Railway Project

SANTA LUCIA RAILWAY BRIDGE

CALCULATION REPORT

DOCUMENT APPROVAL

VERSIO	DESIGNER	SUPERVISOR	ACCEPT.
15.12.2017	Vera Babiczky	Mikko Ilvonen	

REVISION HISTORY

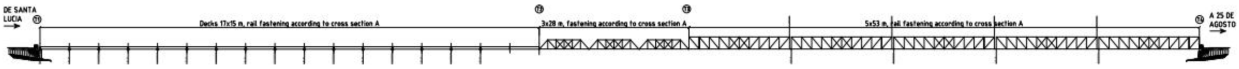
CONTENT

1 SCOPE

Santa Lucia Railway Bridge consists of three different type of cross sections and of three different span lengths. Total length of the bridge is 608 m; consisting of 5x 52 m truss spans, 3x 26 m truss spans and 17x 15 m girder spans.

This report describes the design criteria and capacity of the old structure. Also, the utilizations for the new structures are shown. The chosen solution in pre-engineering is based on experience that the main trusses have capacity, but the secondary structures (cross-girders and longitudinal beams) are problematic mainly in the capacity, fatigue, and functionality of the joints. The known problems of these types of bridges are illustrated in document IRS 77802 (former UIC 778-2) Recommendations for determining the carrying capacity and fatigue risks of existing metallic railway bridges.

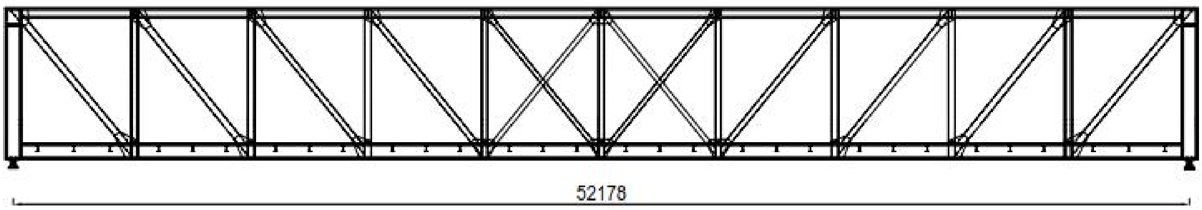
This calculation report is a summary of all calculations executed with FEM-modeling and Structural Analysis and calculations. Its purpose is to show all selections made by the engineer and show the results of the analysis.



Picture 1, Santa Lucia bridge side view

1.1 52 m Lattice/Truss Bridge

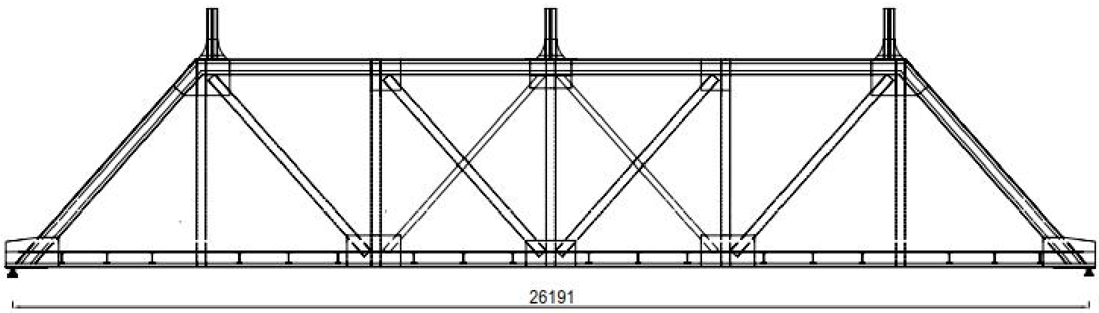
The main goal of this calculation is to show that the old truss structures can be utilized from existing 52,0 m truss sections of the bridge. The main load bearing lattice/truss will be saved as they are and cross beams and longitudinal rail supporting beams will be renewed. There is a possibility to strengthen most critical profiles of truss if more detailed calculations and decisions in the detailed design phase require more safety margins.



Picture 2, Santa Lucia Bridge 52 m

1.2 26 m Lattice/Truss Bridge

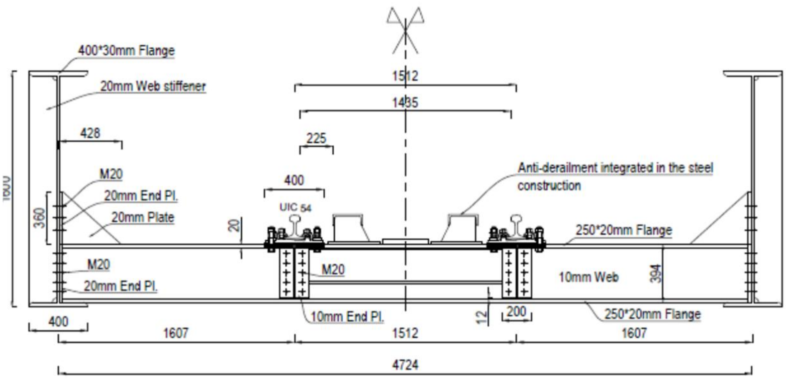
The main goal of this calculation is to show that the old truss structures can be utilized from existing 26,0 m truss sections of the bridge. The main load bearing lattice/truss will be saved as they are and cross beams and longitudinal rail supporting beams will be renewed. There is a possibility to strengthen most critical profiles of truss if more detailed calculations and decisions in the detailed design phase require more safety margins.



Picture 3, Santa Lucia Bridge 26 m

1.3 15 m Girder Bridge

The girder bridge of 15 m span was studied with same actions as the truss sections with replacing of cross beams and longitudinal rail supporting beams. But technical and economical evaluations show that it is more cost effectiveness that all girders spans will be renewed completely. The lifting weight of a single span is suitable for this kind of replacement.



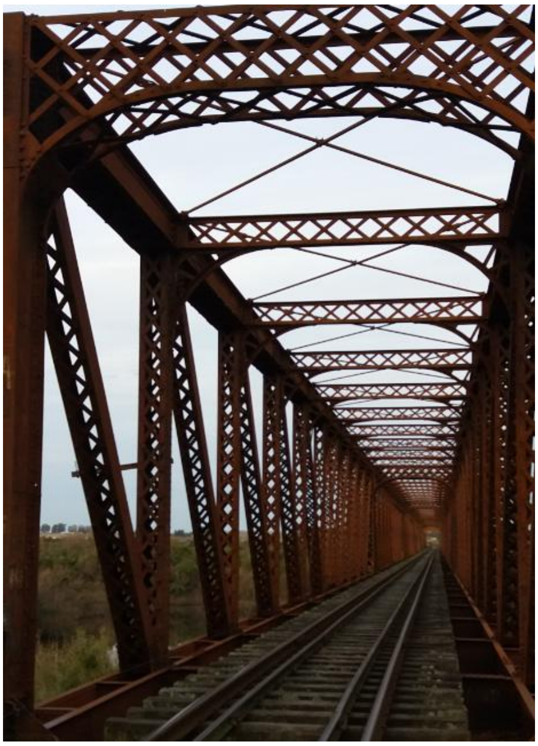
Picture 4, Santa Lucia Bridge 15 m

2 DESIGN CRITERIA

FEM calculations was made with Autodesk® Robot™ Structural Analysis Professional, Version 30.0.0.5913.

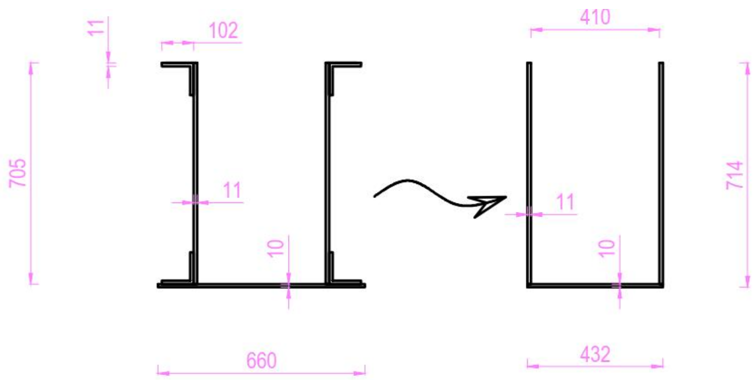
2.1 Structure

Bridge super structure members are complicated truss- and rivet connected profiles. For this calculation FEM-model, simplified profiles were used. Simplifications were made so that function in FEM model equals actual profiles. The simplifications are shown in pictures 6-25 in section 2.1.1.

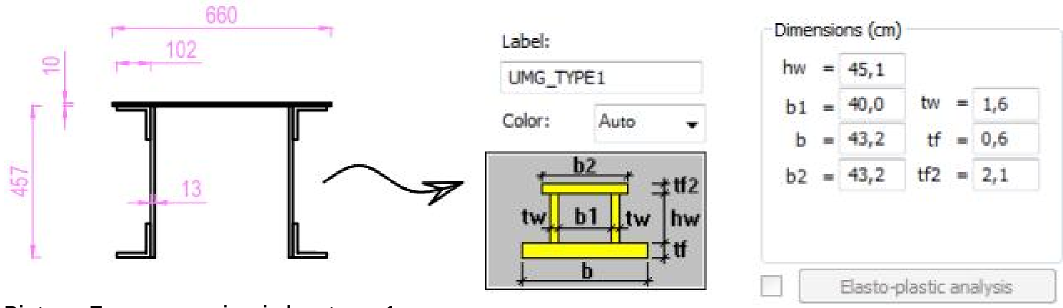


Picture 5, Santa Lucia inside view

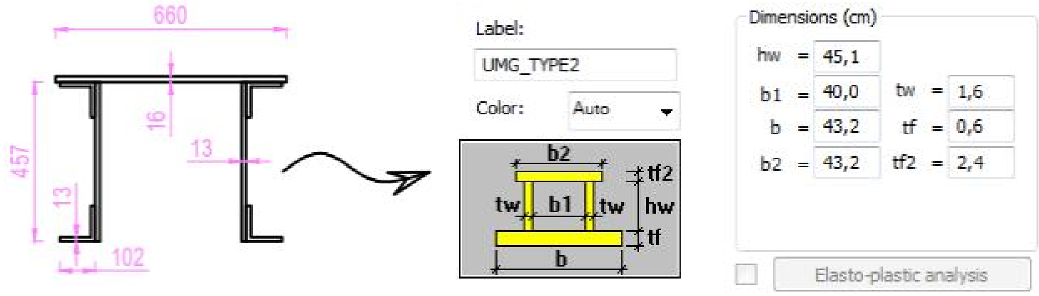
2.1.1 Simplifications for sections in 52 m span truss bridge



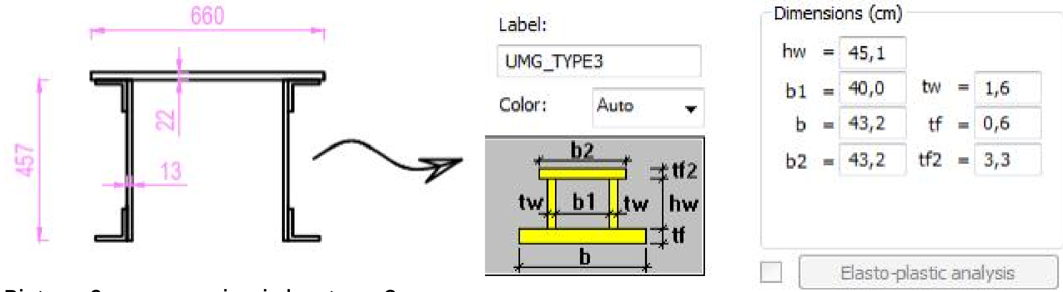
Picture 6, lower main girder



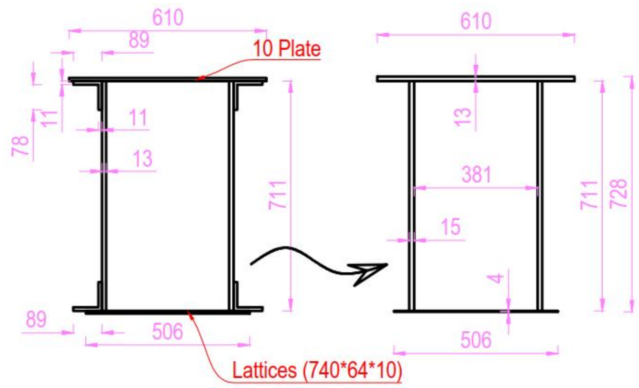
Picture 7, upper main girder, type 1



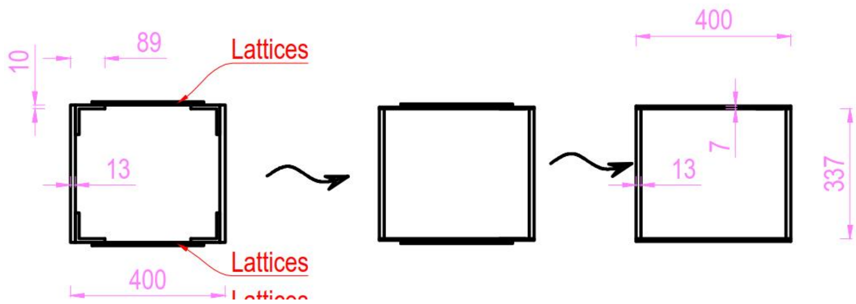
Picture 8, upper main girder, type 2



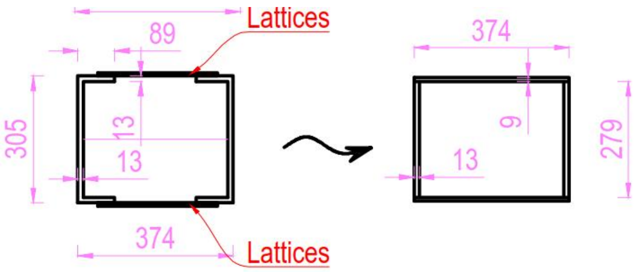
Picture 9, upper main girder, type 3



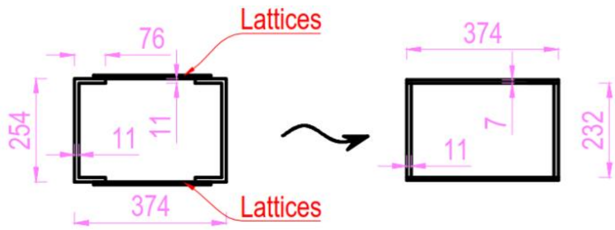
Picture 10, end frame columns



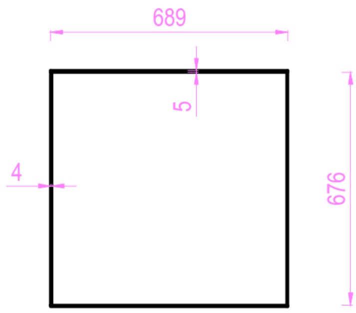
Picture 11, columns, type 1



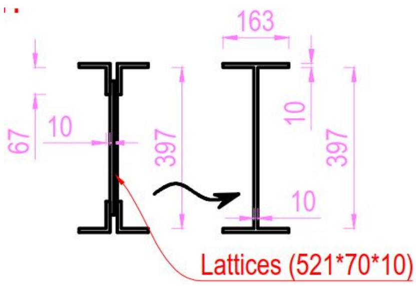
Picture 12, columns, type 2



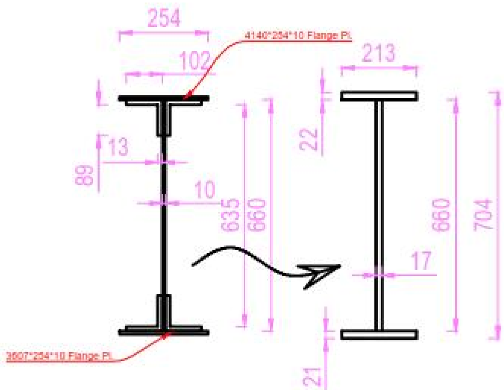
Picture 13, columns, type 3



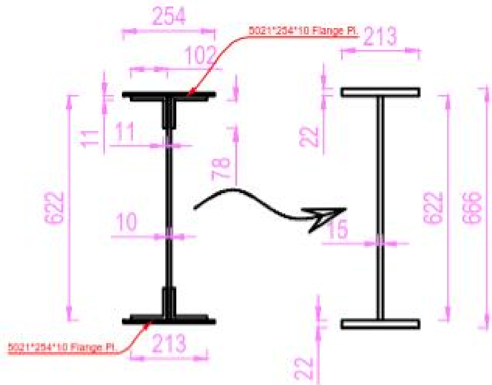
Picture 14, upper end cross beams



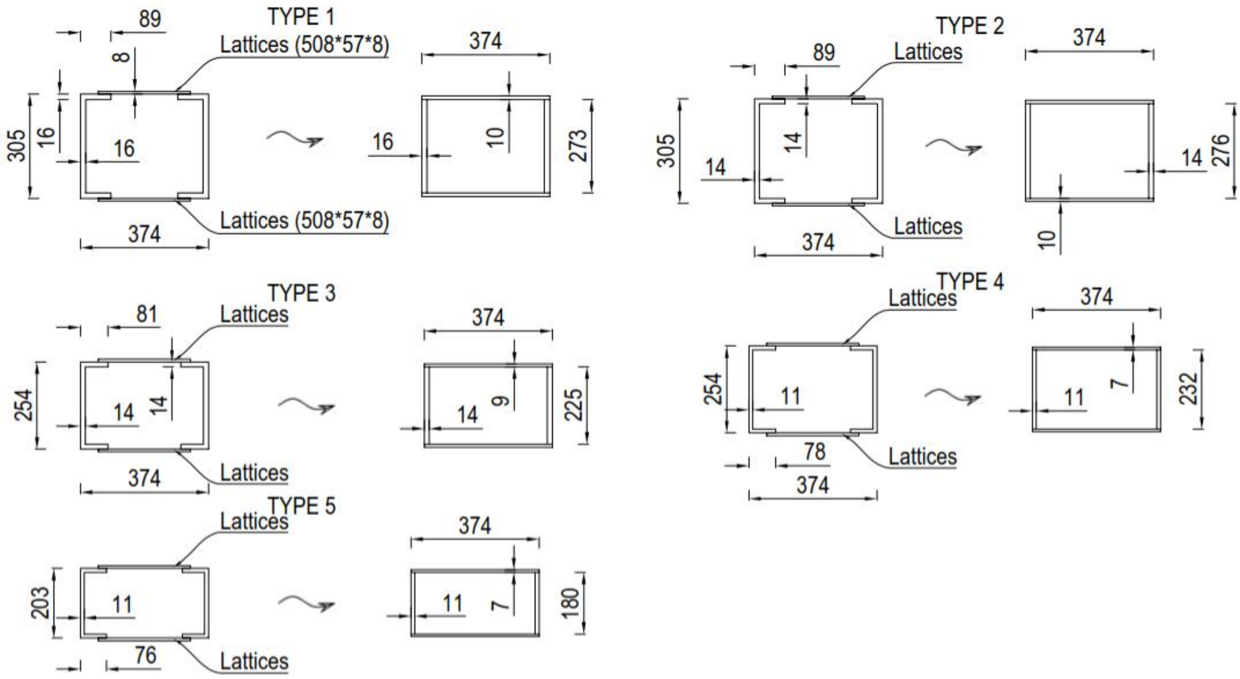
Picture 15, upper cross beams



Picture 16, lower cross beams

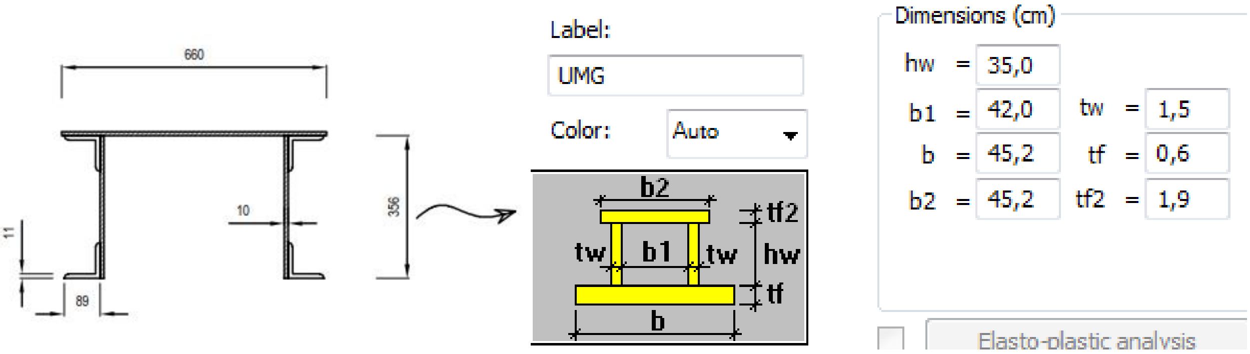


Picture 17, longitudinal beams

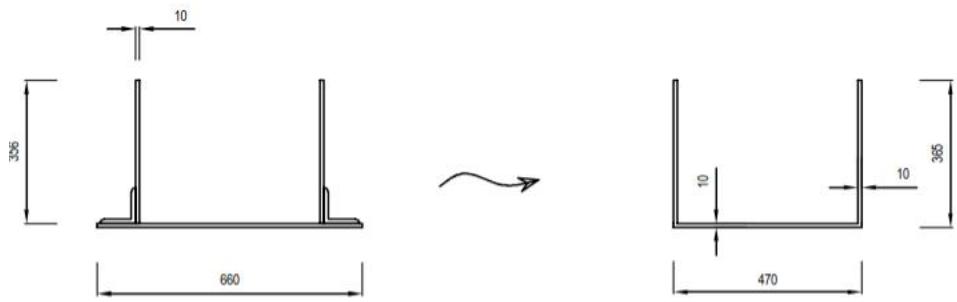


Picture 18, diagonals (5 types)

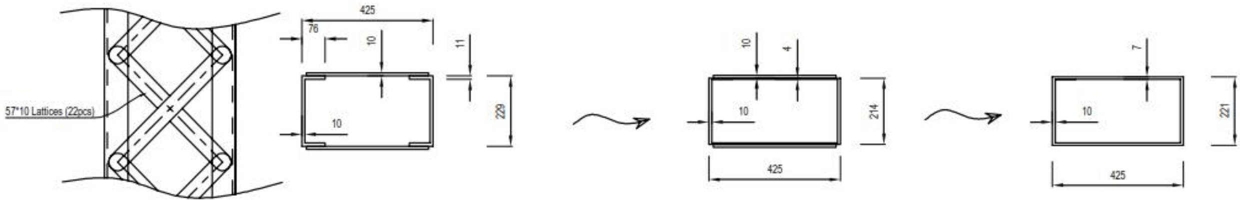
2.1.2 Simplifications for sections in 26 m span truss bridge



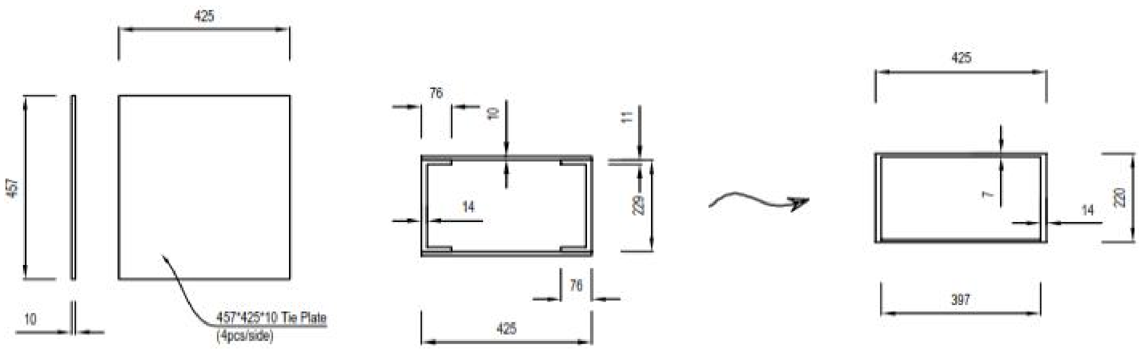
Picture 19, Upper main girder and first diagonals



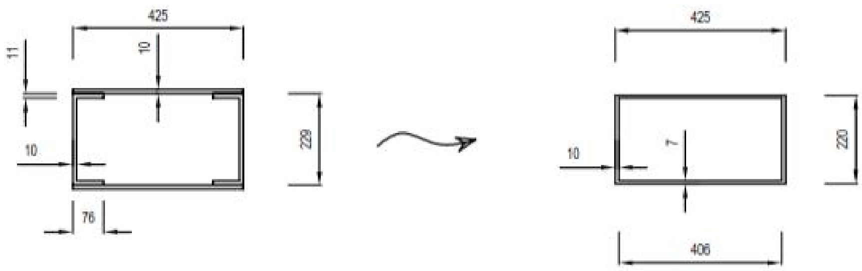
Picture 20, lower main girders



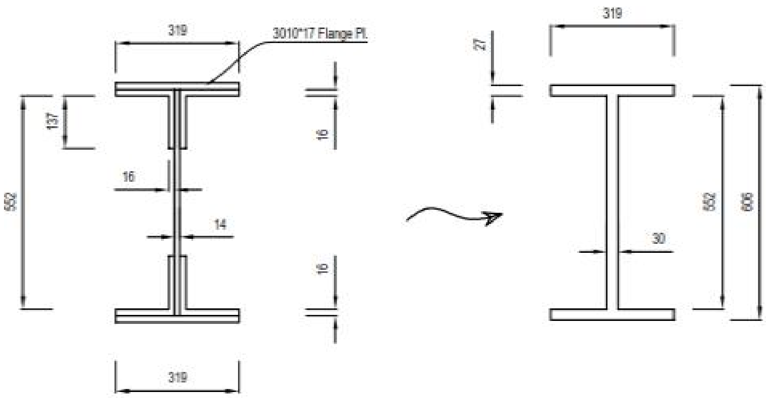
Picture 21, all verticals and columns



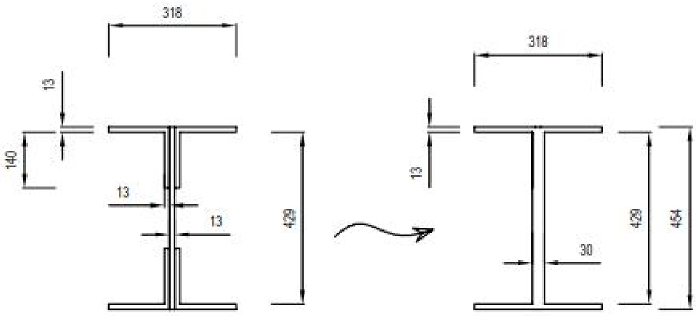
Picture 22, diagonals, type 1



Picture 23, diagonals, type 2



Picture 24, Cross girders



Picture 25, longitudinal girders

2.2 Loads

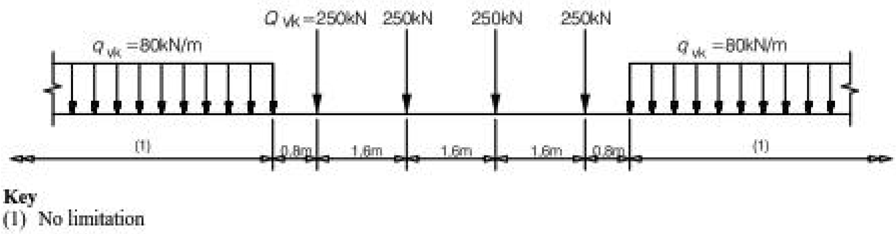
2.2.1 Selfweight / dead load

The Robot Structural Analysis gives weights of structures according to cross sections and selected materials. For the weight of steel is used 7850kg/m3. An additional 1kN/m2 was added for the whole bridge area to act as weight of rails structures.

There is extra weight also because simplified profiles give a larger sum of mass than actual profiles are.
For the comparison, 26 m span bridge total mass according to FEM model 74,9t and according to the old design documents corresponding value is 52,6t. Safety margin for dead load is at least 1.4 times.

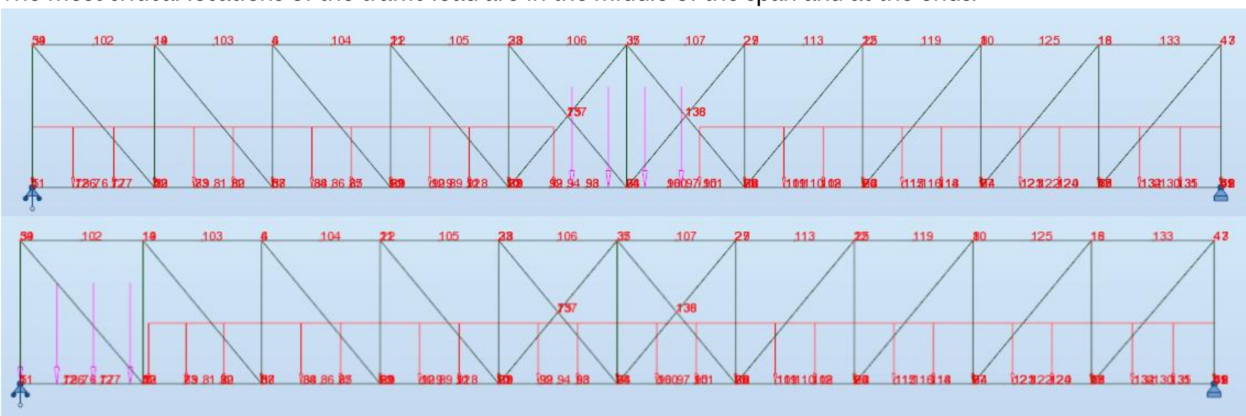
2.2.2 Train load

Train axle load is increased to 22,5 tons. Load is applied according to EN 1991-2, section 6.3.2, load model LM71.



Picture 26, train load model 71

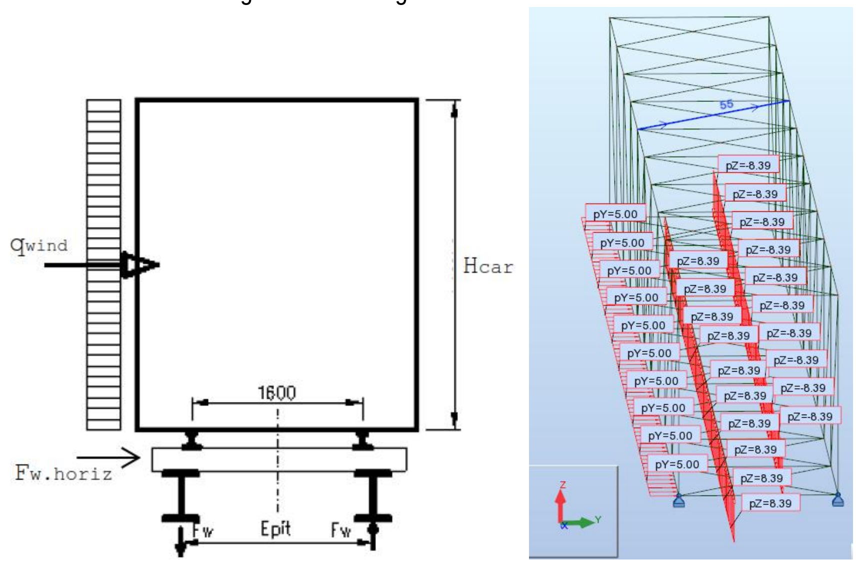
In the calculations, the trains were placed on all locations on the bridge. The load can be anywhere on the bridge. The most critical locations of the traffic load are in the middle of the span and at the ends.



Picture 27, load model 71 applications

2.2.3 Wind load

The applied characteristic wind load is 1 kN/m^2.
The wind effected area in truss bridges is minimal compared to the train area, so wind load is applied for the train cars for the whole length of the bridge.



Picture 28, Wind load

The most critical case for strains in structures is if bridge will be fully loaded at the same time with the wind. Structurally there is no such change that would make bridge behave differently from the last 100 years.

2.3 Load Combinations and combination factors

Load combinations are applied according to EN 1990, table A2.4

|AC> Table A2.4(A) Design values of actions (EQU) (Set A)

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10)	$\gamma_{G,j,sup} G_{k,j,sup}$	$\gamma_{G,j,inf} G_{k,j,inf}$	$\gamma_P P$	$\gamma_{Q,1} Q_{k,1}$		$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$

(*) Variable actions are those considered in Tables A2.1 to A2.3.

NOTE 1 The γ values for the persistent and transient design situations may be set by the National Annex.

For persistent design situations, the recommended set of values for γ are:

$\gamma_{G,sup} = 1,05$
$\gamma_{G,inf} = 0,95^{(1)}$
$\gamma_Q = 1,35$ for road and pedestrian traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1,45$ for rail traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1,50$ for all other variable actions for persistent design situations, where unfavourable (0 where favourable).
γ_P = recommended values defined in the relevant design Eurocode.

For transient design situations during which there is a risk of loss of static equilibrium, $Q_{k,1}$ represents the dominant destabilising variable action and $Q_{k,i}$ represents the relevant accompanying destabilising variable actions.

During execution, if the construction process is adequately controlled, the recommended set of values for γ are:
$\gamma_{G,sup} = 1,05$
$\gamma_{G,inf} = 0,95^{(1)}$
$\gamma_Q = 1,35$ for construction loads where unfavourable (0 where favourable)
$\gamma_Q = 1,50$ for all other variable actions, where unfavourable (0 where favourable)

(1) Where a counterweight is used, the variability of its characteristics may be taken into account, for example, by one or both of the following recommended rules:

— applying a partial factor $\gamma_{G,inf} = 0,8$ where the self-weight is not well defined (e.g. containers);

— by considering a variation of its project-defined position specified proportionately to the dimensions of the bridge, where the magnitude of the counterweight is well defined. For steel bridges during launching, the variation of the counterweight position is often taken equal to ± 1 m.

NOTE 2 For the verification of uplift of bearings of continuous bridges or in cases where the verification of static equilibrium also involves the resistance of structural elements (for example where the loss of static equilibrium is prevented by stabilising systems or devices, e.g. anchors, stays or auxiliary columns), as an alternative to two separate verifications based on Tables A2.4(A) and A2.4(B), a combined verification, based on Table A2.4(A), may be adopted. The National Annex may set the γ values. The following values of γ are recommended:
$\gamma_{G,sup} = 1,35$
$\gamma_{G,inf} = 1,25$
$\gamma_Q = 1,35$ for road and pedestrian traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1,45$ for rail traffic actions, where unfavourable (0 where favourable)
$\gamma_Q = 1,50$ for all other variable actions for persistent design situations, where unfavourable (0 where favourable)
$\gamma_Q = 1,35$ for all other variable actions, where unfavourable (0 where favourable) provided that applying $\gamma_{G,inf} = 1,00$ both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect.

Combination factors according to EN 1990, Table A2.3.

Table A2.3 Recommended values of ψ factors for railway bridges

Actions			ψ_0	ψ_1	ψ_2 4)
Main traffic actions (groups of loads)	gr11 (LM71 + SW/0)	Max. vertical 1 with max. longitudinal	0,80	0,80	0
	gr12 (LM71 + SW/0)	Max. vertical 2 with max. transverse			
	gr13 (Braking/traction)	Max. longitudinal			
	gr14 (Centrifugal/nosing)	Max. lateral			
	gr15 (Unloaded train)	Lateral stability with "unloaded train"			
	gr16 (SW/2)	SW/2 with max. longitudinal			
	gr17 (SW/2)	SW/2 with max. transverse			
	gr21 (LM71 + SW/0)	Max. vertical 1 with max. longitudinal	0,80	0,70	0
	gr22 (LM71 + SW/0)	Max. vertical 2 with max transverse			
	gr23 (Braking/traction)	Max. longitudinal			
	gr24 (Centrifugal/nosing)	Max. lateral			
	gr26 (SW/2)	SW/2 with max. longitudinal			
	gr27 (SW2)	SW/2 with max. transverse			
	gr31 (LM71 + SW/0)	Additional load cases	0,80	0,60	0
Other operating actions	Aerodynamic effects		0,80	0,50	0
	General maintenance loading for non public footpaths		0,80	0,50	0
Wind forces 2)	F_{Wk}		0,75	0,50	0
	F_W**		1,00	0	0
Thermal actions 3)	T_k		0,60	0,60	0,50
Snow loads	$Q_{Sn,k}$ (during execution)		0,8	–	0
Construction loads	Q_c		1,0	–	1,0

1) 0,8 if 1 track only is loaded
0,7 if 2 tracks are simultaneously loaded
0,6 if 3 or more tracks are simultaneously loaded.

2) When wind forces act simultaneously with traffic actions, the wind force ψ_0 F_{Wk} should be taken as no greater than F_W** (see EN 1991-1-4). See A2.2.4(4).

3) See EN 1991-1-5.

4) If deformation is being considered for Persistent and Transient design situations, ψ_2 should be taken equal to 1,00 for rail traffic actions. For seismic design situations, see Table A2.5.

5) Minimum coexistent favourable vertical load with individual components of rail traffic actions (e.g. centrifugal, traction or braking) is 0,5 LM71, etc.

NOTE 5 For specific design situations (e.g. calculation of bridge camber for aesthetics and drainage consideration, calculation of clearance, etc.) the requirements for the combinations of actions to be used may be defined for the individual project.

NOTE 6 For railway bridges, the infrequent value of variable actions is not relevant.

(2) |**AC**> For railway bridges <**AC**|, a unique ψ value should be applied to one group of loads as defined in EN 1991-2, and taken as equal to the ψ value applicable to the leading component of the group.

2.4 Materials

Steel Properties: Yield strength = 220 MPa
Ultimate tensile strength = 370 MPa
E = 205 000 MPa

Assumption is based on a UIC publication IRS 77802 “Assessment of Existing Steel Structures: Recommendations for Estimation of Remaining Fatigue Life”.

Assessment of Existing Steel Structures, Remaining Fatigue Life First edition 2008

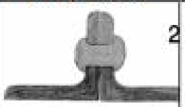
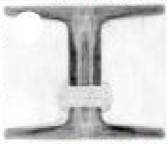
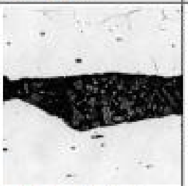
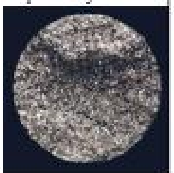
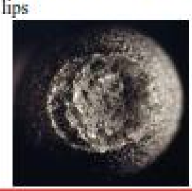
	Cast iron	Wrought steel Puddled steel	Mild steel (19th century)	Mild steel (20th century)
Sulphur-print (Baumann-print)	Content of sulphur is depending on coke quality	 Slag segregation lines containing phosphorus and sulphur	 Core segregation containing phosphorus and sulphur	 Low content of phosphorus and sulphur
Micro-structure ~1:400	 Cast iron with lamellar graphite	 Ferritic, Inhomogeneous grain size distribution, Oxide inclusions, Slag lines	 Ferritic-pearlitic, Increasing grain size from the edge to the core, Oxide and sulphide inclusions	 Homogenous small grain
Chemical analysis	C ≈ 2,0-4,0 % Mn ≈ 0,2-1,2 % Si ≈ 0,3-3,0 % S < ≈ 1,2 % P < ≈ 1,0 %	Very variable C < ≈ 0,08 % Mn < ≈ 0,4 % S < ≈ 0,04 % P < ≈ 0,6 %	**Bessemer / Thomas steel** C ≈ 0,02-0,1 % Mn ≈ 0,3-0,5 % S < ≈ 0,1 % P≈0,04-0,07%(**B**)/ -0,12%(**T**) **Siemens-Martin steel** C ≈ 0,05-0,15 % Mn ≈ 0,2-0,5 % S ≈ 0,02-0,15 % P ≈ 0,03-0,06 %	Low-alloyed steel **(T, SM)** C ≈ 0,1-0,2 % Mn ≈ 0,4-0,5 % Si ≈ 0,01 %
			Blast Process: N>≈0,01%, Hearth Process: N<≈0,01%	
			Bessemer steel: Si>≈0,08%, Thomas steel: Si<≈0,08%	
Tension test	Very brittle, almost no plasticity 	No local necking 	local necking 	Local necking and shear lips 
Tension strength	Old cast iron R_m≈90-135[1] N/mm² ε [2] ≈ 0 %	R_e ≈ 220-310 N/mm² R_m ≈ 280-400 N/mm² ε ≈ 5-20 %	R_e > ≈ 220 N/mm² R_m ≈ 370-440 N/mm² ε > ≈ 20 %	**Low-alloyed steel** R_e ≈ 240-280 N/mm² R_m ≈ 370-450 N/mm² ε ≈ 15-25 %
Specimen after tension test	No local necking 		Local necking 	

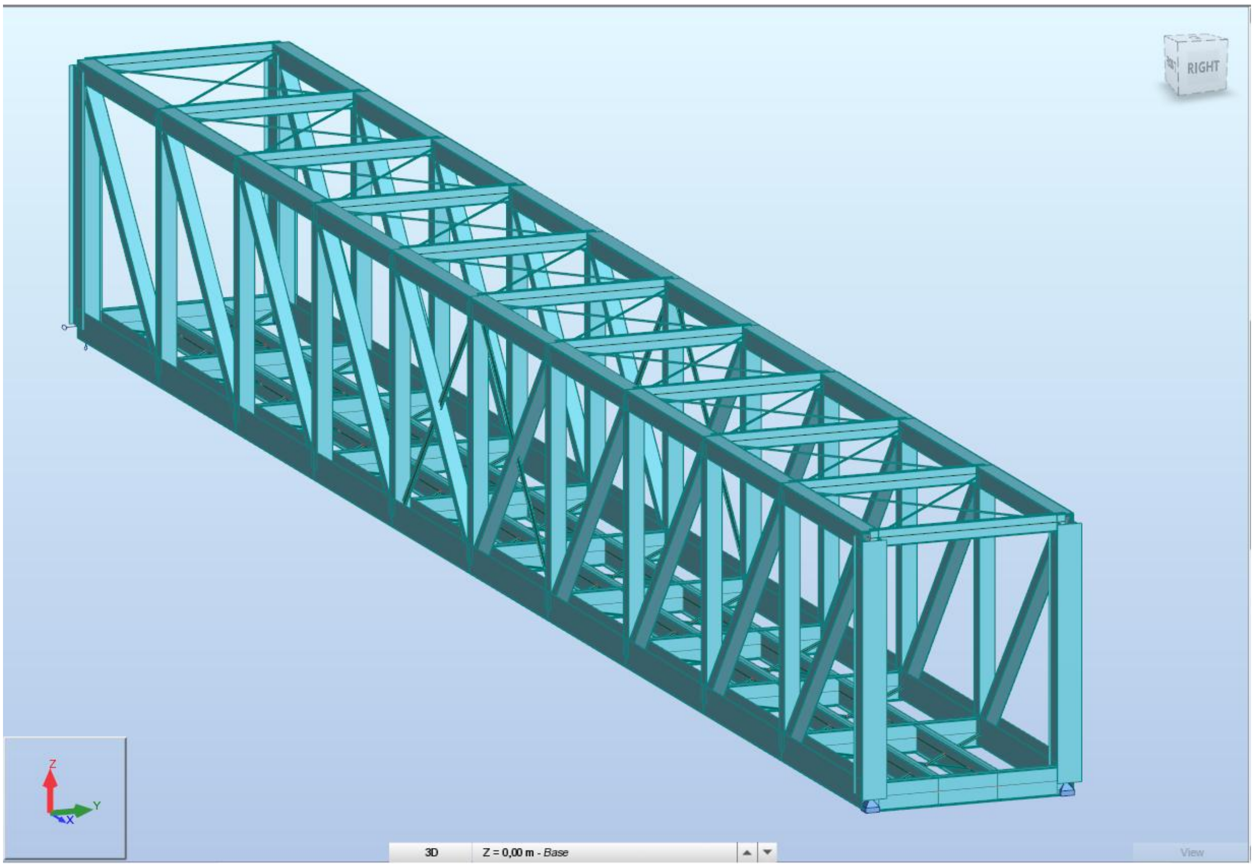
Table 3-2: Information on material characteristics of old iron and steels

[1] Literature [Lit. 96] gives also higher values up to 260 N/mm²; [2] elongation at rupture

Picture 29, steel material properties

3 RESULTS

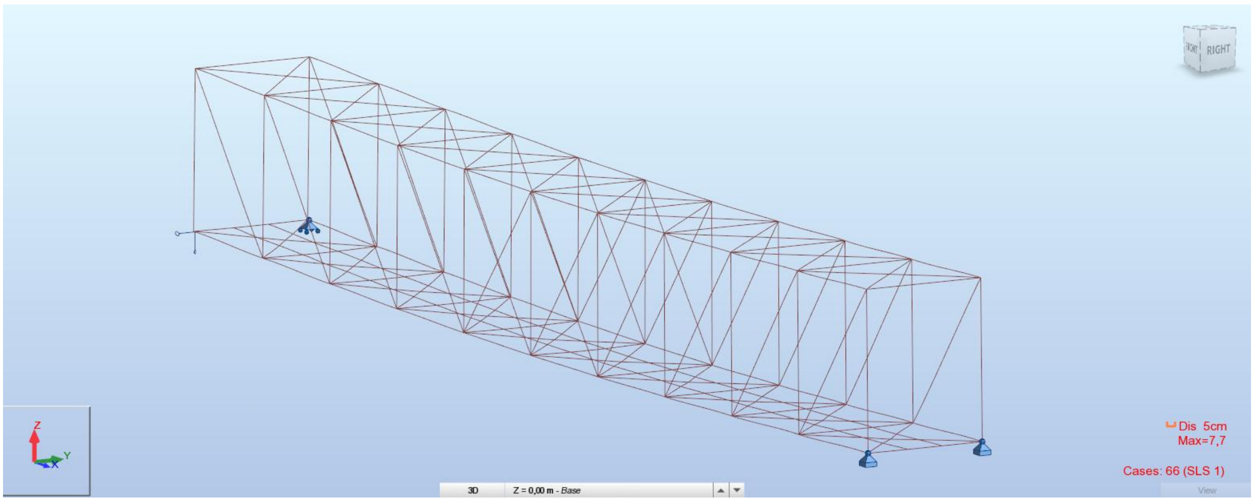
3.1 52 m Span Truss Bridge



Picture 30, View of FEM model

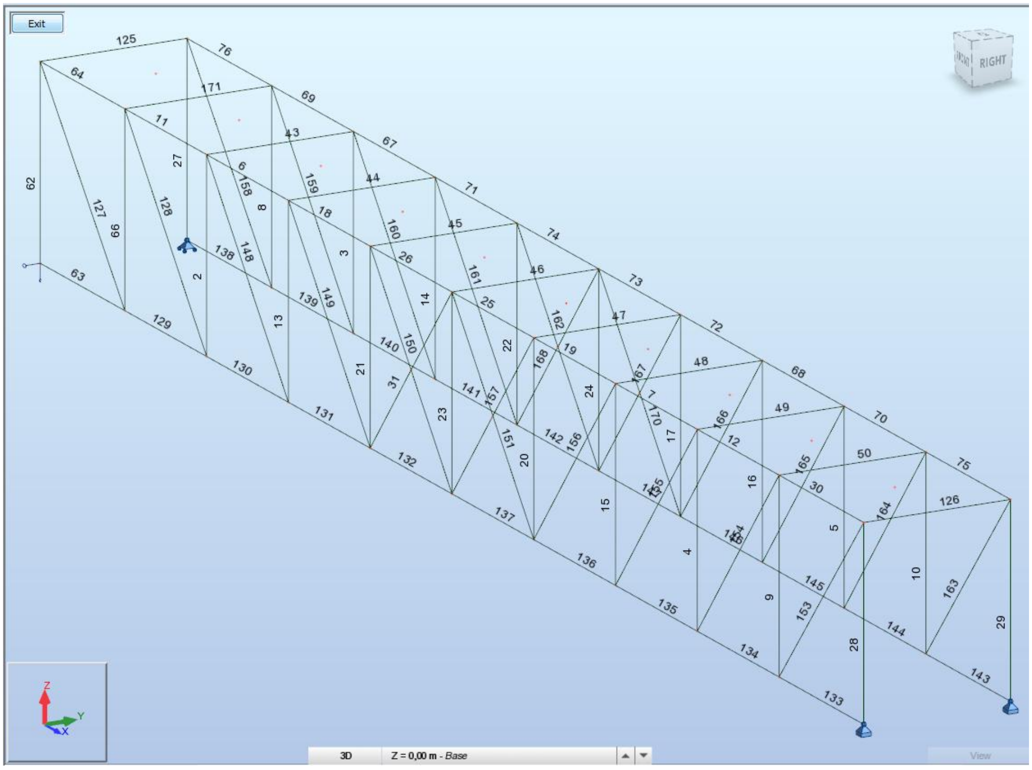
3.1.1 SLS results (Serviceability Limit State)

Total deflection of bridge is 7,7 cm = L/675.



Picture 31, Deflection

3.1.2 ULS results (Ultimate Limit State)



Picture 32, member numbers for profiles that will be utilized from old construction

After analysis, a utilization ratio may be calculated for each member of the bridge truss. The highest utilization for tension member is 0.84 and for compressed member 0.76.

Table 1, printout of utilization of profiles in order of section with highest stress

SFS-EN 1993-1:2005/NA:2007/A1:2014 - Member Verification (ULS) 2to31 43to50 62to64 66to7

Results | Messages

Tension members

Member		Section	Material	Lay	Laz	Ratio	Case
142	OK	CHAN_V_	S220	26.57	22.29	0.84	22 L2.Eq.6.10b/1
147	OK	CHAN_V_1	S220	26.57	22.29	0.84	22 L2.Eq.6.10b/1
132	OK	CHAN_V_1	S220	26.57	22.29	0.84	22 L2.Eq.6.10b/1
137	OK	CHAN_V_1	S220	26.57	22.29	0.84	22 L2.Eq.6.10b/1
153	OK	BOX1_V_2	S220	73.57	54.84	0.80	22 L2.Eq.6.10b/1
158	OK	BOX1_V_2	S220	73.57	54.84	0.80	22 L2.Eq.6.10b/1
127	OK	BOX1_V_2	S220	73.57	54.84	0.80	22 L2.Eq.6.10b/1
163	OK	BOX1_V_2	S220	73.57	54.84	0.80	22 L2.Eq.6.10b/1
26	OK	UMG_TYPE3	S235	29.66	30.78	0.76	22 L2.Eq.6.10b/1
74	OK	UMG_TYPE3	S235	29.66	30.78	0.76	22 L2.Eq.6.10b/1
25	OK	UMG_TYPE3	S235	29.66	30.78	0.76	22 L2.Eq.6.10b/1
73	OK	UMG_TYPE3	S235	29.66	30.78	0.76	22 L2.Eq.6.10b/1
19	OK	UMG_TYPE3	S235	29.66	30.78	0.73	22 L2.Eq.6.10b/1
18	OK	UMG_TYPE3	S235	29.66	30.78	0.73	22 L2.Eq.6.10b/1
71	OK	UMG_TYPE3	S235	29.66	30.78	0.73	22 L2.Eq.6.10b/1
72	OK	UMG_TYPE3	S235	29.66	30.78	0.73	22 L2.Eq.6.10b/1
7	OK	UMG_TYPE2	S235	29.63	29.84	0.72	22 L2.Eq.6.10b/1
6	OK	UMG_TYPE2	S235	29.63	29.84	0.72	22 L2.Eq.6.10b/1
67	OK	UMG_TYPE2	S235	29.63	29.84	0.72	22 L2.Eq.6.10b/1
68	OK	UMG_TYP	S235	29.63	29.84	0.72	22 L2.Eq.6.10b/1
141	OK	CHAN_V_1	[illegible]	[illegible]	[illegible]	[illegible]	L2.Eq.6.10b/1
146	OK	CHAN_V_1	[illegible]	[illegible]	[illegible]	[illegible]	L2.Eq.6.10b/1
131	OK	CHAN_V_1	[illegible]	[illegible]	[illegible]	[illegible]	L2.Eq.6.10b/1
136	OK	CHAN_V_1	S220	26.57	22.29	0.69	22 L2.Eq.6.10b/1
154	OK	BOX1_V_3	S220	70.59	54.61	0.66	22 L2.Eq.6.10b/1
159	OK	BOX1_V_3	S220	70.59	54.61	0.66	22 L2.Eq.6.10b/1

Upper main girders
Compressed.
Profile simplified on a safe side considering
buckling effects

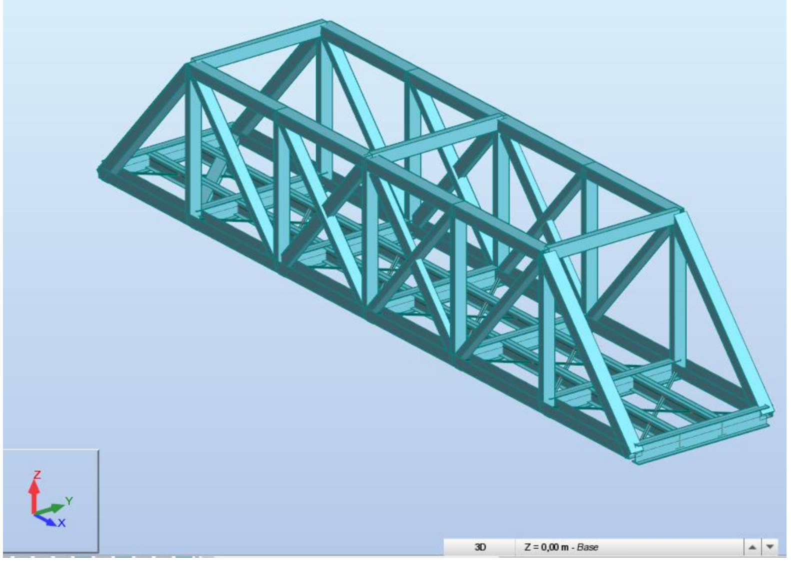
Table 2, printout of utilization of profiles in order of stress.

Member	Section	Material	Lay	Laz	Ratio	Case
142	CHAN_V_1	S220	26.57	22.29	0.84	22 L2.Eq.6.10b/1
147	CHAN_V_1	S220	26.57	22.29	0.84	22 L2.Eq.6.10b/1
132	CHAN_V_1	S220	26.57	22.29	0.84	22 L2.Eq.6.10b/1
137	CHAN_V_1	S220	26.57	22.29	0.84	22 L2.Eq.6.10b/1
153	BOX1_V_2	S220	73.57	54.54	0.80	22 L2.Eq.6.10b/1
158	BOX1_V_2	S220	73.57	54.54	0.80	22 L2.Eq.6.10b/1
127	BOX1_V_2	S220	73.57	54.54	0.80	22 L2.Eq.6.10b/1
163	BOX1_V_2	S220	73.57	54.54	0.80	22 L2.Eq.6.10b/1
26	UMG_TYPE3	S235	29.66	30.75	0.76	22 L2.Eq.6.10b/1
74	UMG_TYPE3	S235	29.66	30.75	0.76	22 L2.Eq.6.10b/1
25	UMG_TYPE3	S235	29.66	30.75	0.76	22 L2.Eq.6.10b/1
73	UMG_TYPE3	S235	29.66	30.75	0.76	22 L2.Eq.6.10b/1
19	UMG_TYPE3	S235	29.66	30.75	0.73	22 L2.Eq.6.10b/1
18	UMG_TYPE3	S235	29.66	30.75	0.73	22 L2.Eq.6.10b/1
71	UMG_TYPE3	S235	29.66	30.75	0.73	22 L2.Eq.6.10b/1
72	UMG_TYPE3	S235	29.66	30.75	0.73	22 L2.Eq.6.10b/1
7	UMG_TYPE2	S235	29.63	29.54	0.72	22 L2.Eq.6.10b/1
6	UMG_TYPE2	S235	29.63	29.54	0.72	22 L2.Eq.6.10b/1
67	UMG_TYPE2	S235	29.63	29.54	0.72	22 L2.Eq.6.10b/1
68	UMG_TYPE2	S235	29.63	29.54	0.72	22 L2.Eq.6.10b/1
141	CHAN_V_1	S220	26.57	22.29	0.69	22 L2.Eq.6.10b/1
146	CHAN_V_1	S220	26.57	22.29	0.69	22 L2.Eq.6.10b/1
131	CHAN_V_1	S220	26.57	22.29	0.69	22 L2.Eq.6.10b/1
136	CHAN_V_1	S220	26.57	22.29	0.69	22 L2.Eq.6.10b/1
154	BOX1_V_3	S220	70.59	54.61	0.66	22 L2.Eq.6.10b/1
159	BOX1_V_3	S220	70.59	54.61	0.66	22 L2.Eq.6.10b/1
128	BOX1_V_3	S220	70.59	54.61	0.66	22 L2.Eq.6.10b/1
164	BOX1_V_3	S220	70.59	54.61	0.66	22 L2.Eq.6.10b/1
9	BOX1_V_7	S220	47.69	37.46	0.66	22 L2.Eq.6.10b/1
66	BOX1_V_7	S220	47.69	37.46	0.66	22 L2.Eq.6.10b/1
10	BOX1_V_7	S220	47.69	37.46	0.66	22 L2.Eq.6.10b/1
8	BOX1_V_7	S220	47.69	37.46	0.66	22 L2.Eq.6.10b/1
155	BOX1_V_4	S220	55.24	55.25	0.58	22 L2.Eq.6.10b/1
160	BOX1_V_4	S220	55.24	55.25	0.58	22 L2.Eq.6.10b/1
165	BOX1_V_4	S220	55.24	55.25	0.58	22 L2.Eq.6.10b/1
145	BOX1_V_4	S220	55.24	55.25	0.58	22 L2.Eq.6.10b/1
12	UMG_TYPE2	S235	29.63	29.54	0.55	22 L2.Eq.6.10b/1
11	UMG_TYPE2	S235	29.63	29.54	0.55	22 L2.Eq.6.10b/1
69	UMG_TYPE2	S235	29.63	29.54	0.55	22 L2.Eq.6.10b/1
70	UMG_TYPE2	S235	29.63	29.54	0.55	22 L2.Eq.6.10b/1
140	CHAN_V_1	S220	26.57	22.29	0.52	22 L2.Eq.6.10b/1
145	CHAN_V_1	S220	26.57	22.29	0.52	22 L2.Eq.6.10b/1
130	CHAN_V_1	S220	26.57	22.29	0.52	22 L2.Eq.6.10b/1
135	CHAN_V_1	S220	26.57	22.29	0.52	22 L2.Eq.6.10b/1
2	BOX1_V_7	S220	47.69	37.46	0.49	22 L2.Eq.6.10b/1
4	BOX1_V_7	S220	47.69	37.46	0.49	22 L2.Eq.6.10b/1
5	BOX1_V_7	S220	47.69	37.46	0.49	22 L2.Eq.6.10b/1

Member	Section	Material	Lay	Laz	Ratio	Case
3	OK BOX1_V_7	S220	47.69	37.46	0.49	22 L2.Eq.6.10b/1
156	OK BOX1_V_5	S220	53.72	54.65	0.45	22 L2.Eq.6.10b/1
161	OK BOX1_V_5	S220	53.72	54.65	0.45	22 L2.Eq.6.10b/1
166	OK BOX1_V_5	S220	53.72	54.65	0.45	22 L2.Eq.6.10b/1
149	OK BOX1_V_5	S220	53.72	54.65	0.45	22 L2.Eq.6.10b/1
143	OK CHAN_V_1	S220	27.49	23.07	0.42	33 L3.Eq.6.10b/2
25	OK BOX3_1	S220	24.51	33.07	0.37	22 L2.Eq.6.10b/1
29	OK BOX3_1	S220	24.51	33.07	0.37	22 L2.Eq.6.10b/1
62	OK BOX3_1	S220	24.51	33.07	0.37	22 L2.Eq.6.10b/1
27	OK BOX3_1	S220	24.51	33.07	0.37	22 L2.Eq.6.10b/1
133	OK CHAN_V_1	S220	27.49	23.07	0.35	22 L2.Eq.6.10b/1
63	OK CHAN_V_1	S220	27.49	23.07	0.35	22 L2.Eq.6.10b/1
138	OK CHAN_V_1	S220	27.49	23.07	0.35	22 L2.Eq.6.10b/1
14	OK BOX1_V_7	S220	47.69	37.46	0.33	22 L2.Eq.6.10b/1
13	OK BOX1_V_7	S220	47.69	37.46	0.33	22 L2.Eq.6.10b/1
15	OK BOX1_V_7	S220	47.69	37.46	0.33	22 L2.Eq.6.10b/1
16	OK BOX1_V_7	S220	47.69	37.46	0.33	22 L2.Eq.6.10b/1
30	OK UMG_TYPE1	S235	30.75	30.51	0.32	22 L2.Eq.6.10b/1
64	OK UMG_TYPE1	S235	30.75	30.51	0.32	22 L2.Eq.6.10b/1
76	OK UMG_TYPE1	S235	30.75	30.51	0.32	22 L2.Eq.6.10b/1
75	OK UMG_TYPE1	S235	30.75	30.51	0.32	22 L2.Eq.6.10b/1
139	OK CHAN_V_1	S220	26.57	22.29	0.26	22 L2.Eq.6.10b/1
129	OK CHAN_V_1	S220	26.57	22.29	0.26	22 L2.Eq.6.10b/1
144	OK CHAN_V_1	S220	26.57	22.29	0.26	22 L2.Eq.6.10b/1
134	OK CHAN_V_1	S220	26.57	22.29	0.26	22 L2.Eq.6.10b/1
162	OK BOX1_V_6	S220	103.53	56.39	0.15	22 L2.Eq.6.10b/1
167	OK BOX1_V_6	S220	103.53	56.39	0.15	22 L2.Eq.6.10b/1
150	OK BOX1_V_6	S220	103.53	56.39	0.15	22 L2.Eq.6.10b/1
157	OK BOX1_V_6	S220	103.53	56.39	0.15	22 L2.Eq.6.10b/1
151	OK 203*25	S220	135.56	1006.73	0.13	22 L2.Eq.6.10b/1
31	OK 203*25	S220	135.56	1006.73	0.13	22 L2.Eq.6.10b/1
170	OK 203*25	S220	135.56	1006.73	0.13	22 L2.Eq.6.10b/1
165	OK 203*25	S220	135.56	1006.73	0.13	22 L2.Eq.6.10b/1
22	OK BOX1_V_7	S220	47.69	37.46	0.11	22 L2.Eq.6.10b/1
21	OK BOX1_V_7	S220	47.69	37.46	0.11	22 L2.Eq.6.10b/1
17	OK BOX1_V_7	S220	47.69	37.46	0.11	22 L2.Eq.6.10b/1
20	OK BOX1_V_7	S220	47.69	37.46	0.11	22 L2.Eq.6.10b/1
46	OK U_crossbeam	S220	30.63	155.61	0.07	22 L2.Eq.6.10b/1
47	OK U_crossbeam	S220	30.63	155.61	0.07	22 L2.Eq.6.10b/1
45	OK U_crossbeam	S220	30.63	155.61	0.07	22 L2.Eq.6.10b/1
44	OK U_crossbeam	S220	30.63	155.61	0.07	22 L2.Eq.6.10b/1
45	OK U_crossbeam	S220	30.63	155.61	0.07	22 L2.Eq.6.10b/1
23	OK BOX1_V_7	S220	47.69	37.46	0.06	22 L2.Eq.6.10b/1
24	OK BOX1_V_7	S220	47.69	37.46	0.06	22 L2.Eq.6.10b/1
43	OK U_crossbeam	S220	30.63	155.61	0.06	22 L2.Eq.6.10b/1
49	OK U_crossbeam	S220	30.63	155.61	0.06	22 L2.Eq.6.10b/1
171	OK U_crossbeam	S220	30.63	155.61	0.04	22 L2.Eq.6.10b/1

Member	Section	Material	Lay	Laz	Ratio	Case
50	OK U_crossbeam	S220	30.63	155.61	0.04	22 L2.Eq.6.10b/1
126	OK U_crossbeam	S220	30.63	155.61	0.02	22 L2.Eq.6.10b/1
125	OK U_crossbeam	S220	30.63	155.61	0.02	22 L2.Eq.6.10b/1

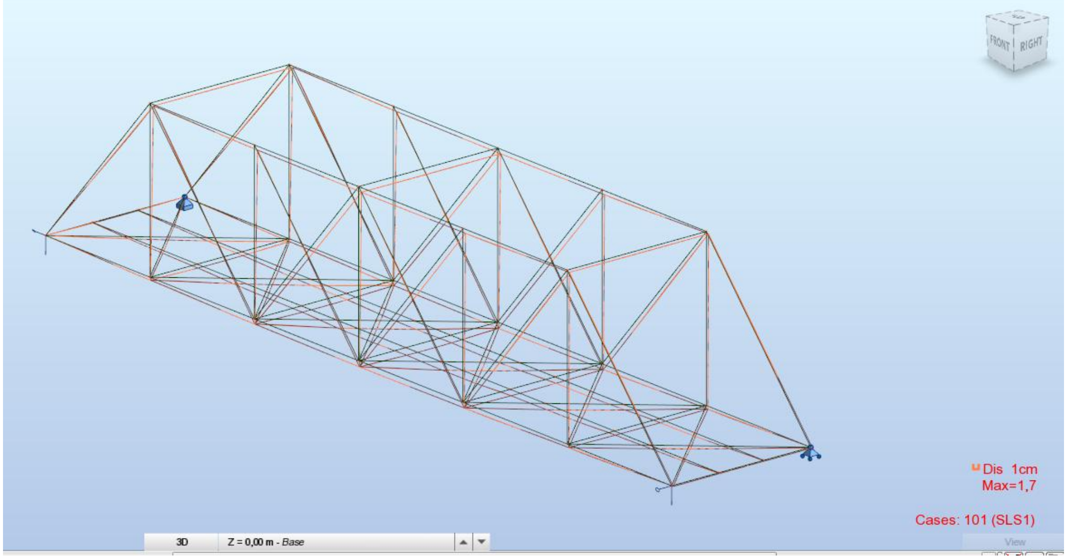
3.2 26 m span truss bridge



Picture 34, View of FEM model

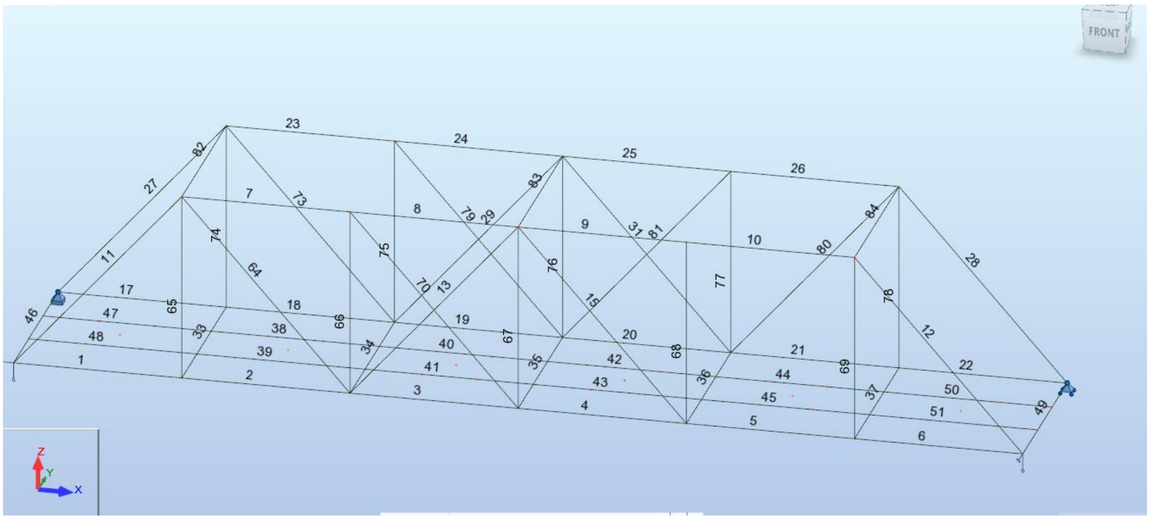
3.2.1 SLS results (Serviceability Limit State)

Total deflection of bridge is 1,7 cm = L/1500



Picture 35, Deflection

3.2.2 ULS results (Ultimate Limit State)



Picture 36, member numbers for profiles that will be utilized from old construction

After analysis, a utilization ratio may be calculated for each member of the bridge truss. The highest utilization ratio for the members is 0,70...076.

Table 2, printout of utilization of profiles in order of section with highest stress

Member			Material	Lay	Laz	Ratio	Case
50	OK	LG	S220	25.23	73.60	0.76	202 L2.Eq.6.10b/1
42	OK	LG	S220	25.23	73.60	0.70	202 L2.Eq.6.10b/1
51	OK	LG	S220	25.23	73.60	0.68	202 L2.Eq.6.10b/1
44	OK	LG	S220	25.23	73.60	0.66	202 L2.Eq.6.10b/1
40	OK	LG	S220	25.23	73.60	0.66	202 L2.Eq.6.10b/1
43	OK	LG	S220	25.23	73.60	0.62	202 L2.Eq.6.10b/1
45	OK	LG	S220	25.23	73.60	0.58	202 L2.Eq.6.10b/1
47	OK	LG	S220	25.23	73.60	0.58	202 L2.Eq.6.10b/1
41	OK	LG	S220	25.23	73.60	0.57	202 L2.Eq.6.10b/1
38	OK	LG	S220	25.23	73.60	0.54	202 L2.Eq.6.10b/1
48	OK	LG	S220	25.23	73.60	0.52	202 L2.Eq.6.10b/1
39	OK	LG	S220	25.23	73.60	0.47	202 L2.Eq.6.10b/1
35	OK	CG	S220	20.11	71.54	0.46	202 L2.Eq.6.10b/1
20	OK	LMG	S220	21.48	35.27	0.41	202 L2.Eq.6.10b/1
19	OK	LMG	S220	21.48	35.27	0.40	202 L2.Eq.6.10b/1
4	OK	LMG	S220	21.48	35.27	0.38	202 L2.Eq.6.10b/1
3	OK	LMG	S220	21.48	35.27	0.37	202 L2.Eq.6.10b/1
36	OK	CG	S220	20.11	71.54	0.36	202 L2.Eq.6.10b/1
25	OK	UMG	S220	30.30	23.93	0.36	202 L2.Eq.6.10b/1
24	OK	UMG	S220	30.30	23.93	0.36	202 L2.Eq.6.10b/1
73	OK	DIAG_1	S220	40.53	74.84	0.35	202 L2.Eq.6.10b/1
80	OK	DIAG_1	S220	40.53	74.84	0.35	202 L2.Eq.6.10b/1
34	OK	CG	S220	20.11	71.54	0.34	202 L2.Eq.6.10b/1
9	OK	UMG	S220	30.30	23.93	0.34	202 L2.Eq.6.10b/1
8	OK	UMG	S220	30.30	23.93	0.34	202 L2.Eq.6.10b/1
71	OK	DIAG_1	S220	40.53	74.84	0.33	202 L2.Eq.6.10b/1
64	OK	DIAG_1	S220	40.53	74.84	0.33	202 L2.Eq.6.10b/1
28	OK	UMG	S220	45.74	36.13	0.33	202 L2.Eq.6.10b/1
27	OK	UMG	S220	45.74	36.13	0.33	202 L2.Eq.6.10b/1
26	OK	UMG	S220	30.30	23.93	0.33	202 L2.Eq.6.10b/1
23	OK	UMG	S220	30.30	23.93	0.33	202 L2.Eq.6.10b/1

Member	Section	Material	Lay	Laz	Ratio	Case
37	CG	S220	20.11	71.54	0.31	202 L2.Eq.6.10b/1
11	UMG	S220	45.74	36.13	0.31	202 L2.Eq.6.10b/1
7	UMG	S220	30.30	23.93	0.31	202 L2.Eq.6.10b/1
12	UMG	S220	45.74	36.13	0.31	202 L2.Eq.6.10b/1
10	UMG	S220	30.30	23.93	0.31	202 L2.Eq.6.10b/1
22	LMG	S220	21.48	35.27	0.27	202 L2.Eq.6.10b/1
6	LMG	S220	21.48	35.27	0.25	202 L2.Eq.6.10b/1
49	CG	S220	20.11	71.54	0.24	202 L2.Eq.6.10b/1
1	LMG	S220	21.48	35.27	0.24	202 L2.Eq.6.10b/1
17	LMG	S220	21.48	35.27	0.23	202 L2.Eq.6.10b/1
46	CG	S220	20.11	71.54	0.21	202 L2.Eq.6.10b/1
21	LMG	S220	21.48	35.27	0.20	202 L2.Eq.6.10b/1
5	LMG	S220	21.48	35.27	0.19	202 L2.Eq.6.10b/1
18	LMG	S220	21.48	35.27	0.17	202 L2.Eq.6.10b/1
2	LMG	S220	21.48	35.27	0.16	202 L2.Eq.6.10b/1
29	DIAG_1	S220	40.53	74.84	0.14	202 L2.Eq.6.10b/1
33	CG	S220	20.11	71.54	0.14	202 L2.Eq.6.10b/1
78	VERT	S220	29.54	53.09	0.13	202 L2.Eq.6.10b/1
74	VERT	S220	29.54	53.09	0.13	202 L2.Eq.6.10b/1
13	DIAG_1	S220	40.53	74.84	0.13	202 L2.Eq.6.10b/1
15	DIAG_1	S220	40.53	74.84	0.13	202 L2.Eq.6.10b/1
65	VERT	S220	29.54	53.09	0.12	202 L2.Eq.6.10b/1
69	VERT	S220	29.54	53.09	0.12	202 L2.Eq.6.10b/1
31	DIAG_1	S220	40.53	74.84	0.12	202 L2.Eq.6.10b/1
77	VERT	S220	29.54	53.09	0.10	202 L2.Eq.6.10b/1
75	VERT	S220	29.54	53.09	0.10	202 L2.Eq.6.10b/1
68	VERT	S220	29.54	53.09	0.09	202 L2.Eq.6.10b/1
81	DIAG_1	S220	40.53	74.84	0.09	202 L2.Eq.6.10b/1
79	DIAG_1	S220	40.53	74.84	0.09	202 L2.Eq.6.10b/1
66	VERT	S220	29.54	53.09	0.09	202 L2.Eq.6.10b/1
72	DIAG_1	S220	40.53	74.84	0.09	202 L2.Eq.6.10b/1
70	DIAG_1	S220	40.53	74.84	0.09	202 L2.Eq.6.10b/1
76	VERT	S220	29.54	53.09	0.08	202 L2.Eq.6.10b/1
67	VERT	S220	29.54	53.09	0.08	202 L2.Eq.6.10b/1
83	VERT	S220	28.99	52.11	0.01	201 L1.Eq.6.10a
82	VERT	S220	28.99	52.11	0.01	201 L1.Eq.6.10a
84	VERT	S220	28.99	52.11	0.01	201 L1.Eq.6.10a

4 CROSS GIRDER-RAIL BEARER JOINT

A connection verification was carried out for the cross girder-rail bearer joint of the 52-m span bridge. The result shows (Appendix 2), that the connection's resistance is not adequate against the design force. The basic requirement is that the resistance is greater than the forces, but analysis shows that (V_{Ed}/V_{Rd}) = 1.194 > 1. In addition, the there are many uncertainties to the calculation, since the condition of these joints, especially the main plates under the cover plates is unknown and not visible.

The document IRS 77802 (former UIC 778-2) "Recommendations for determining the carrying capacity and fatigue risks of existing metallic railway bridges" gives instructions for Fatigue Susceptible Details, which generally have a more unreliable fatigue performance and experience indicates they are more prone to fatigue cracking than or other typical design details in modern bridges.

Typically Fatigue Susceptible Details:

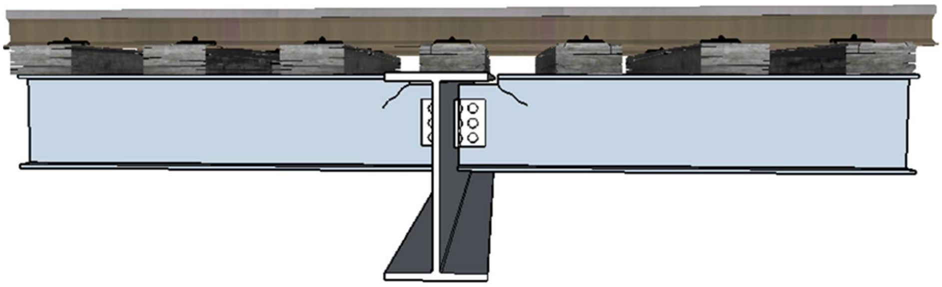
- *are subject to significant cycles of stress from short influence line length load effects that are neglected at the ULS (for example rail bearer joints that are assumed to be pinned joints at ULS subject to cycles of stress from passing individual axles) and or;*
- *are subject to significant cycles of stress from the real "whole bridge" behavior or the real distribution of stresses in complex details and or connections that is neglected at the ULS,*

for example cross girder end joints that have additional stresses induced by the differential global deflection of a bridge (particularly skew bridges) and / or;

- *have additional stress concentration features present that are not present in a similar detail tested to establish the fatigue performance of the detail.*

Examples of Fatigue Susceptible Details

- *An example of a Fatigue Susceptible Detail is a notched rail bearer to cross girder connection, especially where the notch has been flame cut:*



Picture 37. Joint of typical cross girder connection, one example (IRS 77802).

Joints that are Fatigue Susceptible Details include:

- *joints with other geometrical stress concentration features,*
- *misaligned load carrying parts*
- *joints subject to multiple cycles of stress due to the passage of individual axles*

An example of a fatigue susceptible joint is also a rail bearer to cross girder connection with flange plates providing continuity between adjoining rail bearers. This arrangement results in these joints being subject to multiple cycles of stress from the passage of individual axles as well as tension loading effects arising from the floor of a bridge being located below the neutral axis of the bridge superstructure.

Moreover, in case of a fatigue analysis wants to be performed, the dismantling of the joint is needed to gather sufficient information on the existing structure (conditions, presence of cracks in web). To ensure the safety of the structure, changing the critical fatigue sensitive connection parts (cross girders, rail bearers), is a suitable solution.

5 CONCLUSIONS

<u>52 m span</u>

Utilization of profiles for 52 m span truss bridge are 84 % at the most critical section. The engineer has chosen all the assumptions with safety margins. These results show that it is a feasible solution to use the old main trusses and replace only the cross beams and longitudinal rail supporting beams.

The deflection is L/675, less than allowed L/600 according to EN 1990-1, A2.4.4.2.3 (1), [1].

<u>26 m span</u>

The 26 m span truss bridge are also within acceptable level with the proposed heavier load. Utilizations ratios are at the most critical points < 0.76 and the maximum deflection is only L/1500.

<u>15 m span (girder)</u>

Based on economic and technical reasons, the result is to renew entire span(s). In all of these spans the new structure is thought in the pre-engineering phase to be embedded rail, so the height of the secondary structures can be increased for capacity reasons compared to existing situation with wooden sleepers.

Joints of cross-girder – Rail bearer connection

Based on the studies and calculations, shows that the capacity of joints is not sufficient (V_{Ed}/V_{Rd}) = 1.194 > 1. There are many uncertainties to these calculations and to find a solution to save the secondary structures, more detailed analysis is needed and the dismantling of the joint is needed to gather sufficient information on the existing structure (conditions, presence of cracks in web). To ensure the safety of the structure, changing the critical fatigue sensitive connection parts (cross girders, rail bearers), is a suitable solution.

LITERATURE

[1] EN 1990: Basis of Structural design
[2] EN 1991-2: Design of Steel Structures. Part 2: Traffic loads on bridges
[3] EN 1993-1-1: Design of Steel Structures. Part 1-1: General rules and rules for buildings
[4] EN 1993-1-8: Design of Steel Structures. Part 1-8: Design of joints
[5] EN 1993-2: Design of Steel Structures. Part 2: Steel bridges
[6] IRS 77802: Assessment of existing Steel Structures: Recommendations for Estimation of Remaining Fatigue Life; Eurocode Background Documents; JRC Scientific and Technical Reports
[7] Riveted Connections in Historical Metal Structures (1840-1940): Hot Driven Rivets: Technology and Experiments. Quentin Collette, Thesis, Doctor in Engineering, Vilje Universiteit Brussel.

APPENDIX 1: 15 M GIRDER BRIDGE CALCULATION

APPENDIX 2: CONNENCTION CALCULATION

APPENDIX 1

Preliminary calculation

References: EN1993-1-1, Design of steel structures
EN1991-2, Actions on structures - Part 2: Traffic loads on bridges

Material: Steel 355 $f_y := 355 \frac{N}{mm^2}$

$\rho := 7850 \frac{kg}{m^3}$

$g := 9.8 \frac{m}{s^2}$

Cross girder:

Geometry:

I - profile: $H := 394mm$

$t_w := 12mm$

$b_f := 250mm$

$t_f := 24mm$

$h_w := H - 2 \cdot t_f = 346 \cdot mm$

$A_{cross} := h_w \cdot t_w + 2b_f \cdot t_f = 16152 \cdot mm^2$

cross girder length: $l_{cross} := 4.727m$

Dynamic factor

For track with standard maintenance:

$$1 < \phi = \frac{2.16}{\sqrt{L_\phi} - 0.2} + 0.73 < 2$$

Ref. EN1991-
§6.4.5.2 (2)

Detenninant length L.ϕ:

Steel grillage: open deck without ballast bed [b] (for local and transverse stresses)		
3.1	Rail bearers: - as an element of a continuous grillage - simply supported	3 times cross girder spacing Cross girder spacing + 3 m
3.2	Cantilever of rail bearer [a]	3,6m
3.3	Cross girders (as part of cross girder/ continuous rail bearer grillage)	Twice the length of the cross girder
3.4	End cross girders	3,6m [b]
[a] In general all cantilevers greater than 0,50 m supporting rail traffic actions need a special study in accordance with 6.4.6 and with the loading agreed with the relevant authority specified in the National Annex [b] It is recommended to apply Φ_3		

Ref. EN1991-2 §6.4.5.3 Table 6.2

Cross girder length: $l_{cross} = 4.727\,m$

Dinamic factor for cross girder:

$$L_{\phi.cross} := 2 \cdot l_{cross} = 9.454\,m$$

$$\phi_{cross} := \frac{2.16}{\sqrt{\frac{L_{\phi.cross}}{m}} - 0.2} + 0.73 = 1.481$$

Loads:

Self weight:

$$G_{cross} := A_{cross} \cdot \rho \cdot g = 1243 \cdot \frac{N}{m}$$

Rail self weight: $G_{rail} := 1\,\frac{kN}{m}$

Traffic Load: LM71-22,5

$$Q_v := 125kN$$

$$q_v := 40\,\frac{kN}{m}$$

$Q_v := \phi_{cross} \cdot Q_v = 185 \cdot kN$

$q_v := \phi_{cross} \cdot q_v = 59 \cdot \frac{kN}{m}$

For cross girder, I calculated the load, which comes from traffic load and transfered b rails and longitudinal girders to the cross girder.

distance between cross girders: $L_{cross} := 2438mm$

from distributed load - q.v: $F_{traffic.q} := q_v \cdot L_{cross} = 144 \cdot kN$

$n := 1$ number of pont loads, which carried by one cross girder

0 < $L_{cross} - n \cdot 1.6m = 0.838\,m$ < 1.6 m

from point load - Q.v: $F_{traffic.Q} := (n + 1) \cdot Q_v = 370 \cdot kN$

For now I only work with these two cases, where the cross gider first only subjacted to the distributed load (from LM71), then only the concentrated load (from LM71)

$F_{traffic} := \max\left(F_{traffic.q}, F_{traffic.Q}\right) = 370.343 \cdot kN$

Wind load:

height of the car: $H_{car} := 5m$

Mean wind load: $q_{mean} := 1kPa$

Longitudinal girder distance = gauge: $E := 1512mm$

$$f_w := \frac{q_{mean} \cdot H_{car}^{\,2}}{2 \cdot E} = 8.267 \cdot \frac{kN}{m}$$

Load factors:

$\gamma_G := 1.35$ just permanent load

L1.Eq610.a $\gamma_G \cdot G_{cross} = 1.677 \cdot \frac{kN}{m}$ distributed load along girder axis

$\gamma_G \cdot (G_{rail} \cdot L_{cross}) = 3.291 \cdot kN$ concentrated load at 'rail position'

$\gamma_G := 1.25 \quad \gamma_Q := 1.45 \quad \gamma_{wind} := 1.5 \quad \psi_{0i} := 0.75$

L2.Eq610.b

concentrated load at 'rail position'

$$F_1 := \gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot F_{traffic} + \gamma_{wind} \cdot \psi_{0i} \cdot (f_w \cdot L_{cross}) = 563 \cdot kN$$

distributed load along girder axis

$$f := \gamma_G \cdot G_{cross} = 1.553 \cdot \frac{kN}{m}$$

concentrated load at 'rail position'

$$F_2 := \gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot F_{traffic} - \gamma_{wind} \cdot \psi_{0i} \cdot (f_w \cdot L_{cross}) = 517 \cdot kN$$

$\gamma_G := 1.25 \quad \gamma_Q := 1.45 \quad \gamma_{wind} := 1.5 \quad \psi_{0i} := 0.8$

L3.Eq610.b

concentrated load at 'rail position'

$$\gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot \psi_{0i} \cdot F_{traffic} + \gamma_{wind} \cdot (f_w \cdot L_{cross}) = 463 \cdot kN$$

distributed load along girder axis

$$\gamma_G \cdot G_{cross} = 1.553 \cdot \frac{kN}{m}$$

concentrated load at 'rail position'

$$\gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot \psi_{0i} \cdot F_{traffic} - \gamma_{wind} \cdot (f_w \cdot L_{cross}) = 402 \cdot kN$$

I only calculate the design momnet for L2 load combination (the most relevant one)

I assume that the cross girder is a simply supported beam.

cross girder length: $l_{cross} = 4.727\,m$

$$l_1 := \frac{(l_{cross} - E)}{2} = 1.608\,m$$

$$l_2 := l_1 = 1.608\,m$$

Calc of reaction forces: $B_1 := \dfrac{F_1 \cdot l_1 + F_2 \cdot (l_1 + E)}{l_{cross}} = 533 \cdot kN$

$$B_2 := \frac{f \cdot l_{cross}}{2} = 3.671 \cdot kN$$

$$B := B_1 + B_2 = 536 \cdot kN$$

$$A_1 := F_1 + F_2 - B_1 = 5.473 \times 10^5 \, N$$

$$A_2 := B_2 = 3.671 \times 10^3 \, N$$

$$A := A_1 + A_2 = 551 \cdot kN$$

bending moment at midsection:

$$M_{Ed.mid} := f \cdot \frac{l_{cross}{}^2}{8} + \left[A_1 \cdot \frac{l_{cross}}{2} - F_1 \cdot \left(\frac{l_{cross}}{2} - l_1 \right) \right] = 872 \cdot kN \cdot m$$

bending moment at rail position 1:

$$M_{Ed.1} := \left(A_2 \cdot l_1 - f \cdot \frac{l_1{}^2}{2} \right) + (A_1 \cdot l_1) = 884 \cdot kN \cdot m$$

bending moment at rail position 2:

$$M_{Ed.2} := \left[A_2 \cdot (l_1 + E) - f \cdot \frac{(l_1 + E)^2}{2} \right] + \left[A_1 \cdot (l_1 + E) - F_1 \cdot E \right] = 860 \, m \cdot kN$$

check:

$$M_{Ed.3} := \left(B_2 \cdot l_2 - f \cdot \frac{l_2{}^2}{2} \right) + B_1 \cdot l_2 = 860 \, m \cdot kN$$

$$M_{Ed} := \max(M_{Ed.mid}, M_{Ed.1}, M_{Ed.2}) = 884 \cdot kN \cdot m$$

Cross section resistance:

Cross section classification:

$$\varepsilon := \sqrt{\frac{235MPa}{f_y}} = 0.814$$

web: $\frac{h_w}{t_w} = 28.833$

$72 \cdot \varepsilon = 58.58$	Class 1	
$83 \cdot \varepsilon = 67.53$	Class 2	=> Class 1
$124 \cdot \varepsilon = 100.888$	Class 3	

flange: $\frac{\frac{b_f - t_w}{2}}{t_f} = 4.958$

$9 \cdot \varepsilon = 7.323$	Class 1	
$10\varepsilon = 8.136$	Class 2	=> Class 1
$14 \cdot \varepsilon = 11.391$	Class 3	

Cross section is Class 1 - plastic analysis

plastic section modulus: $W_{pl} := b_f \cdot t_f \cdot (H - t_f) + \frac{t_w \cdot h_w^2}{4} = 2579148 \cdot mm^3$

elastic section modulus: $W_{el} := \frac{b_f \cdot H^2}{6} - \frac{(b_f - t_w) \cdot h_w^3}{6 \cdot H} = 2297958 \cdot mm^3$

Moment resistance of cross girder:

$\gamma_{M0} := 1$

$$M_{pl.Rd} := \frac{W_{pl} \cdot f_y}{\gamma_{M0}} = 916 \cdot kN \cdot m$$

$\frac{M_{Ed}}{M_{pl.Rd}} = 0.965$ < 1 OK!

Longitudinal girder/Railbearer:

Geometry:

I - profile:

$$H_{long} := 370mm$$

$$t_w := 10mm$$

$$b_{f.1} := 400mm$$

$$t_{f.1} := 20mm$$

$$b_{f.2} := 200mm$$

$$t_{f.2} := 12mm$$

$$h_w := H_{long} - 2 \cdot t_f = 322 \cdot mm$$

$$A_{long} := h_w \cdot t_w + b_{f.1} \cdot t_{f.1} + b_{f.2} \cdot t_{f.2} = 13620 \cdot mm^2$$

long. girder length: $L_{long} := L_{cross} = 2.438\,m$

Dinamic factor for rail bearer:

$$L_{\phi.rail} := 3 \cdot L_{long} = 7.314\,m$$

$$\phi_{rail} := \frac{2.16}{\sqrt{\frac{L_{\phi.rail}}{m}} - 0.2} + 0.73 = 1.592$$

Loads:

Self weight:

$$G_{long} := A_{long} \cdot \rho \cdot g = 1048 \cdot \frac{N}{m}$$

Rail self weight: $G_{rail} := 1 \frac{kN}{m}$

Traffic Load: LM71-22,5

$$Q_v := 125kN$$

$$q_v := 40 \frac{kN}{m}$$

$$Q_v := \phi_{rail} \cdot Q_v = 199 \cdot kN$$

$$q_v := \phi_{rail} \cdot q_v = 64 \cdot \frac{kN}{m}$$

In this preliminary calculation I'll only check the beam against the maximum positive bendir moment:

During calculation I assume, the longitudinal girder is a simply supported beam.

long. girder length: $L_{cross} = 2.438\,m$

I get the maximum positive bending moment, when the concentrated loads are positiond in the middle of the beam.

$\gamma_G := 1.35$

L1.Eq610.a $\gamma_G \cdot G_{long} + \gamma_G \cdot G_{rail} = 2.765 \cdot \frac{kN}{m}$

$\gamma_G := 1.25$ $\gamma_Q := 1.45$ $\gamma_{wind} := 1.5$ $\psi_{0i} := 0.75$

L2.Eq610.b $n := 1$

$0m < n \cdot 1.6m = 1.6\,m < L_{long} = 2.438\,m$

$F := \gamma_Q \cdot Q_v = 289 \cdot kN$

$f_1 := \gamma_G \cdot G_{long} + \gamma_{wind} \cdot \psi_{0i} \cdot f_w = 10.61 \cdot \frac{kN}{m}$ distributed load

$f_2 := \gamma_G \cdot G_{long} - \gamma_{wind} \cdot \psi_{0i} \cdot f_w = -7.991 \cdot \frac{kN}{m}$ distributed load

$\psi_{0i} := 0.8$

L3.Eq610.b $\gamma_Q \cdot \psi_{0i} \cdot Q_v = 231 \cdot kN$

$\gamma_G \cdot G_{long} + \gamma_{wind} \cdot f_w = 13.711 \cdot \frac{kN}{m}$ distributed load

$\gamma_G \cdot G_{long} - \gamma_{wind} \cdot f_w = -11.091 \cdot \frac{kN}{m}$ distributed load

Calc of reaction forces: $R_y := \frac{(n+1) \cdot F + f_1 \cdot L_{long}}{2} = 301.569 \cdot kN$

Maximum moment at mid span:

long. girder length: $L_{long} = 2.438\,m$

$$l_1 := \frac{(L_{cross} - 1.6m)}{2} = 0.419\,m$$

$$l_2 := l_1 = 0.419\,m$$

$$M_{Ed} := R_y \cdot \frac{L_{long}}{2} - F \cdot \left(\frac{L_{long}}{2} - l_1\right) - f_1 \cdot \frac{L_{long}^2}{8} = 129 \cdot kN \cdot m$$

Cross section classification:

$$\varepsilon := \sqrt{\frac{235MPa}{f_y}} = 0.81$$

web:

$$\frac{h_w}{t_w} = 32.2$$

$72 \cdot \varepsilon = 58.58$	Class 1	
$83 \cdot \varepsilon = 67.53$	Class 2	=> Class 1
$124 \cdot \varepsilon = 100.888$	Class 3	

upper flange:

$$\frac{\frac{b_{f.1} - t_w}{2}}{t_{f.1}} = 9.75$$

$9 \cdot \varepsilon = 7.323$	Class 1	
$10\varepsilon = 8.136$	Class 2	=> Class 3
$14 \cdot \varepsilon = 11.391$	Class 3	
	Class 4	

lower flange:

$$\frac{\frac{b_{f.2} - t_w}{2}}{t_{f.2}} = 7.917$$

$9 \cdot \varepsilon = 7.323$	Class 1	
$10\varepsilon = 8.136$	Class 2	=> Class 2
$14 \cdot \varepsilon = 11.391$	Class 3	
	Class 4	

Cross section is Class 3 - elastic analysis

$\gamma_{M0} :=$

plastic section modulus: $W_{pl} := b_f \cdot t_f \cdot (H_{long} - t_f) + \frac{t_w \cdot h_w^2}{4} = 2335210 \cdot mm^3$

elastic section modulus (from Robot Str. Analysis) : $W_{el} := 1131.77 cm^3$ From Robot Structural Analysis

Moment resistance of cross girder: $f_y = 355 \cdot \frac{N}{mm}$

$$M_{el.Rd} := \frac{W_{el} \cdot f_y}{\gamma_{M0}} = 402 \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{el.Rd}} = 0.321 \quad < \quad 1$$ OK!

Main girder:

Geometry:

I - profile: $H := 1600mm$

$t_w := 20mm$

$b_f := 400mm$

$t_f := 30mm$

$h_w := H - 2 \cdot t_f = 1.54 \times 10^3 \cdot mm$

$A_{main} := h_w \cdot t_w + 2b_f \cdot t_f = 54800 \cdot mm^2$

Span: $L := 14.628m$

Dinamic factor for main girder

$$L_{\phi.main} := L = 14.628\,m$$

$$\phi_{main} := \frac{2.16}{\sqrt{\frac{L_{\phi.main}}{m} - 0.2}} + 0.73 = 1.326$$

Case	Structural element	Determinant length L_Φ
Main girders		
5.1	Simply supported girders and slabs (including steel beams embedded in concrete)	Span in main girder direction

Ref. EN1991-2 §6.4.5.3 Table 6.2

Loads:

Permanent load:

$G_{main} := A_{main} \cdot \rho \cdot g = 4.216 \cdot \frac{kN}{m}$ self weight of main girder

$G_{cross} := \frac{G_{cross}}{2} = 0.621 \cdot \frac{kN}{m}$ self weight of cross girder

$G_{long} = 1.048 \cdot \frac{kN}{m}$ self weight of railbearer

Rail self weight: $G_{rail} := 1 \frac{kN}{m}$

Traffic Load: LM71-22,5

$Q_v := 125kN$

$q_v := 40 \frac{kN}{m}$

$Q_v := \phi_{main} \cdot Q_v = 166 \cdot kN$

$q_v := \phi_{main} \cdot q_v = 53 \cdot \frac{kN}{m}$

Wind load:

height of the car: $H_{car} := 5m$

Mean wind load: $q_{mean} := 1kPa$

Longitudinal girder distance = gauge: $E := 1512mm$

$$f_w := \frac{q_{mean} \cdot H_{car}^2}{2 \cdot E} = 8.267 \cdot \frac{kN}{m}$$

Load factors:

$\gamma_G := 1.35$ just permanent load

L1.Eq610.a distributed load along girder axis

$$\gamma_G \cdot \left(G_{cross} + G_{main} + G_{long} + G_{rail}\right) = 9.295 \cdot \frac{kN}{m}$$

$\gamma_G := 1.25$ $\gamma_Q := 1.45$ $\gamma_{wind} := 1.5$ $\psi_{0i} := 0.75$

L2.Eq610.b

concentrated load at 'rail position'

$$F_2 := \gamma_Q \cdot Q_v \cdot 4 = 961 \cdot kN$$

distributed load along girder axis

$$f_2 := \gamma_G \cdot \left(G_{cross} + G_{main} + G_{long} + G_{rail}\right) + \gamma_Q \cdot q_v + \gamma_{wind} \cdot \psi_{0i} \cdot f_w = 94.81 \cdot \frac{kN}{m}$$

$\gamma_G := 1.25$ $\gamma_Q := 1.45$ $\gamma_{wind} := 1.5$ $\psi_{0i} := 0.8$

L3.Eq610.b

concentrated load at 'rail position'

$$F_3 := \gamma_Q \cdot \psi_{0i} \cdot Q_v \cdot 4 = 769 \cdot kN$$

distributed load along girder axis

$$f_3 := \gamma_G \cdot \left(G_{cross} + G_{main} + G_{long} + G_{rail}\right) + \gamma_Q \cdot \psi_{0i} \cdot q_v + \gamma_{wind} \cdot f_w = 82.529 \cdot \frac{kN}{m}$$

I only calculate the design momnet for L2 load combination (the most relevant one)

I assume that the main girder is a simply supported beam.

$$M_{Ed} := \frac{f_2 \cdot L^2}{8} + \frac{F_2 \cdot L}{4} = 6051 \cdot kN \cdot m$$

Cross section resistance:

Cross section classification:

web: $\frac{h_w}{t_w} = 77$

$72 \cdot \varepsilon = 58.58$ Class 1

$83 \cdot \varepsilon = 67.53$ Class 2 => Class 3

$124 \cdot \varepsilon = 100.888$ Class 3

flange: $\frac{\frac{b_f - t_w}{2}}{t_f} = 6.333$

$9 \cdot \varepsilon = 7.323$ Class 1

$10\varepsilon = 8.136$ Class 2 => Class 1

$14 \cdot \varepsilon = 11.391$ Class 3

Cross section is Class 3 - elastic analysis

plastic section modulus: $W_{pl} := b_f \cdot t_f \cdot (H - t_f) + \frac{t_w \cdot h_w^2}{4} = 30698000 \cdot mm^3$

elastic section modulus: $W_{el} := \frac{b_f \cdot H^2}{6} - \frac{(b_f - t_w) \cdot h_w^3}{6 \cdot H} = 26097883 \cdot mm^3$

$\gamma_{M0} := 1$

Moment resistance of cross girder:

$$M_{el.Rd} := \frac{W_{el} \cdot f_y}{\gamma_{M0}} = 9265 \cdot kN \cdot m$$

$\frac{M_{Ed}}{M_{el.Rd}} = 0.653$ < 1 OK!

Deflection of main girder:

second mometn of area: $I := \frac{h_w{}^3 \cdot t_w}{12} + \left(\frac{b_f}{12}\right) \cdot \left(H^3 - h_w{}^3\right) = 2.088 \times 10^{10} \cdot mm^4$

modulus of elasticity: $E_a := 210000 \frac{N}{mm^2}$

permanent characteristic: $g_k := G_{cross} + G_{main} + G_{long} + G_{rail} = 6.885 \cdot \frac{kN}{m}$

variable characteristic: $q_k := q_v + f_w = 61.304 \cdot \frac{kN}{m}$

$Q_k := 4Q_v = 662.959 \cdot kN$

the calculation of deflection is an approximation on the safe side

$\delta_{max} := \frac{5 \cdot (g_k + q_k) \cdot L^4}{384 \cdot E_a \cdot I} + \frac{Q_k \cdot L^3}{48 \cdot E_a \cdot I} = 19.13 \cdot mm$ < $\frac{L}{400} = 36.57 \cdot mm$

Total weight of structure:

$n_{cross} := 7$

$A_{cross} \cdot l_{cross} \cdot \rho \cdot n_{cross} = 4.195 \times 10^3 \, kg$

$n_{long} := 2$

$A_{long} \cdot L \cdot \rho \cdot n_{long} = 3.128 \times 10^3 \, kg$

$n_{main} := 2$

$A_{main} \cdot L \cdot \rho \cdot n_{main} = 1.259 \times 10^4 \, kg$

Buckiling support girder:

Geometry:

I - profile: $h_b := 246mm$

$t_{w.b} := 10mm$

$b_{f.b} := 120mm$

$t_{f.b} := 13mm$

$h_{w.b} := h_b - 2 \cdot t_{f.b} = 220 \cdot mm$

$A_{buckl} := h_{w.b} \cdot t_{w.b} + 2b_{f.b} \cdot t_{f.b} = 5320 \cdot mm^2$

$l_{buckl} := E = 1.512\, m$

$n_{buckl} := n_{cross} - 1 = 6$

Wind bracing:

$A_w := 102mm \cdot 13mm = 1.326 \times 10^{-3}\, m^2$

$L_w := 6205mm \qquad n_w := 6$

$$G_{TOT} := A_{main} \cdot L \cdot \rho \cdot n_{main} + A_{long} \cdot L \cdot \rho \cdot n_{long} + A_{cross} \cdot l_{cross} \cdot \rho \cdot n_{cross} \cdots + A_{buckl} \cdot l_{buckl} \cdot \rho \cdot n_{buckl} + A_w \cdot L_w \cdot \rho \cdot n_w = 20675 \cdot kg$$

$G_{TOT} \cdot g = 202.62 \cdot kN$

APPENDIX 2

Connection Calculation Report - Crossgirder-railbearer

Material:

Bolt class: 4.6

$$f_{yb} := 240 \frac{N}{mm^2} \qquad f_{ub} := 400 \frac{N}{mm^2}$$

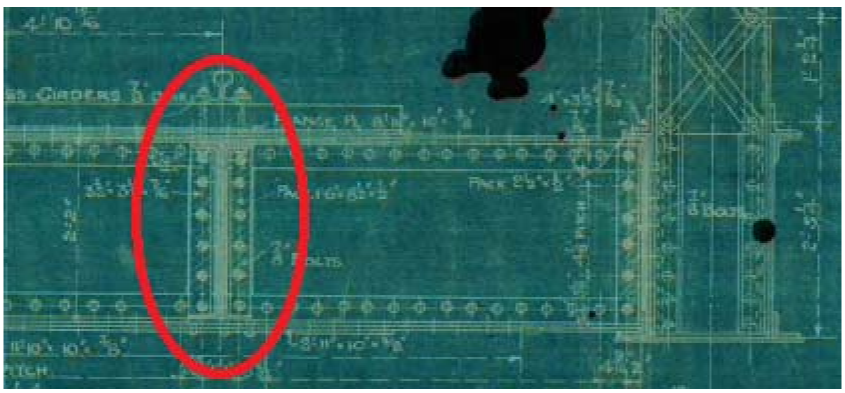
Ref. EN1993-1-8
§3.1.1
Table 3.1

Steel grade: S235

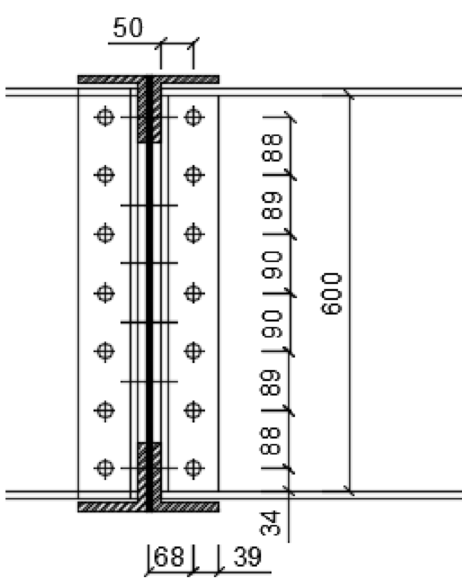
$$f_y := 235 \frac{N}{mm^2} \qquad f_u := 360 \frac{N}{mm^2}$$

Ref. EN1993-1-1
§3.2.3
Table 3.1

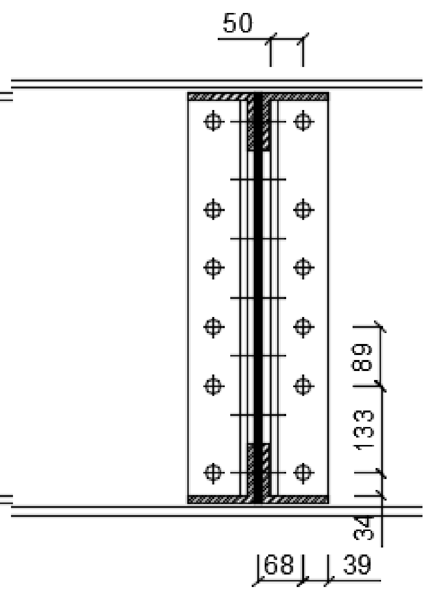
Geometry of joint:



Connection between cross girder and railbearer



Supported beam side



Supporting beam side

$d := 22mm$

$A_b := \frac{d^2 \cdot \pi}{4} = 380 \cdot mm^2$ the gross cross section of the bolt

$d_0 := d + 2mm = 24 \cdot mm$

$p_1 := 89mm$

$z := 68mm$ is the transverse distance from the face of the supporting element to the centre of the bolt group

$n_b := 7$

Partial safety factor for joint:

$\gamma_{M2} := 1.25$ Ref. EN1993-1-8 §2.2 Table 2.1

$\gamma_{M0} := 1$ Ref. EN1993-1-1 §6.1 Note 2B

Supported beam side:

Shear resistance of bolts

Basic requirement: $V_{Ed} \le V_{Rd}$

$$V_{Rd} = \frac{2 \cdot n_b \cdot F_{v.Rd}}{\sqrt{\left(1 + \alpha n_b\right)^2 + \left(\beta \cdot n_b\right)^2}}$$

$$F_{v.Rd} = \frac{\alpha_v \cdot f_{ub} \cdot A}{\gamma_{M2}}$$

Shea resistance per shear plane

Ref. EN1993-1-8 §3.6.1 Table 3.4

for classes 4.6~ 5.6 and 8.8:
α.v = 0,6
- for classes 4.8, 5.8, 6.8 and 10.9:
α.v = 0,5

=> $\alpha_v := 0.6$

For a single vertical line of bolts:

$\alpha := 0$

$$\beta = \frac{6 \cdot z}{n_1 \cdot \left(n_1 + 1\right) \cdot p_1}$$

$n_1 := n_b = 7$

$$\beta := \frac{6 \cdot z}{n_1 \cdot \left(n_1 + 1\right) \cdot p_1} = 0.082$$

$$F_{v.Rd} := \frac{\alpha_v \cdot f_{ub} \cdot A_b}{\gamma_{M2}} = 72.985 \cdot kN$$

$$V_{v.Rd} := \frac{2 \cdot n_b \cdot F_{v.Rd}}{\sqrt{\left(1 + \alpha \cdot n_b\right)^2 + \left(\beta \cdot n_b\right)^2}} = 886.554 \cdot kN$$

Bearing resistance of bolts on the angle cleats

Basic requirement: $V_{Ed} \le V_{Rd}$

$$V_{Rd} = \frac{2 \cdot n_b}{\sqrt{\left(\frac{1 + \alpha \cdot n_b}{F_{b.ver.Rd}}\right)^2 + \left(\frac{\beta \cdot n_b}{F_{b.hor.Rd}}\right)^2}}$$

The vertical bearing resistance of a single bolt on the angle cleat is as follows:

Ref. EN1993-1-8
§3.6.1
Table 3.4

$$F_{b.ver.Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{u.ac} \cdot d \cdot t_{ac}}{\gamma_{M2}}$$

$e_2 := 39mm \quad e_1 := 34mm \quad t_{ac} := 11mm$

$$k_{1.ver} := \min\left(2.8 \cdot \frac{e_2}{d_0} - 1.7, 2.5\right) = 2.5$$

$f_{u.ac} := f_u = 360 \cdot MPa$

$$\alpha_{b.ver} := \min\left(\frac{e_1}{3 \cdot d_0}, \frac{p_1}{3 \cdot d_0} - \frac{1}{4}, \frac{f_{ub}}{f_{u.ac}}, 1\right) = 0.472$$

$$F_{b.ver.Rd} := \frac{k_{1.ver} \cdot \alpha_{b.ver} \cdot f_{u.ac} \cdot d \cdot t_{ac}}{\gamma_{M2}} = 82.28 \cdot kN$$

The horizontal bearing resistance of a single bolt on the angle cleat is as follows:

Ref. EN1993-1-8
§3.6.1
Table 3.4

$$F_{b.hor.Rd} = \frac{k_1 \cdot \alpha_b \cdot f_{u.ac} \cdot d \cdot t_{ac}}{\gamma_{M2}}$$

$$k_{1.hor} := \min\left(2.8 \cdot \frac{e_1}{d_0} - 1.7, 1.4 \cdot \frac{p_1}{d_0} - 1.7, 2.5\right) = 2.267$$

$$\alpha_{b.hor} := \min\left(\frac{e_2}{3\cdot d_0}, \frac{f_{ub}}{f_{u.ac}}, 1\right) = 0.542$$

$$F_{b.hor.Rd} := \frac{k_{1.hor}\cdot\alpha_{b.hor}\cdot f_{u.ac}\cdot d\cdot t_{ac}}{\gamma_{M2}} = 85.571\cdot kN$$

$$V_{b.Rd} := \frac{2\cdot n_b}{\sqrt{\left(\frac{1+\alpha\cdot n_b}{F_{b.ver.Rd}}\right)^2 + \left(\frac{\beta\cdot n_b}{F_{b.hor.Rd}}\right)^2}} = 1009\cdot kN$$

Bearing resistance of bolts on the beam web

Basic requirement: $V_{Ed} \le V_{Rd}$

$e_{2.w} := 50mm \qquad t_w := 9.5mm$

$$V_{Rd} = \frac{n_b}{\sqrt{\left(\frac{1+\alpha\cdot n_b}{F_{b.ver.Rd}}\right)^2 + \left(\frac{\beta\cdot n_b}{F_{b.hor.Rd}}\right)^2}}$$

The vertical bearing resistance:

Ref. EN1993-1-8
§3.6.1
Table 3.4

$$F_{b.ver.Rd.2} = \frac{k_1\cdot\alpha_b\cdot f_{u.w}\cdot d\cdot t_w}{\gamma_{M2}}$$

$$k_{1.ver.2} := \min\left(2.8\cdot\frac{e_{2.w}}{d_0} - 1.7, 2.5\right) = 2.5$$

$$f_{u.w} := f_u = 360\cdot MPa$$

$$\alpha_{b.ver.2} := \min\left(\frac{e_1}{3\cdot d_0}, \frac{p_1}{3\cdot d_0} - \frac{1}{4}, \frac{f_{ub}}{f_{u.w}}, 1\right) = 0.472$$

$$F_{b.ver.Rd.2} := \frac{k_{1.ver.2}\cdot\alpha_{b.ver.2}\cdot f_{u.w}\cdot d\cdot t_w}{\gamma_{M2}} = 71.06\cdot kN$$

The horizontal bearing resistance:

Ref. EN1993-1-8
§3.6.1
Table 3.4

$$F_{b.hor.Rd.2} = \frac{k_1\cdot\alpha_b\cdot f_{u.ac}\cdot d\cdot t_w}{\gamma_{M2}}$$

$$k_{1.hor.2} := \min\left(2.8\cdot\frac{e_1}{d_0} - 1.7, 1.4\cdot\frac{p_1}{d_0} - 1.7, 2.5\right) = 2.267$$

$$\alpha_{b.hor.2} := \min\left(\frac{e_{2.w}}{3\cdot d_0}, \frac{f_{ub}}{f_{u.w}}, 1\right) = 0.694$$

$$F_{b.hor.Rd.2} := \frac{k_{1.hor.2}\cdot\alpha_{b.hor.2}\cdot f_{u.w}\cdot d\cdot t_w}{\gamma_{M2}} = 94.747\cdot kN$$

$$V_{b.Rd.2} := 2\frac{n_b}{\sqrt{\left(\frac{1+\alpha\cdot n_b}{F_{b.ver.Rd.2}}\right)^2 + \left(\frac{\beta\cdot n_b}{F_{b.hor.Rd.2}}\right)^2}} = 914\cdot kN$$

Multiplied by two, because the bearing resistance of the web works against half of the design shear force.

Supporting beam side:

Basic requirement:

$$V_{Ed} \le F_{Rd}$$

$$F_{Rd} = \begin{cases} \sum_n F_{b.Rd} & \text{if } \max(F_{b.Rd}) \le F_{v.Rd} \\ n_s\cdot\min(F_{b.Rd}) & \text{if } \min(F_{b.Rd}) \le F_{v.Rd} \le \max(F_{b.Rd}) \\ 0.8\cdot n_s\cdot F_{v.Rd} & \text{if } F_{v.Rd} \le \min(F_{b.Rd}) \end{cases}$$

Ref. EN1993-1-8 §3.7 (1)

Shear resistance of bolts:

$$F_{v.Rd} = 73\cdot kN$$

Bearing resistance of bolts on the angle cleats

Ref. EN1993-1-8 §3.6.1 Table 3.4

$$F_{b.Rd} = \frac{k_1\cdot\alpha_b\cdot f_{u.ac}\cdot d\cdot t_{ac}}{\gamma_{M2}}$$

For edge bolts: $k_{1.ac} := \min\left(2.8\cdot\frac{e_2}{d_0} - 1.7, 2.5\right) = 2.5$

For end bolts: $\alpha_{b.ac.end} := \min\left(\frac{e_1}{3\cdot d_0}, \frac{f_{ub}}{f_{u.ac}}, 1\right) = 0.472$

For inner bolts: $\alpha_{b.ac.inn} := \min\left(\frac{p_1}{3 \cdot d_0} - \frac{1}{4}, \frac{f_{ub}}{f_{u.ac}}, 1\right) = 0.986$

For end bolts: $F_{b.Rd.end} := \frac{k_{1.ac} \cdot \alpha_{b.ac.end} \cdot f_{u.ac} \cdot d \cdot t_{ac}}{\gamma_{M2}} = 82.28 \cdot kN$

For inner bolts: $F_{b.Rd.inn} := \frac{k_{1.ac} \cdot \alpha_{b.ac.inn} \cdot f_{u.ac} \cdot d \cdot t_{ac}}{\gamma_{M2}} = 171.82 \cdot kN$

$F_{b.Rd.min} := \min(F_{b.Rd.end}, F_{b.Rd.inn}) = 82.28 \cdot kN$

$F_{b.Rd.max} := \max(F_{b.Rd.end}, F_{b.Rd.inn}) = 171.82 \cdot kN$

$n_{b.2} := 6$ number of bolts on supporting beam side

$n_s := 2 \cdot n_{b.2} = 12$

$$F_{Rd} := \left| \begin{array}{l} F_{b.Rd.end} + F_{b.Rd.inn} \ \text{ if } \ F_{b.Rd.max} \le F_{v.Rd} \\ n_s \cdot F_{b.Rd.min} \ \text{ if } \ F_{b.Rd.min} \le F_{v.Rd} \le F_{b.Rd.max} \\ 0.8 \cdot n_s \cdot F_{v.Rd} \ \text{ if } \ F_{v.Rd} \le F_{b.Rd.min} \end{array} \right. = 701 \cdot kN$$

Supported beam side:

Shear resistance of the angle cleats

Basic requirement: $V_{Ed} \le V_{Rd.min}$

$V_{Rd.min} = \min(V_{Rd.g}, V_{Rd.n}, V_{Rd.b})$

Shear resistance of gross section

$$V_{Rd.g} = 2 \cdot \frac{h_{ac} \cdot t_{ac}}{1.27} \cdot \frac{f_{y.ac}}{\sqrt{3} \cdot \gamma_{M0}}$$

Note: The coefficient 1,27 takes into account the reduction in shear resistance due to the presence of the nominal in-plane bending which produces tension in the bolts

$h_{ac} := 600mm$ $t_{ac} = 11 \cdot mm$ $f_{y.ac} := f_y = 235 \cdot MPa$

$$V_{Rd.g} := 2\cdot\frac{h_{ac}\cdot t_{ac}}{1.27}\cdot\frac{f_{y.ac}}{\sqrt{3}\cdot\gamma_{M0}} = 1410\cdot kN$$

Shear resistance of net section

$$V_{Rd.n} = 2\cdot A_{v.net}\cdot\frac{f_{u.ac}}{\sqrt{3}\cdot\gamma_{M2}}$$

$$A_{v.net} := t_{ac}\cdot\left(h_{ac} - n_1\cdot d_0\right) = 4752\cdot mm^2$$

$$V_{Rd.n} := 2\cdot A_{v.net}\cdot\frac{f_{u.ac}}{\sqrt{3}\cdot\gamma_{M2}} = 1580\cdot kN$$

Block tearing resistance

$$V_{Rd.b} = 2\cdot\left(\frac{0.5\cdot f_{u.ac}\cdot A_{nt}}{\gamma_{M2}} + \frac{f_{y.ac}\cdot A_{nv}}{\sqrt{3}\cdot\gamma_{M0}}\right)$$

Ref.
EN1993-1-8
§3.10.2 (2)

$$A_{nt} := t_{ac}\cdot\left(e_2 - 0.5\cdot d_0\right)$$

$$A_{nv} := t_{ac}\cdot\left[h_{ac} - e_1 - \left(n_1 - 0.5\right)\cdot d_0\right]$$

$$V_{Rd.b} := 2\cdot\left(\frac{0.5\cdot f_{u.ac}\cdot A_{nt}}{\gamma_{M2}} + \frac{f_{y.ac}\cdot A_{nv}}{\sqrt{3}\cdot\gamma_{M0}}\right) = 1309\cdot kN$$

$$V_{Rd.min} := \min\left(V_{Rd.g}, V_{Rd.n}, V_{Rd.b}\right) = 1309\cdot kN$$

Supporting beam side:

Shear resistance of the angle cleats

Basic requirement: $V_{Ed} \le V_{Rd.min}$

$$V_{Rd.min} = \min\left(V_{Rd.g}, V_{Rd.n}, V_{Rd.b}\right)$$

Shear resistance of gross section

$$V_{Rd.g} = 2\cdot\frac{h_{ac}\cdot t_{ac}}{1.27}\cdot\frac{f_{y.ac}}{\sqrt{3}\cdot\gamma_{M0}}$$

$$V_{Rd.g.2} := 2 \cdot \frac{h_{ac} \cdot t_{ac}}{1.27} \cdot \frac{f_{y.ac}}{\sqrt{3} \cdot \gamma_{M0}} = 1410 \cdot kN$$

Shear resistance of net section

$$V_{Rd.n} = 2 \cdot A_{v.net} \cdot \frac{f_{u.ac}}{\sqrt{3} \cdot \gamma_{M2}}$$

$$A_{v.net.2} := t_{ac} \cdot \left(h_{ac} - n_{b.2} \cdot d_0\right) = 5016 \cdot mm^2$$

$$V_{Rd.n.2} := 2 \cdot A_{v.net.2} \cdot \frac{f_{u.ac}}{\sqrt{3} \cdot \gamma_{M2}} = 1668 \cdot kN$$

Block tearing resistance

Ref. EN1993-1-8 §3.10.2 (2)

$$V_{Rd.b} = 2 \cdot \left(\frac{0.5 \cdot f_{u.ac} \cdot A_{nt}}{\gamma_{M2}} + \frac{f_{y.ac} \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}}\right)$$

$$A_{nt.2} := t_{ac} \cdot \left(e_2 - 0.5 \cdot d_0\right)$$

$$A_{nv.2} := t_{ac} \cdot \left[h_{ac} - e_1 - \left(n_{b.2} - 0.5\right) \cdot d_0\right]$$

$$V_{Rd.b.2} := 2 \cdot \left(\frac{0.5 \cdot f_{u.ac} \cdot A_{nt.2}}{\gamma_{M2}} + \frac{f_{y.ac} \cdot A_{nv.2}}{\sqrt{3} \cdot \gamma_{M0}}\right) = 1381 \cdot kN$$

$$V_{Rd.min.2} := \min\left(V_{Rd.g.2}, V_{Rd.n.2}, V_{Rd.b.2}\right) = 1381 \cdot kN$$

Shear resistance of the beam web

Shear and block tearing resistance

Basic requirement: $V_{Ed} \le V_{Rd.min}$

$$V_{Rd.min} = \min\left(V_{Rd.g}, V_{Rd.n}, V_{Rd.b}\right)$$

Shear resistance of gross section

$$V_{Rd.g.wb} = A_{v.wb} \cdot \frac{f_{y.b}}{\sqrt{3} \cdot \gamma_{M0}}$$

$$f_{y.b} := f_y = 235 \cdot MPa$$

$h_w := h_{ac} = 600 \cdot mm \quad t_w = 9.5 \cdot mm$

$A_{v.wb} := h_w \cdot t_w = 5700 \cdot mm^2$

$$V_{Rd.g.wb} := A_{v.wb} \cdot \frac{f_{y.b}}{\sqrt{3} \cdot \gamma_{M0}} = 773.361 \cdot kN$$

Shear resistance of net section

$$V_{Rd.n.wb} = A_{v.wb.net} \cdot \frac{f_{u.b}}{\sqrt{3} \cdot \gamma_{M0}}$$

$A_{v.wb.net} := A_{v.wb} - n_b \cdot d_0 \cdot t_w$

$f_{u.b} := f_u = 360 \cdot MPa$

$$V_{Rd.n.wb} := A_{v.wb.net} \cdot \frac{f_{u.b}}{\sqrt{3} \cdot \gamma_{M0}} = 853 \cdot kN$$

Block tearing resistance

$$V_{Rd.b} = 2 \cdot \left(\frac{0.5 \cdot f_{u.ac} \cdot A_{nt}}{\gamma_{M2}} + \frac{f_{y.ac} \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}} \right)$$

Ref. EN1993-1-8 §3.10.2 (2)

$A_{nt.wb} := t_w \cdot (e_{2.w} - 0.5 \cdot d_0) = 361 \cdot mm^2$

$A_{nv.wb} := t_w \cdot \left[e_1 + (n_1 - 1) \cdot p_1 - (n_1 - 0.5) \cdot d_0 \right] = 3.914 \times 10^3 \cdot mm^2$

$$V_{Rd.b.wb} := 2 \cdot \left(\frac{0.5 \cdot f_{u.b} \cdot A_{nt.wb}}{\gamma_{M2}} + \frac{f_{y.b} \cdot A_{nv.wb}}{\sqrt{3} \cdot \gamma_{M0}} \right) = 1166 \cdot kN$$

$V_{Rd.min.wb} := \min(V_{Rd.g.wb}, V_{Rd.n.wb}, V_{Rd.b.wb}) = 773 \cdot kN$

Summary of design checks:

Shear resistance:

Bolt group design

Supported beam side

Shear resistance of bolts: $V_{v.Rd} = 887 \cdot kN$

Bearing resistance of bolts on angle cleats: $V_{b.Rd} = 1009 \cdot kN$

Bearing resistance of bolts on the beam web: $V_{b.Rd.2} = 914 \cdot kN$

Supporting beam side

Resistance: $F_{Rd} = 701 \cdot kN$

Shear resistance of the angle cleats

Supported beam side

Shear resistance: $V_{Rd.min} = 1309 \cdot kN$

Supporting beam side

Shear resistance: $V_{Rd.min.2} = 1381 \cdot kN$

Shear resistance of the beam web

Shear and block tearing resistance

Shear resistance: $V_{Rd.min.wb} = 773 \cdot kN$

$$V_{Rd} := \min\left(V_{v.Rd}, V_{b.Rd}, V_{b.Rd.2}, V_{Rd.min}, V_{Rd.min.2}, V_{Rd.min.wb}, F_{Rd}\right) = 701 \cdot kN$$

$V_{Ed} := 836.32kN$ From Robot Stractural Analysis

$\frac{V_{Ed}}{V_{Rd}} = 1.194$ > 1 The joint is failing due to the shear design force and the critical failure mode is the bearing resistance of the bolts on the angle cleats.

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