Stretto di Messina

Concessionaria per la progettazione, realizzazione e gestione del collegamento stabile tra la Sicilia
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EurolinK

PONTE SULLO STRETTO DI MESSINA

PROGETTO DEFINITIVO

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INDICE

1 Introduction

1.1 Scope of works

This document describes the different support structures applicable for the suspended deck. The structures are designed primary by hand calculations based on reactions from the global IBDAS FE-model. Further more spreadsheets and the local FE-models described in "General Design Principles for Suspended Deck" are used for verifying more complicated connections and supports.

2 Hanger Anchorages

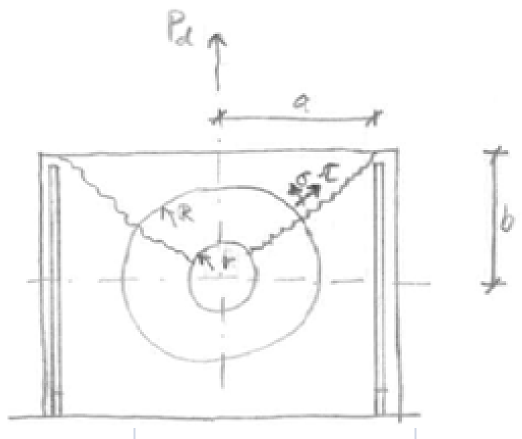
In this chapter, the pull-out strength for anchor and cheek plates in hanger anchorage AP1 to AP6, is verified. This is done in addition to the local FEM-verification of the hanger anchorage in "Design Report - Local FE-models of Suspended deck". This verification is carried out, for the maximum axial hanger forces at the lower hanger anchorage, determined in the global IBDAS model. The relevant load combination used for this verification is the ULS QL load combination 6552, which is specified as combination 2 in "Design Basis" Table 24b. The relevant hanger forces are illustrated in Table 2-1.

Table 2-1 ULS hanger reactions from the global IBDAS model

Hanger no.	**Hanger Anchorage**	**S-coordinate [m]**	**ULS QL [MN]**
60	AP1	0.0	6.13
41	AP2	-570.0	6.02
20	AP3	-1200.0	6.04
7	AP4	-1590.0	6.66
6	AP5a	-1620.0	10.42
5	AP5b	-1680.0	14.13
2	AP6	-1770.0	7.98

Material safety factor γ_{M0} = 1.05.

The geometry of the yielding lines is shown in Figure 2-1.



T = $t_{anchorplate}$ + 2 x $t_{cheekplate}$

t = $t_{anchorplate}$

Figure 2-1 Sketch defining yielding lines for pull-out strength

Table 2-2 Verification of the pull-put strength for the hanger anchorages

Hanger	**AP1**	**AP2**	**AP3**	**AP4**	**AP5a**	**AP5b**	**AP6**
Pd [MN]	6.13	6.02	6.04	6.66	10.42	14.13	7.98
a [mm]	380	380	380	480	730	730	730
b [mm]	300	250	250	360	470	360	470
R [mm]	270	200	200	280	380	280	380
r [mm]	156	97	103	122	171	122	171
T [mm]	120	160	190	230	260	240	260
t [mm]	60	60	70	90	100	100	100
As [mm2]	26529	31772	36270	65140	103162	91314	103162
σ [Mpa]	91	79	70	41	42	69	33
τ [Mpa]	72	52	46	31	27	34	21
σ_{VM} [Mpa]	154	120	105	67	64	91	49
f_{yd} [MPa]	438	438	438	438	438	438	438
UR	0.35	0.27	0.24	0.15	0.15	0.21	0.11

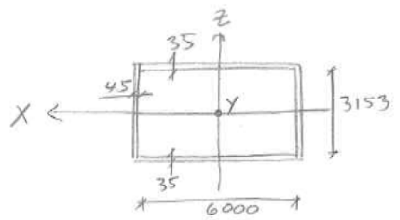
As shown in Table 2-2 the utilisation ratios for the applied hanger forces in Table 2-1 the hanger anchorages have sufficient pull-out capacity.

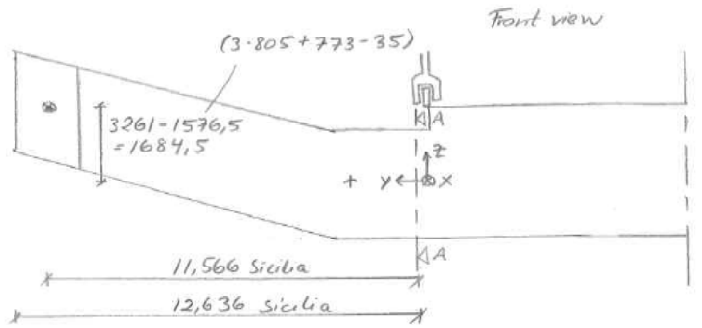
3 Buffers, Lateral Restraints and Bearings at Towers

3.1 Cross Girder Extensions

The connection between the tower and deck is made though a system of buffers. The group of buffers connected to the cross girder T4a is able to transfer a maximum force of 40MN. In the following calculation it has been assumed this the maximum force and a vertical displacement of in 1.5m occurs at the same time. This is a conservative approach. Stiffeners to the skin plates have not been taken in consideration for the capacity of the section, but their position has been studied to get the full effective plates in accord to EN1993-1-5:2006.

For the connection of the buffers to the extended cross girder, reference is made to the calculations made for the towers as they have exact the same geometry.





Section A-A properties:

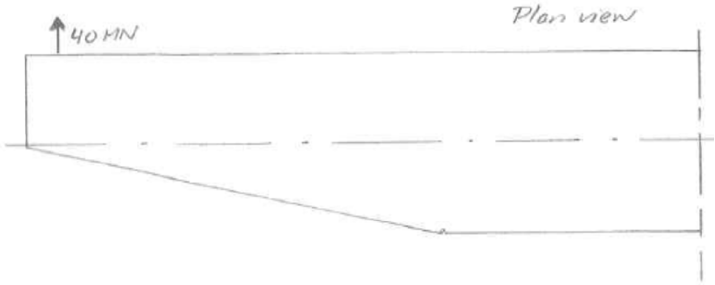
(A with stiff.) $A = 807371 \ mm^2$

$I_x = 1{,}445 \cdot 10^{12} \ mm^4$

$I_z = 4{,}328 \cdot 10^{12} \ mm^4$

$\rho_x = 1611{,}5 \ mm$

$\rho_z = 3045 \ mm$



$M_y = 40 \ MN \cdot 1{,}6845 m = \underline{67{,}38 \ MNm}$

$M_z = 40 \ MN \cdot 11{,}566 \ m = \underline{-462{,}64 \ MNm}$

F_v is transferred in the web

$\alpha = atan\left(\frac{1{,}5}{16}\right) = 0{,}093 \ rad$

Force component in vertical direction:

$F_v = \sin\alpha \cdot 40 \ MN = \sin(0{,}093) \cdot 40 = 3{,}71 \ MN$

$M_x = 11{,}566 \ m \cdot 3{,}71 \ MN = \underline{-43{,}0 \ MNm}$

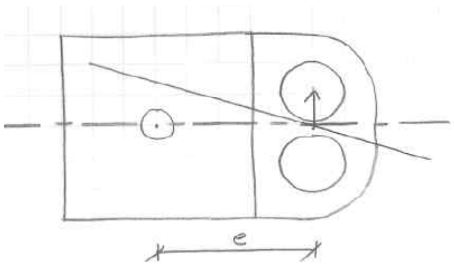
Dead load:

$G = A \cdot \rho_{steel} = 0{,}71 \ m^2 \cdot 0{,}077 \ MN/m$

$G = 0{,}055 \ MN/m$

$M_{x,G} = \frac{1}{2} G \ell^2 = \frac{1}{2} \cdot 0{,}055 \ MN/m \cdot 12{,}636^2 = \underline{4{,}36 \ MNm}$

$M_{y,1,Fv} = 3{,}71 \ MN \cdot \frac{6{,}000 \ m}{2} = \underline{11{,}13 \ MNm}$



$e = 2766\ mm$

$M_{y,2,Fv} = 3{,}71\ MN \cdot 2{,}77\ m = \underline{10{,}3\ MNm}$

$\sigma_{y,Mx} = \frac{M_{x,G} + M_y}{I_x} = \frac{4{,}36 + 43{,}0}{1{,}445} \cdot 1{,}6 = \underline{52{,}8\ MPa}$

$\sigma_{y,Mz} = \frac{M_z}{I_z} \cdot y_z = \frac{462{,}64}{4{,}328} \cdot 3{,}05 = \underline{325{,}5\ MPa}$

Shear due to torsion:

$\tau_t = \frac{M_y + M_{y,1,Fv} + M_{y,2,Fv}}{2 A_m t} = \frac{67{,}38 + 11{,}13 + 10{,}3}{2 \cdot (6{,}000 + 0{,}045) \cdot (3{,}153 + 0{,}035) \cdot t} = \begin{cases} t = 35\ mm: \underline{65{,}8\ MPa} \\ t = 45\ mm: 51{,}2\ MPa \end{cases}$

Shear:

Shear from the load of 40 MN is taken by the flanges:

$\tau_x = \frac{40}{2 \cdot 6{,}090 \cdot 0{,}035} = \underline{93{,}8\ MPa}$

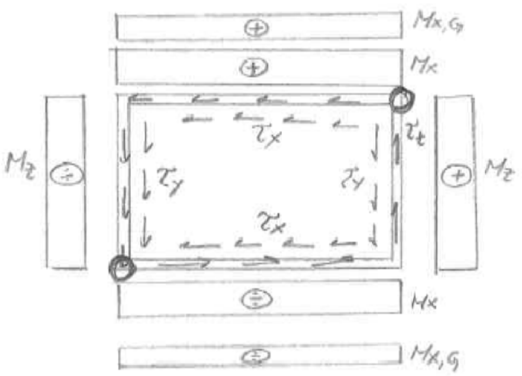
Shear from dead load and the force component F_v, is taken by the webs:

$\tau_y = \frac{0{,}055\ MN/m \cdot 12{,}636\ m + 3{,}71\ MN}{2 \cdot 3{,}153 \cdot 0{,}045\ m^2} = \underline{15{,}5\ MPa}$

Check of cross section:

$$\sqrt{\sigma_y^2+3\tau^2} = \sqrt{(52,8+325,5)^2+3\cdot(65,8+93,8)^2} = 468,5\,MPa$$

$\gamma_{M0} = 1,0$ (ALS)



$$UR = \frac{\sigma_{Ed}}{f_y/\gamma_{M0}} = \frac{468,5}{460/1,0} = \underline{1,02}$$

The linear stress utilisation ratio is 1.02 in the shown points. In this accidental load case this exceeding of yield stress is within a reasonable level for local plastic redistributing of stresses.

3.2 Corbels for Support of Drop in Span

At the location of bearing A3 a reaction of F=8.8 MN is transferred to the corbels for supporting the drop-in span. The corbels will transfer a moment and a shear force into the cross section of the cross girder T4a and T4b. These forces need to be absorbed locally in the section - and since the plate thicknesses in the cross girder are the same, see Figure 3-1, the section here has been verified for the bearing reaction.

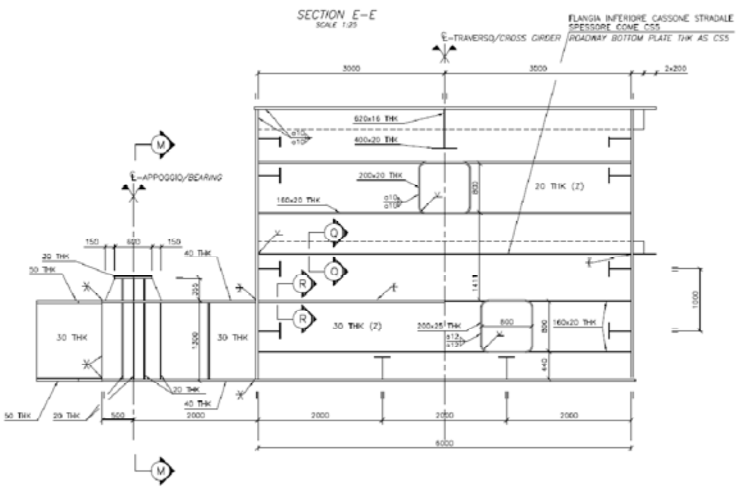
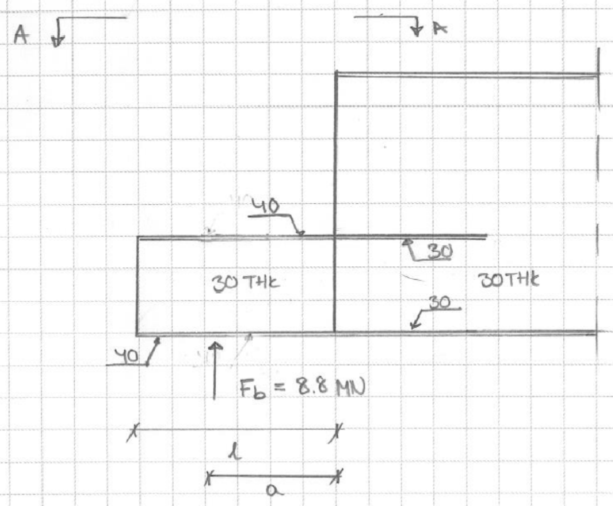
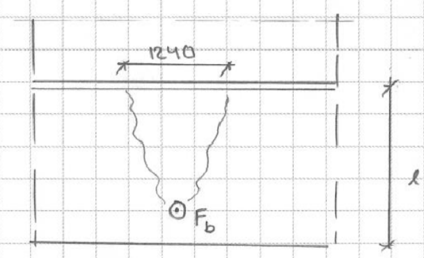


Figure 3-1 Corbels at drop-in span

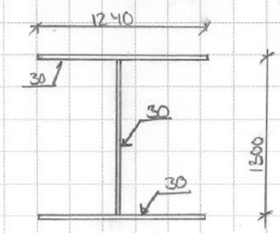
The cross section properties has been calculated based on the assumption that the point load from the bearing is distributed over a distance of 1240 mm defining the I-beam that needs to carry the load, see illustrations below:



Section A-A (with distribution)



Cross section:



Properties:

A = 0.113 m² ; A_{web} = 0.039 m²

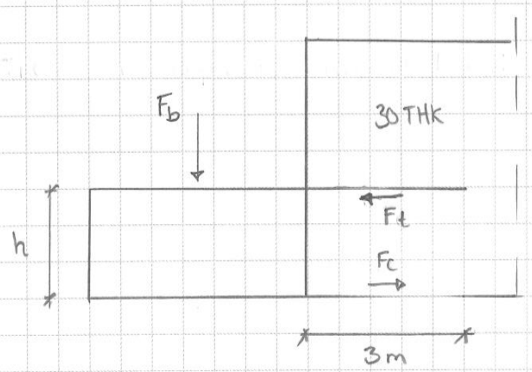
I = 0.031 m⁴ (flanges only)

Forces: $M = F_b \times a$, $a = 2000\ mm$ $\Rightarrow$ $M = 17.6\ MNm$, $V = 8.8\ MN$

Stresses: $\tau = \frac{V}{A_w} = \frac{8.8\ MN}{0.039\ m^2} = 225\ MPa$ ok!

$$\sigma_M = \frac{M}{I} \times z = \frac{17.6\ MNm}{0.031\ m^4} \cdot 0.65\ m = 370\ MPa \quad ok!$$

The bending moment introduced will act as a force couple in the top and bottom flange of the local section considered. The force in the top plate needs to be transferred as shear in the cross girder diaphragms. The diaphragm has a thickness of $t = 30\ mm$ (se figure above). The force needs to be distributed over a length of 3m.



$$F_t = F_c = \frac{M}{h} = \frac{17.6\ MNm}{1.3\ m} = 13.5\ MN$$

Half the force F_t needs to be transferred over a shear area of:

$$A = 3.0\ m \times 0.03\ m = 0.09\ m^2$$

The shear stress in the diaphragm from this local effect becomes:

$$\tau = \frac{F_t/2}{A} = \frac{6.77\ MN}{0.9\ m^2} = 75\ MPa < \frac{f_{yd}}{\sqrt{3}} = 253\ MPa \quad ok!$$

3.3 Support Arrangement and cut out for Expansion Joint

The end of the 60m roadway drop-in span, is designed to accommodate the bearing reactions (A1 and A2) and geometrical requirements of the expansion joint. The global ULS bearing reactions and movements are applied according to the results from the global IBDAS model.

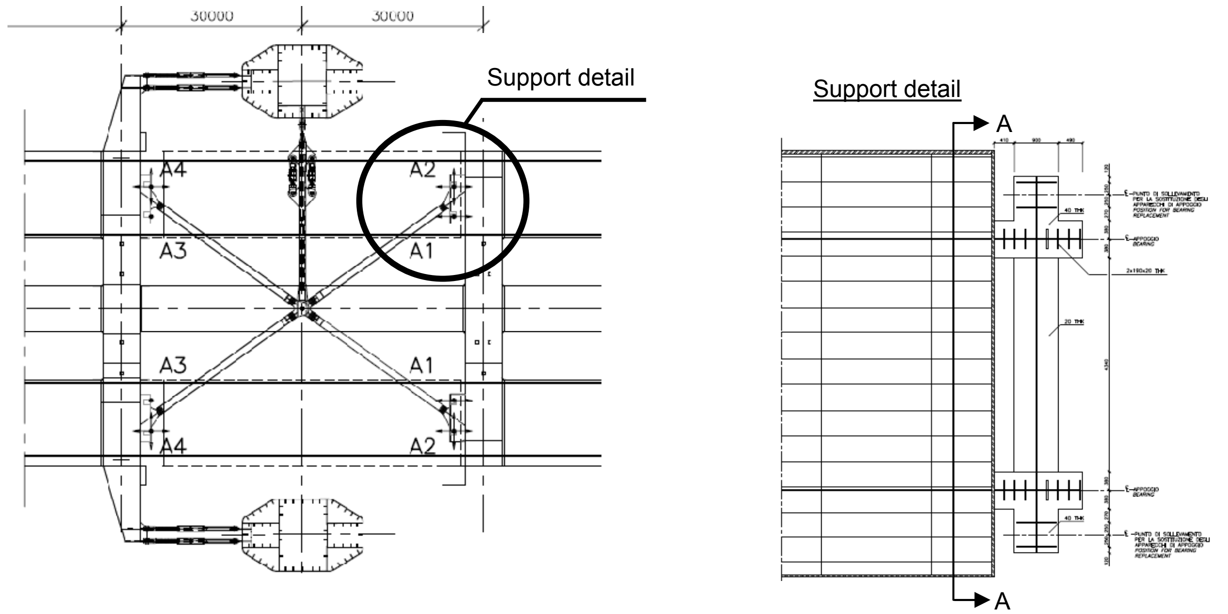


Figure 3-2 Plan view and support detail of 60m drop in span

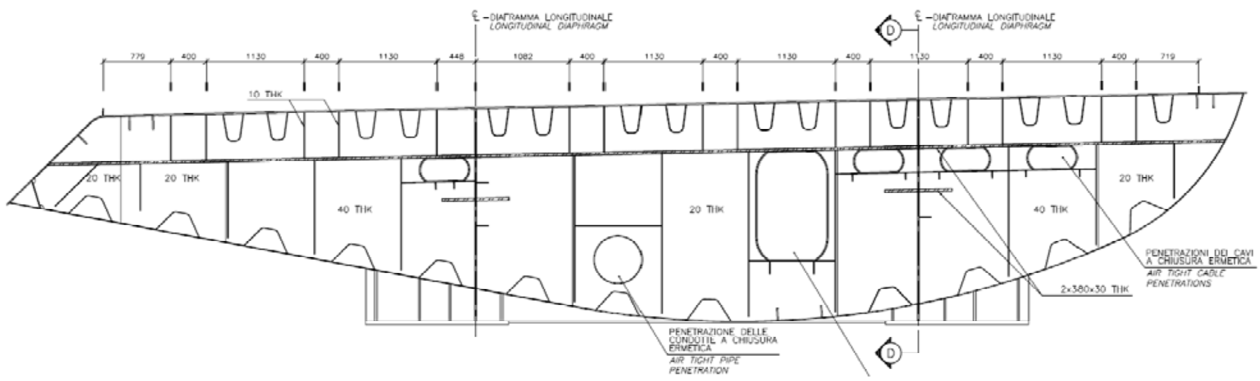


Figure 3-3 Section A-A at cut out for expansion joint

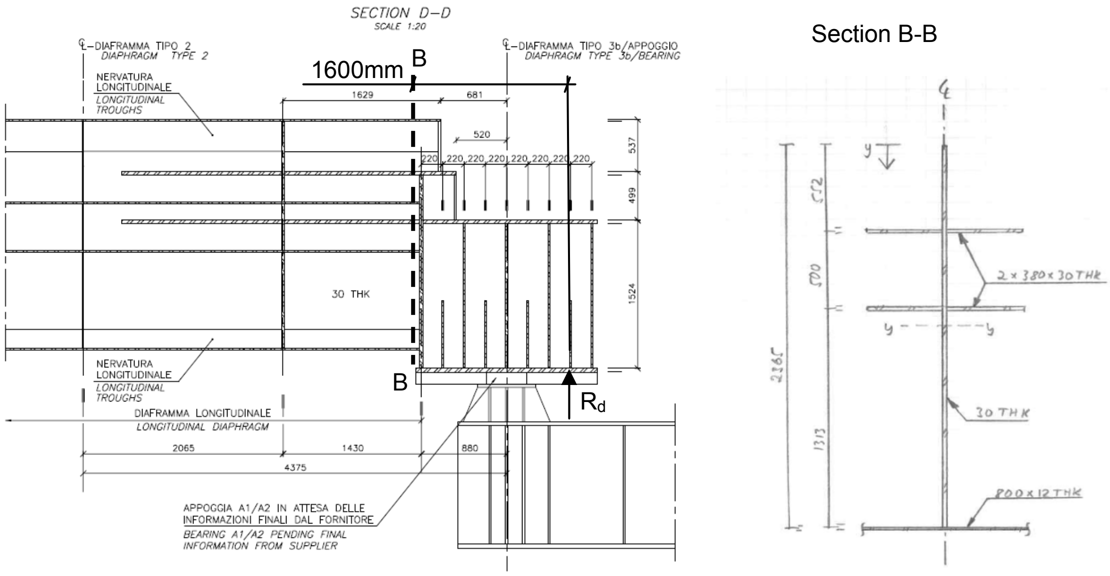


Figure 3-4 Section D-D and section B-B at cut out for expansion joint

The maximum movement of the bearing in longitudinal direction is U_{ULS} = +/- 700 mm. The vertical ULS action from the bearing according the global IBDAS model is R_d = 8.8 MN, which gives the section forces in section B-B:

Vd = 8.8 MN, Md = Vd x 1.60m = 14.1 MNm

Section properties and verification is shown in Figure 3-5

Steel area:

$$A_s = 2 \cdot 2 \cdot 380 \cdot 30 + 2365 \cdot 30 + 800 \cdot 12 = \quad = \underline{126 \cdot 10^3\ mm^2}$$

Distance to centre of gravity:

$$y = \frac{2 \cdot 380 \cdot 30 \cdot 552 + 2 \cdot 380 \cdot 30 \cdot 1052 + 2365^2 \cdot 30 \cdot 0{,}5 + 12 \cdot 800 \cdot 2365}{A_s} =$$

$$= \underline{1136\ mm}$$

Moment of inertia:

$$I_y = \frac{2365^3 \cdot 30}{12} + 2365 \cdot 30 \cdot \left(\frac{2365}{2} - 1136\right)^2 +$$

$$+ \frac{2 \cdot 2 \cdot 380 \cdot 30^3}{12} + 2 \cdot 380 \cdot 30 \cdot (1136 - 552)^2 +$$

$$+ 2 \cdot 380 \cdot 30 \cdot (1136 - 1052)^2 + \frac{800 \cdot 12^3}{12} +$$

$$+ 800 \cdot 12 \cdot (2365 - 1136)^2 = \quad = \underline{55{,}7 \cdot 10^9\ mm^4}$$

$$W_{min} = \frac{I_y}{2365 - y} \quad = \underline{45{,}3 \cdot 10^6\ mm^3}$$

$$\sigma_{max} = \frac{Md}{W_{min}} = \quad = \underline{311\ MPa}$$

$$\tau_{max} = \frac{Vd}{2365 \cdot 30} = \quad = \underline{124\ MPa}$$

$$\sigma_{vm} = \sqrt{\sigma^2 + 3\tau^2} = \quad = \underline{378\ MPa}$$

$$\le \frac{460}{1{,}05} = 438\ MPa$$

ok!

Figure 3-5 Section properties and verification of cross section B-B

3.4 Vertical Support of Triangular Strut Structure

At the centre of the 60 m railway span, a vertical support is located for the triangular strut structure (D2). The vertical support is shown in Figure 3-6. The global ULS bearing reactions and rotations are applied according to the results from the global IBDAS model.

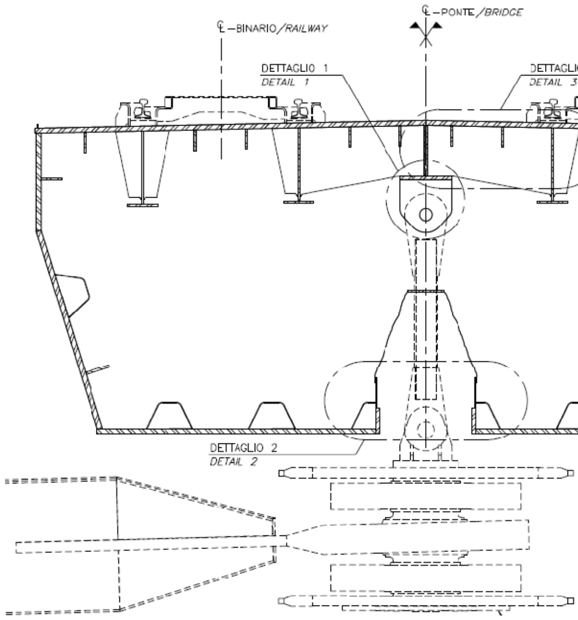


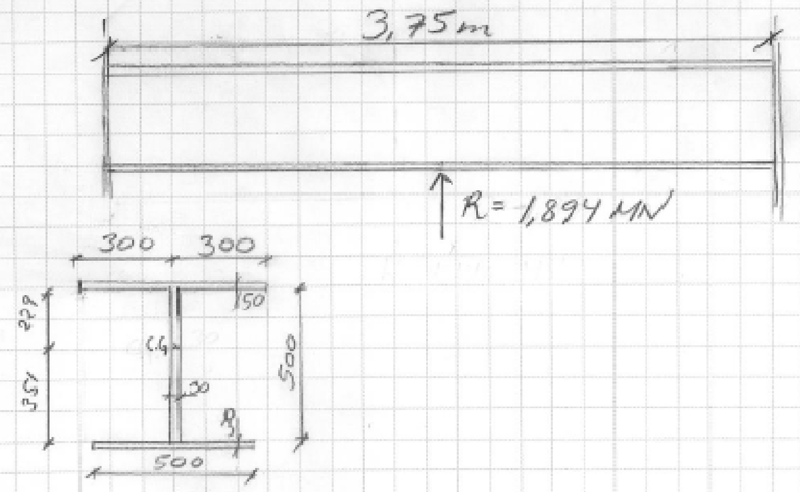
Figure 3-6 Vertical support of triangular strut structure

The loads in the transverse support are calculated in IBDAS and given in Table 3-1. It is seen that the maximum force is a lifting force of -1.89 MN.

Table 3-1 Loads in transverse support

ULS	c3d0 (static - free/free)				c1d0 (static - fixed/fixed)				c3d2 (static response spectrum - free/free)				c1d2 (static response spectrum - fixed/fixed)			
	Sicila		Calabria		Sicila		Calabria		Sicila		Calabria		Sicila		Calabria	
case	max [MN]	min [MN]	max [MN]	min [MN]	max [MN]	min [MN]	max [MN]	min [MN]	max [MN]	min [MN]	max [MN]	min [MN]	max [MN]	min [MN]	max [MN]	min [MN]
1	-0.570		-0.570		-0.570		-0.570		-0.570		-0.570		-0.570		-0.570	
6900	-0.007	-0.919	-0.008	-0.925	0.044	-1.066	0.035	-1.063								
6941	-0.072	-0.916	-0.070	-0.930	-0.054	-1.022	-0.632	-0.817								
6902									0.286	-1.303	0.192	-1.215	0.801	-1.894	0.756	-1.856
temp	0.035	-0.057	0.035	-0.057	0.224	-0.340	0.230	-0.349								

In the following the normal stresses are calculated in the top and the bottom of the T-beam for tension and compression, respectively. A part of the deck plate corresponding to 10 times the plate thickness to each side of the cross section is calculated as being a part of the effective cross section.



$M_{max} = -\frac{1}{4} \cdot 1{,}894 \cdot 3{,}75 = -1{,}776\ MNm$

$I = 0{,}00334\ m^4$

$y = 351\ mm$

The negative bending moment gives the following compression stress and tension stress in the bottom and top of the cross section, respectively.

$\sigma_c = \frac{-1{,}776\ MNm}{0{,}00334\ m^4} \cdot 0{,}351\ m = \underline{187\ MPa}$

$\sigma_t = \frac{1{,}776\ MNm}{0{,}00334\ m^4} \cdot 0{,}229\ m = \underline{122\ MPa}$

The maximum shear force is:

$Q = 0{,}5 \cdot 1{,}894\ MN = 0{,}947\ MN$

$\tau = \frac{0{,}947\ MN}{(0{,}5 \cdot 0{,}03)\ m} = 63\ MPa$

von Mises $\sigma_{v.M} = \sqrt{187^2 + 3 \cdot 63^2} = \underline{217\ MPa}$

It is seen that the stress level is acceptable using steel S355 since 217 MPa < 338 MPa.

4 Bearings and Expansion Joint at Terminal Structures

4.1 Introduction

The suspended deck structure is supported on the terminal superstructures at the bearings A5, A8, A9, A10 and A11, see Figure 4-1.

Three bearings (2xA5 and A8) are located at the entry of the terminal structure underneath the last cross girder, where the roadway girders end. The railway girder is however protruding longer into the terminal structure and bearings for vertical loads are located at two locations - bearings 2xA9 and A10/A11 at the end of the railway girder.

In the transverse direction the railway and road girder is guided by bearings A11 and A8. The two bearings A5 and A9 are furthermore experiencing high uplift forces.

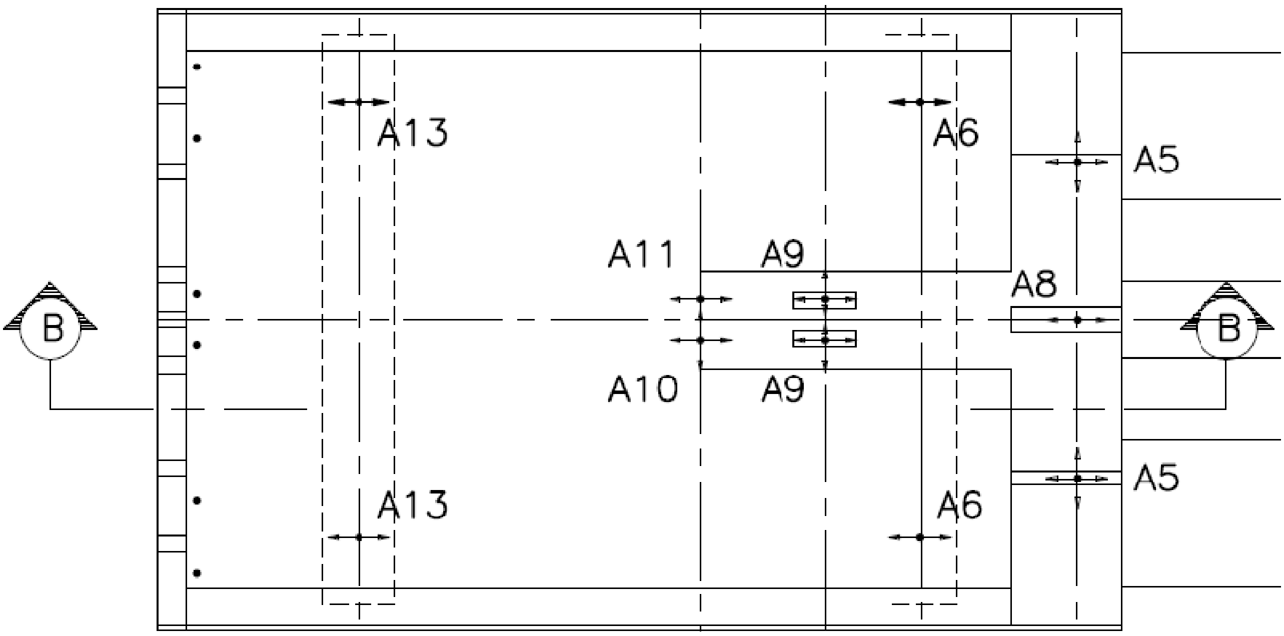


Figure 4-1 Bearing layout at terminal structure on Sicilia side

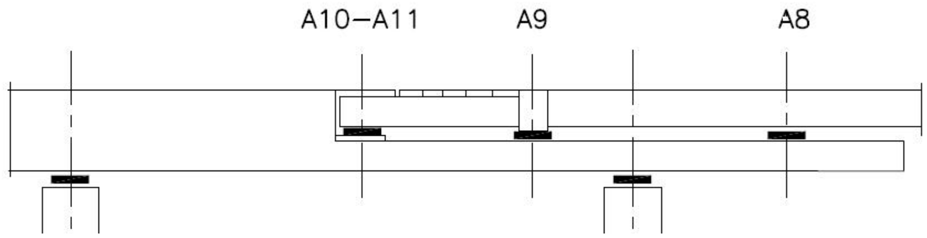


Figure 4-2 Bearing layout at terminal structure on Sicilia side - side view

4.2 Bearings at Entry of Terminal Structure

4.2.1 Bearings at Railway Extension

The bearings at the railway girder extension are located in two lines due to comfort requirements of the railway track line. The bearings are designed to accommodate the reactions including lifting forces and geometrical requirements according to the results from the global IBDAS model.

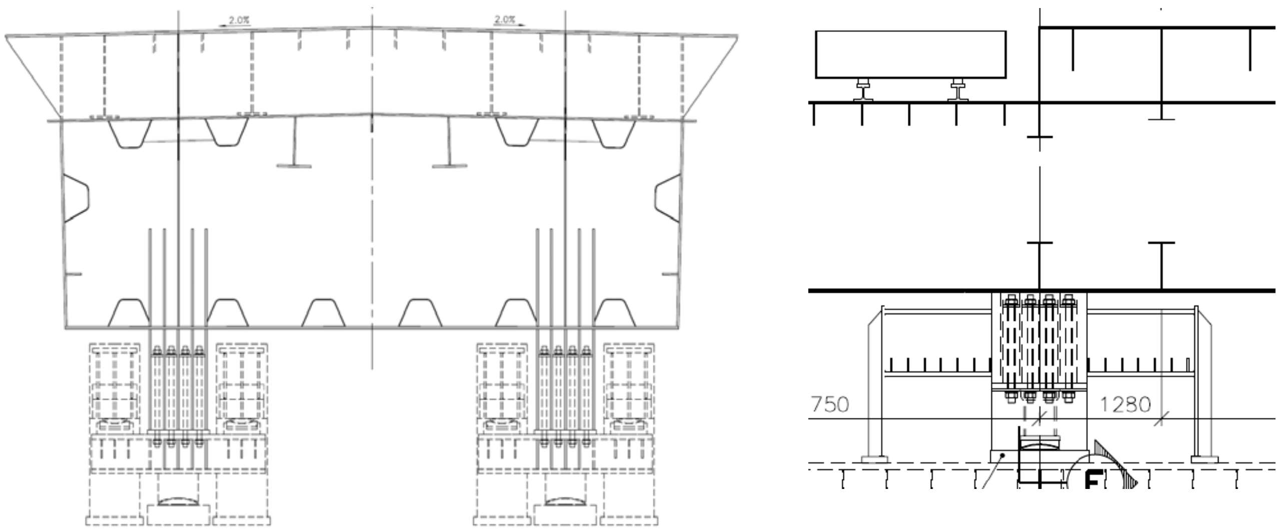
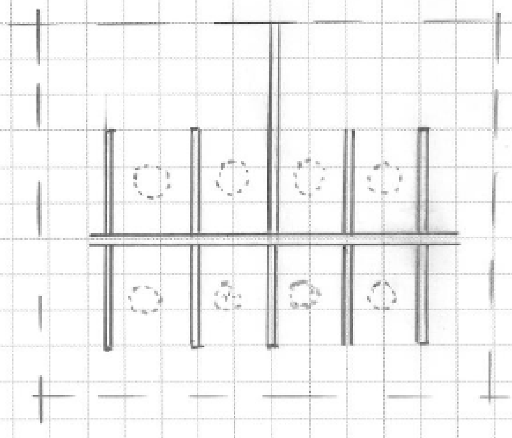


Figure 4-3 Bearing 2 x A9 at railway extension with anti lifting device

Bearing A9 is verified in the following.

At bearing location A9 the bearing structure is connected to the railway girder in a bolted joint. The stiffening plates in the girder act as backing for the bearing reaction $F_b = 5.8$ MN (Lift).
In the following the size of bolts and stiffeners are verified.

In the joint is a total of 8 bolts located as illustrated below: (see also attached figures)



$$N_b = \frac{5.8\ \text{MN}}{8} = 0.73\ \text{MN} \quad (F_{t,R})$$

The bolts are of type 10.9 having

$f_u = 1000$ MPa ; $f_y = 900$ MPa

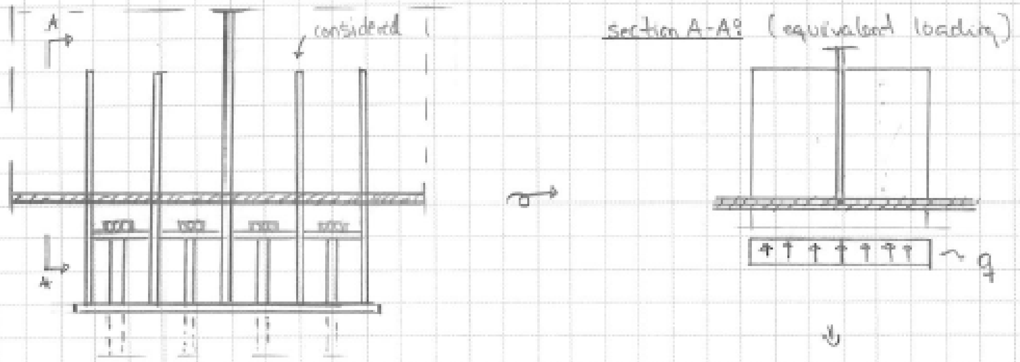
The tension strength of the bolt is $F_{t,R} = 0.9\, f_{ubd} \cdot A$

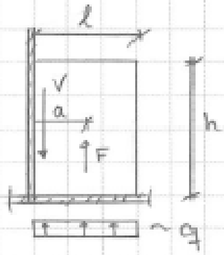
$\gamma_{m2} = 1.25 \Rightarrow f_{ub,d} = 800$ MPa

$$A = \frac{F_{t,R}}{0.9 \cdot f_{ubd}} = \frac{0.73\ \text{MN}}{0.9 \cdot 960\ \text{MPa}} = 0.001\ \text{m}^2 = 1006\ \text{mm}^2 \Rightarrow d = 35.8\ \text{mm}$$

Thus 8 × M42 bolts are used, having a $d_i = 36.5$ mm.

At the bearing location A9 also a large compressiv force occurs of $F_{b,c}$ = 8,9 MN (ULS). The backing plates has been verified for the shear and bending introduced by this compressiv force, see below.





The bolts are supported by plates introducing a uniform load in the backing plates as illustrated.

This load q has to be taken as a shear force V at the connection to the diaphragm, and due to the excentricity of the connection a local moment will be introduced $M = F \cdot a$, where $F = q \cdot l$; $V = F$

The distributed load in the considered plate is equal to the load $F_{b,c}$ distributed over the 5 stiffening panels (10 local panels as illustrated) ⇒ F = 0.89 MN

The plates have the following properties: h = 550 mm l = 300 mm t = 14 mm

$A = 7700\ mm^2$; $I = 1.94 \cdot 10^8\ mm^4$, $\tau = \frac{V}{A} = \frac{0.89 MN}{0.0077 m^2} = 115.6\ MPa$

$\sigma_M = \frac{M}{I} \cdot \frac{h}{2} = 189.4\ MPa$

σ_{VM} = 275 MPa < 355 MPa ok!

Note: It has conservatively been assumed that the diaphragm does not take any load.

COWI

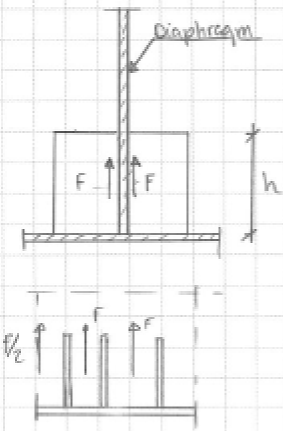
The backing stiffeners are welded to the diaphragm, thus meaning that the diaphragm needs to withstand the shear force from in total 4 of the local panels $\Rightarrow$ $F_{dia} = 0.89$ MN

The diaphragm is of steel grade S355 and has a thickness of 10mm.

The stress in the diaphragm from the local loading then becomes:

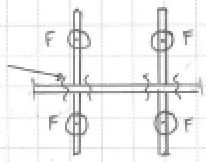
If h = 550 the stress becomes:

$$\tau = \frac{F_{dia}}{h \times 10mm} = 161.8 \text{ MPa} < \frac{355}{\sqrt{3}} = 195 \text{ MPa} \quad \text{Ok!}$$



On the drawings the backing plates are indicated with a height of ~ 900 mm. However these are to be changed to 550 mm.

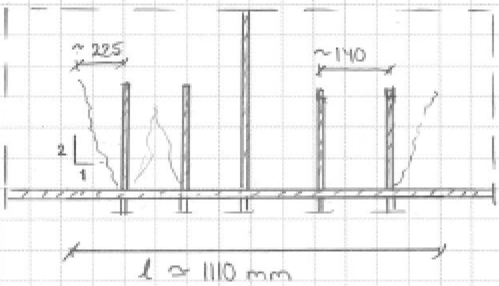
Section in the diaphragm to transfere F as shear



The compressive force from the bearing reaction in the backing plate needs to be transferred as a compressiv normal force.

A distribution of 1:2 of the force from the outer plates is assumed in the calculation, see illustration:

The length of distribution is then determined to ≈ 1110mm.



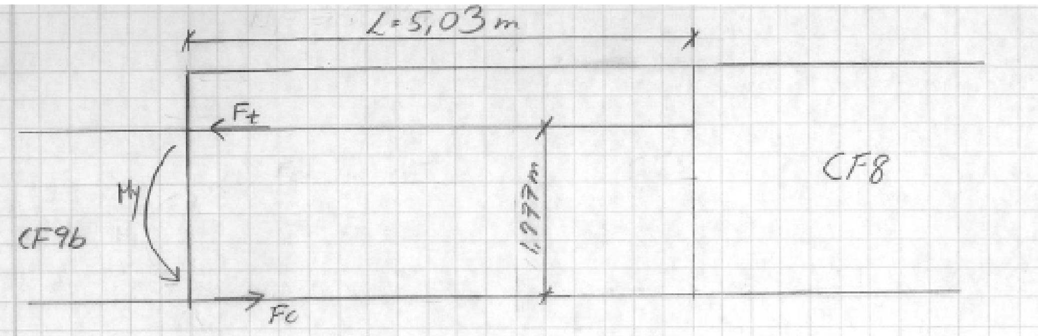
Assuming the diaphragm is changed to steel grade S460 the needed thickness is calculated. The compressiv force is $F_{b,c} = 8.9$ MN

$$f_{yd} \geq \frac{F_{b,c}}{\ell \cdot t} \Rightarrow t \geq \frac{F_{b,c}}{f_{yd} \cdot \ell} = 0{,}018 \Rightarrow t = 20\,\text{mm}$$

The diaphragm is then changed to steel grade S460 and a thickness of t = 20 mm. This are to be included on the drawings.

Drawings will be corrected according to the calculations above. The height of the stiffeners is changed from 900 mm to 550 mm. The thickness of the diaphragm is changed to 20mm and the steel grade is S460.

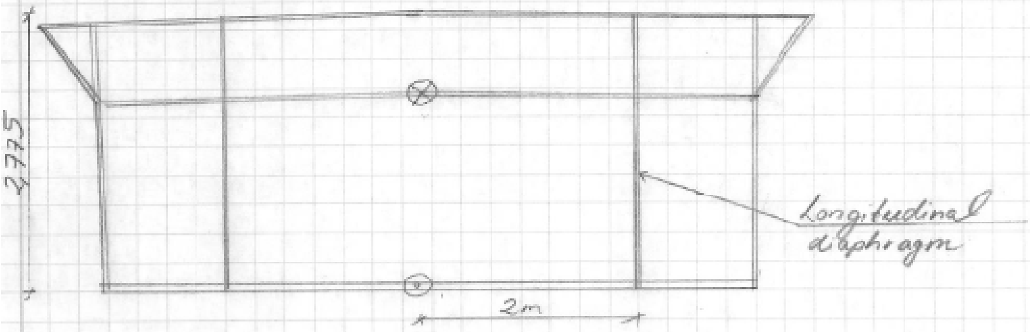
The bending moment at bearing A9 is taken from IBDAS in s-coordinate -1842.38. The minimum bending moment is My = -18.7 MNm. The bending moment is transferred as a force couple in the deck plate and bottom plate as shown in the sketch below.



The bending moment M_y is divided in a force couple transfered in the deck plate and bottom plate as a tension and compression force couple:

$M_y = -18{,}7$

$F_t = F_c = \frac{-18{,}7}{1{,}977} = \underline{9{,}5\ MN}$



The normal stress is calulated in the deck plate:

$\sigma_N = \frac{4{,}75\ MN}{(2 \cdot 0{,}016)m^2} = \underline{148\ MPa}$

The normal force is transferred as shear in the longitudinal diaphragms over a length of 5,03m. The shear stress in the diaphragm becomes:

$\tau = \frac{4{,}75\ MN}{(5{,}03 \cdot 0{,}012)m^2} = \underline{79\ MPa}$

$79\ MPa < \frac{f_{yd}}{\sqrt{3}} = 195\ MPa.$

In Figure 4-4 a figure of bearing A10 and A11 is shown. The bearings are verified in the following.

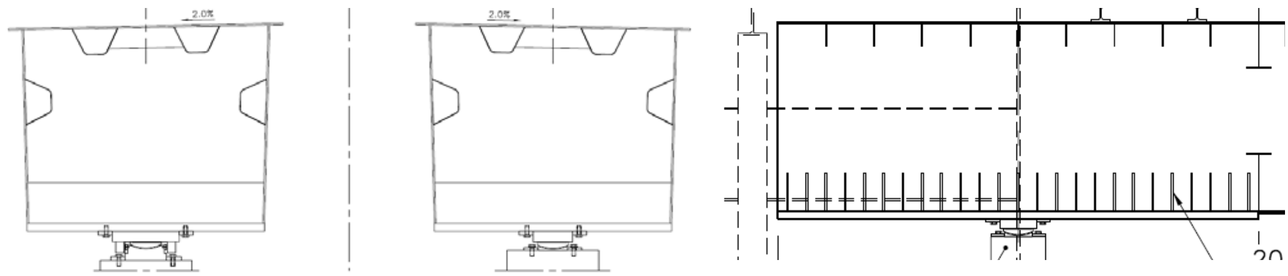


Figure 4-4 Bearing A10 and A11 at end of railway extension

At the location of bearing 10 and 11 the reaction from these on the railway girder bottom plate has been considered in the following.

The loads from the bearing has been calculated to $F_V = 2.5$ MN ; $F_T = 5.0$ MN.

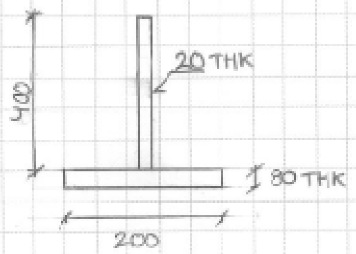
The railway girder is at this location stiffened by plates with a spacing of 200 mm and dimensions as given on the figure below.

The bottom plate has a thickness of 80 mm.

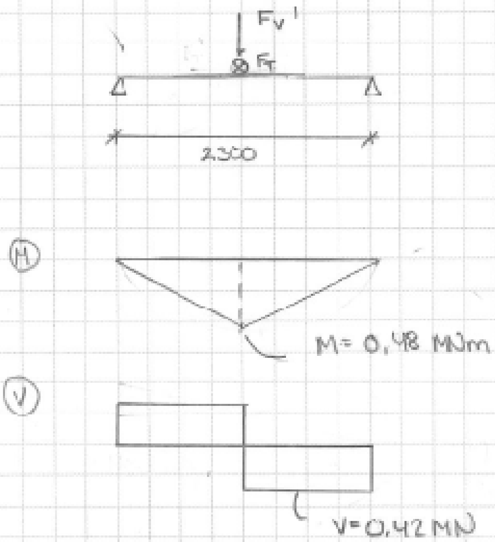
As it can be seen on the figure below the load from the bearing is directly transferred to 3 stiffeners (conservativ consideration).

One stiffener is considered with contributary bottom plate and 1/3 of the bearing load. The equivalent T-beam is considered as simple supported between the webs of the railway girder.

Cross section:



Statical system:



Applied loads:

$F_V' = 0{,}83$ MN

$F_T' = 1{,}67$ MN

Properties:

$A = 400 \times 20 + 200 \times 80 = 24000\ mm^2$

$S = 400 \times 20 \times 380 + 200 \times 80 \times 40 = 2880000\ mm^3$

$y = 120\ mm$

$I = \frac{1}{12} \cdot (20 \times 400^3 + 200 \times 80^3) + 400 \times 20 \times (480 - y)^2 + 80 \times 200\,(y - 80/2)^2 = 4.2 \cdot 10^8\ mm^4$

Due to the location of the neutral axis is close to the bottom plate no local effect of the transverse force F_T has been accounted for.

The stresses in the beam becomes:

$\sigma_M = \frac{M}{I} \times (480 - y) = 409\ MPa$

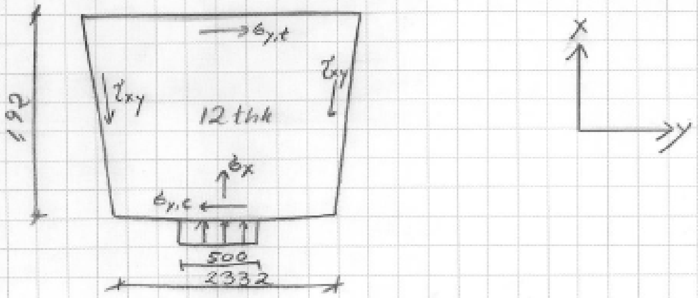
$\tau = \frac{N}{A} = 17.5\ MPa$

$\sigma_{VM} = 410\ MPa < 438\ MPa$ ok!

* Note: The transverse force in the bearing F_T will be taken in the bottom - this additional stress of approx. 110 MPa here will not have any significant effect since the stress from bending in the bottom plate is around 100 MPa. $\sigma_{VM} \approx 190\ MPa$

It should be noticed that the steel grade is changed to S460.

The stresses in diaphragm type 5c at the railway extension are calculated in the following. The reaction force at the bearing is 2.5 MN. It is assumed that one third of the reaction force in the bearing is transferred to the diaphragm.



Reaction force at bearing: 2,5 MN

Reaction at diaphragm: $\dfrac{\frac{1}{3}\cdot 2{,}5\ MN}{0{,}5\ m} = 1{,}67\ MN/m$

$M = -0{,}434\ MNm$

$I_y = \frac{1}{12}\cdot 0{,}012 \cdot 1{,}92^3 = 0{,}0071\ m^4$

Normal stress in y-direction:

(tension in top/compression in bottom)

$\sigma_{y,c} = \dfrac{-0{,}434}{0{,}0071}\cdot(1{,}92/2) = -58\ MPa$

Normal stress in x-direction:

$\sigma_x = \dfrac{0{,}83}{2{,}332\cdot 0{,}012} = 30\ MPa$

Shear at edges

$\tau_{xy} = \dfrac{0{,}83\ MN}{2\cdot 1{,}92\cdot 0{,}012} = 18\ MPa$

Von Mises, bottom of plate: $\sigma_{v,bot} = \sqrt{30^2+58^2-30\cdot 58} = 77\ MPa$

Von Mises, edge of plate: $\sigma_{v,edge} = \sqrt{30^2+3\cdot 18^2} = 43\ MPa$

Local contact pressure: $\dfrac{0{,}83\ MN}{0{,}5\,m\cdot 0{,}012\,m} = 138\ MPa$

COWI

The stresses are acceptable for steel S355 since 283MPa < 338MPa.

4.3 Roadway Expansion Joint

The roadway expansion joints are of the modular type with two movable sides. At present the design requirements are only geometrical so that the joint can be constructed and that it can accommodate the ULS movement.

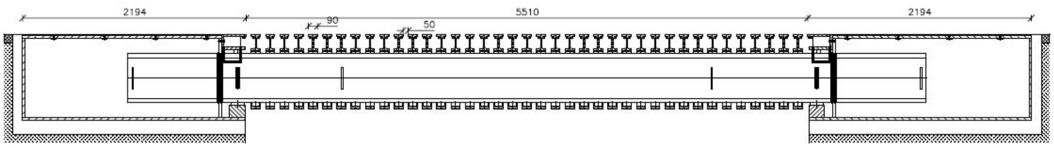


Figure 4-5 Elevation of roadway expansion joint

4.4 Railway Expansion Joints

The railway expansion joints are longitudinally moveable stock rails type which can facilitate large longitudinal and rotational movements. The support for the railway expansion joints are designed to accommodate the reactions geometrical requirements of the expansion joint.

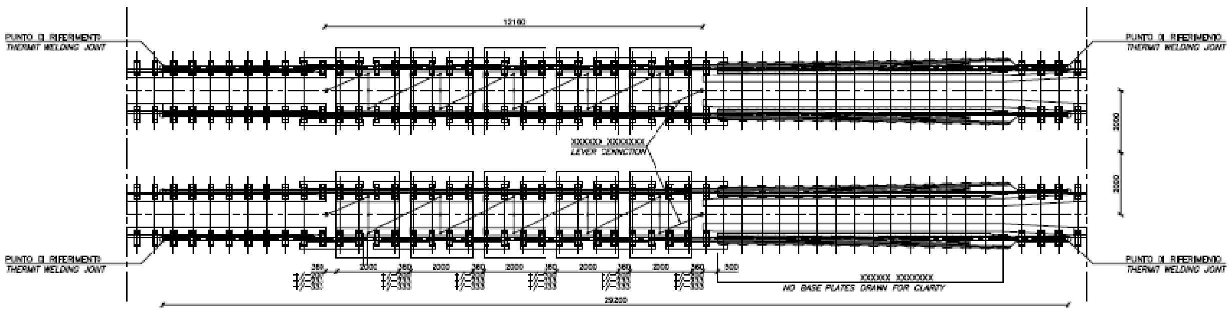


Figure 4-6 Plan of railway expansion joint

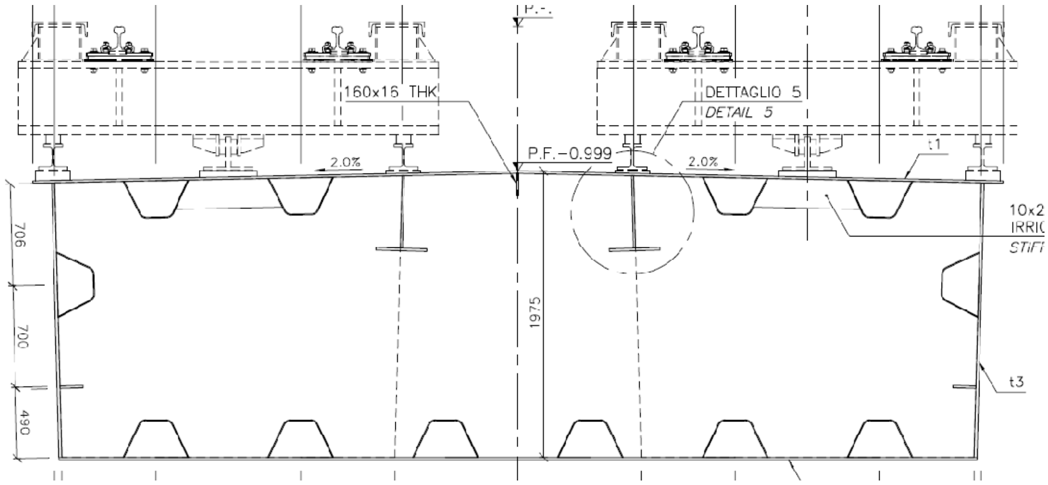
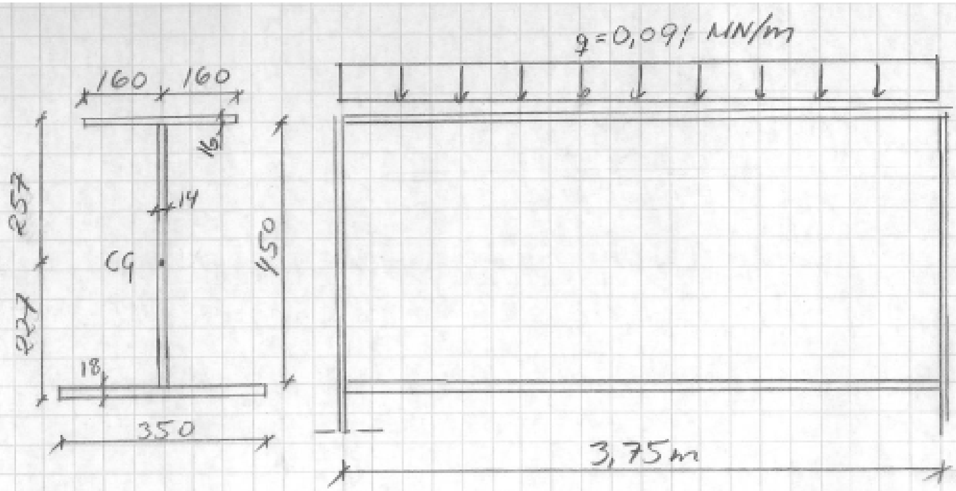


Figure 4-7 Section in railway expansion joint

The T-beam shown in Figure 4-7 transfers the rail load. A verification of the T-beam is given in the following assuming that a part of the deck plate is effective corresponding to ten times the plate thickness.



The T-beam is assumed to transfer the rail load from one track over a length of 3.75m between the diaphragms.

The maximum rail load is taken as the axle load of a LM71. The distance between the axles is 1,6m which gives a uniform distributed load on the T-beam of:

$$q = \frac{0{,}275\ MN \cdot 1{,}06}{2 \cdot 1{,}6m} = 0{,}091\ MN/m$$

$$M = \frac{1}{8} \cdot 0{,}091 \cdot 3{,}75^2 = 0{,}16\ MNm$$

$$I = 0{,}725 \cdot 10^{-3}\ m^4$$

$$y = 0{,}227\ m$$

The positive bending moment gives the following tension stresses and compression stresses in the bottom and top of the cross section, respectively:

$$\sigma_t = \frac{0{,}16}{0{,}725 \cdot 10^{-3}} \cdot 0{,}227 = \underline{50\ MPa}$$

$$\sigma_c = \frac{-0{,}16}{0{,}725 \cdot 10^{-3}} \cdot 0{,}257 = \underline{-57\ MPa}$$

The maximum shear force is:

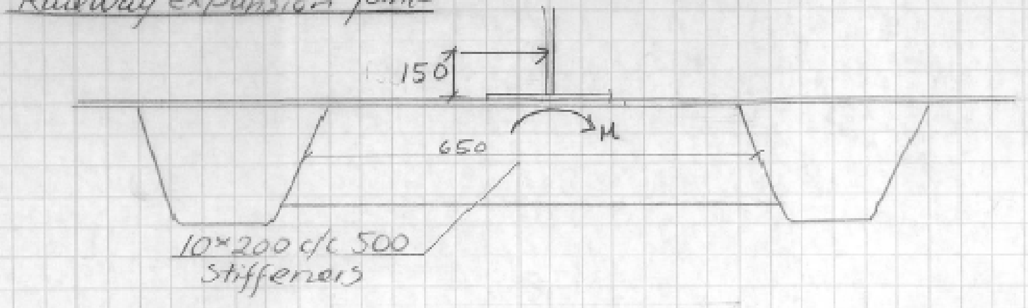
$$Q = 0{,}091 \cdot 3{,}75 \cdot 0{,}5 = 0{,}17\ MN$$

$$\tau = \frac{0{,}17\ MN}{(0{,}450 \cdot 0{,}014)m^2} = 27\ MPa$$

$$27\ MPa < \frac{f_{yd}}{\sqrt{3}} = 195\ MPa.$$

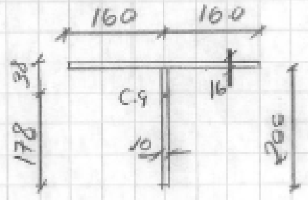
A stress check of the transverse stiffeners between the troughs under the rail system will be done in the following. The dimension of the stiffeners is 10x200, c/c 500. The stiffeners shall be capable of transferring the horizontal nosing force from the trains of 100kN acting on top of the rail contributing with a moment in the stiffener. The wind load is estimated to give a contribution of 10kN. An α-factor of 1.1 is applied for LM71 and furthermore the load is multiplied with a load factor of 1.45.

Railway expansion joints



$F = 0{,}110\ MN \cdot 1{,}1 \cdot 1{,}45 = 0{,}175\ MN$

A part of the deck plate is calculated as being effective corresponding to a width of ten times the plate thickness to each side of the stiffener.



$I = 0{,}236 \cdot 10^{-4}\ m^4$

The moment to be transferred in the stiffener is:

$M = 0{,}175\ MN \cdot 0{,}183 = \underline{0{,}032\ MNm}$

The normal stress is calculated as:

$\sigma_t = \dfrac{0{,}032}{0{,}236 \cdot 10^{-4}} \cdot 0{,}038 = \underline{52\ MPa}$

$\sigma_c = \dfrac{-0{,}032}{0{,}236 \cdot 10^{-4}} \cdot 0{,}178 = \underline{-241\ MPa}$

Check of shear stresses

The shear force is:

$$Q = -\frac{M}{L} = \frac{-0{,}032}{0{,}650} = 0{,}05\ MN$$

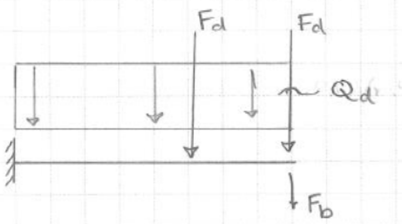
$$\tau = \frac{0{,}05\ MN}{(0{,}2 \cdot 0{,}010)\ m^2} = 25\ MPa$$

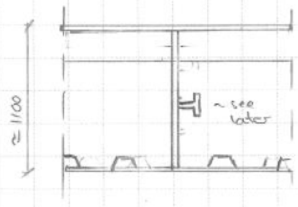
$$\sigma_{vre} = \sqrt{241^2 + 3 \cdot 25^2} = \underline{244\ MPa < 338\ MPa} \quad OK.$$

5 Extension of Roadway Deck at Cross Overs/Service Areas

At cross overs and service areas an extension has been made to widen the roadway girder, refer drawings (attached). The extension will act as a cantilevered beam in the main part (center) - and it is at this location the section is verified.

- Loading:
  - A section of 3.75 m is considered. It is conservatively considered to be fully loaded over the entire surface by max traffic loading:





(Beam) Section

$Q = 9\,kN/m^2$

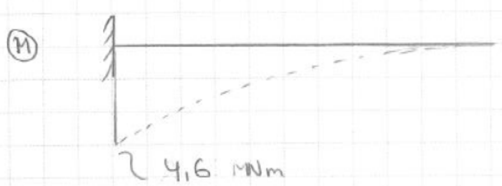
$F = 300\,kN$

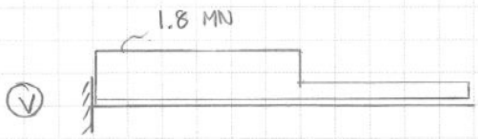
$F_d = 300\,kN \cdot 1{,}35 \cdot 1{,}1 = 446\,kN$

$Q_d = 9\,kN/m^2 \cdot 1{,}35 \cdot 1{,}0 = 12{,}15\,kN/m^2$

$F_b = 750\,kN$ (bearing reaction) ($\Rightarrow$ $F_b = 700$ but not changed here)

- Forces:
  The forces in the beam has been calculated using winbeam, see below. The forces becomes:





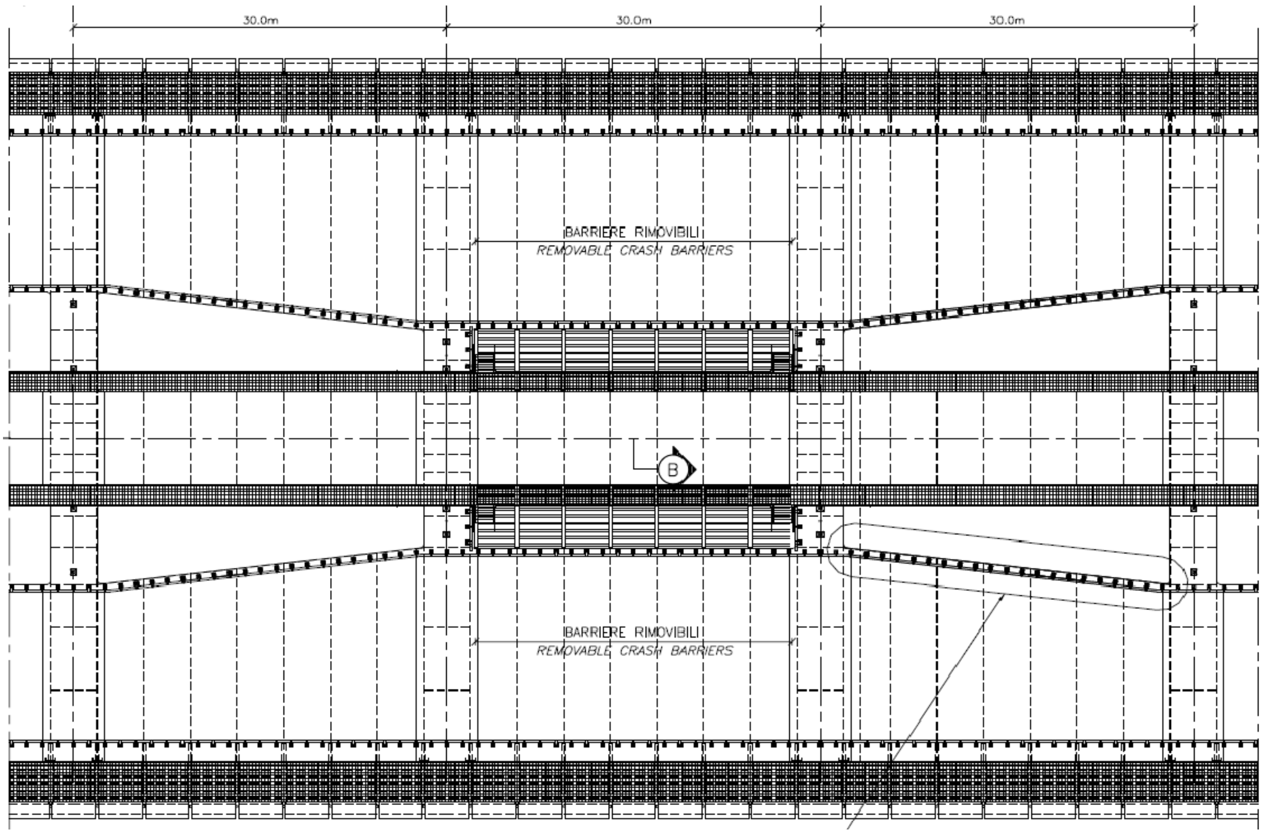


Figure 5-1 Roadway extension at cross overs and service areas, service areas shown

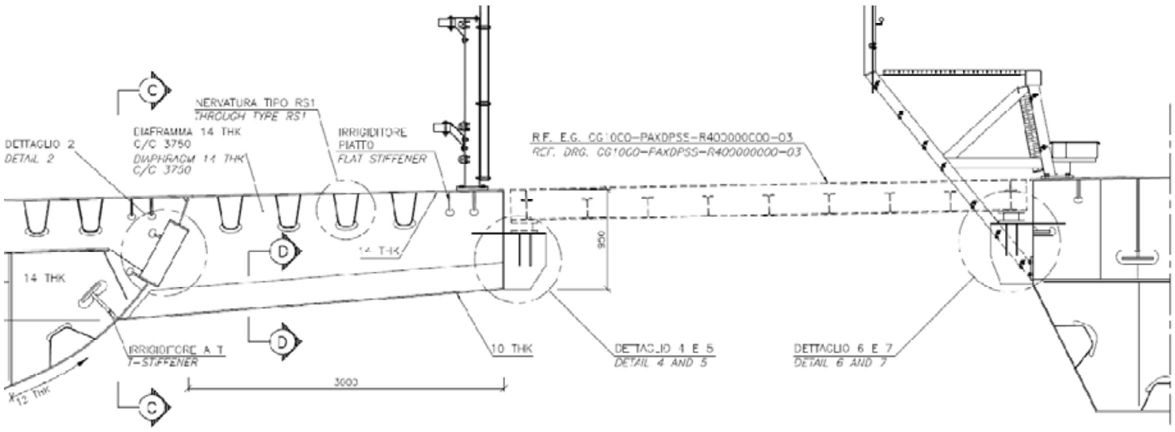


Figure 5-2 Cross section of roadway extension, service area shown

· Properties:

To simplify the calculation the trough-stiffeners are equivalated with a plate thickness "smeared" over the web and bottom flanges.

It is assumed that only the bottom flange throughs contribute to the bending in the considered direction.

The plate thicknesses are as follows:

Top flange: $t_t = 14$ mm

Bottom flange: $t_b = 10$ mm

Web : $t_w = 14$ mm

Additional : $t_a = 3$mm

Eq. bottom fl. : $t_b' = t_b + t_a = 13$ mm

Total of 3 stiffeners on 3,75 m between diaphragms $A_{tot} =$

$A = 0.112\ m^2$

$S = 0.064\ m^3$

$Y_s = 0{,}571\ m$

$I = 0.032\ m^4$

· Stresses:

$$\sigma_M = \frac{M}{I} \cdot Y_s = \frac{4.6\ MNm}{0.032\ m^4} \cdot 0{,}571\,m = 82\ MPa$$

$$\tau = \frac{V}{A} = \frac{1.8\ MN}{0{,}112\ m^2} = 16\ MPa$$

- Due to the small stresses it is not needed to consider the effect of calculating "effectiv properties". The plate thicknesses can even be reduced. However they are keept as shown.

COWI

Since the extension will act as a simple supported beam between diaphragms, and the loading in this area is smaller than the one just considered - this need not to be verified for stresses.

- Stability:

It is now investigated if the stability of the cross over is sufficient. Local plate and panel buckling is considered - this is done using the spreadsheet ADVEPS. The critical stresses are calculated to: (for traffic direction)

$\sigma_{cr,\,plate} = 257$ MPa ; $\sigma_{cr,\,stiffener} = 238$ MPa ; $\sigma_{cr,\,panel} = 211$ MPa

- Since the stresses in the cross section does not exceed any of these there are no problems with local instability. (Tension in top flange => no considered)

- The critical stresses for the transverse direction (global s-axis) are calculated for the top flange to:

$\sigma_{cr,\,plate} = 345$ MPa ; $\sigma_{cr,\,stiffener} = 278$ MPa ; $\sigma_{cr,\,panel} = 271$ MPa

↳ thus there will be no stability problem in the compressiv flange for load in this direction.

- Combined stresses:

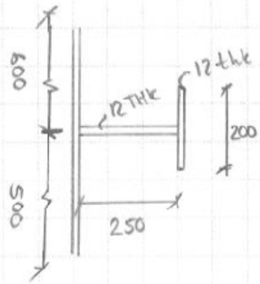
Since the plates will have stresses in two directions the combination is considered - conservatively using twice the stresses in the worst direction.

$\sigma_{cr} = \sqrt{\sigma_1^2 + \sigma_2^2 - \sigma_1 \cdot \sigma_2 + 3 \cdot \tau^2} \simeq \underline{144 \text{ MPa}}$ ok!

· Web

Also the web needs to be investigated for stability, which also are done using Advers..

The critical stresses for the web/diaphragm is calculated for the following stiffener and location:



Assuming a max plate width of 600 mm the stresses becomes:

$\sigma_{cr, plate}$ = 287 mm ; $\sigma_{cr, panel}$ = 285 MPa

$\sigma_{cr, web}$ = 355 ; $\sigma_{cr, flange}$ = 355 } for the stiffener

↳ again those stresses are much heigher than the actual stresses so no further check needed. No stability problems.

- It is investigated if the diaphragm stiffener can be neglected. If a plate of 1200 mm is considered without any stiffeners the critical stress becomes: $\sigma_{cr, plate}$ = 166 MPa ok!

So it is decided not to add a stiffener to the diaphragm.

5.1 Backing in Roadway Girder

The moment produced from traffic from the cantilevered road extension needs to be distributed fully to the roadgirder.

The moment can be split in a force couple producing tension in the top and compression in the bottom flange. Since there are no backing in the roadgirder behind the bottom flange of the extension a T-stiffener is applied to distribute the compressiv force. The dimensions needed for the T-stiffener is considered in the following.

- Load:

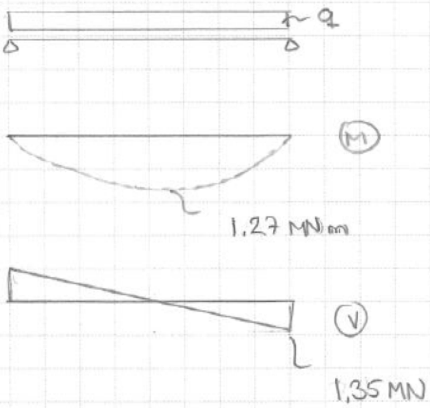
The moment is calculated to $M = 4.6$ MNm, so assuming a height of the section of 1100 mm the forces becomes $F_c = \frac{M}{h} \approx \underline{4.2 \text{ MN}}$

Distributed over the length 3.75 m $\Rightarrow q = \frac{4.2 \text{ MN}}{3.75} = \underline{1.12 \text{ MN/m}}$

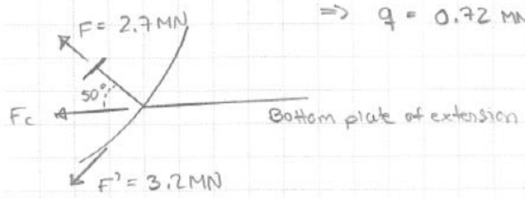
(due to the webs will contribute aswell - these loads could have been reduced 10%.

- The stiffener has an angle of 50° to the bottom plate direction - so the force component in the stiffener direction becomes:

$F = F_c \cdot \cos(50°) = 2.7$ MN (thus 3.2 MN in bottom plate)



$\Rightarrow q = 0.72$ MN/m

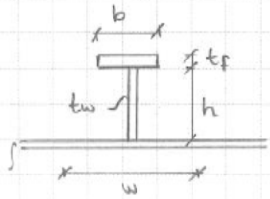


Forces:

The forces in the T-stiffener is illustrated in figures above.

• Properties :

Having a T-stiffener with the following dimensions :



$h = 500$, $b = 350$, $t_f = 20$, $t_w = 22$ mm

Contributing plate from roadgirder $w = 2 \times 15 \times t_p$; $t_p = 12$ mm

↳ plate increased locally from 8mm

$A = 25920 \text{ mm}^2$; $S = 6583520 \text{ mm}^3$; $Y_s = 254$ mm

$I = 1.22 \cdot 10^9$ mm^4

Stresses :

$\tau = \frac{V}{A} = 52$ MPa $\qquad \sigma_M = \frac{M}{I} \cdot Y_s = 281$ MPa $\qquad \Rightarrow \sigma_{VM} = 295$ MPa

(If bottom plate $t_p = 10 \Rightarrow \sigma_{VM} = 331$ MPa)

The stress increase in the bottomplate will in this area be

around $\sigma_N = \frac{3.2 \text{ MN}}{3.75 \times 0.012 \text{ m}} \simeq$ 71 MPa

Diaphragm : (assuming distribution to both sides of stiffener web, so ⅓ and ⅔)

The diaphragms in the roadgirder will at the location of the cross overs carry additional 2×1.35 MN in shear. This will demand an increase in plate thickness locally, so $V = 2.7$MN to be transferred over 360 mm

$\tau = \frac{V}{A} \Rightarrow 340 \text{MPa} = \frac{2.7 \text{MN}}{0.36 \times t} \times \frac{2}{3} \Rightarrow t = 16$ mm

It is then choosen to use an increase locally to 18 mm and the next panel to $t = 14$ mm to make a nice transition.

Since the applied-stiffener for backing can not be 500mm in height due to obstruction of passage to the manhole for inspection it has in the design been chosen to decrees the size and change to steel grade S460. The height has been reduced and the thicknesses changed, see Figure 5-3.

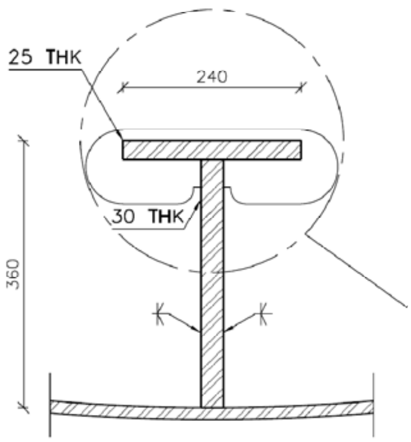


Figure 5-3 T-stiffener add as backing in the roadway girder, cross over location only

Using the shown dimensions the properties and stresses has been calculated to:

A= 24480 mm^2

Y= 184.2 mm

I= $0.61 \cdot 10^9$ mm^4

σ_m = 385 MPa ; τ= 55 MPa; σ_{VM} = 397 MPa < 438 MPa OK!

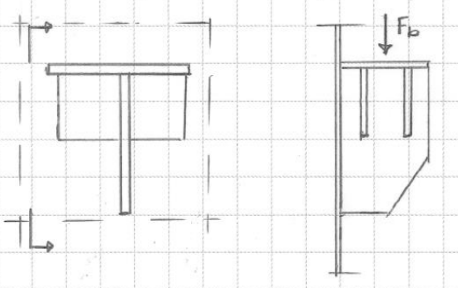
Different plate thicknesses have been used at the service area due to less loading. The calculations are similar and thus not stated here.

5.2 Consoles

At the Cross Overs and Service Areas the crossing bridge is placed on bearings located on steel consoles. Those consoles are welded to the extension of the roadway girder, the railway girder or cross girders, refer drawings.

In the following the consoles at cross overs are considered, since these are heaviest loaded. The bearing reactions at this location is calculated to $F_b = 700$ kN pr. bearing (ULS). (from calculation by IACU)

The consoles are constructed as T-beams with torsional supportings / distribution plate as illustrated.



The height of the webs are (approx.)

- At roadway $h \approx 380$ mm
- At railway $h \approx 665$ mm

$t_{web} = 14$ mm

$t_{flange} = 14$ mm

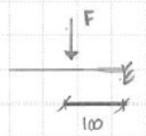
$t_{d.pl.} = 10$ mm

The principle is to transfer the load to the girder diaphragms. So the distribution plates helps trasferring the load from the top flange and through the webs into the diaphragm as shear and moment.

Distribution plates:

The plates act as cantilevered beams (neglecting top flange) so

$F = \frac{F_b}{4} \approx 190\ \text{kN}$ (only 175) $\Rightarrow M = 190\,\text{kN} \times 100\,\text{mm} = 0.019\ \text{MNm}$



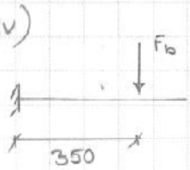
$I = \frac{1}{12} \cdot 10 \cdot 300^3 = 2.25 \cdot 10^{-5}\ \text{m}^4 \Rightarrow \sigma = \frac{M}{I} \cdot z$

$= \frac{0.019\ \text{MNm}}{2.25 \cdot 10^{-5}} \cdot 0.15\,\text{m} = 127\ \text{MPa}$ ok!

The web transferes the shear force to the diaphragm so:

$A = 380\ \text{mm} \cdot 14\ \text{mm} = 5320\ \text{mm}^2 \Rightarrow \tau = \frac{V}{A} = \frac{750\ \text{kN}}{5320\ \text{mm}^2} = 141\ \text{MPa}$ ok

The console acts as a cantilevered T-beam with the bearing reaction introducing a moment of: (considered as point load - conservativ)



Assuming a certain excentricity in the load the moment in the T-beam becomes: $M = 700\ \text{kN} \times 350\ \text{mm} \approx 0.245\ \text{MNm}$

The properties of the beam: $A = 400 \times 14 + 380 \times 14 = 10920\ \text{mm}^2$

$S = 380 \times 14 \cdot 210 + 400 \times 14 \times 427 = 3178000\ \text{mm}^3$

$Y_s = \frac{S}{A} = 291\ \text{mm}$

$I = \frac{1}{12} \cdot (380^3 \cdot 14 + 400 \times 14^3) +$

$380 \times 14\ (190 - 291)^2 + 400 \times 14 \cdot (387 - 291)^2$

$= 169987712.8\ \text{mm}^4$

So the stress becomes: $(z = 334)$ $\quad \sigma_M = \frac{M}{I} \cdot Y_s = 420\ \text{MPa}$ ok!

Steel grade S460 used. $f_{yd} = 438\ \text{MPa} \Rightarrow UR = 0.96$

COWI

Due to the force couple produced by the moment in the consoles a backing plate has been applied for the top flange to ensure full distribution to the diaphragm.

No calculation has been made for the consoles located at the railway, since the moment of inertia of the consoles are larger here (same plate thicknesses) and thus the capacity is sufficient at this location.

Considering the service area the bearing reaction is F_b=150kN, due to this small load no calculation shown for these here. All plates at the service area can be 10mm thick and steel grade S355 can be used.

6 Back-up for Secondary Structures

6.1 Service Lane

The primary backup of the cantilevered beam for the service lane is the diaphragms for every 3.75m. To ensure an acceptable stress flow at the support, the diaphragms are increased to a plate thickness of 12mm in the area shown as t4 in Figure 6-1.

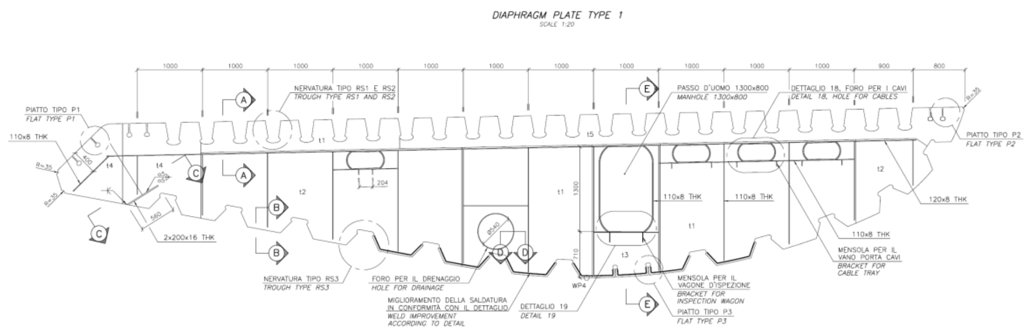


TABELLA 1
SPESSORE LAMIERE E CLASSI ACCIAIO
TABLE 1
PLATE THICKNESS AND STEEL GRADE

CASSONE STRADALE TIPO *ROADWAY GIRDER TYPE*	CS1 – CS8
DIAFRAMMA TIPO *DIAPHRAGM TYPE*	1
t1	10
t2	8
t3	16
t4	12
t5	15
CLASSE ACCIAIO/*STEEL GRADE*	S355

Figure 6-1 Plate thicknesses in roadway diaphragm

The local FE-model for the roadway girder shown in Figure 6-2 is used for stress verification of this area. The model is described in detail in "General Design Principles for Suspended Deck".

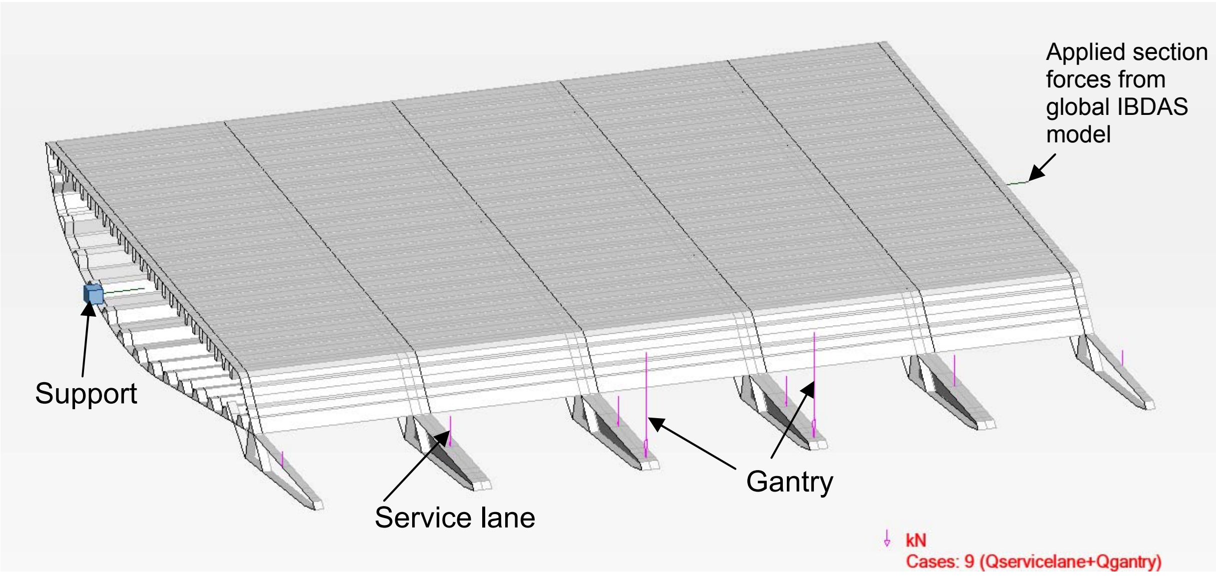


Figure 6-2 Local FE-model for service lane back-up

The model shown in Figure 6-2 is supported in the same way as described in "General Design Principles for Suspended Deck" to simulate the right boundary conditions. The global position of rail and roadway traffic load is fixed for maximising the bending moment My+ in the middle of the model. The forces shown in Figure 6-2 is ULS nodal forces of 209kN for the gantry, applied in the centreline of the rail, and ULS traffic load on the service lane applied as a nodal force of 60kN in the middle of the service lane.

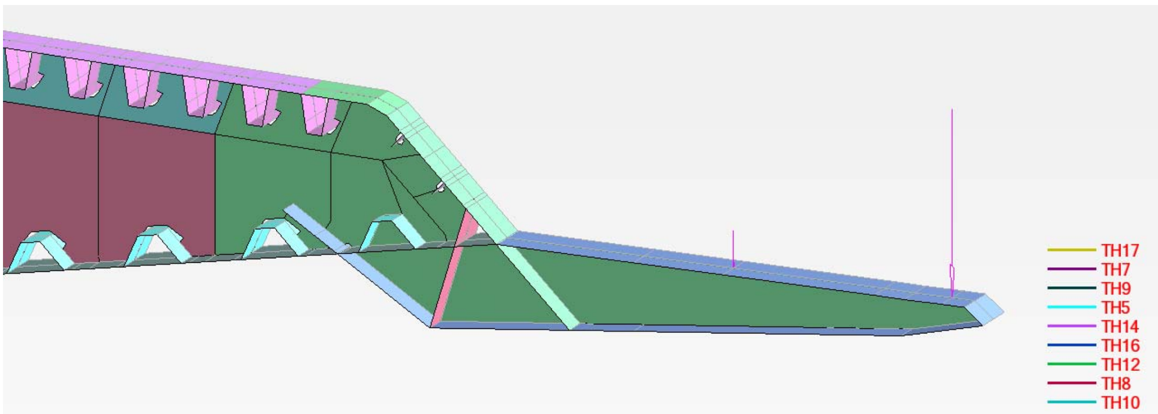


Figure 6-3 Plate thicknesses in Local FE-model for service lane back-up

The ULS load combination used for the stress verification of the cantilevered beam is:

$$1.25xPP+1.5xPN+1.35xQ_{gantry}+1.35xQL_{servicelane}+1.00xQL_{roadway\ traffic}$$

The von Mises stresses for the ULS combination is shown in Figure 6-4.

Figure 6-4 Von Mises stresses for ULS load combination

The steel grade of the shell elements in the diaphragm corresponds to S355. Stress limit has been limited to the yielding stress of f_{yk}/γ_{m0} which correspond to 355/1.05=338MPa.

6.2 Crash Barriers

The back-up steel for the crash barrier is shown in Figure 6-5.

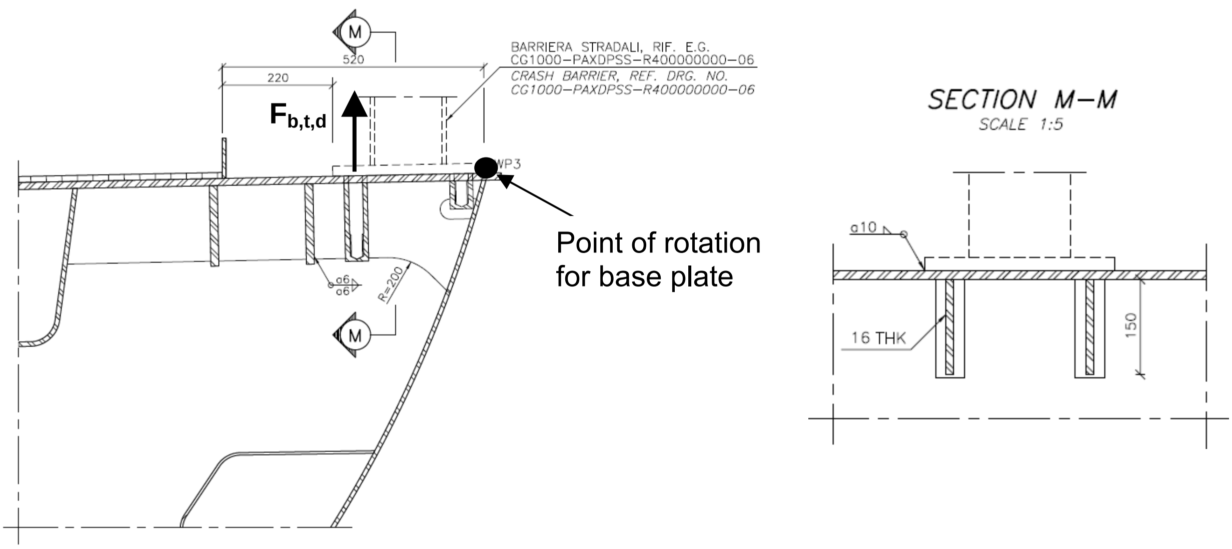


Figure 6-5 Applied load and geometry of back-up steel for crash barriers

Tensile strength of a M24 bolt in quality 10.9 with rolled thread:

$F_{b,t} = 0.9 \times A_s \times f_{ub} / \gamma_{M2} = 0.9 \times 353 \times 1000 / 1.25 = 254\text{kN}$

To ensure robustness for the back-up steel for the crash barrier the force is applied with a safety factor of $F_{b,t,d} = 1.2 \times F_{b,t} = 305\text{kN}$. Furthermore this extra safety factor will ensure that the bolts will break before the roadway girder and back-up will be damaged.

A part of the local FE-model for the roadway girder shown in Figure 6-6 is used for verification of this connection. The model is described in detail in "General Design Principles for Suspended Deck".

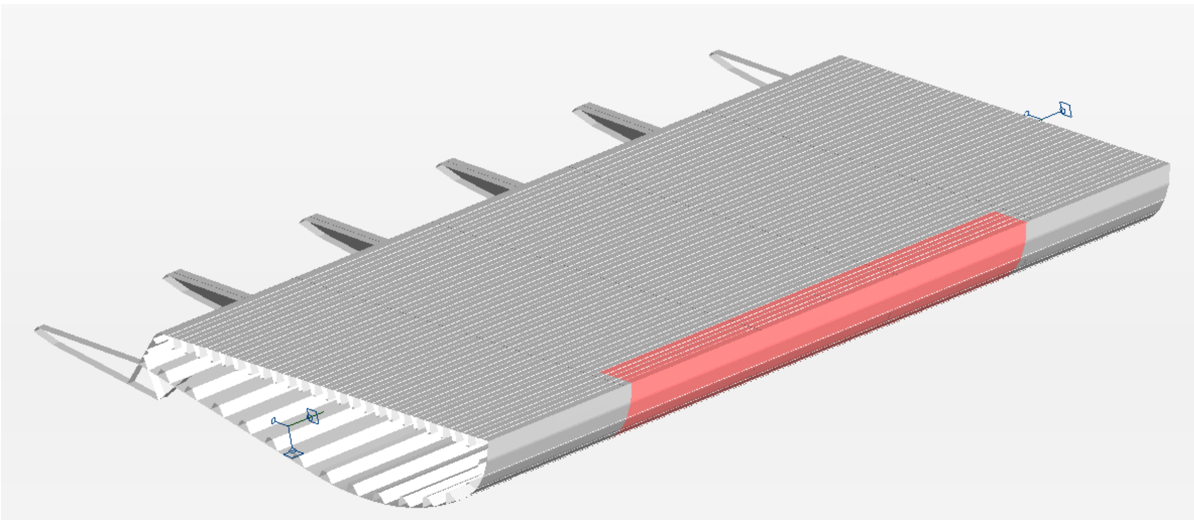


Figure 6-6 Part of the local FE-model used for verification of support of the crash barrier

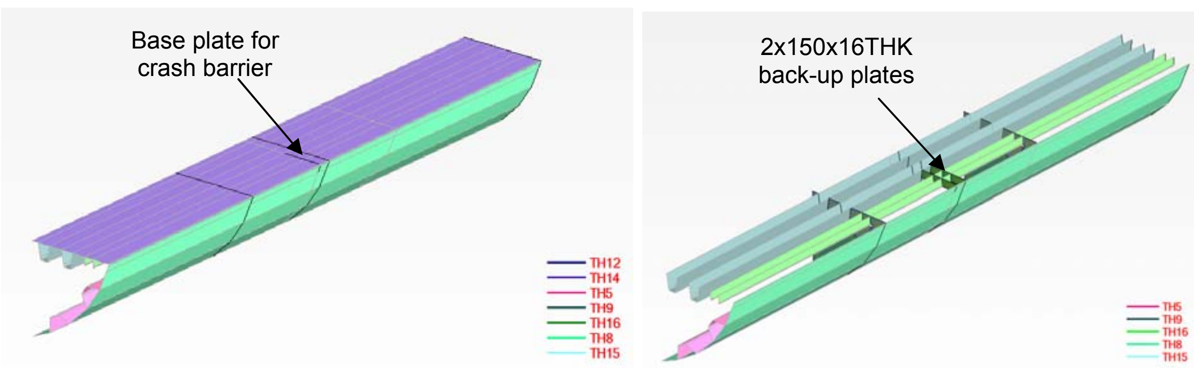


Figure 6-7 Plate thicknesses in local FE-model for verification of support of the crash barrier

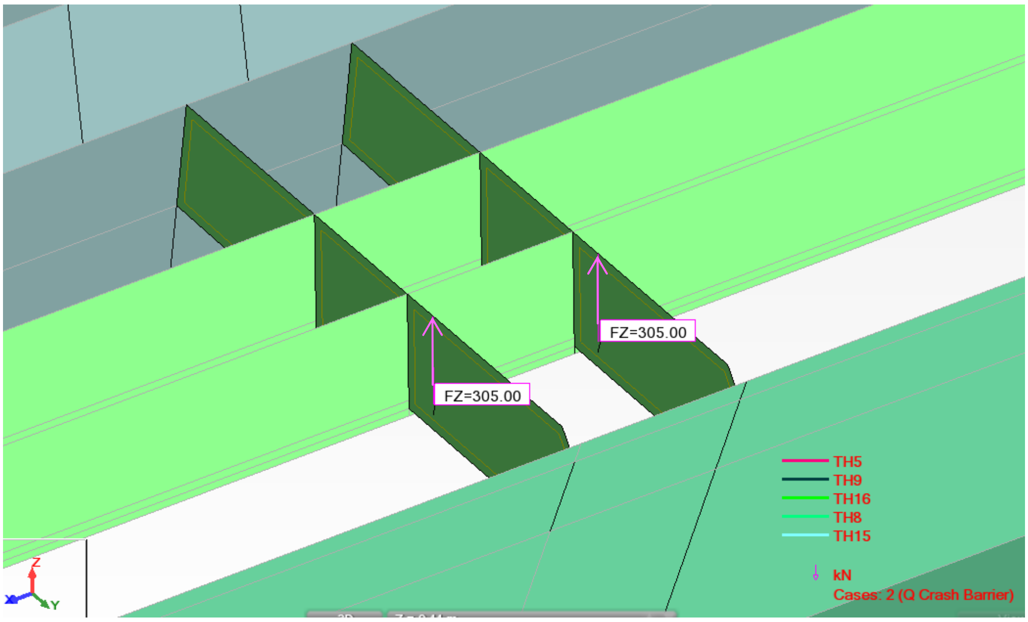


Figure 6-8 Applied $F_{b,t,d}$ load on the back-up plates

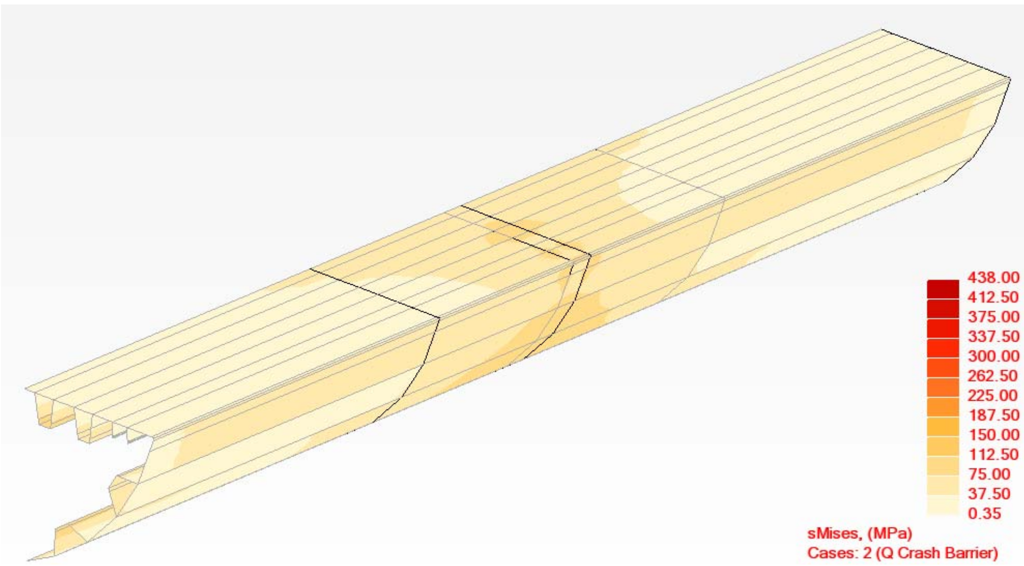


Figure 6-9 Overall Von Mises stresses in local FE-model

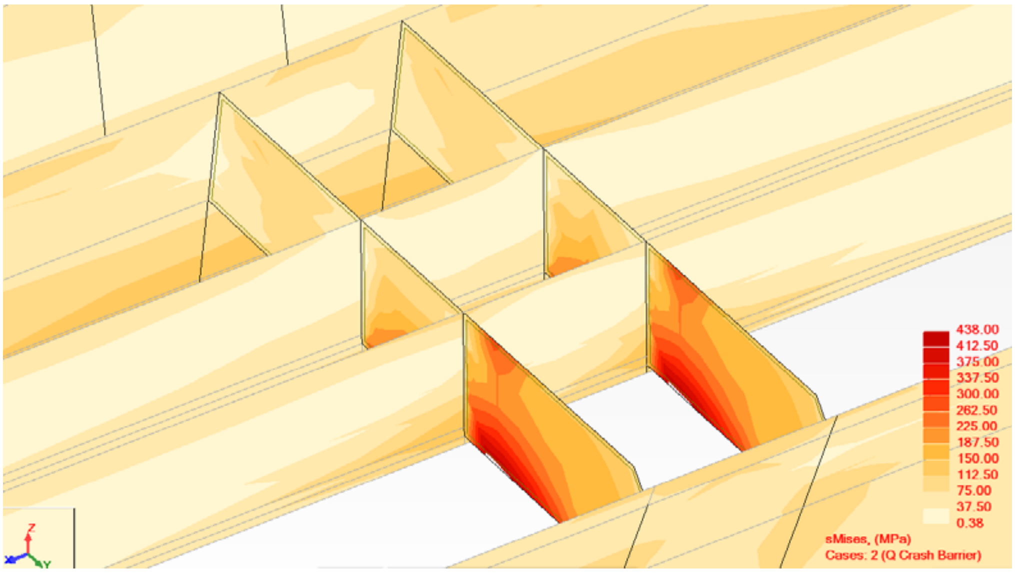


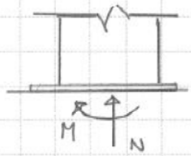
Figure 6-10 Von Mises stresses in local FE-model for verification of support of the crash barrier

The steel grade of stiffener shell elements corresponds to S460. Stress limit has been limited to the yielding stress of f_{yk}/γ_{m0} which correspond to 460/1.05 = 438MPa.

6.3 Portal for Road Signs

In order to support the roadway portals the following consoles have been designed.

The loads from the portal are to be transferred to the roadway girder – this needs intermediate diaphragms locally. The loads from the portal is calculated by IACU to:



$N_z = 30$ kN $\qquad M_y = 400$ kNm

$M_z = 40$ kNm (considered neglectable)

<u>Statical system:</u>

The consoles are designed as a closed box since the moment needs to be transferred as torsion. To ensure that the moment is transferred to the closed box internal stiffeners have been provided. These act like simple supported beams subjected to a vertical force (N) and a bending moment (M_y).

Since two internal stiffeners are provided they take half the force each.

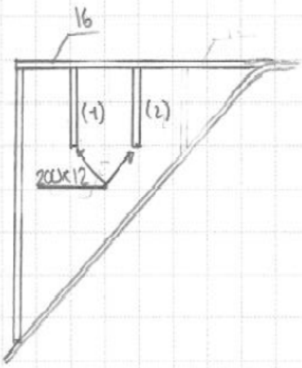
(a 1.2 safety factor has been applied to account for any assymetric loading – however this is highly unlikely to occure)

The calculation is in the following split up for the two types of consoles.

Type 1: At hanger

Type 2: Towards railway

Type 1:



- Support beams: Only one is considered (1), and considered as a T-beam spaning 800 mm

For the properties the base plate has not been considered (conservative):

$A = 200 \times 12 + 320 \times 16 = 7520\ mm^2$ (2×10× pl. thk as tributary plate)

$S = 319360\ mm^2$

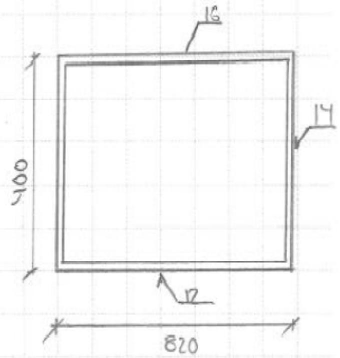
Forces:

$Y = 42.5\ mm$

$V = 0.3\ MN$; $M = 0.12\ MNm$

$I = 27168699\ mm^4$

$\sigma_{vm} = 200\ MPa$ ok! (Reaction at support 300 kN)



The torsion in the console is considered at a "mean section"

$A_0 = 400 \times 820_{mm} \cong 0.33\ m^2$

$M_T = 600\ kN \times 820\ mm \cong 0.5\ MNm$

$\tau = \dfrac{M_T}{2 \cdot A_0 \cdot t_m}$ $t_m = 14\ mm$ $\Rightarrow \tau = 54\ MPa$

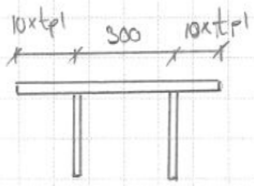
$\Rightarrow \sigma_{vm,\,top} = 220\ MPa$ ok!

- Reducing the web plate so "$t_m = 12$" $\Rightarrow \sigma_{vm} = 230\ MPa$ ok!

Type 2 :

Support beams : For the type 2 console the support beams are similar to type 1, however the top plate is changed to 14 mm $\Rightarrow$ $\sigma_1 = \underline{242 \text{ MPa}}$

(again neglecting the effect of the base plate)



Considering the support beams acting together knowing that the plate between the stiffeners (300 mm) are fully effectiv the stress will be reduced to $\sigma_1 = 236$ MPa . ok!

The torsion gives : $A_0 = 820 \times 350 \simeq 0{,}287 \text{ m}^2$ (mean height used)

and having $t_m \simeq 10$ mm

$$\tau = \frac{0.5 \text{ MNm}}{2 \cdot A_0 \cdot t_m} = \underline{87 \text{ MPa}} \quad (M_T \text{ similar to type 1})$$

ok!

- Since the console act as a cantilevered beam - biaxial stress condition needs to be considered.

For the cantilever : $M = 30 \text{ kN} \times 0.68 \text{ m} = 0.02 \quad (+\ 0.04 \text{ MNm})$

$I = 6.3 \times 10^{-4} \text{ m}^4 \Rightarrow \sigma_2 = \underline{12 \text{ MPa}}$

$$\sigma_{VM} = (236^2 + 12^2 + 236 \times 12 + 3 \times 87^2)^{1/2} \simeq \underline{285 \text{ MPa} < 355 \text{ MPa}} \quad \text{ok!}$$

6.4 Light Masts

Geometry of base plates and back-up steel for the lighting masts is shown in Figure 6-11.

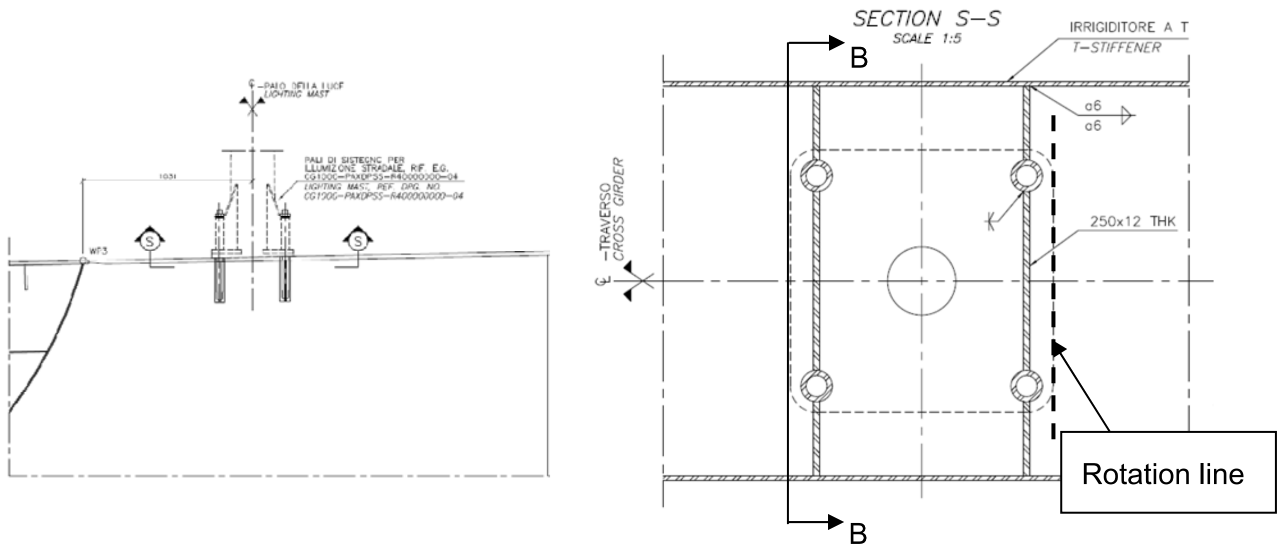
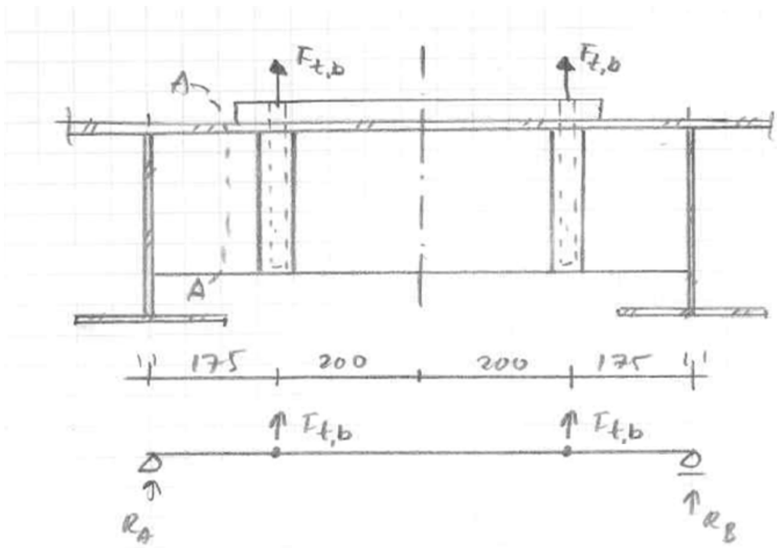


Figure 6-11 Light mast base plate and back-up steel in cross girder

Section B-B:

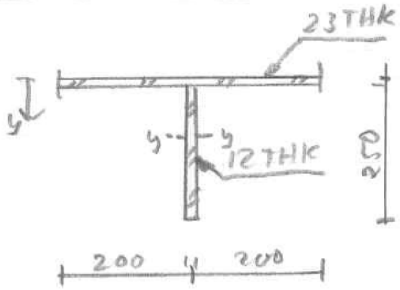


$F_{t,b,d} = 250\,kN$

$R_A = 250\,kN = V_{sd} = \qquad = \underline{250\,kN}$

$M_{max} = 0{,}175 \cdot 250 = \qquad = \underline{43.8\,kNm}$

Section A-A



$$A_s = 12 \cdot 227 + 23 \cdot 400 = 11.9 \cdot 10^3 \, mm^2$$

$$y = \frac{23 \cdot 400 + 12 \cdot 227 \cdot \frac{250}{2}}{A_s} = 29.4 \, mm$$

$$I_y = \frac{12 \cdot 227^3}{12} + \left(\frac{250}{2} - 29.4\right)^2 \cdot 12 \cdot 227 + \frac{23^3 \cdot 400}{12} + 29.4^2 \cdot 23 \cdot 400 = 450 \cdot 10^6 \, mm^4$$

$$W_{min} = \frac{I_y}{250 - y} = 204 \cdot 10^3 \, mm^3$$

$$\sigma_{max} = \frac{M_{max}}{W_{min}} = 215 \, MPa$$

$$\tau_{max} = \frac{V_d}{12 \cdot 250} = 83 \, MPa$$

$$\sigma_{vM} = \sqrt{\sigma^2 + 3\tau^2} = 259 \, MPa$$

$$< \frac{460}{1.05} = 438 \, MPa$$

ok!

6.5 Walkway along Railway including railway crash barriers

The back-up steel for the walkway along the railway crash barriers is shown in Figure 6-12.

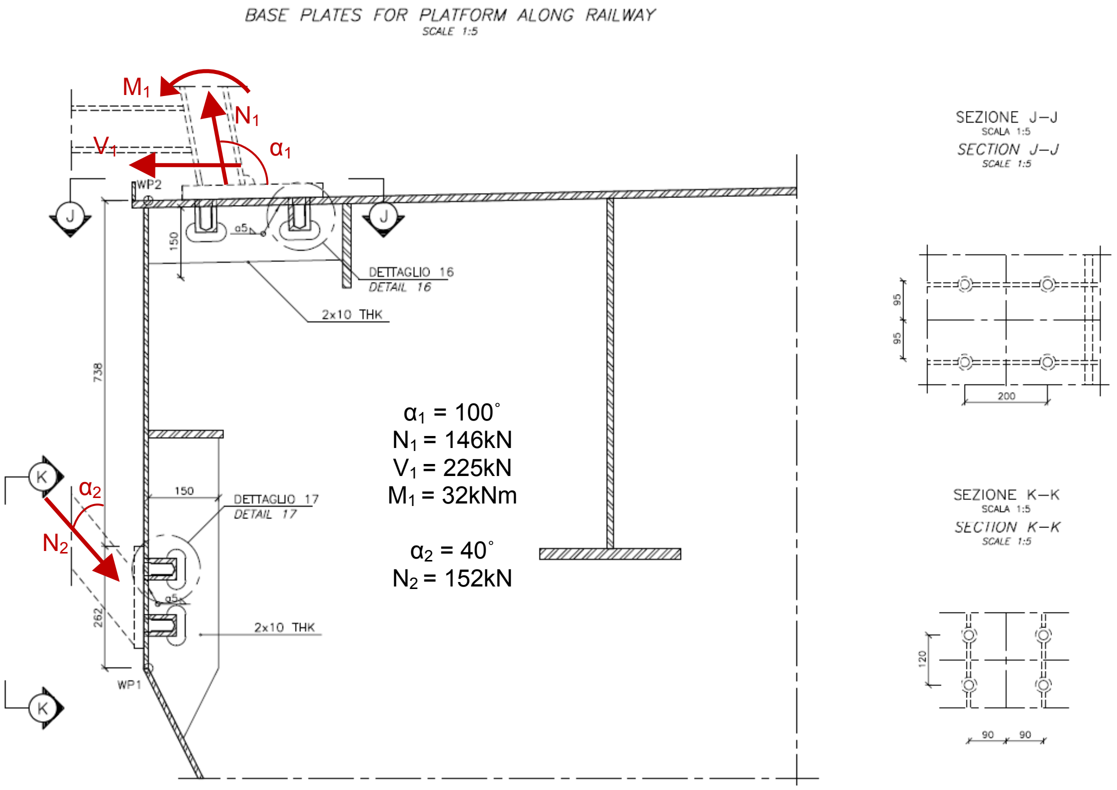


Figure 6-12 Applied load and geometry of back-up steel for walkway along railway

A part of the local FE-model for the railway girder shown in Figure 6-13 and Figure 6-14 is used for verification of this connection. The model is described in detail in "General Design Principles for Suspended Deck".

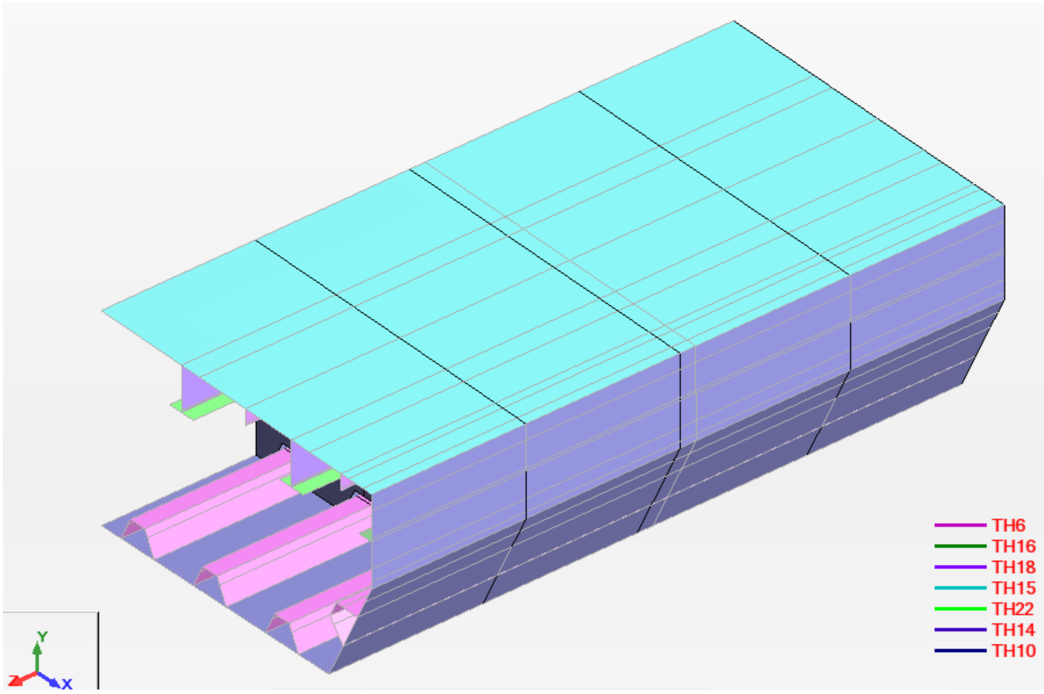


Figure 6-13 Plate thicknesses for the local FE-model of the walkway along railway

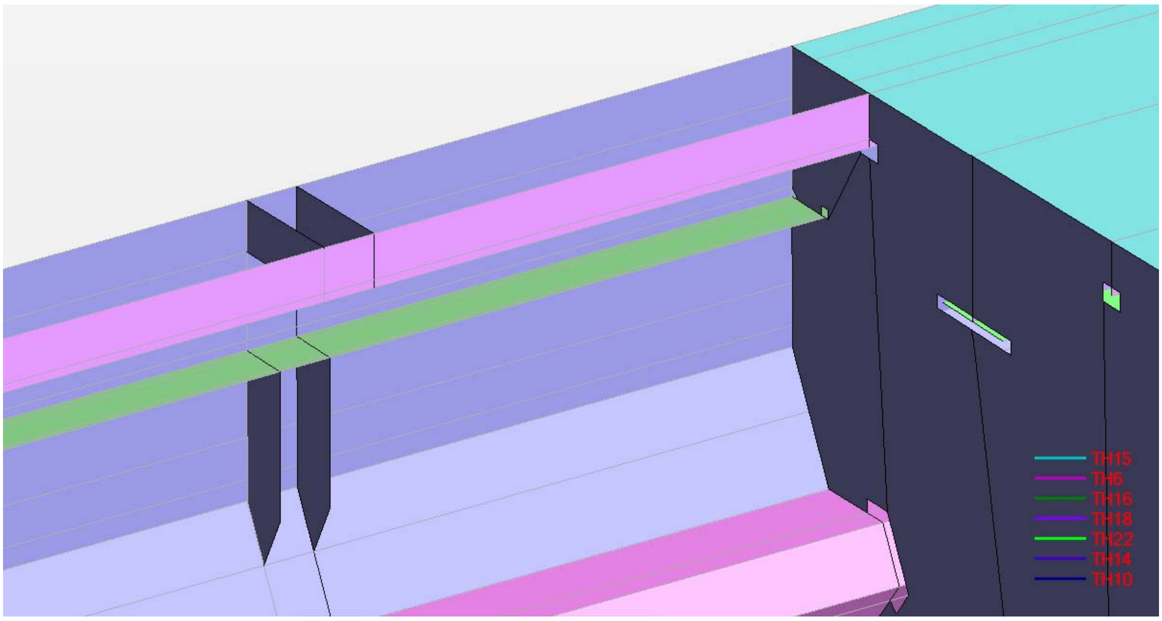


Figure 6-14 Plate thicknesses for back-up of the base plates inside the girder

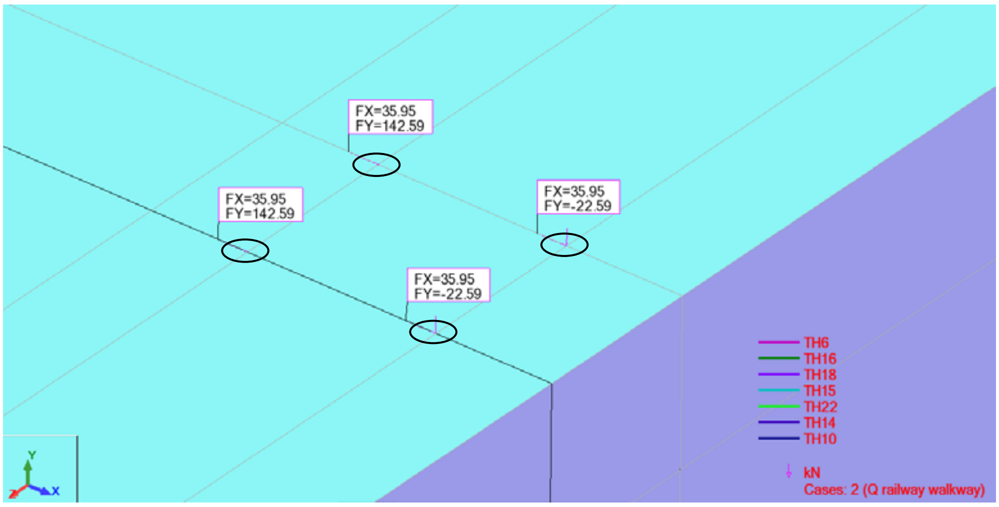


Figure 6-15 Load applied on the top base plate for the walkway along railway

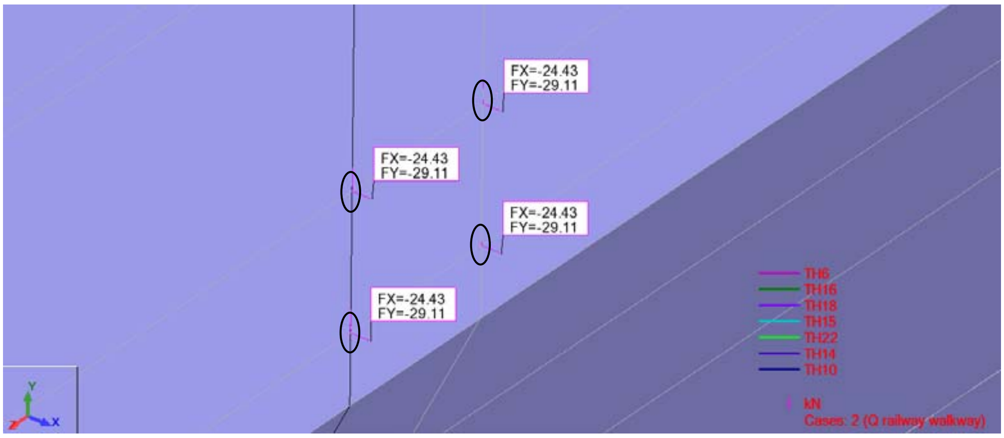


Figure 6-16 Load applied on the side base plate for the walkway along railway

The load combination used for this verification is 1.0 x the load case shown in Figure 6-12 and 1.35xPP.

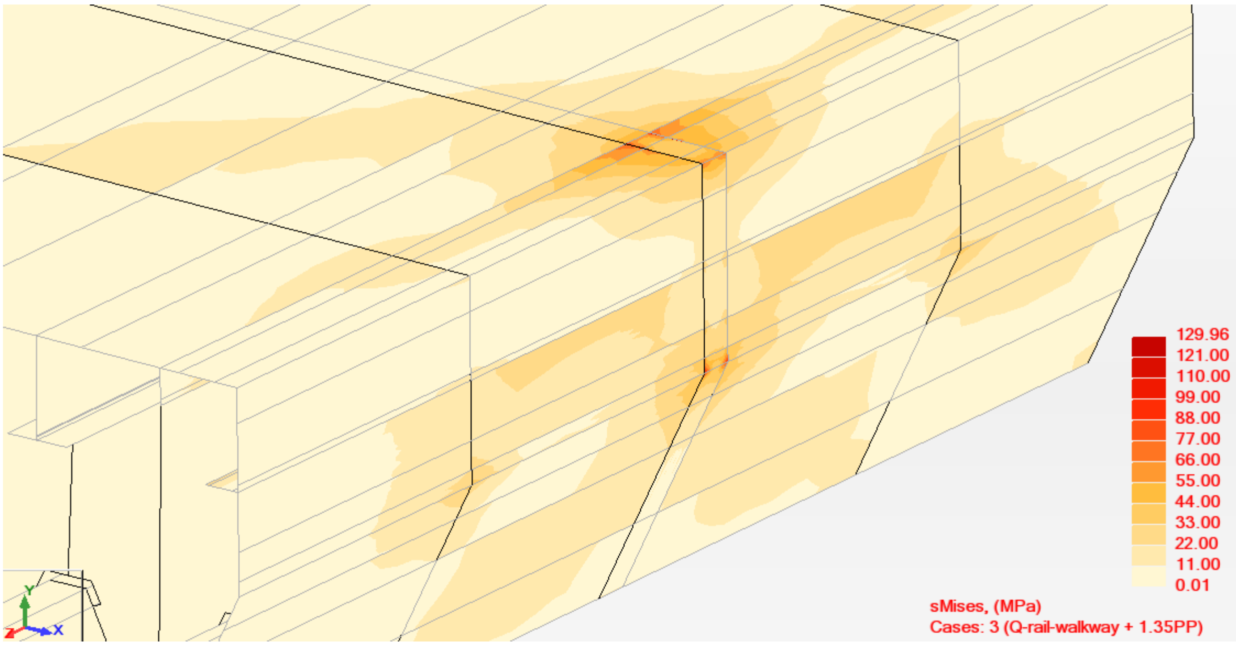


Figure 6-17 Outer maximum von Mises stresses

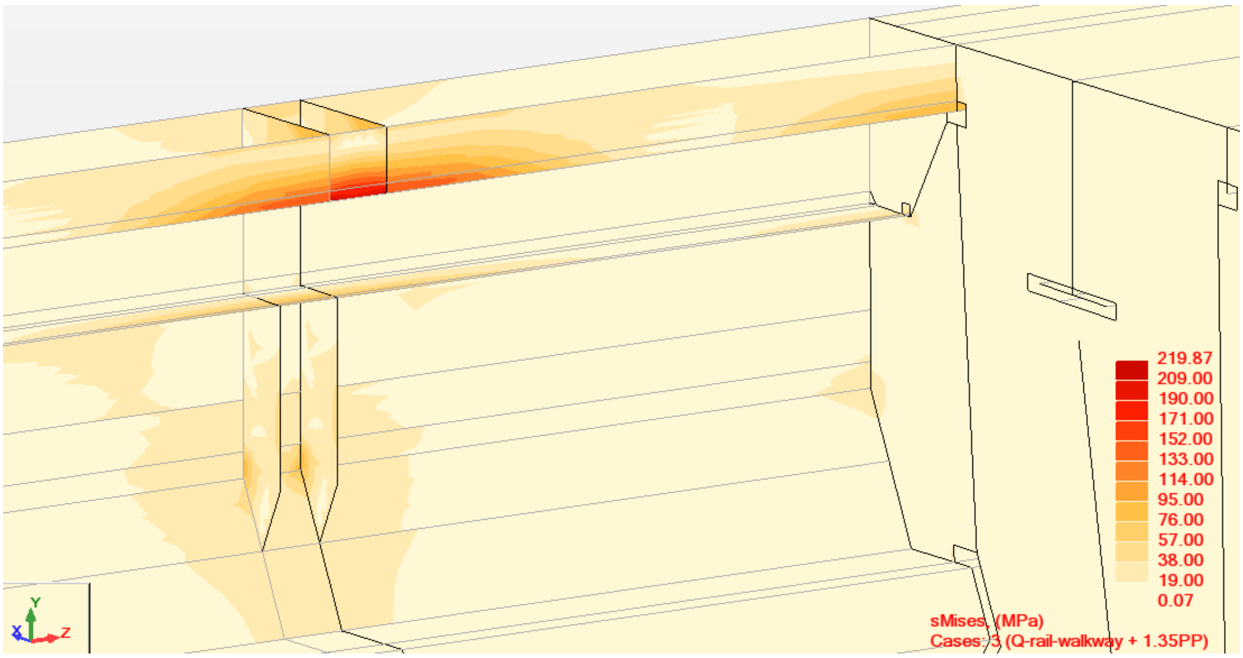


Figure 6-18 Maximum von Mises stresses for back-up of base plates inside the girder

The steel grade of the shell elements corresponds to S355. Design yielding stress limit f_{yk}/γ_{m0} which correspond to 355/1.05=338MPa.