Railway Project

Rio Yi RAILWAY BRIDGE

CALCULATION REPORT

DOCUMENT APPROVAL

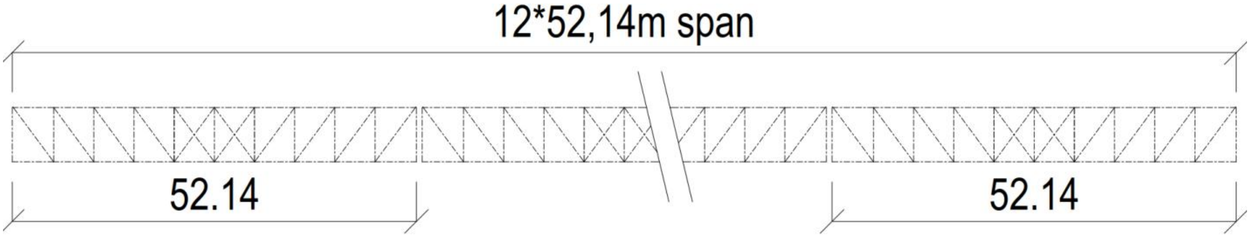
VERSIO	DESIGNER	SUPERVISOR	ACCEPT.
15.12.2017	Vera Babiczky	Mikko Ilvonen	

REVISION HISTORY

CONTENT

1 SCOPE

Yi Bridge consists of a 52m span length steel truss bridge. Total length of the bridge is 634 m.



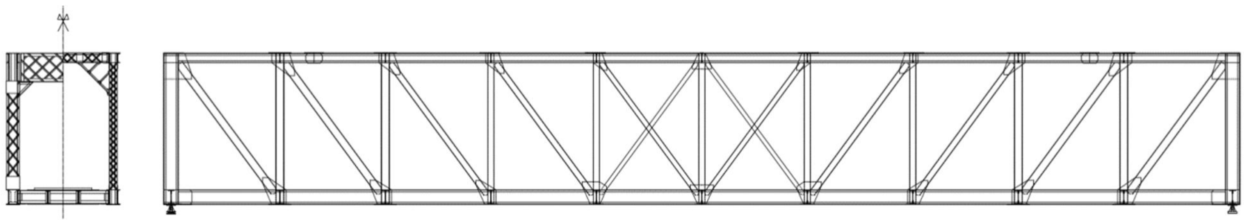
Picture 1, Yi bridge side view

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 the FEM-modelling and all calculations executed with Robot Structural Analysis. Its purpose is to show all selections made by the engineer and show the results of the analysis.

1.1 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, Yi Bridge

2 DESIGN CRITERIA

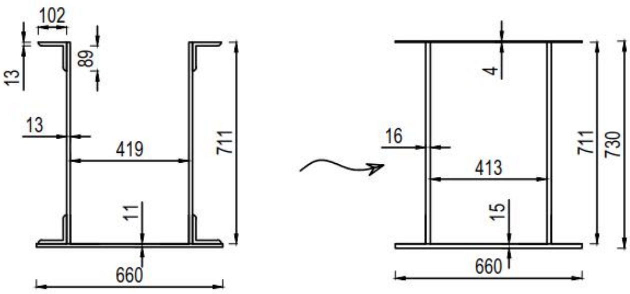
FEM calculations was made with Autodesk® Robot™ Structural Analysis Professional, Version 30.0.0.5913.

2.1 Structure

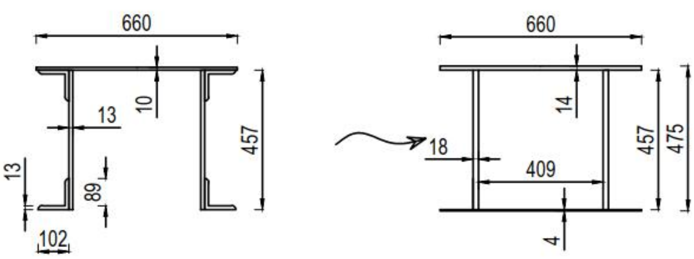
Bridge super structure members are complicated profiles of angles and plates with rivet connection. For building a FEM-model, simplified profiles were used. Simplifications were made so that function in FEM model equals actual profiles. The simplifications are shown in pictures 3-12 in section 2.1.1.

Picture 3, Yi Bridge inside view

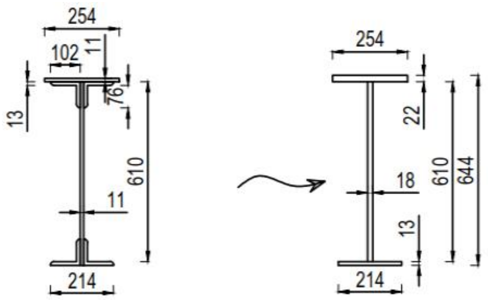
2.1.1 Simplifications for sections



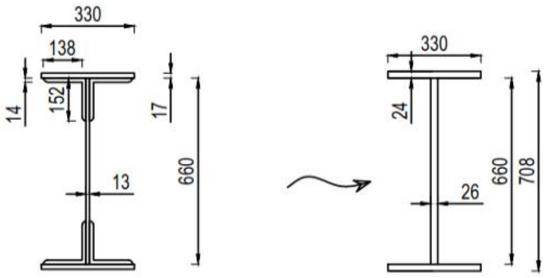
Picture 4, Lower main girder



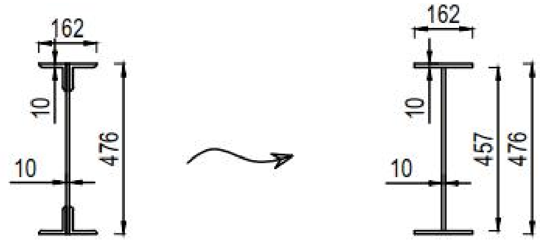
Picture 5, Upper main girder, type 1



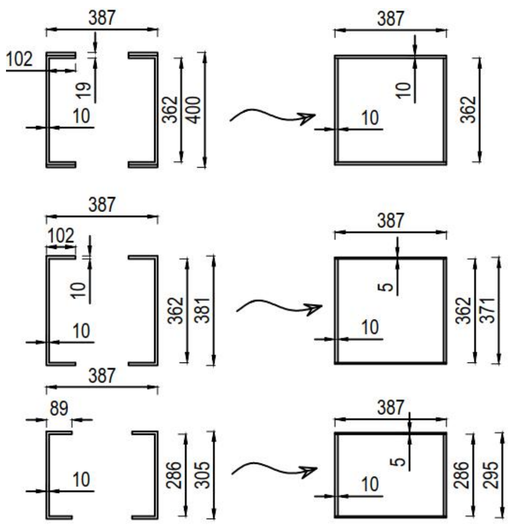
Picture 6, Longitudinal girder



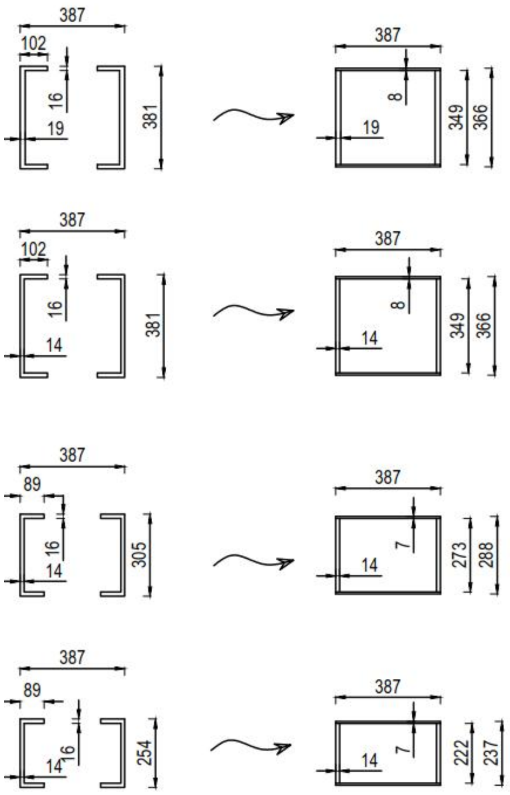
Picture 7, Cross girder



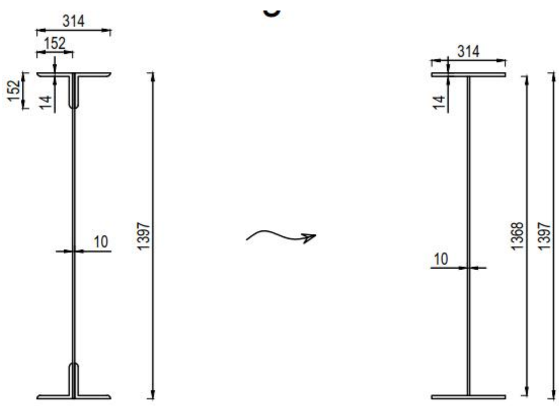
Picture 8, Upper cross bracing



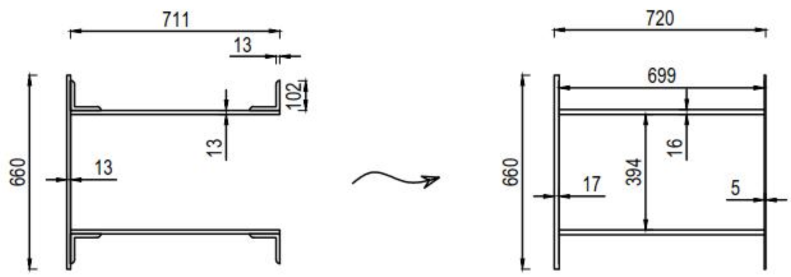
Picture 9, Columns – 3 types



Picture 10, Diagonals – 4 types



Picture 11, Upper end cross bracing



Picture 12, End frame

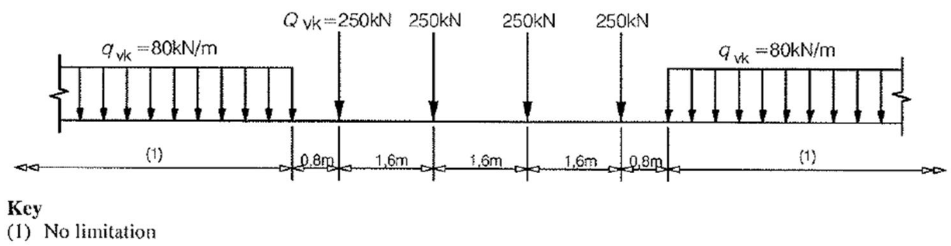
2.2 Loads

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.

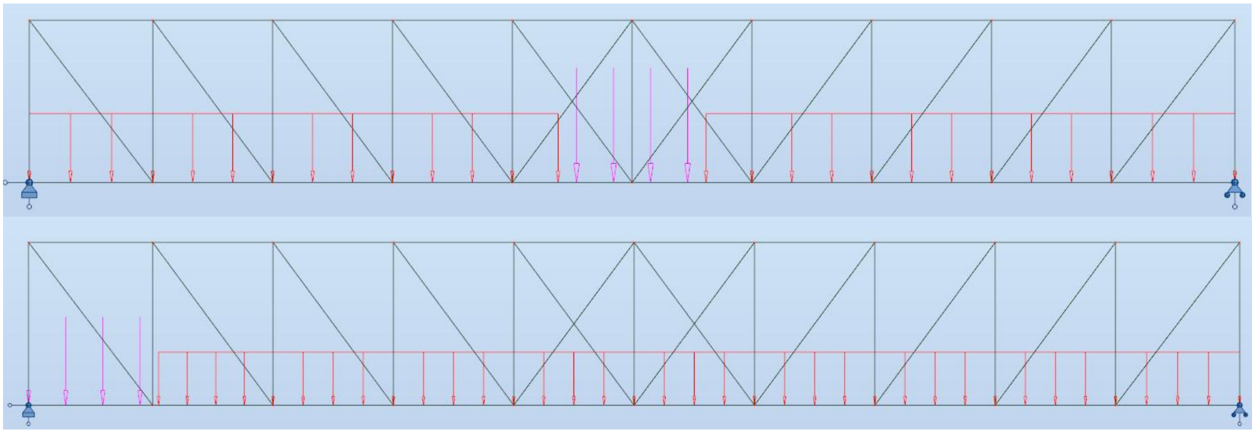
2.2.2 Train load

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

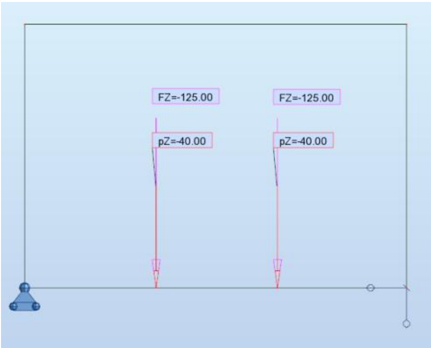


Picture 13, Train load model 71

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 14, Load model 71 applications – side view

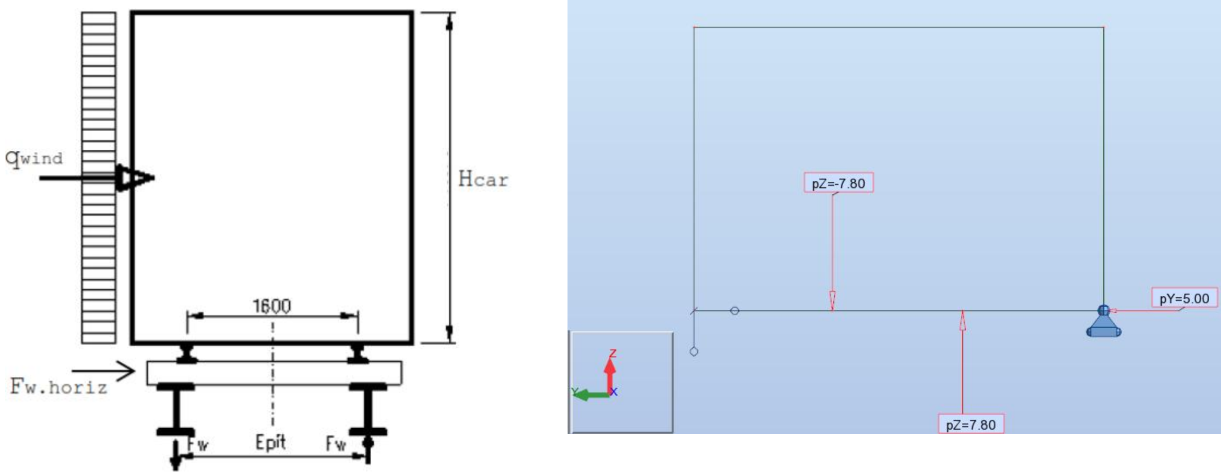


Picture 15, Load model 71 application – front view

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 16, 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			
	gr12 (LM71 + SW/0)	Max. vertical 2 with max. transverse			
	gr13 (Braking/traction)	Max. longitudinal			
	gr14 (Centrifugal/nosing)	Max. lateral	0,80	0,80	0
	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			
	gr22 (LM71 + SW/0)	Max. vertical 2 with max transverse			
	gr23 (Braking/traction)	Max. longitudinal	0,80	0,70	0
	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.

ULS Load combinations:

L1. Eq.610a
    1,35*Seflweight
L2. Eq.610b/1
    1,25*Seflweight+1,45*TrafficLoad+1,50*0,75*Wind
L3. Eq.610b/2
    1,25*Seflweight+1,50*Wind+1,45*0,8*TrafficLoad

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

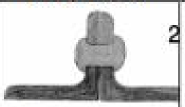
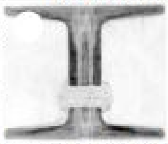
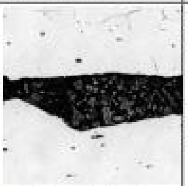
	Cast iron	Wrought steel Puddled steel	Mild steel (19th century)	Mild steel (20th century)
Sulphur-print (Baumann-print)	Content of sulphur is depending on coke quality	 Slag segregation lines containing phosphorus and sulphur	 Core segregation containing phosphorus and sulphur	 Low content of phosphorus 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 	

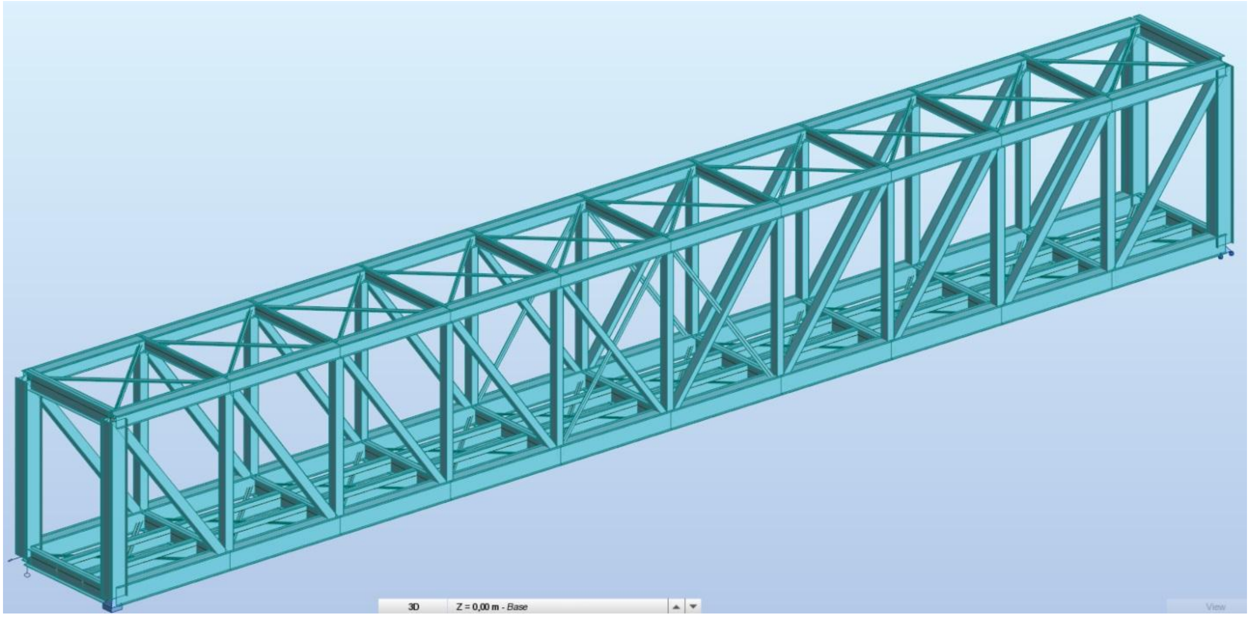
Table 3-2: Information on material characteristics of old iron and steels

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

Picture 17, steel material properties

3 RESULTS

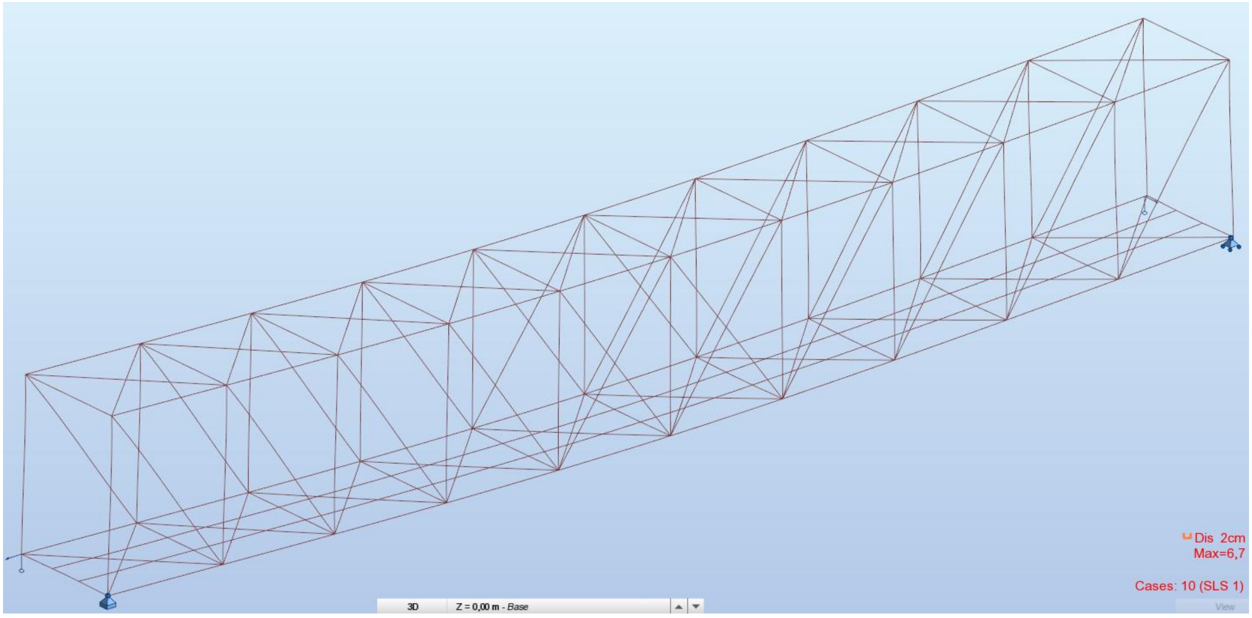
3.1 FEM Model



Picture 18, View of FEM model

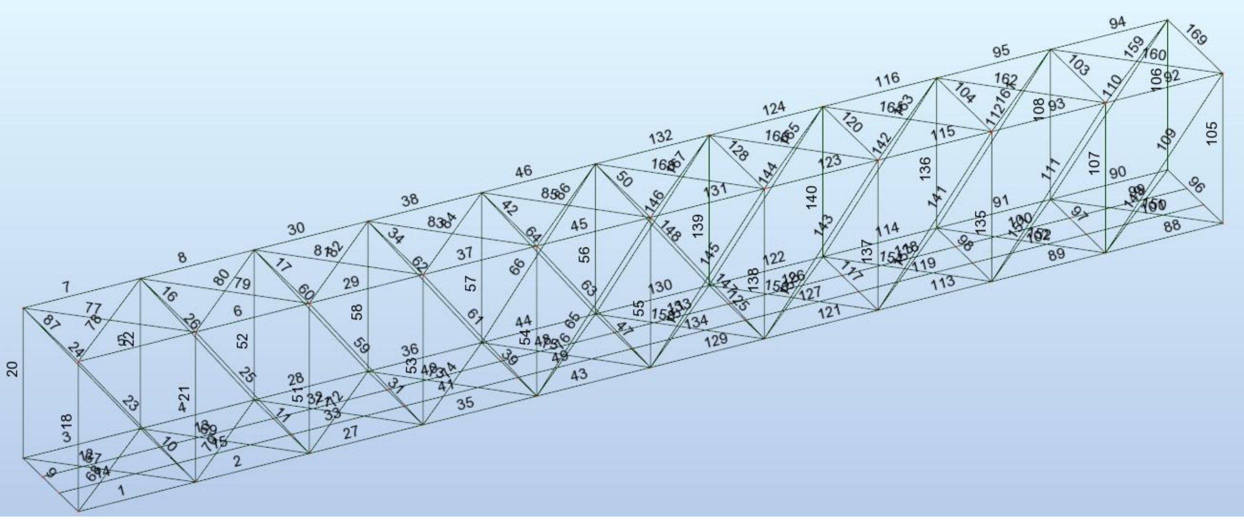
3.1.1 SLS results (Serviceability Limit State)

Total deflection of bridge is 6,7cm = L/776.
Deflection is less than allowed L/600 for railway bridges according to EN 1990-1, A2.4.4.2.3 (1), [1].



Picture 19, Deflection

3.1.2 ULS results (Ultimate Limit State)



Picture 20, member numbers for profiles that will be utilized

After analysis, utilization ratio may be calculated for each member of the bridge truss.

3.1.2.1 Lower main girder utilization

The highest utilization ratio for the members is 0,44
The critical load combination was L2: 1,25*Seflweight+1,45*TrafficLoad+1,50*0,75*Wind, and the traffic load's point loads were placed at midspan.

[13:09:2017] : [13:54:36] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Lower main girder

Results | Messages

Member		Section	Material	Lay	Laz	Ratio	Case
130 Simple bar_1	OK	Lower main gi	STEEL	19.29	25.18	0.44	6 L2.Eq610b/1-mids
44 Simple bar_44	OK	Lower main gi	STEEL	19.29	25.18	0.44	6 L2.Eq610b/1-mids
43 Simple bar_43	OK	Lower main gi	STEEL	19.29	25.18	0.40	6 L2.Eq610b/1-mids
129 Simple bar_1	OK	Lower main gi	STEEL	19.29	25.18	0.40	6 L2.Eq610b/1-mids
122 Simple bar_1	OK	Lower main gi	STEEL	19.29	25.18	0.37	6 L2.Eq610b/1-mids
36 Simple bar_36	OK	Lower main gi	STEEL	19.29	25.18	0.37	6 L2.Eq610b/1-mids
35 Simple bar_35	OK	Lower main gi	STEEL	19.29	25.18	0.34	6 L2.Eq610b/1-mids
121 Simple bar_1	OK	Lower main gi	STEEL	19.29	25.18	0.33	6 L2.Eq610b/1-mids
114 Simple bar_1	OK	Lower main gi	STEEL	19.29	25.18	0.28	6 L2.Eq610b/1-mids
28 Simple bar_28	OK	Lower main gi	STEEL	19.29	25.18	0.27	6 L2.Eq610b/1-mids
27 Simple bar_27	OK	Lower main gi	STEEL	19.29	25.18	0.26	6 L2.Eq610b/1-mids
113 Simple bar_1	OK	Lower main gi	STEEL	19.29	25.18	0.25	6 L2.Eq610b/1-mids
88 Simple bar_88	OK	Lower main gi	STEEL	19.88	25.96	0.17	6 L2.Eq610b/1-mids
3 Simple bar_3	OK	Lower main gi	STEEL	19.88	25.96	0.16	6 L2.Eq610b/1-mids
91 Simple bar_91	OK	Lower main gi	STEEL	19.29	25.18	0.16	6 L2.Eq610b/1-mids
2 Simple bar_2	OK	Lower main gi	STEEL	19.29	25.18	0.15	6 L2.Eq610b/1-mids
1 Simple bar_1	OK	Lower main gi	STEEL	19.88	25.96	0.15	6 L2.Eq610b/1-mids
4 Simple bar_4	OK	Lower main gi	STEEL	19.29	25.18	0.14	6 L2.Eq610b/1-mids
90 Simple bar_90	OK	Lower main gi	STEEL	19.88	25.96	0.14	6 L2.Eq610b/1-mids
89 Simple bar_89	OK	Lower main gi	STEEL	19.29	25.18	0.13	6 L2.Eq610b/1-mids

Calc. Note | Close | Help

Calculation points
Division: n = 3
Extremes: none
Additional: none

Table 1, Utilization of lower main girder in order of utilization ratio

3.1.2.2 Upper main girder utilization

The highest utilization ratio for the members is 0,86
The critical load combination was L2: 1,25*Seflweight+1,45*TrafficLoad+1,50*0,75*Wind, and the traffic load's point loads were placed at midspan.

[13:09:2017] : [13:53:17] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Upper main girder

Results | Messages

Calc. Note | Close | Help

Member		Section	Material	Lay	Laz	Ratio	Case
131 Simple bar_1	OK	Upper main gir	STEEL	29.87	25.37	0.86	6 L2.Eq610b/1-mids
45 Simple bar_45	OK	Upper main gir	STEEL	29.87	25.37	0.86	6 L2.Eq610b/1-mids
46 Simple bar_46	OK	Upper main gir	STEEL	29.87	25.37	0.83	6 L2.Eq610b/1-mids
132 Simple bar_1	OK	Upper main gir	STEEL	29.87	25.37	0.83	6 L2.Eq610b/1-mids
123 Simple bar_1	OK	Upper main gir	STEEL	29.87	25.37	0.79	6 L2.Eq610b/1-mids
37 Simple bar_37	OK	Upper main gir	STEEL	29.87	25.37	0.79	6 L2.Eq610b/1-mids
38 Simple bar_38	OK	Upper main gir	STEEL	29.87	25.37	0.77	6 L2.Eq610b/1-mids
124 Simple bar_1	OK	Upper main gir	STEEL	29.87	25.37	0.77	6 L2.Eq610b/1-mids
115 Simple bar_1	OK	Upper main gir	STEEL	29.87	25.37	0.68	6 L2.Eq610b/1-mids
29 Simple bar_29	OK	Upper main gir	STEEL	29.87	25.37	0.68	6 L2.Eq610b/1-mids
30 Simple bar_30	OK	Upper main gir	STEEL	29.87	25.37	0.67	6 L2.Eq610b/1-mids
116 Simple bar_1	OK	Upper main gir	STEEL	29.87	25.37	0.66	6 L2.Eq610b/1-mids
93 Simple bar_93	OK	Upper main gir	STEEL	29.87	25.37	0.48	6 L2.Eq610b/1-mids
8 Simple bar_8	OK	Upper main gir	STEEL	29.87	25.37	0.48	6 L2.Eq610b/1-mids
6 Simple bar_6	OK	Upper main gir	STEEL	29.87	25.37	0.48	6 L2.Eq610b/1-mids
95 Simple bar_95	OK	Upper main gir	STEEL	29.87	25.37	0.47	6 L2.Eq610b/1-mids
7 Simple bar_7	OK	Upper main gir	STEEL	30.80	26.16	0.29	8 L4.Eq610b/1-atsu
94 Simple bar_94	OK	Upper main gir	STEEL	30.80	26.16	0.28	6 L2.Eq610b/1-mids
92 Simple bar_92	OK	Upper main gir	STEEL	30.80	26.16	0.28	6 L2.Eq610b/1-mids
5 Simple bar_5	OK	Upper main gir	STEEL	30.80	26.16	0.27	8 L4.Eq610b/1-atsu

Calculation points
Division: n = 3
Extremes: none
Additional: none

Table 2, Utilization of upper main girder in order of utilization ratio

3.1.2.3 Cross beams' utilization

The highest utilization ratio for the members is 0,71
The critical load combination was L4: 1,25*Seflweight+1,45*TrafficLoad+1,50*0,75*Wind, and the traffic load's point loads were placed at support.

[13:09:2017] : [13:52:15] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Cross girders

Results | Messages

Calc. Note | Close | Help

Member		Section	Material	Lay	Laz	Ratio	Case
10 Simple bar_10	OK	Cross girder	STEEL	18.00	74.45	0.71	8 L4.Eq610b/1-atsu
97 Simple bar_97	OK	Cross girder	STEEL	18.00	74.45	0.66	6 L2.Eq610b/1-mids
11 Simple bar_11	OK	Cross girder	STEEL	18.00	74.45	0.61	6 L2.Eq610b/1-mids
98 Simple bar_98	OK	Cross girder	STEEL	18.00	74.45	0.61	6 L2.Eq610b/1-mids
31 Simple bar_31	OK	Cross girder	STEEL	18.00	74.45	0.51	6 L2.Eq610b/1-mids
117 Simple bar_1	OK	Cross girder	STEEL	18.00	74.45	0.51	6 L2.Eq610b/1-mids
96 Simple bar_96	OK	Cross girder	STEEL	18.00	74.45	0.49	6 L2.Eq610b/1-mids
9 Simple bar_9	OK	Cross girder	STEEL	18.00	74.45	0.47	6 L2.Eq610b/1-mids
39 Simple bar_39	OK	Cross girder	STEEL	18.00	74.45	0.42	6 L2.Eq610b/1-mids
125 Simple bar_1	OK	Cross girder	STEEL	18.00	74.45	0.42	6 L2.Eq610b/1-mids
47 Simple bar_47	OK	Cross girder	STEEL	18.00	74.45	0.41	6 L2.Eq610b/1-mids

Calculation points
Division: n = 3
Extremes: none
Additional: none

Table 3, Utilization of cross beams in order of utilization ratio

3.1.2.4 Rail bearers' utilization

The highest utilization ratio for the members is 0,27
The critical load combination was L4: 1,25*Seflweight+1,45*TrafficLoad+1,50*0,75*Wind, and the traffic load's point loads were placed at support.

[13:09:2017] : [13:50:44] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Longitudinal

Results | Messages

Calc. Note | Close | Help

Member		Section	Material	Lay	Laz	Ratio	Case
12 Simple bar_12	OK	Longitudinal gi	STEEL	22.14	116.11	0.27	8 L4.Eq610b/1-atsu
14 Simple bar_14	OK	Longitudinal gi	STEEL	22.14	116.11	0.24	8 L4.Eq610b/1-atsu
15 Simple bar_15	OK	Longitudinal gi	STEEL	21.47	112.64	0.21	8 L4.Eq610b/1-atsu
133 Simple bar_1	OK	Longitudinal gi	STEEL	21.47	112.64	0.21	6 L2.Eq610b/1-mids
48 Simple bar_48	OK	Longitudinal gi	STEEL	21.47	112.64	0.21	6 L2.Eq610b/1-mids
13 Simple bar_13	OK	Longitudinal gi	STEEL	21.47	112.64	0.20	8 L4.Eq610b/1-atsu
101 Simple bar_1	OK	Longitudinal gi	STEEL	22.14	116.11	0.19	6 L2.Eq610b/1-mids
102 Simple bar_1	OK	Longitudinal gi	STEEL	21.47	112.64	0.19	6 L2.Eq610b/1-mids
100 Simple bar_1	OK	Longitudinal gi	STEEL	21.47	112.64	0.19	6 L2.Eq610b/1-mids
99 Simple bar_99	OK	Longitudinal gi	STEEL	22.14	116.11	0.18	6 L2.Eq610b/1-mids
134 Simple bar_1	OK	Longitudinal gi	STEEL	21.47	112.64	0.18	6 L2.Eq610b/1-mids
49 Simple bar_49	OK	Longitudinal gi	STEEL	21.47	112.64	0.18	6 L2.Eq610b/1-mids
33 Simple bar_33	OK	Longitudinal gi	STEEL	21.47	112.64	0.16	6 L2.Eq610b/1-mids
119 Simple bar_1	OK	Longitudinal gi	STEEL	21.47	112.64	0.15	6 L2.Eq610b/1-mids
118 Simple bar_1	OK	Longitudinal gi	STEEL	21.47	112.64	0.14	6 L2.Eq610b/1-mids
126 Simple bar_1	OK	Longitudinal gi	STEEL	21.47	112.64	0.13	6 L2.Eq610b/1-mids
32 Simple bar_32	OK	Longitudinal gi	STEEL	21.47	112.64	0.13	6 L2.Eq610b/1-mids
40 Simple bar_40	OK	Longitudinal gi	STEEL	21.47	112.64	0.13	6 L2.Eq610b/1-mids
41 Simple bar_41	OK	Longitudinal gi	STEEL	21.47	112.64	0.13	6 L2.Eq610b/1-mids
127 Simple bar_1	OK	Longitudinal gi	STEEL	21.47	112.64	0.13	6 L2.Eq610b/1-mids

Calculation points
Division: n = 3
Extremes: none
Additional: none

Table 4, Utilization of rail bearers in order of utilization ratio

3.1.2.5 Diagonals' utilization

The highest utilization ratio for the members is 0,59
The critical load combination was L2: 1,25*Seflweight+1,45*TrafficLoad+1,50*0,75*Wind, and the traffic load's point loads were placed at midspan.

[13:09:2017] : [13:57:04] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Diagonals

Results | Messages

Calc. Note | Close | Help

Member		Section	Material	Lay	Laz	Ratio	Case
24 Simple bar_24	OK	Diagonals/1	STEEL	67.47	53.56	0.59	6 L2.Eq610b/1-mids
110 Simple bar_1	OK	Diagonals/1	STEEL	67.47	53.56	0.59	6 L2.Eq610b/1-mids
109 Simple bar_1	OK	Diagonals/1	STEEL	67.47	53.56	0.58	6 L2.Eq610b/1-mids
23 Simple bar_23	OK	Diagonals/1	STEEL	67.47	53.56	0.57	6 L2.Eq610b/1-mids
26 Simple bar_26	OK	Diagonals/2	STEEL	63.96	53.92	0.54	6 L2.Eq610b/1-mids
112 Simple bar_1	OK	Diagonals/2	STEEL	63.96	53.92	0.54	6 L2.Eq610b/1-mids
111 Simple bar_1	OK	Diagonals/2	STEEL	63.96	53.92	0.54	6 L2.Eq610b/1-mids
25 Simple bar_25	OK	Diagonals/2	STEEL	63.96	53.92	0.53	6 L2.Eq610b/1-mids
60 Simple bar_60	OK	Diagonals/3	STEEL	80.37	54.55	0.50	6 L2.Eq610b/1-mids
141 Simple bar_1	OK	Diagonals/3	STEEL	80.37	54.55	0.49	6 L2.Eq610b/1-mids
142 Simple bar_1	OK	Diagonals/3	STEEL	80.37	54.55	0.49	6 L2.Eq610b/1-mids
59 Simple bar_59	OK	Diagonals/3	STEEL	80.37	54.55	0.49	6 L2.Eq610b/1-mids
62 Simple bar_62	OK	Diagonals/3	STEEL	80.37	54.55	0.34	6 L2.Eq610b/1-mids
143 Simple bar_1	OK	Diagonals/3	STEEL	80.37	54.55	0.34	6 L2.Eq610b/1-mids
144 Simple bar_1	OK	Diagonals/3	STEEL	80.37	54.55	0.33	6 L2.Eq610b/1-mids
61 Simple bar_61	OK	Diagonals/3	STEEL	80.37	54.55	0.33	6 L2.Eq610b/1-mids
64 Simple bar_64	OK	Diagonals/4	STEEL	95.68	55.80	0.15	6 L2.Eq610b/1-mids
145 Simple bar_1	OK	Diagonals/4	STEEL	95.68	55.80	0.15	6 L2.Eq610b/1-mids
146 Simple bar_1	OK	Diagonals/4	STEEL	95.68	55.80	0.15	6 L2.Eq610b/1-mids
63 Simple bar_63	OK	Diagonals/4	STEEL	95.68	55.80	0.14	6 L2.Eq610b/1-mids
65 Simple bar_65	OK	Flat diagonals	STEEL	148.76	1078.50	0.00	5 L1.Eq610a
66 Simple bar_66	OK	Flat diagonals	STEEL	148.76	1078.50	0.00	5 L1.Eq610a
147 Simple bar_1	OK	Flat diagonals	STEEL	148.76	1078.50	0.00	5 L1.Eq610a
148 Simple bar_1	OK	Flat diagonals	STEEL	148.76	1078.50	0.00	5 L1.Eq610a

Calculation points
Division: n = 3
Extremes: none
Additional: none

Table 5, Utilization of diagonals in order of utilization ratio

3.1.2.6 Columns' utilization

The highest utilization ratio for the members is 0,66
The critical load combination was L2: 1,25*Seflweight+1,45*TrafficLoad+1,50*0,75*Wind, and the traffic load's point loads were placed at midspan.

[13:09:2017] : [13:57:51] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Columns

Member	Section	Material	Lay	Laz	Ratio	Case
135 Simple bar_1 OK	Column/2	STEEL	49.71	43.04	0.66	6 L2.Eq610b/1-mids
22 Simple bar_22 OK	Column/1	STEEL	46.07	45.61	0.65	6 L2.Eq610b/1-mids
107 Simple bar_1 OK	Column/1	STEEL	46.07	45.61	0.64	6 L2.Eq610b/1-mids
108 Simple bar_1 OK	Column/1	STEEL	46.07	45.61	0.64	6 L2.Eq610b/1-mids
51 Simple bar_51 OK	Column/2	STEEL	49.71	43.04	0.64	6 L2.Eq610b/1-mids
21 Simple bar_21 OK	Column/1	STEEL	46.07	45.61	0.62	6 L2.Eq610b/1-mids
52 Simple bar_52 OK	Column/2	STEEL	49.71	43.04	0.62	6 L2.Eq610b/1-mids
137 Simple bar_1 OK	Column/3	STEEL	60.75	44.15	0.62	6 L2.Eq610b/1-mids
136 Simple bar_1 OK	Column/2	STEEL	49.71	43.04	0.60	6 L2.Eq610b/1-mids
53 Simple bar_53 OK	Column/3	STEEL	60.75	44.15	0.60	6 L2.Eq610b/1-mids
58 Simple bar_58 OK	Column/3	STEEL	60.75	44.15	0.55	6 L2.Eq610b/1-mids
140 Simple bar_1 OK	Column/3	STEEL	60.75	44.15	0.53	6 L2.Eq610b/1-mids
138 Simple bar_1 OK	Column/3	STEEL	60.75	44.15	0.38	6 L2.Eq610b/1-mids
54 Simple bar_54 OK	Column/3	STEEL	60.75	44.15	0.37	6 L2.Eq610b/1-mids
57 Simple bar_57 OK	Column/3	STEEL	60.75	44.15	0.30	6 L2.Eq610b/1-mids
139 Simple bar_1 OK	Column/3	STEEL	60.75	44.15	0.29	6 L2.Eq610b/1-mids
55 Simple bar_55 OK	Column/3	STEEL	60.75	44.15	0.22	6 L2.Eq610b/1-mids
56 Simple bar_56 OK	Column/3	STEEL	60.75	44.15	0.15	6 L2.Eq610b/1-mids

Calculation points
Division: n = 3
Extremes: none
Additional: none

Table 6, Utilization of columns in order of utilization ratio

3.1.2.7 End columns' utilization

The highest utilization ratio for the members is 0,53
The critical load combination was L4: 1,25*Seflweight+1,45*TrafficLoad+1,50*0,75*Wind, and the traffic load's point loads were placed at support.

[13:09:2017] : [13:58:18] SFS-EN 1993-1:2005/NA:2007/A1:2014 - End columns

Member	Section	Material	Lay	Laz	Ratio	Case
20 Simple bar_20 OK	End column	STEEL	26.73	35.14	0.53	8 L4.Eq610b/1-atsu
106 Simple bar_1 OK	End column	STEEL	26.73	35.14	0.42	6 L2.Eq610b/1-mids
18 Simple bar_18 OK	End column	STEEL	26.73	35.14	0.38	8 L4.Eq610b/1-atsu
105 Simple bar_1 OK	End column	STEEL	26.73	35.14	0.30	6 L2.Eq610b/1-mids

Calculation points
Division: n = 3
Extremes: none

Table 7, Utilization of end columns in order of utilization ratio

3.1.2.8 Upper cross bracings' utilization

The highest utilization ratio for the members is 0,12
The critical load combination was L2: 1,25*Seflweight+1,45*TrafficLoad+1,50*0,75*Wind, and the traffic load's point loads were placed at midspan.

[13:09:2017] : [13:59:29] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Upper cross bracing

Member	Section	Material	Lay	Laz	Ratio	Case
120 Simple bar_1 OK	Upper cross b	STEEL	27.22	163.23	0.12	6 L2.Eq610b/1-mids
34 Simple bar_34 OK	Upper cross b	STEEL	27.22	163.23	0.12	6 L2.Eq610b/1-mids
128 Simple bar_1 OK	Upper cross b	STEEL	27.22	163.23	0.12	6 L2.Eq610b/1-mids
42 Simple bar_42 OK	Upper cross b	STEEL	27.22	163.23	0.12	6 L2.Eq610b/1-mids
87 Simple bar_87 OK	Upper End cro	STEEL	18.45	86.68	0.12	9 L5.Eq610b/2-atsu
50 Simple bar_50 OK	Upper cross b	STEEL	27.22	163.23	0.12	6 L2.Eq610b/1-mids
17 Simple bar_17 OK	Upper cross b	STEEL	27.22	163.23	0.10	6 L2.Eq610b/1-mids
104 Simple bar_1 OK	Upper cross b	STEEL	27.22	163.23	0.10	6 L2.Eq610b/1-mids
169 Simple bar_1 OK	Upper End cro	STEEL	18.45	86.68	0.09	7 L3.Eq610b/2-mids
103 Simple bar_1 OK	Upper cross b	STEEL	27.22	163.23	0.07	6 L2.Eq610b/1-mids
16 Simple bar_16 OK	Upper cross b	STEEL	27.22	163.23	0.07	6 L2.Eq610b/1-mids

Calculation points
Division: n = 3
Extremes: none
Additional: none

Table 8, Utilization of upper cross bracings in order of utilization ratio

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.

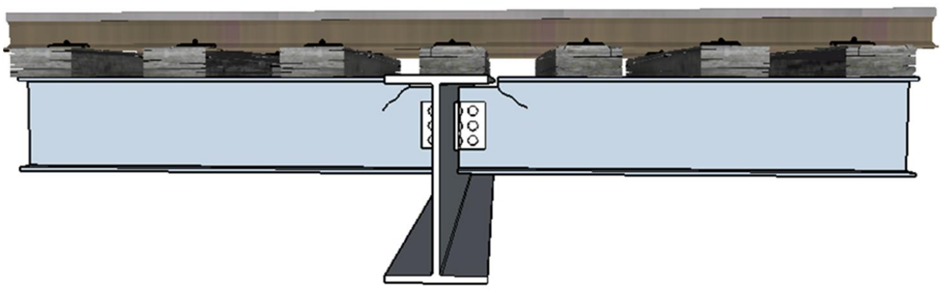
The document IRS 77802 (former UIC 778-2) "Recommendations for determining the carrying capacity and fatigue risks of existing metallic railway bridges" gives instructions for Fatigue Susceptible Details, which generally have a more unreliable fatigue performance and experience indicates they are more 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

52 m span

Deflection of the bridge is less than allowed L/600 for railway bridges according to EN 1990-1, A2.4.4.2.3 (1), [1].

Ultimate limit state verification shows that structures are feasible for higher loading.
Utilization of profiles for truss bridge are 86 % at the most critical section. The engineer has chosen all the assumptions with safety margins.

Joints of cross-girder – Rail bearer connection

For Cross beams and rail bearers it is recommended to be renewed for structural reasons due to rail system update, even with results showing maximum utilization of 71%. However, the joints of these secondary girders are more critical than the tension or stress of the girder materials.

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 Struct9ures: 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: N.A. (NOT APPLICABLE)

APPENDIX 2: CONNENCTION CALCULATION

APPENDIX 2

Connection Calculation Report - Crossgirder-railbearer

Material:

Bolt class: 4.6

Ref. EN1993-1-8
§3.1.1
Table 3.1

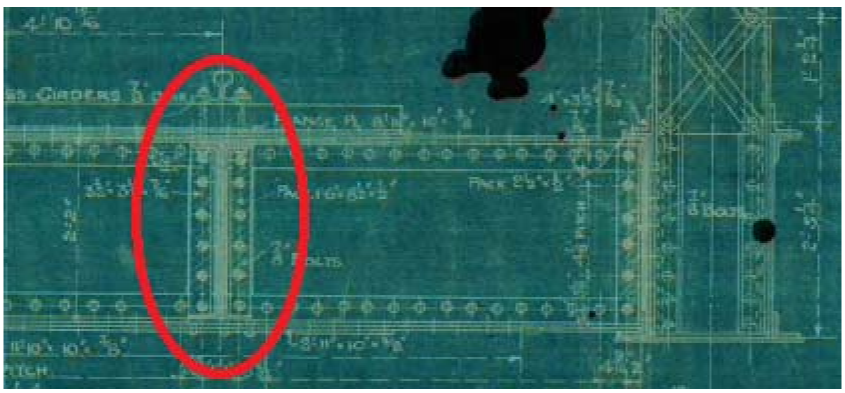
$$f_{yb} := 240 \frac{N}{mm^2} \qquad f_{ub} := 400 \frac{N}{mm^2}$$

Steel grade: S235

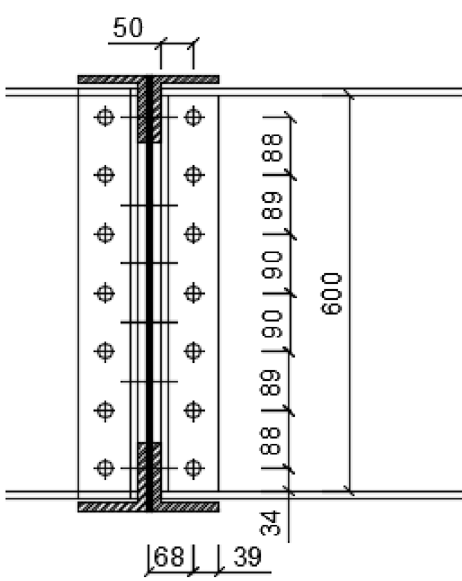
Ref. EN1993-1-1
§3.2.3
Table 3.1

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

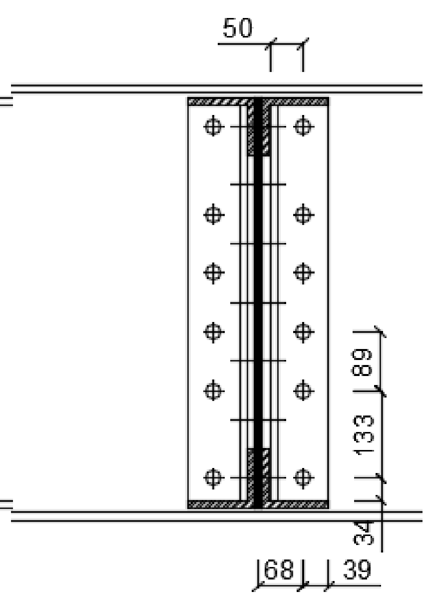
Geometry of joint:



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

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

Ref. EN1993-1-8 §3.6.1 Table 3.4

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

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

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

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

Ref.
EN1993-1-8
§3.10.2 (2)

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

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

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

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

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

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

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

Ref.
EN1993-1-8
§3.10.2 (2)

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

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

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

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

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)

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

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

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

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

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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