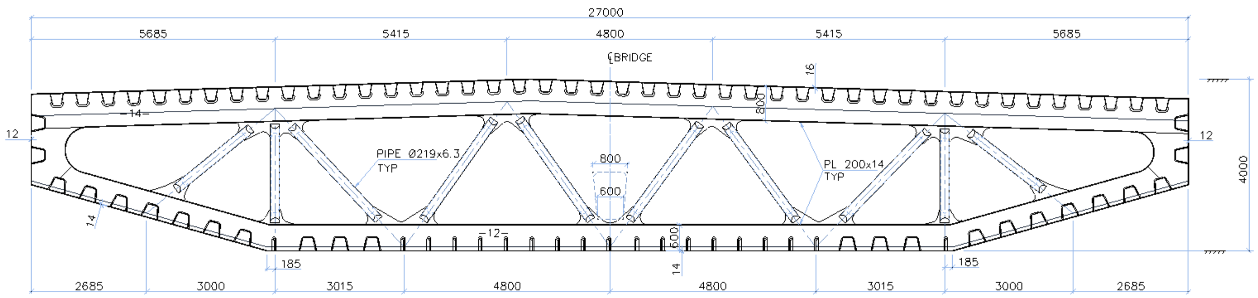


Figure 1-2 Typical transverse truss for axis 3 - 8, column width of 9600 mm

2 Design requirements for manhole in SVV handbooks

According to SVV handbook N400, the outline of a manhole shall pass the truss as shown below.

With t < 200 mm the minimum values shall be as follows: h_{min} = 1400 mm and k_{min} = 400 mm.

4.6.2 Dører og mannhull i vertikale flater

Krav til minimum høyde h fra overkant bunnplate, oppbygd trappetrinn eller repos til overkant åpning skal være som angitt i tabell 4.1. Kravet avhenger av konstruksjonsdelens tykkelse (t).

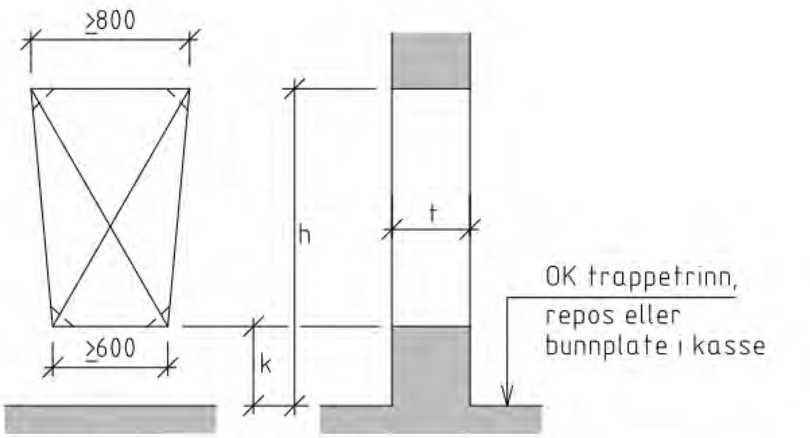
t	< 1000	1000 < 2000	2000 < 4000	≥ 4000
h	≥ 1400	≥ 1600	≥ 1800	≥ 2000

Tabell 4.1: Høydekrav for dører og mannhull i vertikale flater. Alle mål i mm.

Dører skal ha åpning med bredde ≥ 800 mm.

Mannhull skal ha minimum fri bredde i henhold til figur 4.12. For åpning med minimumsmål skal vouter ikke være større enn 100 x 100 mm, alternativt avrundet med radius R ≤ 200 mm.

Terskelhøyde (*k*) målt fra overkant bunnplate, trappetrinn eller repos skal være:
- for veggtykkelse t ≤ 200 mm skal *k* ≤ 400 mm
- for veggtykkelse t > 200 mm skal *k* ≤ 200 mm



Figur 4.12: Typiske mål for dører og mannhull i vertikale flater. Breddereduksjon i åpningens bunn gjelder ikke dører, kun for mannhull.

For tverrskott, tverrbærere osv. oppbygd som fagverk skal mannhullets minstekontur kunne passere gjennom fagverket uten konflikt.

Figure 2-1 Extract from handbook N400

The outline for the manhole is shown in figures below where the walkway is situated on a 30 mm grating placed on top of the bottom plate bulb stiffeners as shown on the figures below.

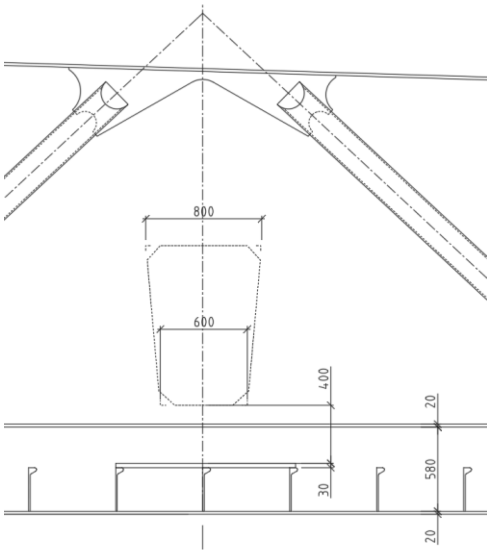


Figure 2-2 Outline for manhole in Transverse truss for 7200 mm column

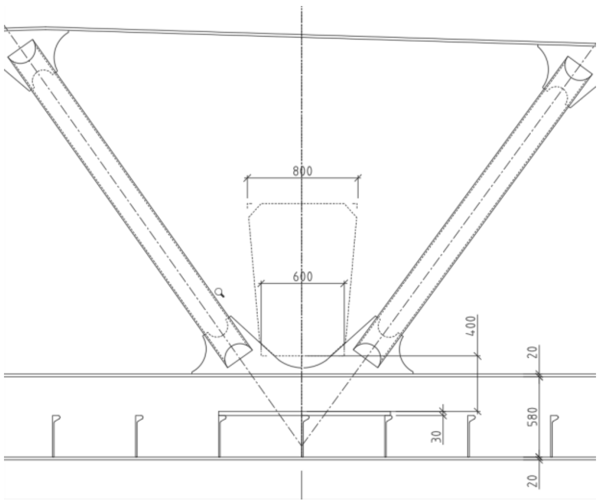


Figure 2-3 Outline for manhole in Transverse truss for 9600 mm column

3 Analysis of Transverse Trusses

3.1 General

The transverse trusses are analyzed with use of the FEM-program Staad.Pro V8i.

Two different analyses are performed, one for the transverse trusses from axis 9, with a column width of 7200 mm and one for the transverse trusses for axis 3 – 8, with a column width of 9600 mm.

The Staad-models are built up as frame models with beam elements. The size of the truss elements is ø219x6.3. The flanges are modelled as unsymmetrical wide flange-profiles where the outer skin of the bridge girder is included as a flange.

The upper deck plate of the bridge girder is in compression at midspan, while the lower skin plate is in compression above the bridge columns. In the calculations of effective flange width, it is conservatively assumed that both the upper and lower skin plates are in compression. This reduces the effective flange width to 371 mm for the upper beam and 459 mm for the lower beam. The effective flange width of the side beam is 326 mm.

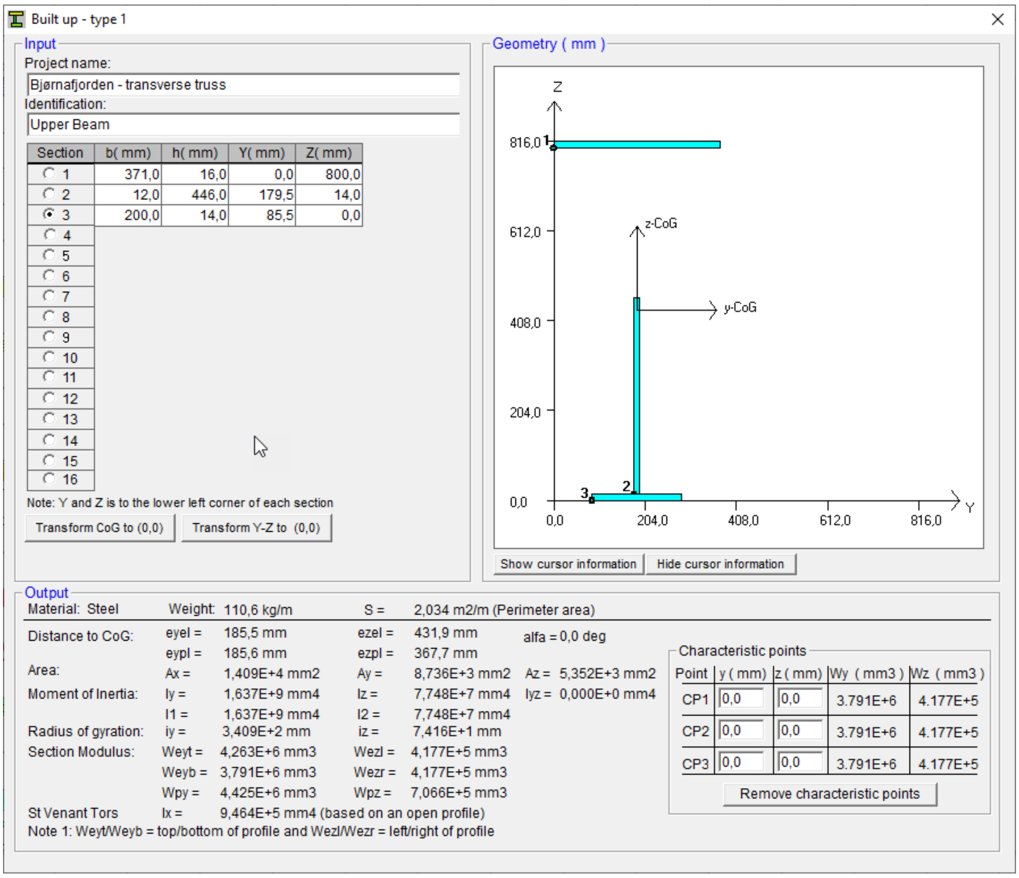
The flange width is taken as the minimum value based on sections 4.4 and 9.1 of NS-EN 1993-1-5, as shown in the calculations below.

Effective flange width according to NS-EN 1993-1-5:2006+NA:2009

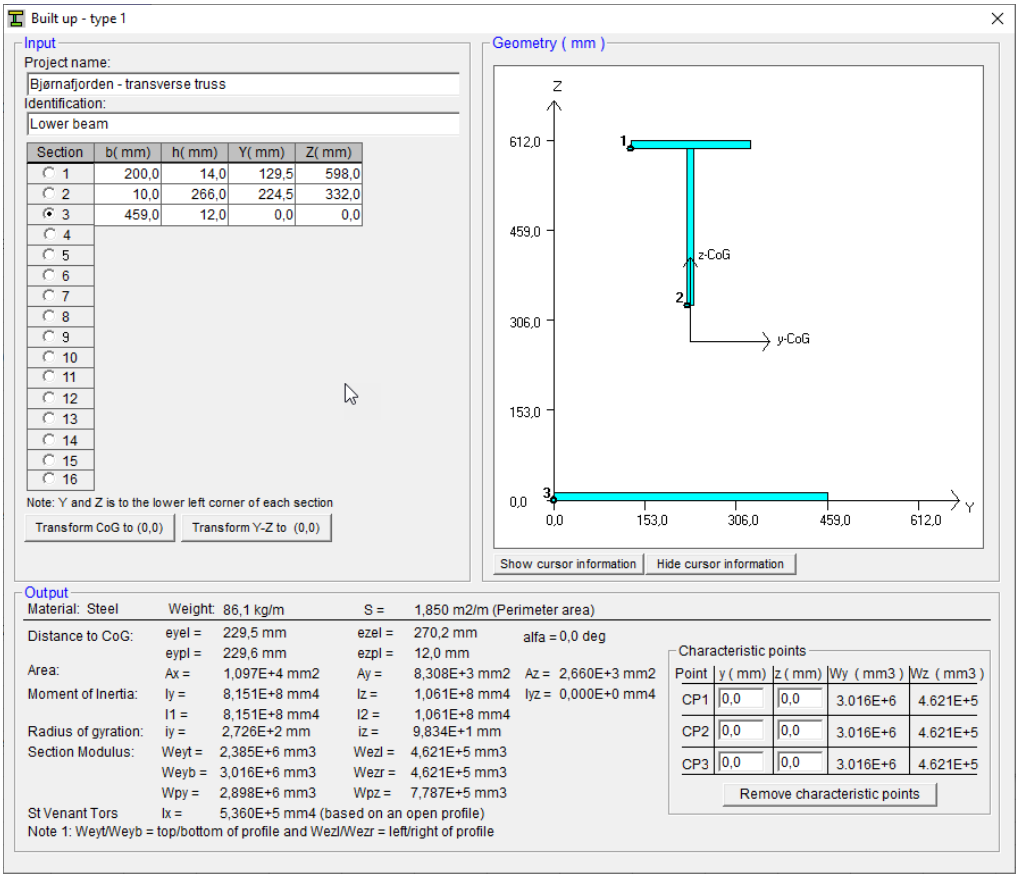
$f_y := 420 \cdot MPa$	yield stress
$b := 4.0 \cdot m$	plate width
$\varepsilon_1 := \sqrt{\frac{235MPa}{f_y}}$	yield stress factor
$k_\sigma := 4$	plate buckling factor
$\psi := 1$	stress ratio factor

Upper beam	**Lower beam**	**Side beam**	
$t_u := 16 \cdot mm$	$t_l := 20 \cdot mm$ *when in compression	$t_s := 14 \cdot mm$	plate thickness
$t_{wu} := 12mm$	$t_{wl} := 10mm$	$t_{ws} := 12mm$	web thickness of transverse stiffener
Max flange width according to section 4.4			
$\lambda_{pu} := \frac{\frac{b}{t_u}}{28.4 \cdot \varepsilon_1 \cdot \sqrt{k_\sigma}} = 5.884$	$\lambda_{pl} := \frac{\frac{b}{t_l}}{28.4 \cdot \varepsilon_1 \cdot \sqrt{k_\sigma}} = 4.707$	$\lambda_{ps} := \frac{\frac{b}{t_s}}{28.4 \cdot \varepsilon_1 \cdot \sqrt{k_\sigma}} = 6.725$	plate buckling slenderness
$\rho_u := \frac{\lambda_{pu} - 0.055 \cdot (3 + \psi)}{\lambda_{pu}^2} = 0.164$	$\rho_l := \frac{\lambda_{pl} - 0.055 \cdot (3 + \psi)}{\lambda_{pl}^2} = 0.203$	$\rho_s := \frac{\lambda_{ps} - 0.055 \cdot (3 + \psi)}{\lambda_{ps}^2} = 0.144$	reduction factor
$b_{ue4.4} := b \cdot \rho_u = 0.654\,m$	$b_{le4.4} := b \cdot \rho_l = 0.81\,m$	$b_{se4.4} := b \cdot \rho_s = 0.575\,m$	effective plate width due to buckling
Max flange width according to section 9.1			
$b_{ue9.1} := 2 \cdot 15 \cdot \varepsilon_1 \cdot t_u + t_{wu} = 0.371\,m$	$b_{le9.1} := 2 \cdot 15 \cdot \varepsilon_1 \cdot t_l + t_{wl} = 0.459\,m$	$b_{se9.1} := 2 \cdot 15 \cdot \varepsilon_1 \cdot t_s + t_{ws} = 0.326\,m$	max plate width
Flange width for calculations			
$b_{ue} := \min(b_{ue4.4}, b_{ue9.1}) = 0.371\,m$	$b_{le} := \min(b_{le4.4}, b_{le9.1}) = 0.459\,m$	$b_{se} := \min(b_{se4.4}, b_{se9.1}) = 0.326\,m$	effective plate width for calculations

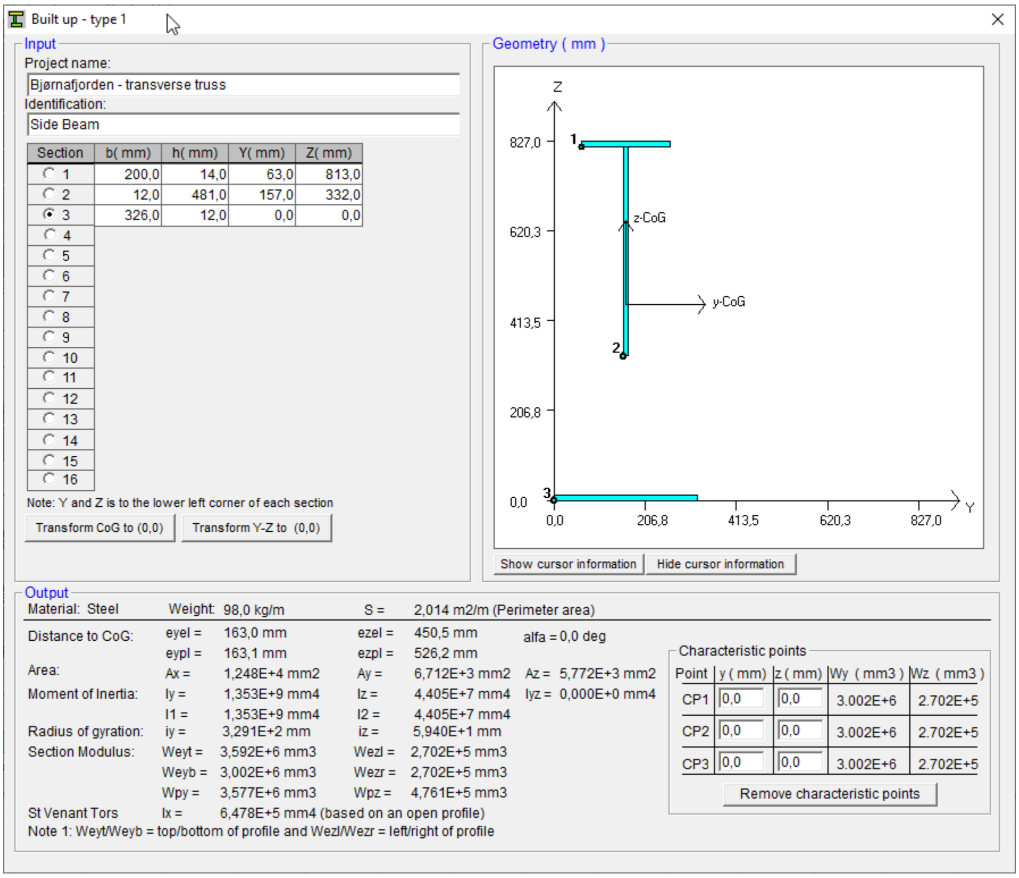
The following profile for the upper beam is used in the Staad-analysis:


Built up - type 1
Input
Project name:
Bjørnafjorden - transverse truss
Identification:
Upper Beam
Section | b(mm) | h(mm) | Y(mm) | Z(mm)
1 | 371,0 | 16,0 | 0,0 | 800,0
2 | 12,0 | 446,0 | 179,5 | 14,0
3 | 200,0 | 14,0 | 85,5 | 0,0
4 5 6 7 8 9 10 11 12 13 14 15 16
Note: Y and Z is to the lower left corner of each section
Transform CoG to (0,0)
Transform Y-Z to (0,0)
Geometry (mm)
Z
816,0
612,0
408,0
204,0
0,0
0,0 204,0 408,0 612,0 816,0
Y
z-CoG
y-CoG
1
2
3
Show cursor information
Hide cursor information
Output
Material: Steel Weight: 110,6 kg/m S = 2,034 m2/m (Perimeter area)
Distance to CoG: eyel = 185,5 mm ezel = 431,9 mm alfa = 0,0 deg
eypl = 185,6 mm ezpl = 367,7 mm
Area: Ax = 1,409E+4 mm2 Ay = 8,736E+3 mm2 Az = 5,352E+3 mm2
Moment of Inertia: Iy = 1,637E+9 mm4 Iz = 7,748E+7 mm4 Iyz = 0,000E+0 mm4
I1 = 1,637E+9 mm4 I2 = 7,748E+7 mm4
Radius of gyration: iy = 3,409E+2 mm iz = 7,416E+1 mm
Section Modulus: Weyt = 4,263E+6 mm3 Wezl = 4,177E+5 mm3
Weyb = 3,791E+6 mm3 Wezr = 4,177E+5 mm3
Wpy = 4,425E+6 mm3 Wpz = 7,066E+5 mm3
St Venant Tors Ix = 9,464E+5 mm4 (based on an open profile)
Note 1: Weyt/Weyb = top/bottom of profile and Wezl/Wezr = left/right of profile
Characteristic points
Point | y (mm) | z (mm) | Wy (mm3) | Wz (mm3)
CP1 | 0,0 | 0,0 | 3.791E+6 | 4.177E+5
CP2 | 0,0 | 0,0 | 3.791E+6 | 4.177E+5
CP3 | 0,0 | 0,0 | 3.791E+6 | 4.177E+5
Remove characteristic points

The following profile for the lower beam is used in the Staad-analysis:



The following profile for the side beam is used in the Staad-analysis:



3.2 Loads

Only dead loads of the bridge girder and traffic loads on the orthotropic deck plate are included.

3.2.1 Dead loads

Self-weight of the steel members is automatically calculated in the Staad-analysis. Additional dead loads of the steel which is not included in the model is applied as calculated on the following pages.

Upper flange of truss

$$A_{steel} := \frac{600mm \cdot 16mm + (290mm \cdot 2 + 185mm) \cdot 8mm}{600mm} = 0.026\,m$$ area of stiffener and deck plate per m

$$L_{span} := 4.0 \cdot m$$ deck plate span

$$A_{flange} := 371mm \cdot 16mm = 5.936 \times 10^{-3}\,m^2$$ area of flange included in Staad-analysis

$$g_{steel} := 77\frac{kN}{m^3}$$ weight of steel

$$q_{steel} := \left(A_{steel} \cdot L_{span} - A_{flange}\right) \cdot g_{steel} = 7.6 \cdot \frac{kN}{m}$$ dead weight of steel per m

$$q_{asphalt} := 2.0 \cdot \frac{kN}{m^2} \cdot L_{span} = 8.0 \cdot \frac{kN}{m}$$ weight of asphalt according to hb.400

$$q_{dead.weight} := q_{steel} + q_{asphalt} = 15.6 \cdot \frac{kN}{m}$$ dead weight on upper flange

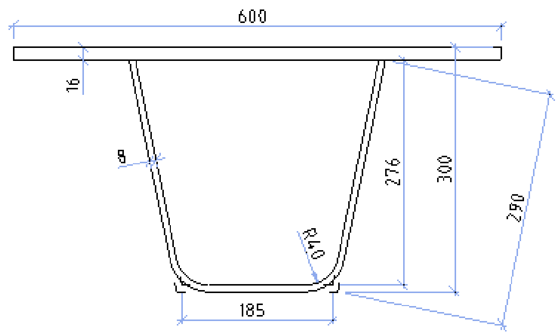


Figure 3-1 Upper flange with stiffeners cc 600 mm

Lower flange of truss

$A_{steel} := \dfrac{750mm \cdot 12mm + (314mm \cdot 2 + 230mm) \cdot 6mm}{750mm} = 0.019\,m$ area of stiffener and deck plate per m

$L_{span} := 4.0 \cdot m$ deck plate span

$A_{flange} := 459mm \cdot 12mm = 5.508 \times 10^{-3}\,m^2$ area of flange included in Staad-analysis

$g_{steel} := 77 \dfrac{kN}{m^3}$ weight of steel

$q_{steel} := \left(A_{steel} \cdot L_{span} - A_{flange}\right) \cdot g_{steel} = 5.4 \cdot \dfrac{kN}{m}$ dead weight of steel per m

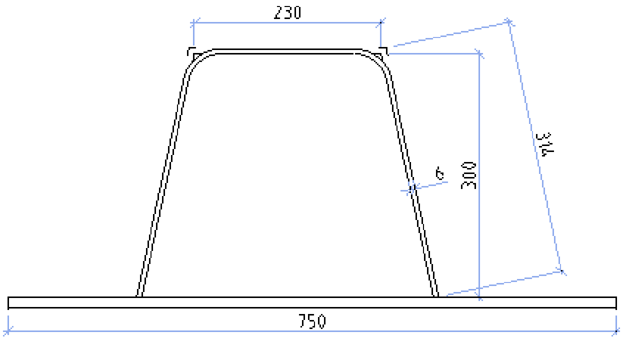


Figure 3-2 Lower flange with stiffeners cc 750 mm

Side beam of truss

$A_{steel} := \dfrac{750mm \cdot 12mm + (314mm \cdot 2 + 230mm) \cdot 6mm}{750mm} = 0.019\,m$ area of stiffener and deck plate per m

$L_{span} := 4.0 \cdot m$ deck plate span

$A_{flange} := 326mm \cdot 12mm = 3.912 \times 10^{-3}\,m^2$ area of flange included in Staad-analysis

$g_{steel} := 77 \dfrac{kN}{m^3}$ weight of steel

$q_{steel} := \left(A_{steel} \cdot L_{span} - A_{flange}\right) \cdot g_{steel} = 5.5 \cdot \dfrac{kN}{m}$ dead weight of steel per m

Transverse trusses in bridge girder

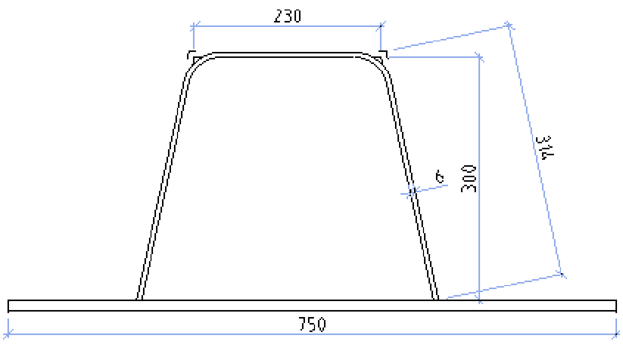


Figure 3-3 Web plate with stiffeners cc 750 mm

3.2.2 Traffic loads

The traffic loads are applied according to NS-EN 1991-2, load model 1.

Traffic loads for load model 1
According to NS-EN 1991-2:2003+NA:2010, section 4.3.2

Point loads

$P_1 := 2 \cdot \frac{300kN}{2} = 300 \cdot kN$ boggie load in line 1

$P_2 := 2 \cdot \frac{200kN}{2} = 200 \cdot kN$ boggie load in line 2

$P_3 := 2 \cdot \frac{100kN}{2} = 100 \cdot kN$ boggie load in line 3

Distrubuted loads

$q_1 := 9 \frac{kN}{m^2} \cdot 0.6 \cdot L_{span} = 21.6 \cdot \frac{kN}{m}$ load in line 1

$q_2 := 2.5 \frac{kN}{m^2} \cdot L_{span} = 10 \cdot \frac{kN}{m}$ load in line 2, 3, 4, foot path and remaining area

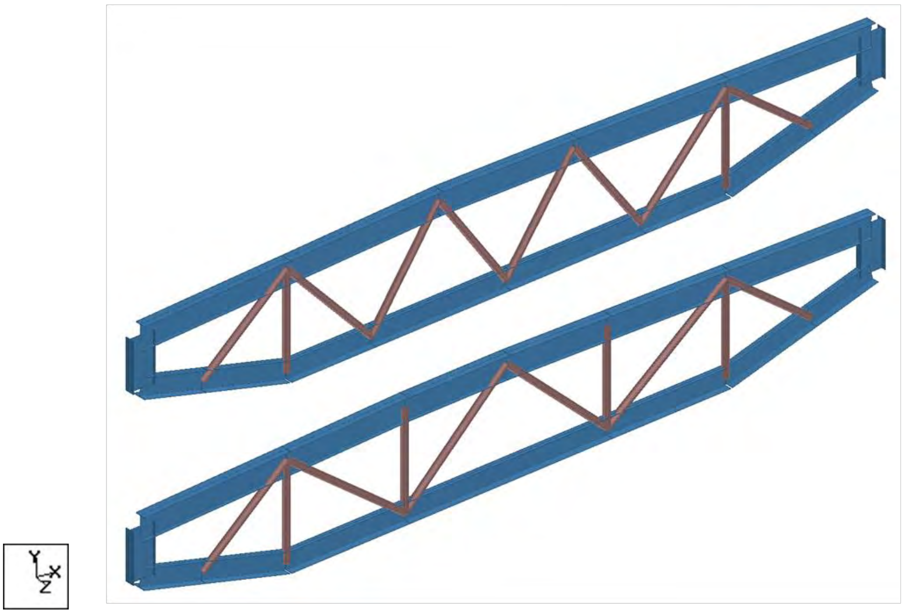
3.2.3 ULS load combination

1.2 x dead load + 1.35 x traffic load

3.3 Staad-analysis for Transverse Trusses

3.3.1 Model

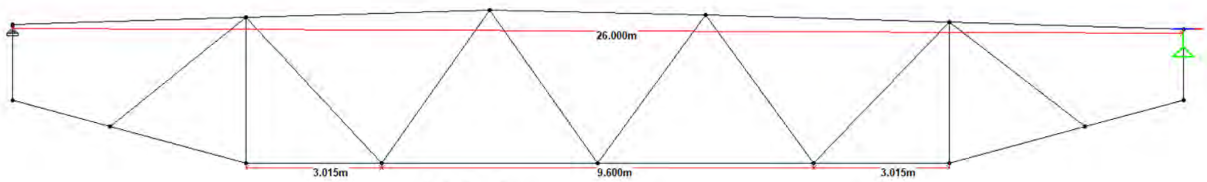
Span length for the frame is assumed to be between centre lines for the wide flange-profiles following the outer skin of the steel box. The frame is simply supported at each end as shown below.



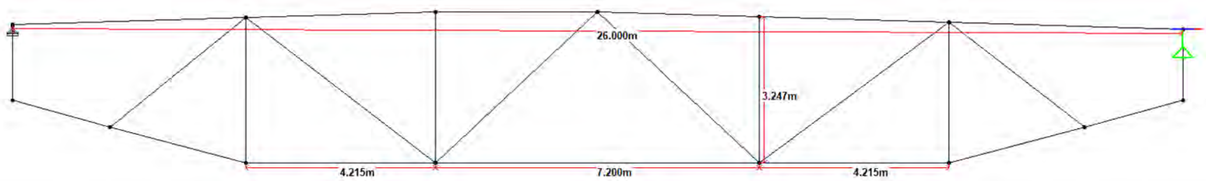
Axis 3 – 8
For column width of 9600 mm

Axis 9 –
For column width of 7200 mm

Figure 3-4 3D-view of Staad-model



Axis 3 – 8
For column width of 9600 mm



Axis 9
For column width of 7200 mm

Figure 3-5 Staad-model

3.3.2 Loads

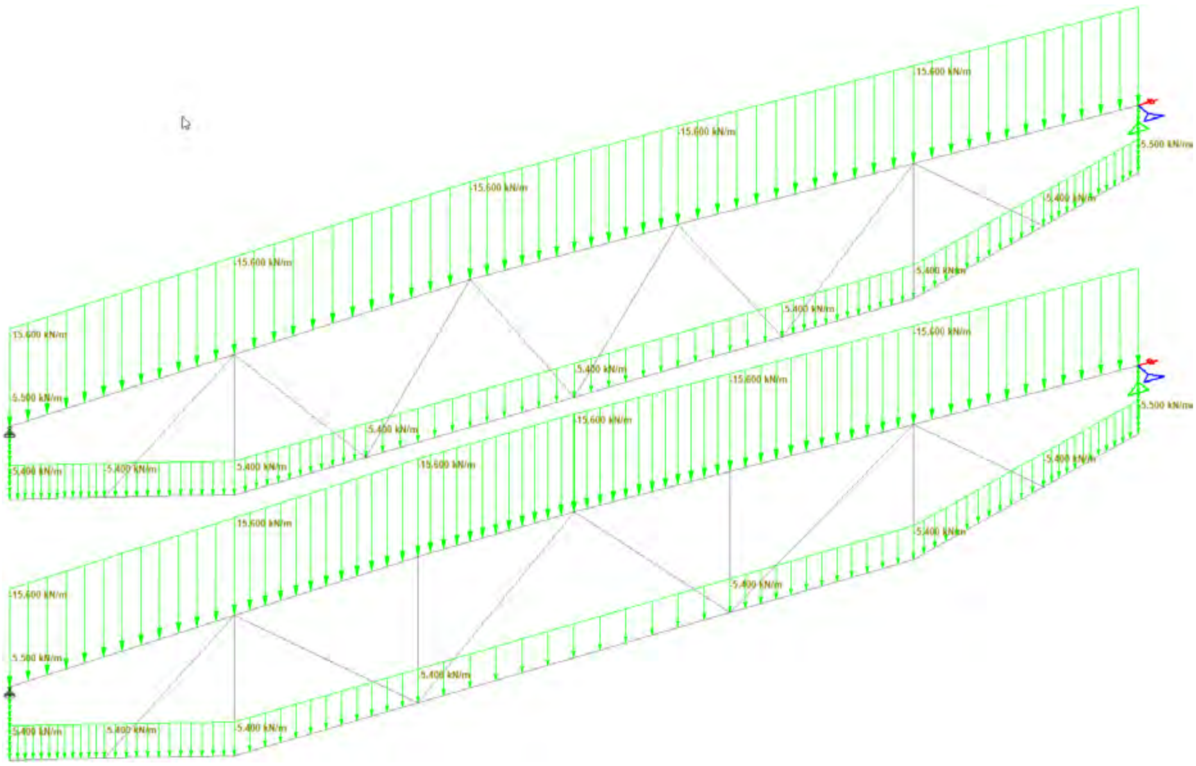


Figure 3-6 Dead loads of bridge birder. Self-weight of beams is also included.

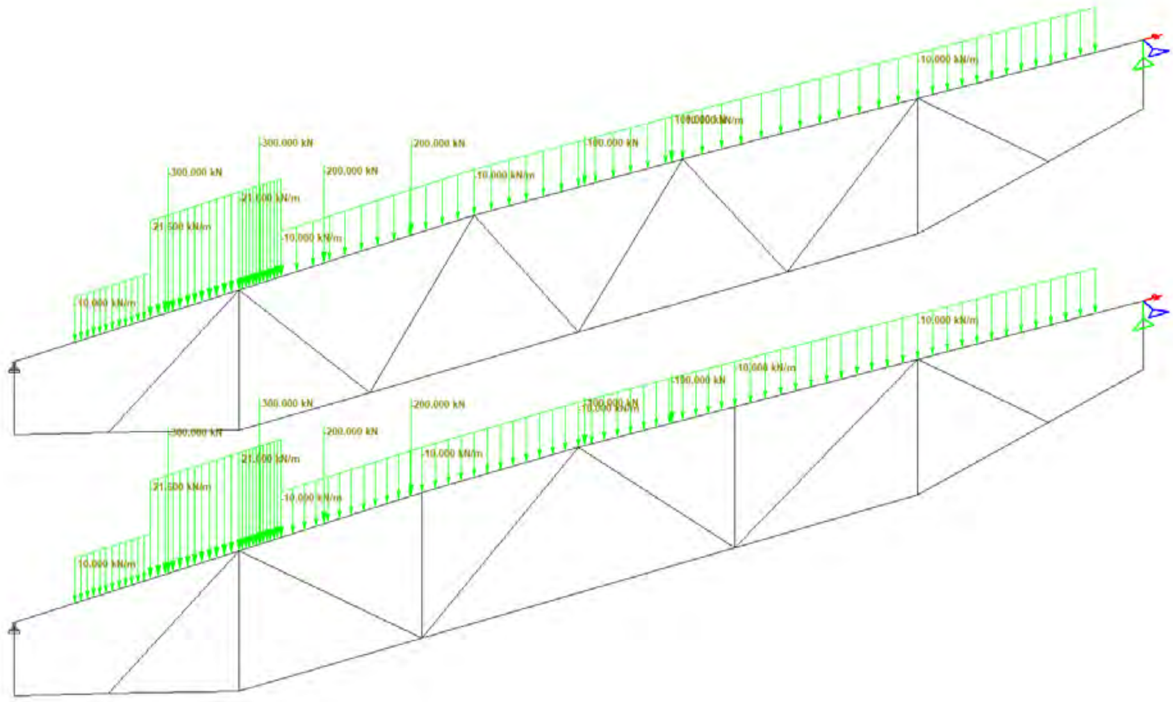


Figure 3-7 Traffic loads on bridge girder with maximum traffic in lane 1

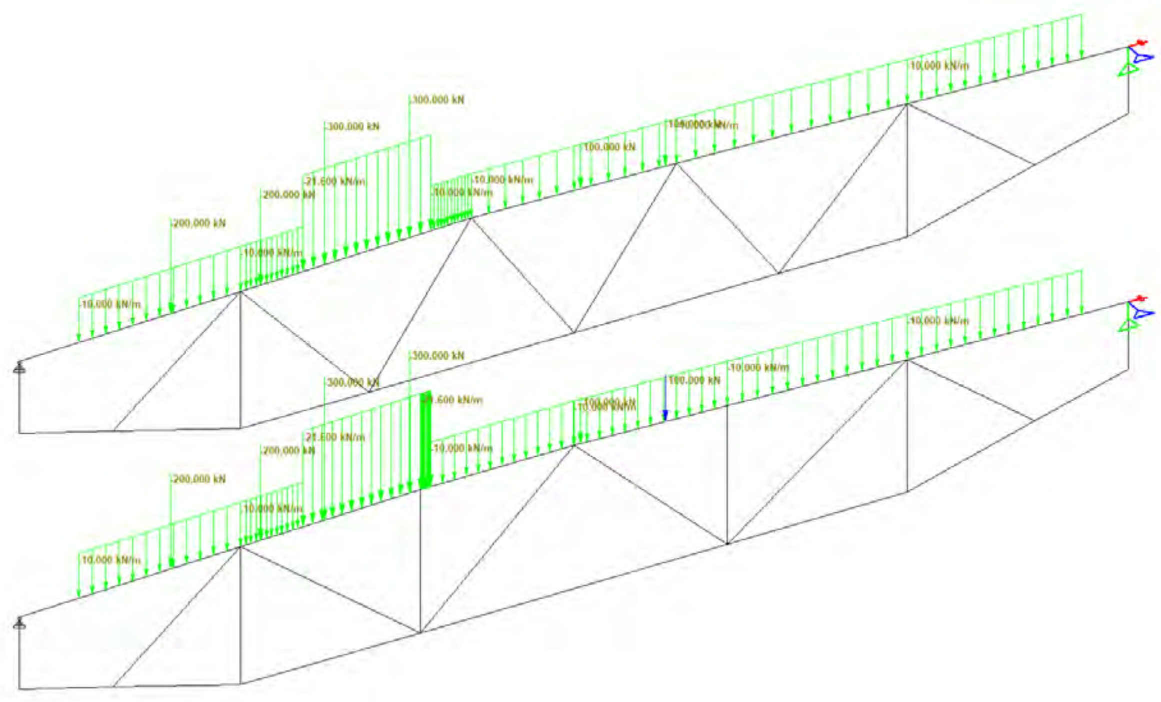


Figure 3-8 Traffic loads on bridge girder with maximum traffic in lane 2

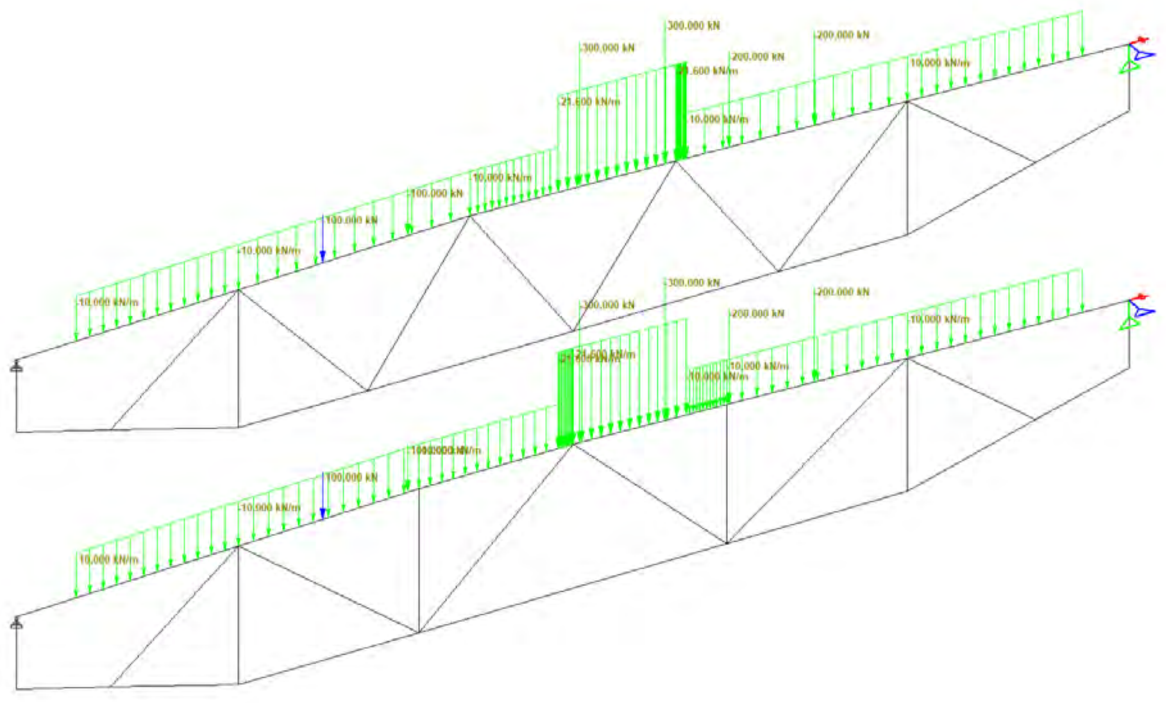


Figure 3-9 Traffic loads on bridge girder with maximum traffic in lane 3

Transverse trusses in bridge girder

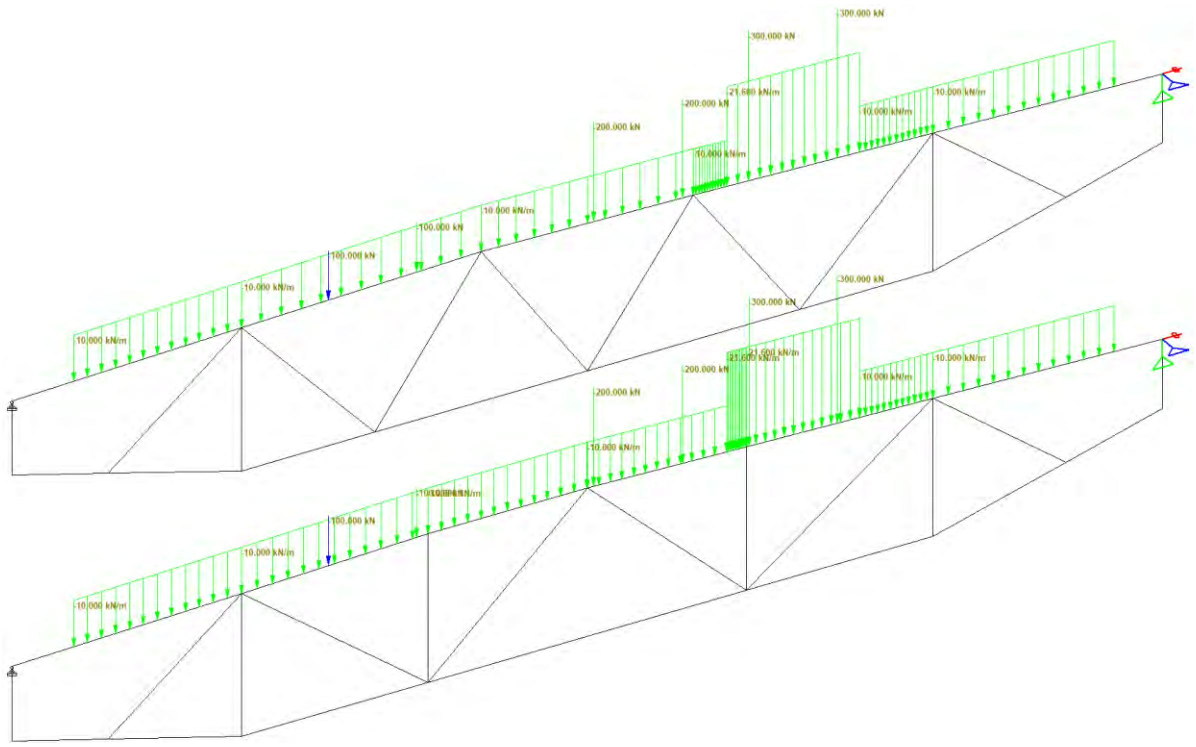


Figure 3-10 Traffic loads on bridge girder with maximum traffic in lane 4

3.3.3 Sectional forces and utilization factors

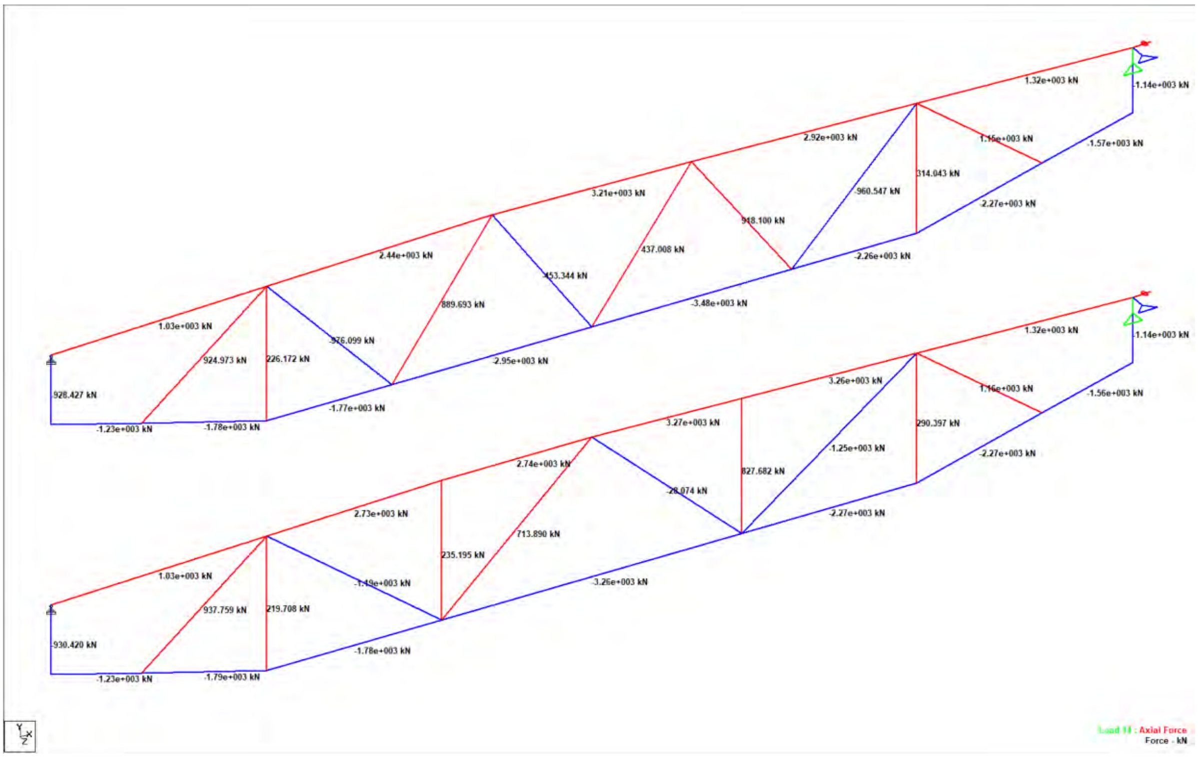


Figure 3-11 ULS axial forces for load combination 1

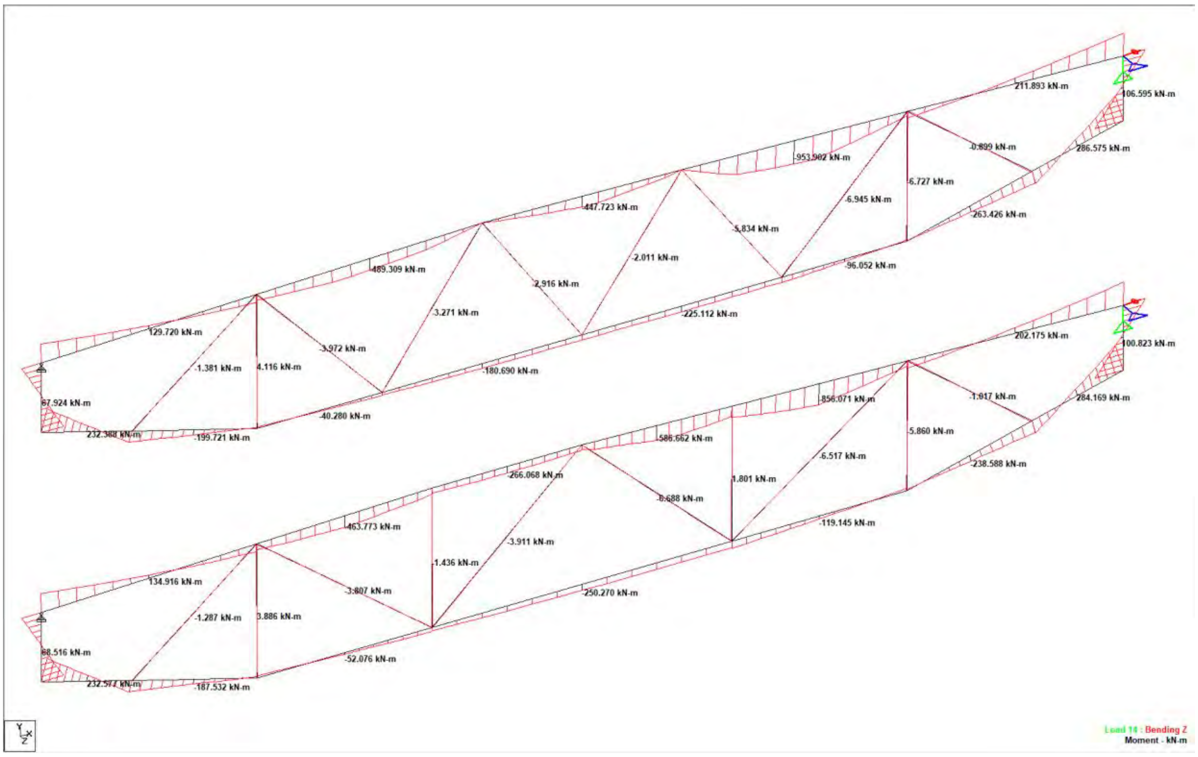


Figure 3-12 ULS bending forces

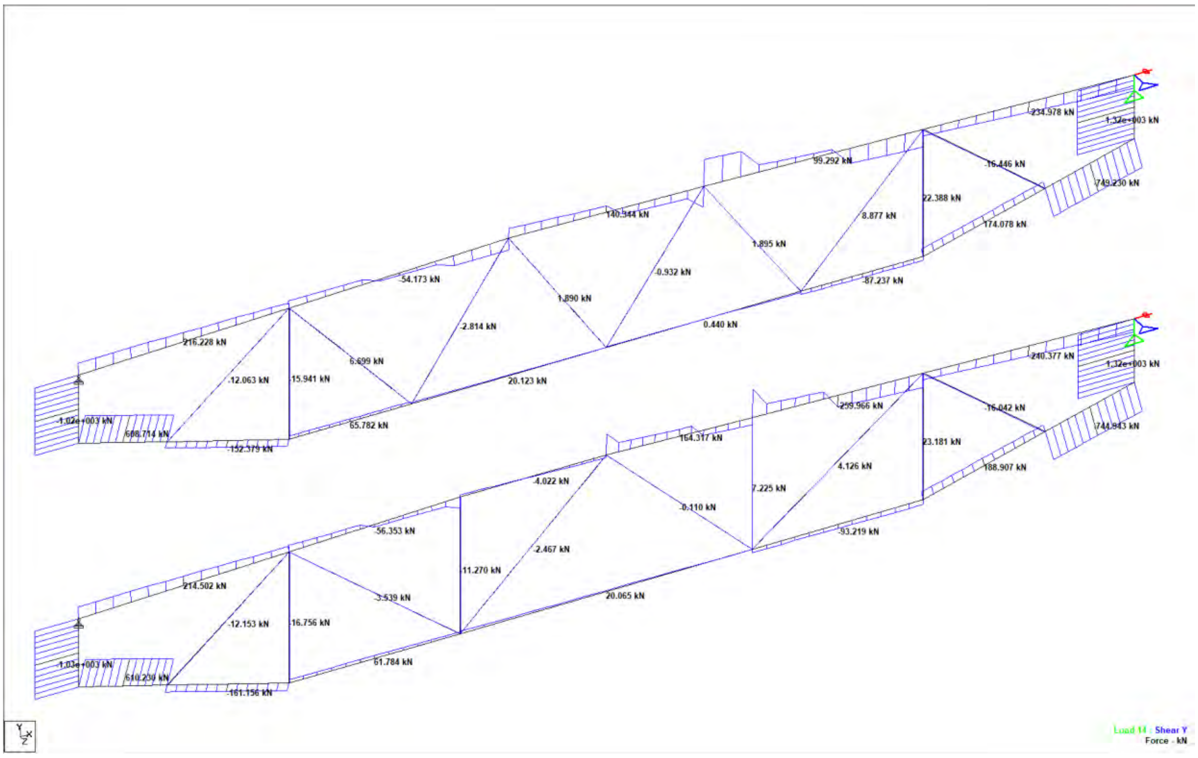


Figure 3-13 ULS shear forces

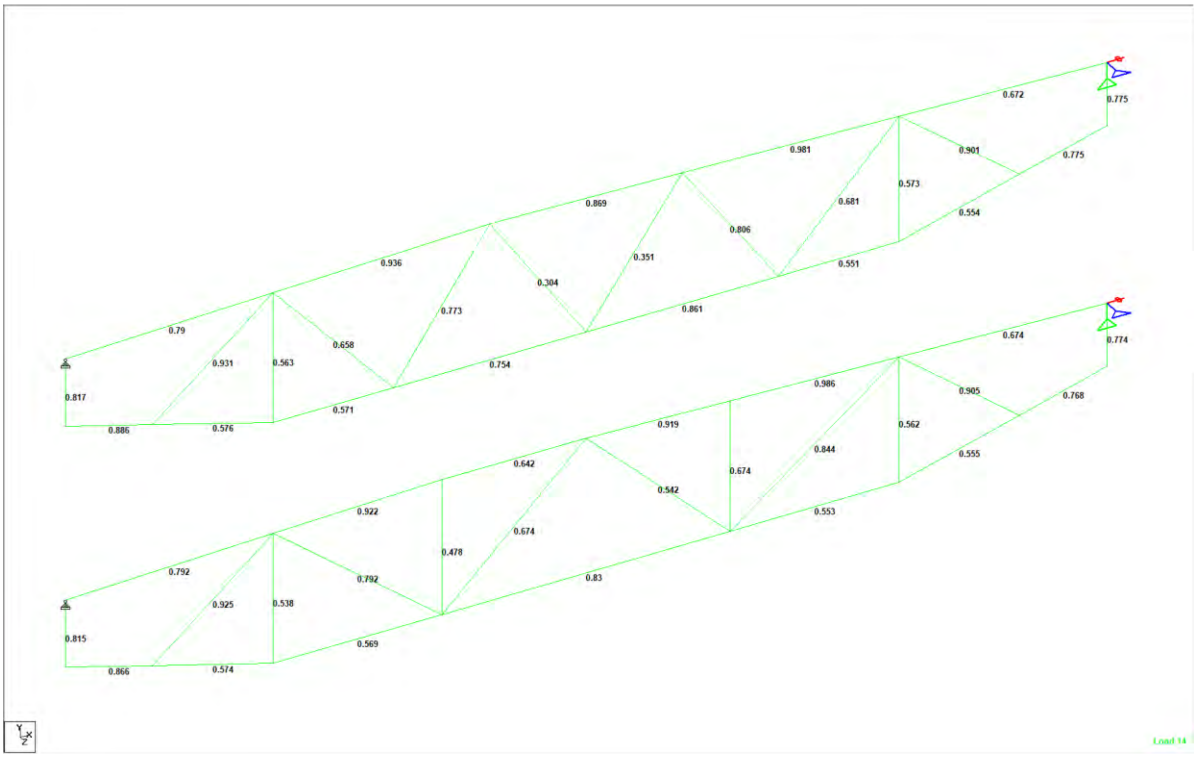


Figure 3-14 Utilization factors of truss members

4 Comparison with local FE-model of box girder

The utilization of the transverse trusses attained from the Staad-analyses are high, even when ignoring environmental loads, but it can be shown that the design approach presented in this memo, where it is assumed that all vertical loads are carried by the outer webs of the bridge girder, is a very conservative representation of the bridge girders load-carrying capacity. This can be seen in from the local FE-model of the bridge girder, refer to memo 13-NOT-099 – FEM analysis of bridge girder and column. Some relevant plots from the memo are shown in the following, where it is noted that vertical loads are also carried by the longitudinal trapezoidal stiffeners in the girder. This demonstrates that the load-carrying capacity is increased compared to the simplified support conditions used in Staad, proving sufficient capacity to carry additional environmental loads.

In Figure 4-1, the axial force in the truss elements of the transverse trusses are shown for the ULS load combination with maximum traffic in lane 1. The maximum tensile force is 40.7 kN while the maximum compression force is -142.1 kN. Figure 4-2 also show the axial forces in the truss elements, but the trapezoidal stiffeners are removed from the FE-model, resembling the Staad-analyses presented in this memo. For this case, the loads in the truss elements are significantly higher. The maximum tensile force is 662 kN, which is more than 16 times larger than the maximum tensile force in the model which includes the longitudinal stiffeners. The maximum compression force has increased by a factor of 4.6, to -605.7 kN.

The mentioned observation illustrates how the longitudinal stiffeners help to carry the vertical loads. In addition, it is observed that traffic loads are evenly distributed and carried by several transverse trusses when the longitudinal stiffeners are included in the FE-model, compared to only one transverse truss being heavily loaded when the stiffeners are removed.

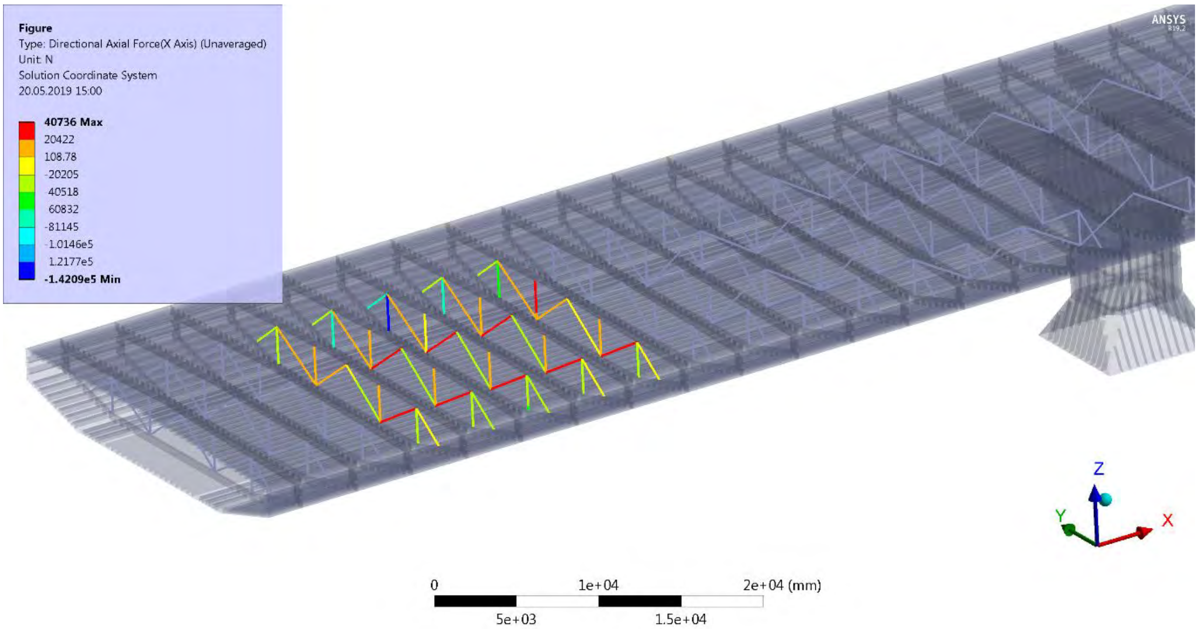


Figure 4-1 Axial force in truss elements

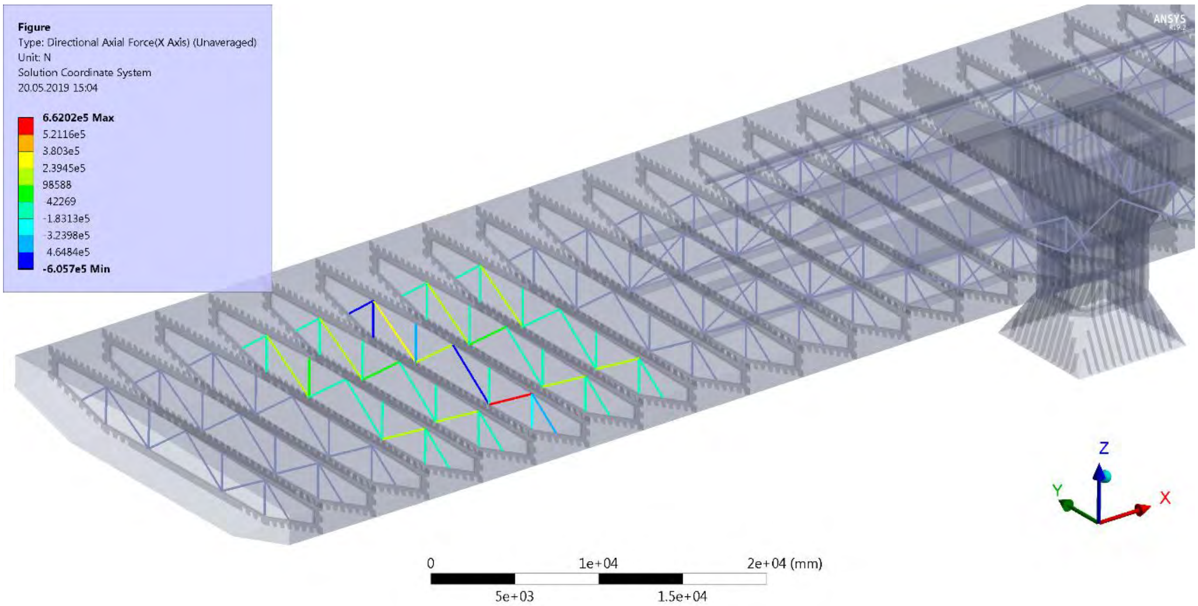


Figure 4-2 Axial force in truss elements - trapezoidal stiffeners removed

Another observation that can be done, justifying the conservativity of the chosen support conditions in the Staad-analyses, is the value of shear stress in the web plates of the bridge girder. In the FE-model where the longitudinal stiffeners are included, the maximum shear stress is only 6.5 MPa as shown in Figure 4-3. The maximum shar stress increases to 125 MPa when the stiffeners are removed. This is shown in Figure 4-4.

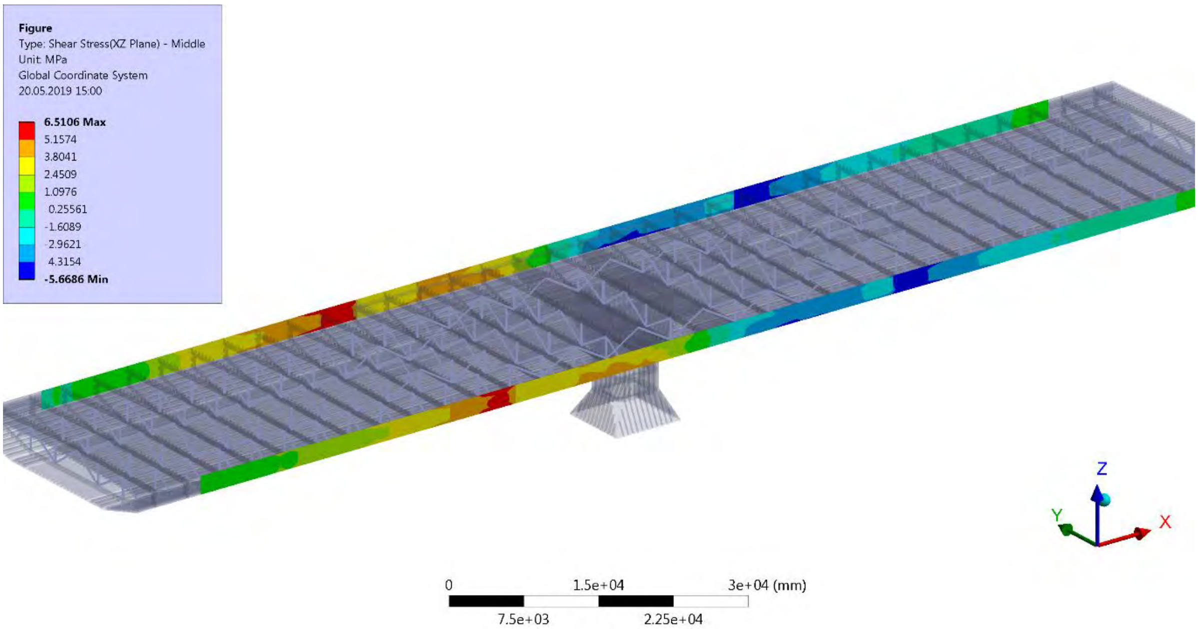


Figure 4-3 Shear stress in webs plates

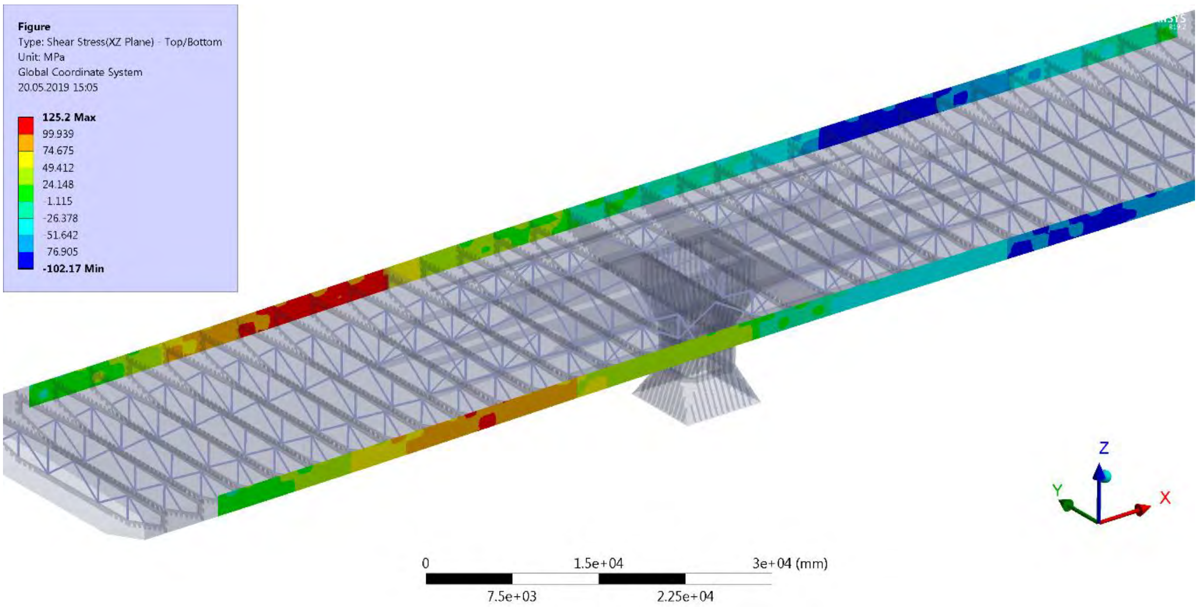


Figure 4-4 Shear stress in web plates - trapezoidal stiffeners removed

Considering the presented comparisons of the two different local FE-models, it is affirmed that designing the transverse truss with the assumption that all dead loads and traffic loads are carried from the orthotropic deck plate to the webs in the box girder is conservative. This assures that there is sufficient capacity in the transverse trusses to also carry the environmental loads which are not included in the Staad-analyses.

5 Additional requirements for transverse trusses

In order to provide a rigid support for the longitudinal stiffened plates, the transverse stiffeners must not only be designed for strength, but also for stiffness. The stiffness design criteria are given in section 9.2.1 of NS-EN 1993-1-5:2006 + NA:2009. In addition, limitations for cut outs in the stiffeners are given in section 9.2.4 of the same Eurocode.

The calculations on the following pages show that the transverse trusses satisfy the mentioned requirements.

The criteria related to stiffness are based on the relative stiffness between the plates which are supported by the transverse stiffener, and the transverse stiffener itself. For the bottom plate, only the check of the transverse stiffener above support is included. This is because the thickness of the bottom plate above support is larger than the thickness of the bottom plate at midspan, requiring a larger stiffness of the transverse stiffener.

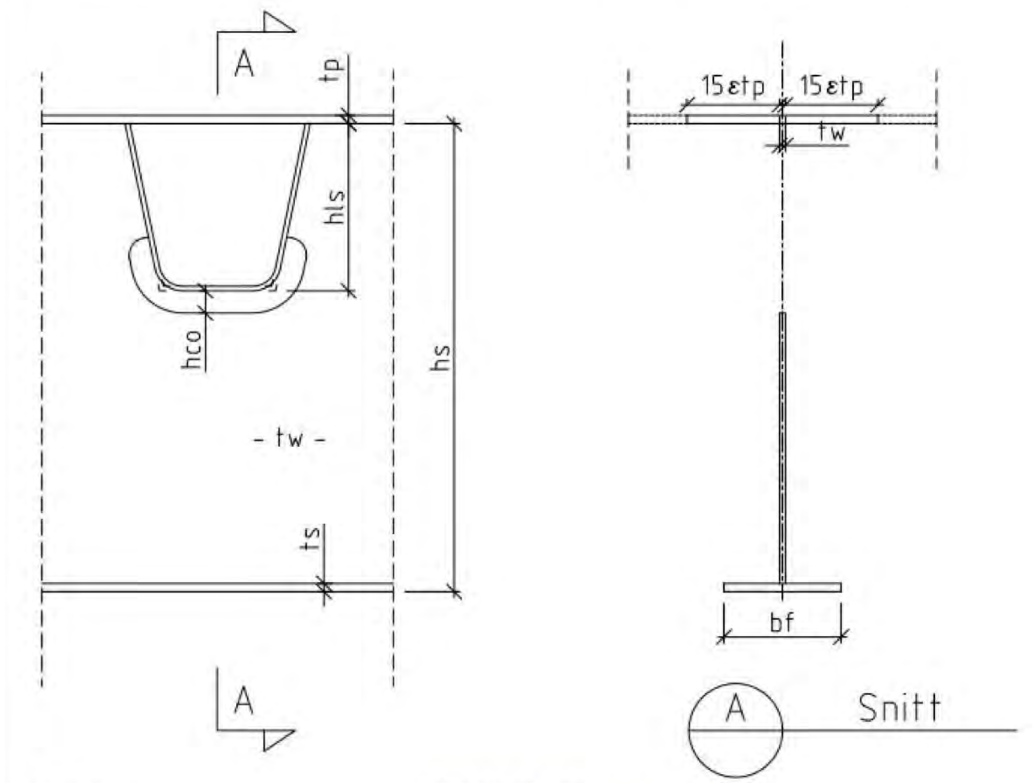
NS-EN 1993-1-5 9 Stiffeners and detailing
Section 9.2 Direct stresses

Check of transverse stiffener in top plate

Input

$f_y := 420MPa$	Yield strength
$E_{mod} := 210000MPa$	Modulus of elasticity
$\gamma_{M0} := 1.1$	Partial factor for reistance of cross-section
$\gamma_{M1} := 1.1$	Partial factor for reistance of members to instability
$\eta := 1.2$	Factor used in calculation of shear resistance in accordance with National Annex for NS-EN 1993-1-5 clause NA5.1.(2)
$a_1 := 4035mm$	Length of panel 1 adjacent to the transverse stiffener
$a_2 := 4035mm$	Length of panel 2 adjacent to the transverse stiffener
$b := 5685mm$	Width of plate - largest distance between supports in transverse frame
$A_{eff} := 0.124m^2$	Effective compressive area of panel 1 and panel 2 within width b
$b_G := 3015mm$	Min length of transverse stiffener between webs (or longitudinal diaphragms)

AMC *Prosjekt: Bjørnafjorden phase 5* *Side: ___2___*



$t_p := 16mm$	Thickness of panel
$h_{ls} := 286mm$	Height of longitudinal stiffener in panel
$h_{co} := 40mm$	Additional cut-out height in transverse stiffener
$t_w := 12mm$	Web thickness of transverse stiffener
$h_s := 800mm$	Height of transverse stiffener
$t_s := 14mm$	Thickness flange of transverse stiffener
$b_f := 200mm$	Width of flange in transverse stiffener

Filnavn:NS-EN 1993-1-5 Stiffeners and detailing - transverse stiffener in top *Dato:14.05.2019* *Sign: EDM*

Calculations

$$\varepsilon_s := \sqrt{\frac{235MPa}{f_y}} = 0.75$$

$$b_{eff\_p} := 15 \cdot \varepsilon_s \cdot t_p = 180 \cdot mm$$

$$b_p := 2 \cdot b_{eff\_p} + t_w = 371 \cdot mm$$

Effective flange width in panel

Gross section

$$h_w := h_s - t_s = 786 \cdot mm$$

Web height in transverse stiffener

$$A_s := b_p \cdot t_p + h_w \cdot t_w + b_f \cdot t_s = 18169 \cdot mm^2$$

Gross area of transverse stiffener

$$y_{st} := \frac{b_p \cdot \frac{t_p^2}{2} + h_w \cdot t_w \cdot \left(t_p + \frac{h_w}{2}\right) \dots + b_f \cdot t_s \cdot \left(t_p + h_w + \frac{t_s}{2}\right)}{A_s} = 340 \cdot mm$$

CoG of transverse stiffener from above panel

$$I_{st} := \frac{1}{12} \cdot \left(b_p \cdot t_p^3 + h_w^3 \cdot t_w + b_f \cdot t_s^3\right) \dots + b_p \cdot t_p \cdot \left(y_{st} - \frac{t_p}{2}\right)^2 \dots + h_w \cdot t_w \cdot \left(-y_{st} + t_p + \frac{h_w}{2}\right)^2 \dots + b_f \cdot t_s \cdot \left(-y_{st} + t_p + h_w + \frac{t_s}{2}\right)^2 = 1.801 \times 10^9 \cdot mm^4$$

Second moment of area for the gross section of the transverse stiffener

$$e_{max} := \max\left(y_{st}, h_s + t_p - y_{st}\right) = 476 \cdot mm$$

Maximum distance from the extreme fibre of the stiffener to the centroid of the stiffener

$$e_{min} := \min\left(y_{st}, h_s + t_p - y_{st}\right) = 340 \cdot mm$$

Minimum distance from the extreme fibre of the stiffener to the centroid of the stiffener

Cross section class check of gross stiffener section

Compression flange

$$class_{flange} := \begin{cases} \text{"1-3"} & \text{if } \dfrac{\dfrac{b_f}{2} - t_w}{t_s} \le 14 \cdot \varepsilon_s \\ 4 & \text{otherwise} \end{cases} = \text{"1-3"}$$

Web in bending

$z_1 := t_p + h_w - y_{st} = 462 \cdot mm$ Distance from CoG in stiffener to extreme web fiber in compression

$z_2 := y_{st} - t_p = 324 \cdot mm$ Distance from CoG in steffener to extreme web fiber in tension

$$\psi := \frac{-z_2}{z_1} = -0.70$$

$$class_{web} := \begin{cases} \text{"1-3"} & \text{if } \dfrac{h_w}{t_w} \le \dfrac{42 \cdot \varepsilon_s}{0.67 + 0.33 \cdot \psi} \wedge \psi > -1 \\ \text{"1-3"} & \text{if } \dfrac{h_w}{t_w} \le 62 \cdot \varepsilon_s \cdot (1 - \psi) \cdot \sqrt{-\psi} \wedge \psi \le -1 \\ 4 & \text{otherwise} \end{cases} = \text{"1-3"}$$

Net section

$h_{eff\_w.net} := h_s - h_{ls} - h_{co} - t_s = 460 \cdot mm$ Net web height of transverse stiffener

$A_{s.net} := b_p \cdot t_p + h_{eff\_w.net} \cdot t_w + b_f \cdot t_s = 14257 \cdot mm^2$ Net area of transverse stiffener

$$y_{st.net} := \frac{b_p \cdot \dfrac{t_p^2}{2} + h_{eff\_w.net} \cdot t_w \cdot \left(t_p + h_{ls} + h_{co} + \dfrac{h_{eff\_w.net}}{2}\right) \dots + b_f \cdot t_s \cdot \left(t_p + h_{ls} + h_{co} + h_{eff\_w.net} + \dfrac{t_s}{2}\right)}{A_{s.net}} = 384 \cdot mm$$

CoG of net stiffener from above plate

$$I_{st.net} := \frac{1}{12} \cdot \left(b_p \cdot t_p^3 + h_{eff\_w.net}^3 \cdot t_w + b_f \cdot t_s^3\right) \dots + b_p \cdot t_p \cdot \left(y_{st.net} - \frac{t_p}{2}\right)^2 \dots + h_{eff\_w.net} \cdot t_w \cdot \left(-y_{st.net} + t_p + h_{ls} + h_{co} + \frac{h_{eff\_w.net}}{2}\right)^2 \dots + b_f \cdot t_s \cdot \left(-y_{st.net} + t_p + h_{ls} + h_{co} + h_{eff\_w.net} + \frac{t_s}{2}\right)^2 = 1.638 \times 10^9 \cdot mm^4$$

Second moment of area for the net section of the transverse stiffener

$e_{net} := h_s + t_p - y_{st.net} = 432 \cdot mm$ Distance from the underside of the flange plate to the neutral axis of the net section

Minimum requirement for transverse stiffener in accordance with cluase 9.2.1(5)

$$w_0 := \frac{\min(a_1, a_1, b)}{300} = 13.45 \cdot mm$$

Initial imperfection of transverse stiffener

$$u := \max\left(1, \frac{\pi^2 \cdot E_{mod} \cdot e_{max}}{\frac{f_y \cdot 300 \cdot b}{\gamma_{M1}}}\right) = 1.52$$

$$N_{Ed} := \frac{f_y}{\gamma_{M0}} A_{eff} = 47 \cdot MN$$

Compressive force in adjacent panels. Conservatively set equal to capacity of adjacent panels

$$\sigma_{fac} := 1.0$$

Aspect ratio between elastic ctirical stresses for column- and plate-like buckling of adjacent panels. Conservatively set to 1.0

$$\sigma_m := \sigma_{fac} \cdot \left[\frac{N_{Ed}}{b} \cdot \left(\frac{1}{a_1} + \frac{1}{a_2}\right)\right] = 4.13 \cdot MPa$$

$$I_{st.min} := \frac{\sigma_m}{E_{mod}} \cdot \left(\frac{b}{\pi}\right)^4 \cdot \left(1 + w_0 \cdot \frac{300}{b} \cdot u\right) = 4.376 \times 10^8 \cdot mm^4$$

Minimum required stiffness of transverse stiffener

$$\frac{I_{st.min}}{I_{st}} = 0.24$$

$$eq_{9.1} := \text{if}\left(I_{st} \geq I_{st.min}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$$

Limtation of cut-out in transverse stiffener web in accordance with clause 9.2.4(4)

$$cl_{9.2.4\_4} := \text{if}\left[\left(h_{ls} + h_{co}\right) \leq 0.6\left(h_s + t_p\right), \text{"OK"}, \text{"NOT OK"}\right] = \text{"OK"}$$

Shear resistance of the stiffener web in accordance with clause 9.2.4(5)

$$V_{Ed} := \frac{I_{st.net} \cdot f_y \cdot \pi}{e_{net} \cdot \gamma_{M0} \cdot b_G} = 1507 \cdot kN$$

Shear force to be resisted by the gross web adjacent to the cut-out

$$k_\tau := \begin{vmatrix} 5.34 + 4 \cdot \left(\frac{h_w}{b_G}\right)^2 & \text{if } \frac{b_G}{h_w} \geq 1 \\ 4 + 5.34 \cdot \left(\frac{h_w}{b_G}\right)^2 & \text{otherwise} \end{vmatrix} = 5.61$$

$$\lambda_{w.s} := \frac{h_w}{37.4 \cdot t_w \cdot \varepsilon_s \cdot \sqrt{k_\tau}} = 0.99$$

$$\chi_w := \begin{vmatrix} \eta & \text{if } \lambda_{w.s} < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_{w.s}} & \text{if } \frac{0.83}{\eta} \leq \lambda_{w.s} < 1.08 \\ \frac{1.37}{0.7 + \lambda_{w.s}} & \text{otherwise} \end{vmatrix} = 0.84$$

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1746 \cdot kN$$

$$eq_{9.5} := \text{if}\left(V_{bw.Rd} \geq V_{Ed}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$$

NS-EN 1993-1-5 9 Stiffeners and detailing
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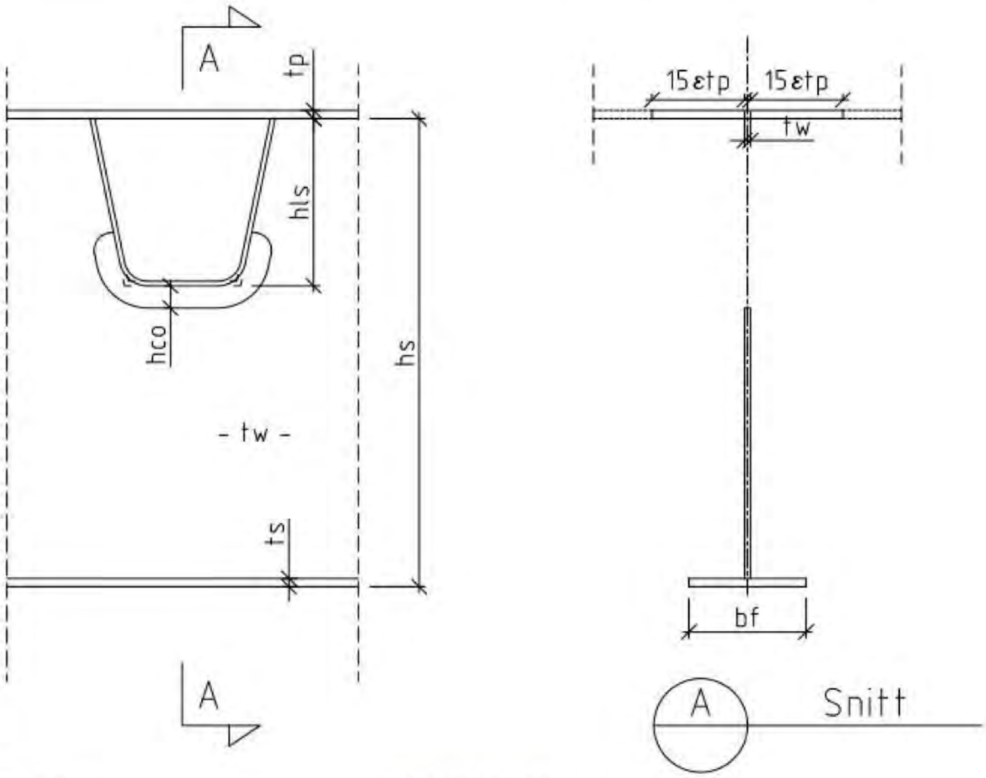
Check of transverse stiffener in bottom plate of cross section type S1 - transverse truss for column width 7200 mm

Input

$f_y := 420\text{MPa}$	Yield strength
$E_{mod} := 210000\text{MPa}$	Modulus of elasticity
$\gamma_{M0} := 1.1$	Partial factor for reistance of cross-section
$\gamma_{M1} := 1.1$	Partial factor for reistance of members to instability
$\eta := 1.2$	Factor used in calculation of shear resistance in accordance with National Annex for NS-EN 1993-1-5 clause NA5.1.(2)
$a_1 := 4035\text{mm}$	Length of panel 1 adjacent to the transverse stiffener
$a_2 := 4035\text{mm}$	Length of panel 2 adjacent to the transverse stiffener
$b := 7200\text{mm}$	Width of plate - largest distance between supports in transverse frame
$A_{eff} := 0.189\text{m}^2$	Effective compressive area of panel 1 and panel 2 within width b
$b_G := 4215\text{mm}$	Min length of transverse stiffener between webs (or longitudinal diaphragms)

AMC | **Prosjekt: Bjørnafjorden phase 5** | **Side: ___2___**



$t_p := 20\text{mm}$ — Thickness of panel

$h_{ls} := 308\text{mm}$ — Height of longitudinal stiffener in panel

$h_{co} := 20\text{mm}$ — Additional cut-out height in transverse stiffener

$t_w := 10\text{mm}$ — Web thickness of transverse stiffener

$h_s := 600\text{mm}$ — Height of transverse stiffener

$t_s := 14\text{mm}$ — Thickness flange of transverse stiffener

$b_f := 200\text{mm}$ — Width of flange in transverse stiffener

Filnavn:NS-EN 1993-1-5 Stiffeners and detailing - transverse stiffener in bottom panel | *Dato:16.05.2019* | *Sign: EDM*

Calculations

$$\varepsilon_s := \sqrt{\frac{235MPa}{f_y}} = 0.75$$

$$b_{eff\_p} := 15 \cdot \varepsilon_s \cdot t_p = 224 \cdot mm$$

$$b_p := 2 \cdot b_{eff\_p} + t_w = 459 \cdot mm$$

Effective flange width in panel

Gross section

$$h_w := h_s - t_s = 586 \cdot mm$$

Web height in transverse stiffener

$$A_s := b_p \cdot t_p + h_w \cdot t_w + b_f \cdot t_s = 17836 \cdot mm^2$$

Gross area of transverse stiffener

$$y_{st} := \frac{b_p \cdot \frac{t_p^2}{2} + h_w \cdot t_w \cdot \left(t_p + \frac{h_w}{2}\right) \ldots + b_f \cdot t_s \cdot \left(t_p + h_w + \frac{t_s}{2}\right)}{A_s} = 204 \cdot mm$$

CoG of transverse stiffener from above panel

$$I_{st} := \frac{1}{12} \cdot \left(b_p \cdot t_p^3 + h_w^3 \cdot t_w + b_f \cdot t_s^3\right) \ldots + b_p \cdot t_p \cdot \left(y_{st} - \frac{t_p}{2}\right)^2 \ldots + h_w \cdot t_w \cdot \left(-y_{st} + t_p + \frac{h_w}{2}\right)^2 \ldots + b_f \cdot t_s \cdot \left(-y_{st} + t_p + h_w + \frac{t_s}{2}\right)^2 = 1.051 \times 10^9 \cdot mm^4$$

Second moment of area for the gross section of the transverse stiffener

$$e_{max} := \max\left(y_{st}, h_s + t_p - y_{st}\right) = 416 \cdot mm$$

Maximum distance from the extreme fibre of the stiffener to the centroid of the stiffener

$$e_{min} := \min\left(y_{st}, h_s + t_p - y_{st}\right) = 204 \cdot mm$$

Minimum distance from the extreme fibre of the stiffener to the centroid of the stiffener

Filnavn:NS-EN 1993-1-5 Stiffeners and detailing - transverse stiffener in bottom panel *Dato:16.05.2019* *Sign: EDM*

Cross section class check of gross stiffener section

Compression flange

$$class_{flange} := \begin{cases} \text{"1-3"} & \text{if } \dfrac{\dfrac{b_f}{2} - t_w}{t_s} \le 14 \cdot \varepsilon_s \\ 4 & \text{otherwise} \end{cases} = \text{"1-3"}$$

Web in bending

$z_1 := t_p + h_w - y_{st} = 402 \cdot mm$ Distance from CoG in stiffener to extreme web fiber in compression

$z_2 := y_{st} - t_p = 184 \cdot mm$ Distance from CoG in steffener to extreme web fiber in tension

$$\psi := \frac{-z_2}{z_1} = -0.46$$

$$class_{web} := \begin{cases} \text{"1-3"} & \text{if } \dfrac{h_w}{t_w} \le \dfrac{42 \cdot \varepsilon_s}{0.67 + 0.33 \cdot \psi} \wedge \psi > -1 \\ \text{"1-3"} & \text{if } \dfrac{h_w}{t_w} \le 62 \cdot \varepsilon_s \cdot (1 - \psi) \cdot \sqrt{-\psi} \wedge \psi \le -1 \\ 4 & \text{otherwise} \end{cases} = \text{"1-3"}$$

Net section

$h_{eff\_w.net} := h_s - h_{ls} - h_{co} - t_s = 258 \cdot mm$ Net web height of transverse stiffener

$A_{s.net} := b_p \cdot t_p + h_{eff\_w.net} \cdot t_w + b_f \cdot t_s = 14556 \cdot mm^2$ Net area of transverse stiffener

$$y_{st.net} := \frac{b_p \cdot \dfrac{t_p^2}{2} + h_{eff\_w.net} \cdot t_w \cdot \left(t_p + h_{ls} + h_{co} + \dfrac{h_{eff\_w.net}}{2}\right) \dots + b_f \cdot t_s \cdot \left(t_p + h_{ls} + h_{co} + h_{eff\_w.net} + \dfrac{t_s}{2}\right)}{A_{s.net}} = 209 \cdot mm$$

CoG of net stiffener from above plate

$$I_{st.net} := \frac{1}{12} \cdot \left(b_p \cdot t_p^3 + h_{eff\_w.net}^3 \cdot t_w + b_f \cdot t_s^3\right) \dots + b_p \cdot t_p \cdot \left(y_{st.net} - \frac{t_p}{2}\right)^2 \dots + h_{eff\_w.net} \cdot t_w \cdot \left(-y_{st.net} + t_p + h_{ls} + h_{co} + \frac{h_{eff\_w.net}}{2}\right)^2 \dots + b_f \cdot t_s \cdot \left(-y_{st.net} + t_p + h_{ls} + h_{co} + h_{eff\_w.net} + \frac{t_s}{2}\right)^2 = 1.020 \times 10^9 \cdot mm^4$$

Second moment of area for the net section of the transverse stiffener

$e_{net} := h_s + t_p - y_{st.net} = 411 \cdot mm$ Distance from the underside of the flange plate to the neutral axis of the net section

Minimum requirement for transverse stiffener in accordance with cluase 9.2.1(5)

$$w_0 := \frac{\min(a_1, a_1, b)}{300} = 13.45 \cdot mm$$

Initial imperfection of transverse stiffener

$$u := \max\left(1, \frac{\pi^2 \cdot E_{mod} \cdot e_{max}}{\frac{f_y \cdot 300 \cdot b}{\gamma_{M1}}}\right) = 1.04$$

$$N_{Ed} := \frac{f_y}{\gamma_{M0}} A_{eff} = 72 \cdot MN$$

Compressive force in adjacent panels. Conservatively set equal to capacity of adjacent panels

$$\sigma_{fac} := 1.0$$

Aspect ratio between elastic ctirical stresses for column- and plate-like buckling of adjacent panels. Conservatively set to 1.0

$$\sigma_m := \sigma_{fac} \cdot \left[\frac{N_{Ed}}{b} \cdot \left(\frac{1}{a_1} + \frac{1}{a_2}\right)\right] = 4.97 \cdot MPa$$

$$I_{st.min} := \frac{\sigma_m}{E_{mod}} \cdot \left(\frac{b}{\pi}\right)^4 \cdot \left(1 + w_0 \cdot \frac{300}{b} \cdot u\right) = 1.035 \times 10^9 \cdot mm^4$$

Minimum required stiffness of transverse stiffener

$$\frac{I_{st.min}}{I_{st}} = 0.98$$

$$eq_{9.1} := \text{if}\left(I_{st} \geq I_{st.min}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$$

Limtation of cut-out in transverse stiffener web in accordance with clause 9.2.4(4)

$$cl_{9.2.4\_4} := \text{if}\left[\left(h_{ls} + h_{co}\right) \leq 0.6\left(h_s + t_p\right), \text{"OK"}, \text{"NOT OK"}\right] = \text{"OK"}$$

Shear resistance of the stiffener web in accordance with clause 9.2.4(5)

$$V_{Ed} := \frac{I_{st.net} \cdot f_y \cdot \pi}{e_{net} \cdot \gamma_{M0} \cdot b_G} = 706 \cdot kN$$

Shear force to be resisted by the gross web adjacent to the cut-out

$$k_\tau := \begin{cases} 5.34 + 4 \cdot \left(\frac{h_w}{b_G}\right)^2 & \text{if } \frac{b_G}{h_w} \geq 1 \\ 4 + 5.34 \cdot \left(\frac{h_w}{b_G}\right)^2 & \text{otherwise} \end{cases} = 5.42$$

$$\lambda_{w.s} := \frac{h_w}{37.4 \cdot t_w \cdot \varepsilon_s \cdot \sqrt{k_\tau}} = 0.90$$

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_{w.s} < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_{w.s}} & \text{if } \frac{0.83}{\eta} \leq \lambda_{w.s} < 1.08 \\ \frac{1.37}{0.7 + \lambda_{w.s}} & \text{otherwise} \end{cases} = 0.92$$

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1191 \cdot kN$$

$$eq_{9.5} := \text{if}\left(V_{bw.Rd} \geq V_{Ed}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$$

AMC *Prosjekt: Bjørnafjorden phase 5* *Side: __1__*

NS-EN 1993-1-5 9 Stiffeners and detailing
Section 9.2 Direct stresses

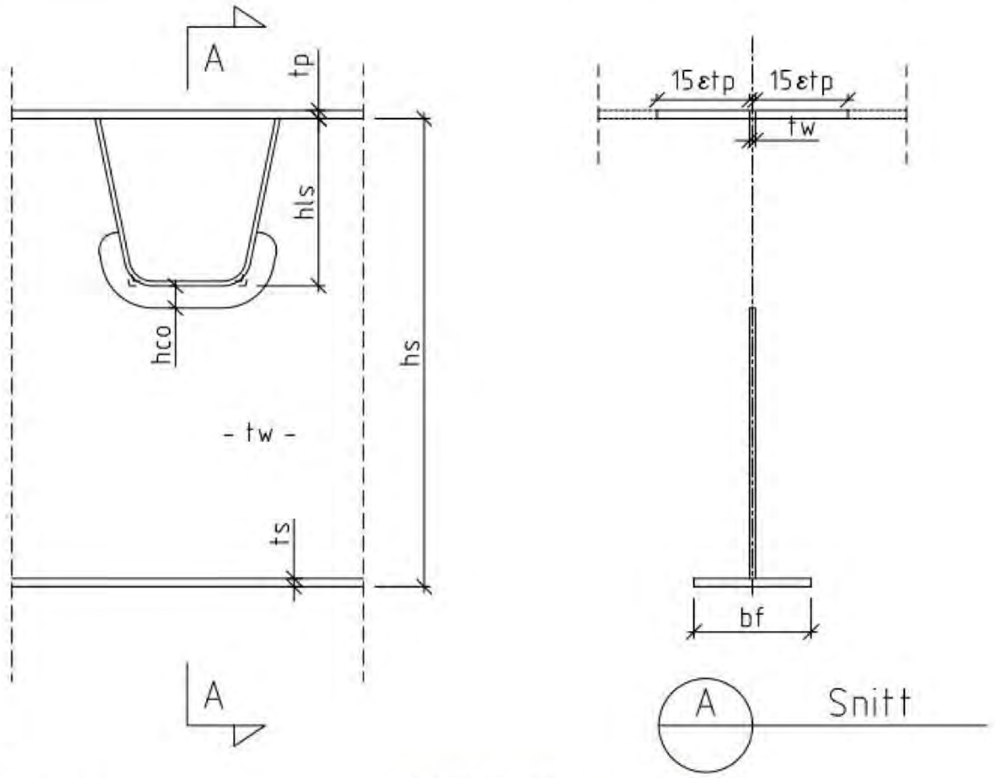
Check of transverse stiffener i bottom plate of cross section type S2 - transverse truss for column width 9600 mm

Input

$f_y := 420MPa$	Yield strength
$E_{mod} := 210000MPa$	Modulus of elasticity
$\gamma_{M0} := 1.1$	Partial factor for reistance of cross-section
$\gamma_{M1} := 1.1$	Partial factor for reistance of members to instability
$\eta := 1.2$	Factor used in calculation of shear resistance in accordance with National Annex for NS-EN 1993-1-5 clause NA5.1.(2)
$a_1 := 4035mm$	Length of panel 1 adjacent to the transverse stiffener
$a_2 := 4035mm$	Length of panel 2 adjacent to the transverse stiffener
$b := 4800mm$	Width of plate - largest distance between supports in transverse frame
$A_{eff} := 0.139m^2$	Effective compressive area of panel 1 and panel 2 within width b
$b_G := 3015mm$	Min length of transverse stiffener between webs (or longitudinal diaphragms)

Filnavn:NS-EN 1993-1-5 Stiffeners and detailing - transverse stiffener in bottom panel *Dato:14.05.2019* *Sign: EDM*

AMC *Prosjekt: Bjørnafjorden phase 5* *Side: __2__*



$t_p := 22mm$	Thickness of panel
$h_{ls} := 308mm$	Height of longitudinal stiffener in panel
$h_{co} := 20mm$	Additional cut-out height in transverse stiffener
$t_w := 10mm$	Web thickness of transverse stiffener
$h_s := 600mm$	Height of transverse stiffener
$t_s := 14mm$	Thickness flange of transverse stiffener
$b_f := 200mm$	Width of flange in transverse stiffener

Filnavn:NS-EN 1993-1-5 Stiffeners and detailing - transverse stiffener in bottom panel *Dato:14.05.2019* *Sign: EDM*

Calculations

$$\varepsilon_s := \sqrt{\frac{235\text{MPa}}{f_y}} = 0.75$$

$$b_{eff\_p} := 15 \cdot \varepsilon_s \cdot t_p = 247 \cdot \text{mm}$$

$$b_p := 2 \cdot b_{eff\_p} + t_w = 504 \cdot \text{mm}$$

Effective flange width in panel

Gross section

$$h_w := h_s - t_s = 586 \cdot \text{mm}$$

Web height in transverse stiffener

$$A_s := b_p \cdot t_p + h_w \cdot t_w + b_f \cdot t_s = 19741 \cdot \text{mm}^2$$

Gross area of transverse stiffener

$$y_{st} := \frac{b_p \cdot \frac{t_p^2}{2} + h_w \cdot t_w \cdot \left(t_p + \frac{h_w}{2}\right) \ldots + b_f \cdot t_s \cdot \left(t_p + h_w + \frac{t_s}{2}\right)}{A_s} = 187 \cdot \text{mm}$$

CoG of transverse stiffener from above panel

$$I_{st} := \frac{1}{12} \cdot \left(b_p \cdot t_p^3 + h_w^3 \cdot t_w + b_f \cdot t_s^3\right) \ldots + b_p \cdot t_p \cdot \left(y_{st} - \frac{t_p}{2}\right)^2 \ldots + h_w \cdot t_w \cdot \left(-y_{st} + t_p + \frac{h_w}{2}\right)^2 \ldots + b_f \cdot t_s \cdot \left(-y_{st} + t_p + h_w + \frac{t_s}{2}\right)^2 = 1.120 \times 10^9 \cdot \text{mm}^4$$

Second moment of area for the gross section of the transverse stiffener

$$e_{max} := \max\left(y_{st}, h_s + t_p - y_{st}\right) = 435 \cdot \text{mm}$$

Maximum distance from the extreme fibre of the stiffener to the centroid of the stiffener

$$e_{min} := \min\left(y_{st}, h_s + t_p - y_{st}\right) = 187 \cdot \text{mm}$$

Minimum distance from the extreme fibre of the stiffener to the centroid of the stiffener

Cross section class check of gross stiffener section

Compression flange

$$\text{class}_{\text{flange}} := \begin{cases} \text{"1-3"} & \text{if } \dfrac{\frac{b_f}{2} - t_w}{t_s} \le 14 \cdot \varepsilon_s \\ 4 & \text{otherwise} \end{cases} = \text{"1-3"}$$

Web in bending

$z_1 := t_p + h_w - y_{st} = 421 \cdot \text{mm}$ Distance from CoG in stiffener to extreme web fiber in compression

$z_2 := y_{st} - t_p = 165 \cdot \text{mm}$ Distance from CoG in steffener to extreme web fiber in tension

$$\psi := \frac{-z_2}{z_1} = -0.39$$

$$\text{class}_{\text{web}} := \begin{cases} \text{"1-3"} & \text{if } \dfrac{h_w}{t_w} \le \dfrac{42 \cdot \varepsilon_s}{0.67 + 0.33 \cdot \psi} \wedge \psi > -1 \\ \text{"1-3"} & \text{if } \dfrac{h_w}{t_w} \le 62 \cdot \varepsilon_s \cdot (1 - \psi) \cdot \sqrt{-\psi} \wedge \psi \le -1 \\ 4 & \text{otherwise} \end{cases} = 4.00$$

Effective section

$$k_\sigma := 7.81 - 6.29 \cdot \psi + 9.78 \cdot \psi^2 = 11.77$$

$$\lambda_w := \frac{\frac{h_w}{t_w}}{28.4 \cdot \varepsilon_s \cdot \sqrt{k_\sigma}} = 0.80$$

$$\rho_w := \frac{\lambda_w - 0.188}{\lambda_w^2} = 0.95$$

$b_{we1} := 0.4 \rho_w \cdot z_1 = 161 \cdot \text{mm}$

$b_{we2} := 0.6 \rho_w \cdot z_1 = 241 \cdot \text{mm}$

$b_{we3} := z_2 + b_{we2} = 406 \cdot \text{mm}$

$A_{s.eff} := A_s - h_w \cdot (1 - \rho_w) \cdot t_w = 19466 \cdot \text{mm}^2$ Effective area of transverse stiffener

$$y_{st.eff} := \frac{b_p \cdot \frac{t_p^2}{2} + b_{we3} \cdot t_w \cdot \left(t_p + \frac{b_{we3}}{2}\right) \ldots + b_{we1} \cdot t_w \cdot \left(t_p + h_w - \frac{b_{we1}}{2}\right) + b_f \cdot t_s \cdot \left(t_p + h_w + \frac{t_s}{2}\right)}{A_s} = 183 \cdot \text{mm}$$

CoG of effective transverse stiffener from above panel

Filnavn:NS-EN 1993-1-5 Stiffeners and detailing - transverse stiffener in bottom panel *Dato:14.05.2019* *Sign: EDM*

$$I_{st.eff} := \frac{1}{12}\cdot\left(b_p\cdot t_p^3 + b_{we3}^3\cdot t_w + b_{we1}^3\cdot t_w + b_f\cdot t_s^3\right) ... + b_p\cdot t_p\cdot\left(y_{st} - \frac{t_p}{2}\right)^2 ... + b_{we3}\cdot t_w\cdot\left(-y_{st} + t_p + \frac{b_{we3}}{2}\right)^2 ... + b_{we1}\cdot t_w\cdot\left(-y_{st} + t_p + h_w - \frac{b_{we1}}{2}\right)^2 ... + b_f\cdot t_s\cdot\left(-y_{st} + t_p + h_w + \frac{t_s}{2}\right)^2 = 1.108\times 10^9\cdot mm^4$$

Second moment of area for the effective section of the transverse stiffener

$$e_{max.eff} := \max\left(y_{st.eff}, h_s + t_p - y_{st.eff}\right) = 439\cdot mm$$

Maximum distance from the extreme fibre of the effective stiffener to the centroid of the stiffener

$$e_{min.eff} := \min\left(y_{st.eff}, h_s + t_p - y_{st.eff}\right) = 183\cdot mm$$

Minimum distance from the extreme fibre of the effective stiffener to the centroid of the stiffener

Net section

$$h_{eff\_w.net} := h_s - h_{ls} - h_{co} - t_s = 258\cdot mm$$

Net web height of transverse stiffener

$$A_{s.net} := b_p\cdot t_p + h_{eff\_w.net}\cdot t_w + b_f\cdot t_s = 16461\cdot mm^2$$

Net area of transverse stiffener

$$y_{st.net} := \frac{b_p\cdot\frac{t_p^2}{2} + h_{eff\_w.net}\cdot t_w\cdot\left(t_p + h_{ls} + h_{co} + \frac{h_{eff\_w.net}}{2}\right) ... + b_f\cdot t_s\cdot\left(t_p + h_{ls} + h_{co} + h_{eff\_w.net} + \frac{t_s}{2}\right)}{A_{s.net}} = 187\cdot mm$$

CoG of net stiffener from above plate

$$I_{st.net} := \frac{1}{12}\cdot\left(b_p\cdot t_p^3 + h_{eff\_w.net}^3\cdot t_w + b_f\cdot t_s^3\right) ... + b_p\cdot t_p\cdot\left(y_{st.net} - \frac{t_p}{2}\right)^2 ... + h_{eff\_w.net}\cdot t_w\cdot\left(-y_{st.net} + t_p + h_{ls} + h_{co} + \frac{h_{eff\_w.net}}{2}\right)^2 ... + b_f\cdot t_s\cdot\left(-y_{st.net} + t_p + h_{ls} + h_{co} + h_{eff\_w.net} + \frac{t_s}{2}\right)^2 = 1.091\times 10^9\cdot mm^4$$

Second moment of area for the net section of the transverse stiffener

$$e_{net} := h_s + t_p - y_{st.net} = 435\cdot mm$$

Distance from the underside of the flange plate to the neutral axis of the net section

Minimum requirement for transverse stiffener in accordance with cluase 9.2.1(5)

$$w_0 := \frac{\min(a_1, a_1, b)}{300} = 13.45 \cdot mm$$

Initial imperfection of transverse stiffener

$$u := \max\left(1, \frac{\pi^2 \cdot E_{mod} \cdot e_{max.eff}}{\frac{f_y \cdot 300 \cdot b}{\gamma_{M1}}}\right) = 1.66$$

$$N_{Ed} := \frac{f_y}{\gamma_{M0}} A_{eff} = 53 \cdot MN$$

Compressive force in adjacent panels. Conservatively set equal to capacity of adjacent panels

$$\sigma_{fac} := 1.0$$

Aspect ratio between elastic ctirical stresses for column- and plate-like buckling of adjacent panels. Conservatively set to 1.0

$$\sigma_m := \sigma_{fac} \cdot \left[\frac{N_{Ed}}{b} \cdot \left(\frac{1}{a_1} + \frac{1}{a_2}\right)\right] = 5.48 \cdot MPa$$

$$I_{st.min} := \frac{\sigma_m}{E_{mod}} \cdot \left(\frac{b}{\pi}\right)^4 \cdot \left(1 + w_0 \cdot \frac{300}{b} \cdot u\right) = 3.403 \times 10^8 \cdot mm^4$$

Minimum required stiffness of transverse stiffener

$$\frac{I_{st.min}}{I_{st.eff}} = 0.31$$

$$eq_{9.1} := \text{if}\left(I_{st.eff} \geq I_{st.min}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$$

Limtation of cut-out in transverse stiffener web in accordance with clause 9.2.4(4)

$$cl_{9.2.4\_4} := \text{if}\left[\left(h_{ls} + h_{co}\right) \leq 0.6\left(h_s + t_p\right), \text{"OK"}, \text{"NOT OK"}\right] = \text{"OK"}$$

Shear resistance of the stiffener web in accordance with clause 9.2.4(5)

$$V_{Ed} := \frac{I_{st.net} \cdot f_y \cdot \pi}{e_{net} \cdot \gamma_{M0} \cdot b_G} = 998 \cdot kN$$

Shear force to be resisted by the gross web adjacent to the cut-out

$$k_\tau := \begin{cases} 5.34 + 4 \cdot \left(\frac{h_w}{b_G}\right)^2 & \text{if } \frac{b_G}{h_w} \geq 1 \\ 4 + 5.34 \cdot \left(\frac{h_w}{b_G}\right)^2 & \text{otherwise} \end{cases} = 5.49$$

$$\lambda_{w.s} := \frac{h_w}{37.4 \cdot t_w \cdot \varepsilon_s \cdot \sqrt{k_\tau}} = 0.89$$

$$\chi_w := \begin{cases} \eta & \text{if } \lambda_{w.s} < \frac{0.83}{\eta} \\ \frac{0.83}{\lambda_{w.s}} & \text{if } \frac{0.83}{\eta} \leq \lambda_{w.s} < 1.08 \\ \frac{1.37}{0.7 + \lambda_{w.s}} & \text{otherwise} \end{cases} = 0.93$$

$$V_{bw.Rd} := \frac{\chi_w \cdot f_y \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = 1199 \cdot kN$$

$$eq_{9.5} := \text{if}\left(V_{bw.Rd} \geq V_{Ed}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$$

6 Conclusion

With the given geometry and chosen truss size of ø219x6.3, the maximum utilization member ratio is 0.931 for the truss elements and 0.986 for the upper beam. As discussed in section 4, these high utilizations are acceptable based on the very conservative representation of the support conditions in the Staad-analyses.

Concept development, floating bridge
E39 Bjørnafjorden

Appendix K – Enclosure 4

10205546-13-NOT-085
End of Bridge Girder at Abutment North

MEMO

PROJECT	Concept development, floating bridge E39 Bjørnafjorden	DOCUMENT CODE	10205546-13-NOT-085
CLIENT	Statens vegvesen	ACCESSIBILITY	Restricted
SUBJECT	End of Bridge Girder at Abutment North	PROJECT MANAGER	Svein Erik Jakobsen
TO	Statens vegvesen	PREPARED BY	Dag Ivar Ytreberg / Tore Aas
COPY TO		RESPONSIBLE UNIT	AMC

SUMMARY

This memo summarizes the preliminary design of the end of the Bridge Girder at the Abutment North.

Rev. 1 is an update for the preferred option K12. Old plots no longer relevant are deleted.

1 Introduction

The Bridge Girder is fixed to the Abutment North due to structural and functional reasons. The sectional forces in the Bridge Girder, in this end part of the bridge, have the largest sectional forces in the floating part of the bridge. The end part of the Bridge Girder towards the North Abutment needs therefore to be reinforced compared to the typical section of the Bridge Girder.

2 Sectional forces in Bridge Girder from global analysis version-06

The maximal forces used in the preliminary design was taken from global analysis version-06 for K11. Load combination ULS 3 and ship impact from deckhouse was evaluated. The forces are shown below.

K11-06

K11 ULS 3	N	Mz	My	T	Vz	Vy
N+	78	-1811	15	-25	-11	-8
N-	-78	1763	446	3	-15	7
Mz+	**-32**	2996	**-9**	**-6**	**-10**	**11**
Mz-	29	-3254	581	-12	-18	-12
My+	18	-1730	915	-16	-23	-5
My-	-19	780	-330	-11	-6	4

K11-06

K11 Deckhouse	N	Mz	My	T	Vz	Vy
N+	80	2962	267	14	-12	18
N-	-66	-2286	243	-5	-13	-10
Mz+	**21**	**4319**	**257**	**-1**	**-11**	**33**
Mz-	-42	-4208	242	5	-13	-17
My+	16	3101	308	121	-9	15
My-	-15	-1271	192	-123	-14	-10

For Ship Impact add My=+250 and Vz=-12 for self weight

Figure 2-1 Maximal forces for K11 from global analysis version-06

1	15.08.2019	Final issue	T. Aas	P.N. Larsen	S.E. Jakobsen
0	29.03.2019	Status 2 issue	D.I. Ytreberg	P.N. Larsen	S.E. Jakobsen
REV.	DATE	DESCRIPTION	PREPARED BY	CHECKED BY	APPROVED BY

3 Design

3.1 General design basics

Since the sectional forces are much larger in the north end of the Bridge Girder than in other parts, especially the moment about strong axis, it is reasonable to expand the width of the Bridge Girder towards the end.

It is also necessary to have a larger area in the end section so that it is enough space for all the pre-tensioned tendons that are needed. The pre-tensioned tendons will press the Bridge girder against the Abutment North so that the Bridge Girder will be fixed to the Abutment North in all load combinations.

3.2 The shape of the Bridge Girder

The general Bridge Girder section is used except the last 52 m of the Bridge Girder towards the Abutment North. From 52 m to 48 m from the end the Bridge Girder changes from the general shape to a rectangular shape. From 44 m to 4 m from the end the rectangular shape increases in width from 27 m to 36 m. The last 4 m from the end the shape has the same width of 36 m.

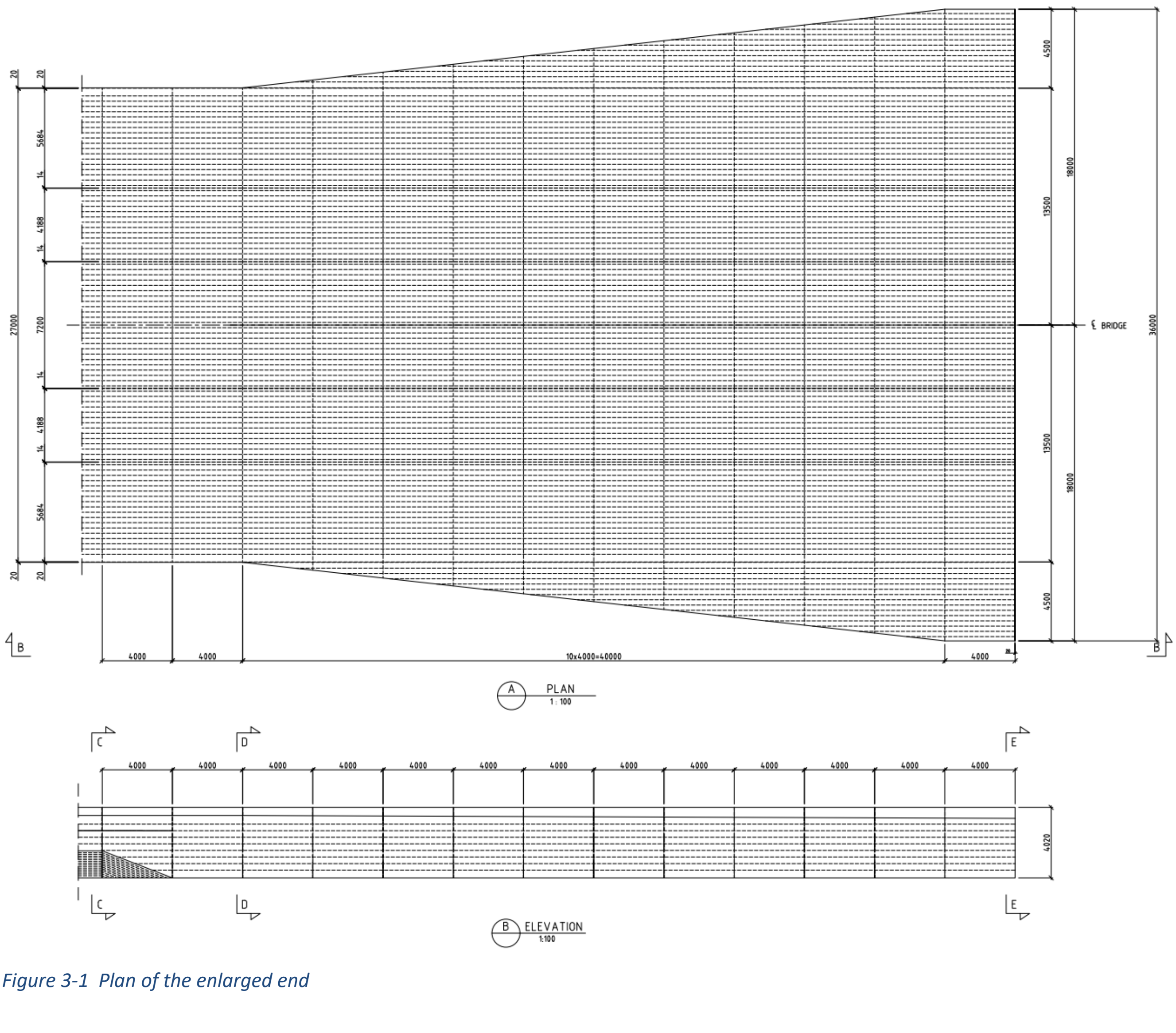


Figure 3-1 Plan of the enlarged end

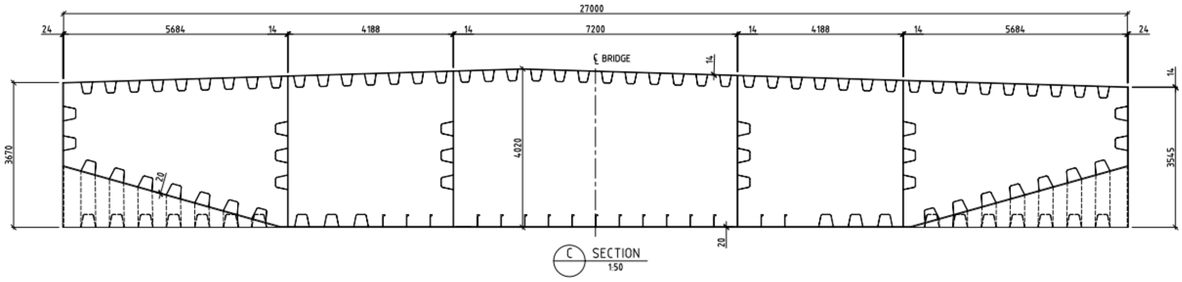


Figure 3-2 Section of Bridge Girder

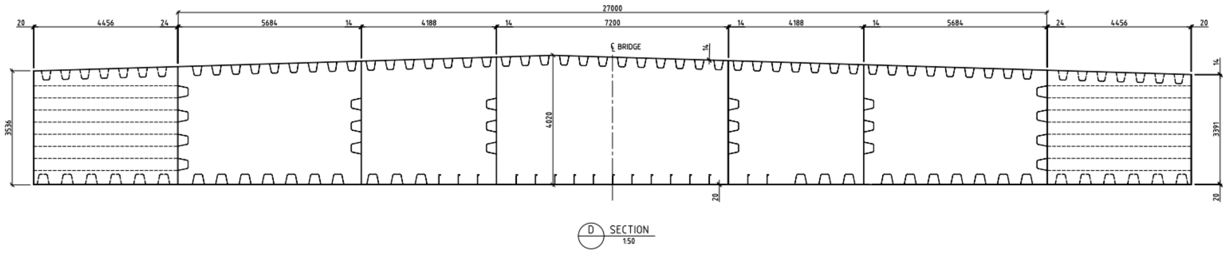


Figure 3-3 Section of Bridge Girder

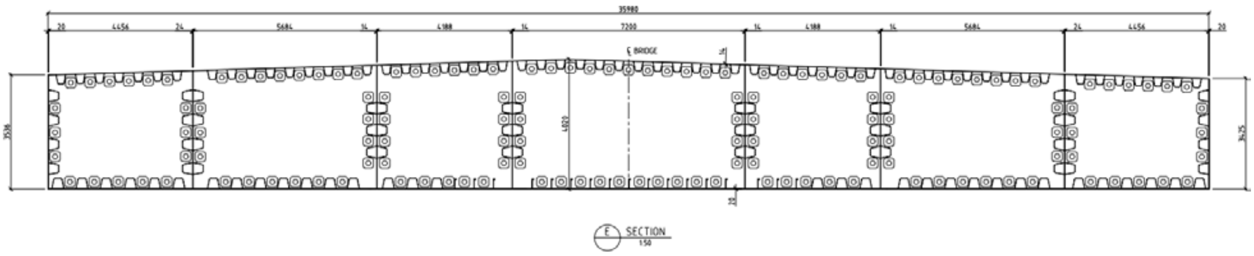


Figure 3-4 Section of Bridge Girder at the end section

3.3 Details in the end section

The end section will support 143 pre-tensioned tendons type 6-22 that will press the Bridge girder against the Abutment North. The minimum distance is 600 mm between each of the tendons. The load in the tendons will be taken by a thick transversal bulkhead, the two nearby stiffeners and the nearest flange or web in the Bridge Girder.

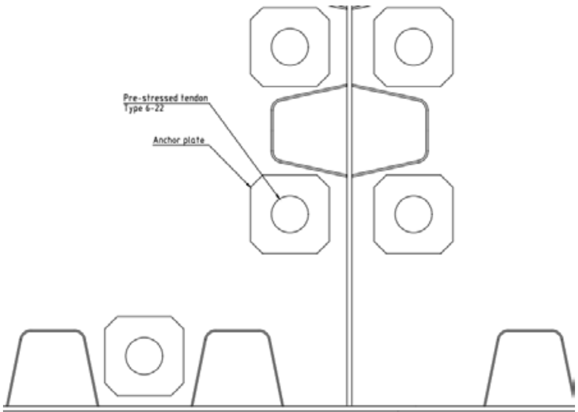


Figure 3-5 Details of end section

4 Sectional forces in Bridge Girder from global analysis version-07

In the latest global analysis version-07 for K11, the bending moments about strong- and weak axis both are increased with approx. 35%. The forces are shown below.

K11-07

K11 ULS 3	N	Mz	My	T	Vz	Vy
N+	64	-1908	-147	-2	-12	-7
N-	-64	1831	660	3	-19	7
Mz+	**-35**	**4393**	**-131**	**52**	**-12**	**15**
Mz-	26	-4336	855	-99	-22	-15
My+	21	-1744	1213	-136	-26	-6
My-	-34	-538	-450	7	-8	-1

K11-07

K11 Deckhouse	N	Mz	My	T	Vz	Vy
N+	78	256	262	23	-10	13
N-	80	3615	267	14	-8	24
Mz+	54	5435	330	25	-4	40
Mz-	-43	-5842	239	-5	-14	-29
My+	41	3091	354	53	5	13
My-	-29	-1977	165	-24	-25	-9

For Ship Impact add My=+250 and Vz=-12 for self weight

Figure 4-1 Maximal forces for K11 from global analysis version-07

The sectional forces for K12, K13 and K14 are also increased in global analysis version-07, but not so much as K11 for moments about strong axis. The table below show the maximal moments about strong axis with associated forces.

Skipsstøt		N	Mz	My	T	Vz	Vy
K11-07	Mz+	57	5498	292	4	-8	38
K11-07	Mz-	-43	**-5842**	239	-5	-26	-29
K12-05	Mz+	52	**4851**	274	6	-10	35
K12-05	Mz-	-39	-3494	211	-10	-13	-30
K13-06	Mz+	31	4774	266	1	-9	36
K13-06	Mz-	-34	-3004	227	-2	-15	-22
K14-06	Mz+	55	4566	267	4	-10	36
K14-06	Mz-	-29	-3514	191	-29	-14	-25

ULS 3		N	Mz	My	T	Vz	Vy
K11-07	Mz+	-35	**4393**	-131	52	15	-12
K11-07	Mz-	26	-4336	**855**	-99	-15	-22
K12-05	Mz+	-30	2393	-8	77	10	-13
K12-05	Mz-	25	-2381	596	-16	-11	-19
K13-06	Mz+	-122	2483	633	44	8	-19
K13-06	Mz-	94	-2407	129	-92	-10	-15
K14-06	Mz+	-15	2744	101	46	12	-14
K14-06	Mz-	16	**-2854**	**702**	-35	-12	-21

Figure 4-2 Maximal forces about strong axis with associated forces