# Railway Project

## Rio Negro RAILWAY BRIDGE

### CALCULATION REPORT

DOCUMENT APPROVAL

| VERSIO | DESIGNER | SUPERVISOR | ACCEPT. |
|---|---|---|---|
| 15.12.2017 | Vera Babiczky | Mikko Ilvonen | |

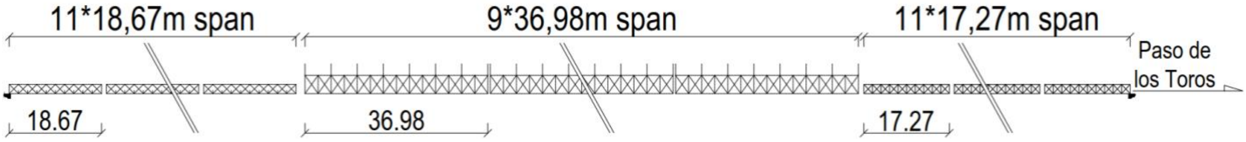
REVISION HISTORY

| | | | |
|---|---|---|---|
| | | | |

# CONTENT

# 1 SCOPE

Rio Negro Bridge consists of three different types of cross sections and three different span lengths. Total length of the bridge is 704 m; consisting of 11x 18,67 m spans, 9x 37 m spans and 11x 17,27 m spans.



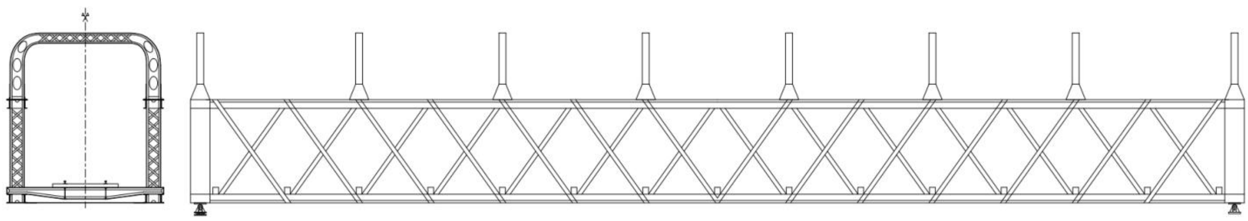

Picture 1, Rio Negro bridge side view

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 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.

## 1.1 37 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.

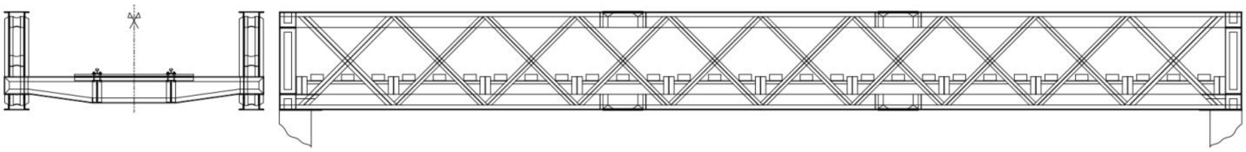


Picture 2, Rio Negro Bridge 37 m

## 1.2 18,67 m Truss Bridge

The main goal of this calculation is to show that the old truss structures could be utilized from existing 19 m truss sections of the bridge.

The girder bridge of 19 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. The pre-engineering solution is to replace these spans.

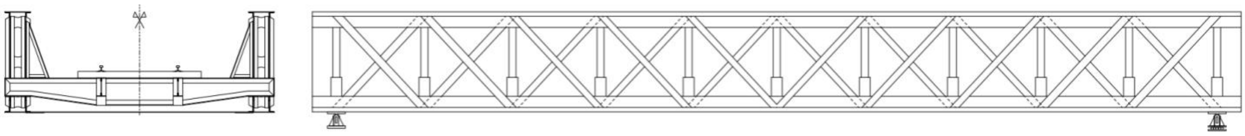


Picture 3, Rio Negro Bridge 19 m

## 1.3 17 m Truss Bridge

The main goal of this calculation is to show that the old truss structures could be utilized from existing 17 m truss sections of the bridge.

The girder bridge of 17 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. The pre-engineering solution is to replace these spans.



Picture 4, Rio Negro 17 m

# 2 DESIGN CRITERIA

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

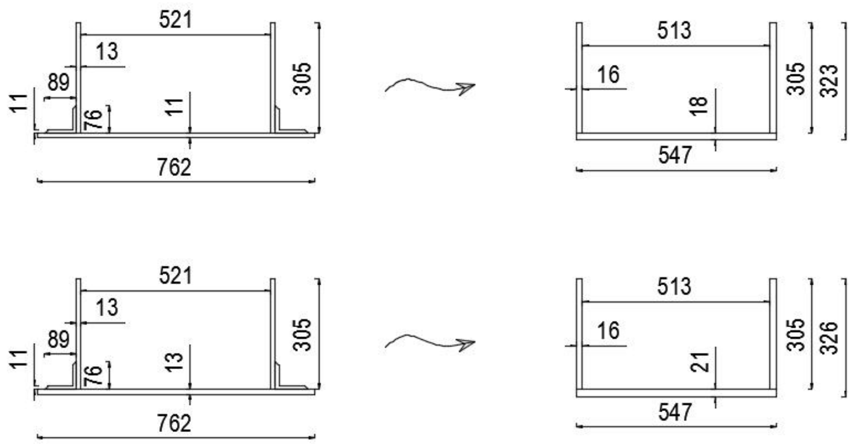
## 2.1 Structure

Bridge super structure members are complicated 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 6-24 in section 2.1.1.

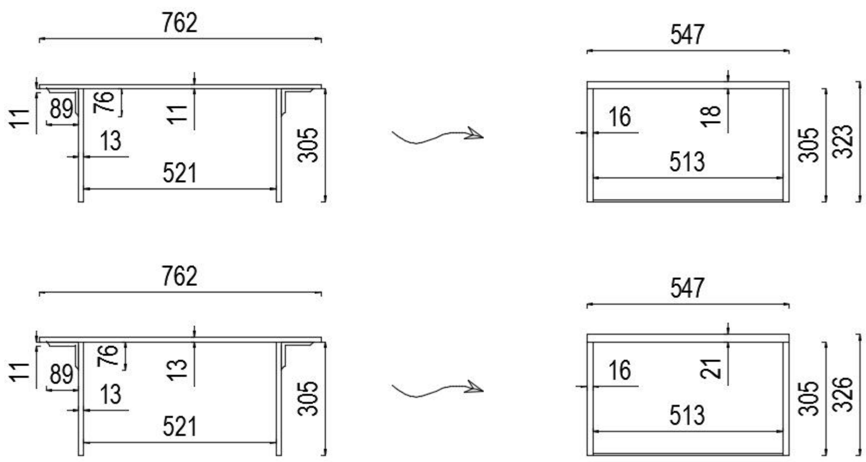
The calculations are based on an estimated geometry from insufficient quality old drawings, photos and limited site visit information. Before making final design, all structures need to be verified on site in order to gather the missing data.

Picture 5, Rio Negro inside view

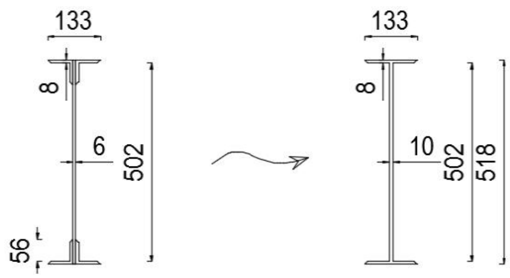
### 2.1.1 Simplifications for sections in 37 m span bridge



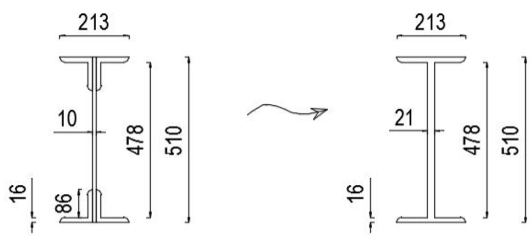

Picture 6, Lower main girder



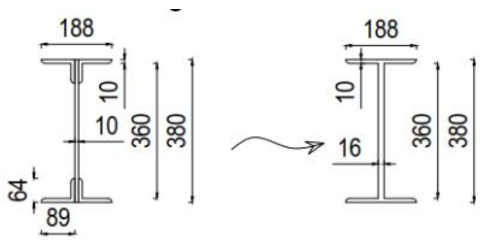

Picture 7, Upper main girder, type 1



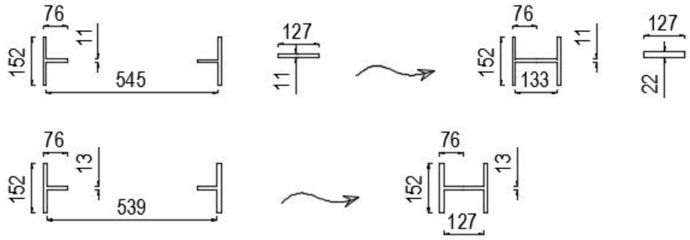

Picture 8, Longitudinal girder



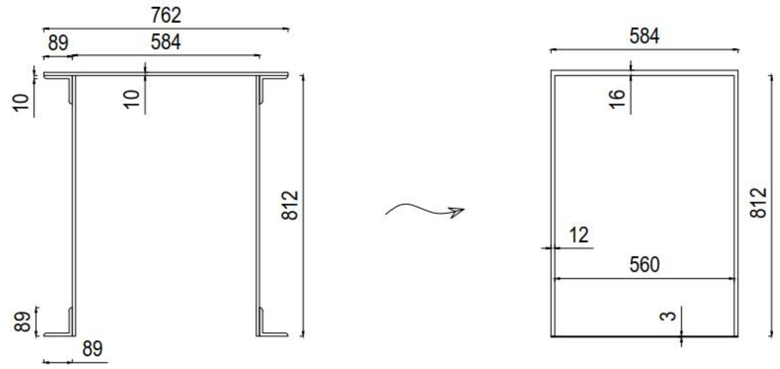

Picture 9, Cross girder




Picture 10, Upper cross bracing

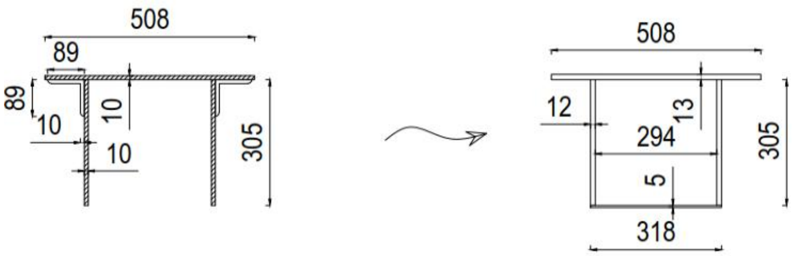



Picture 11, Diagonals

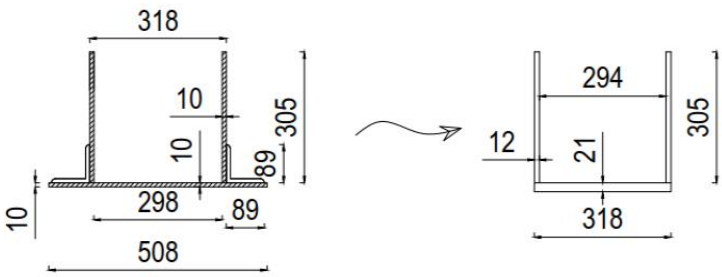



Picture 12, End frame

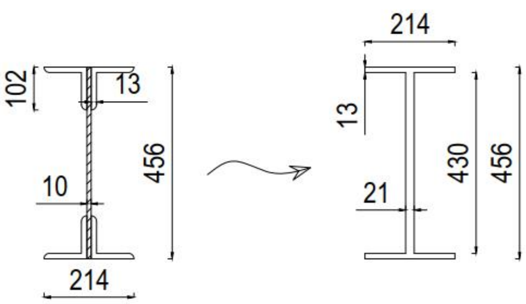
### 2.1.2 Simplifications for sections in 18,67 m span bridge



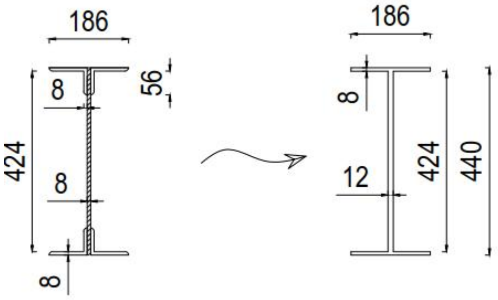

Picture 13, Upper main girder



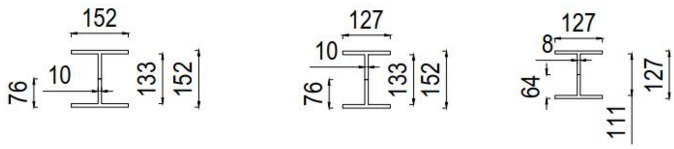

Picture 14, Lower main girder



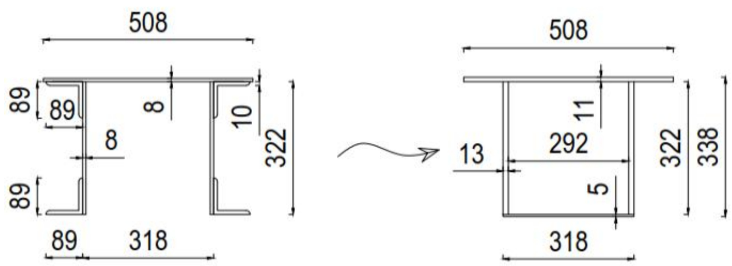

Picture 15, Cross girder




Picture 16, Longitudinal girder

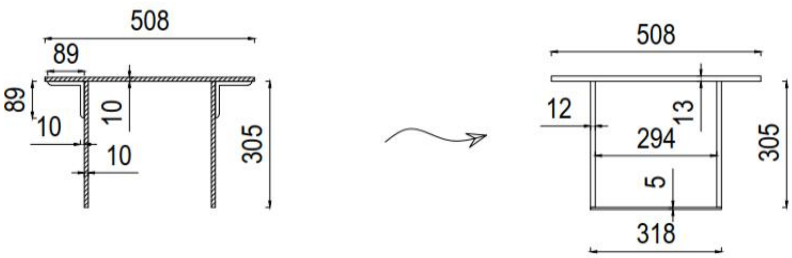



Picture 17, Diagonals, 3 types

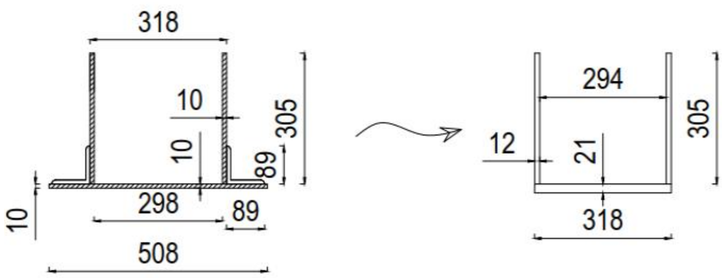



Picture 18, End frame

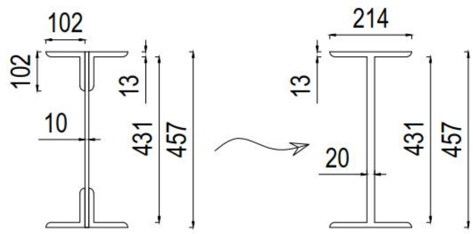
### 2.1.3 Simplifications for sections in 17 m span bridge



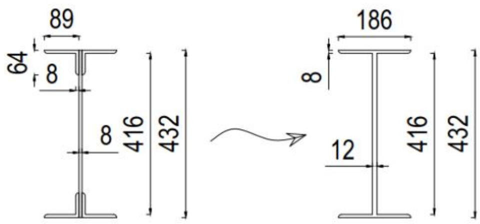

Picture 19, Upper main girder



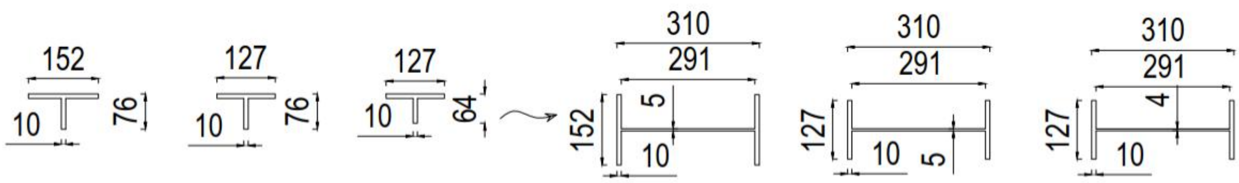

Picture 20, Lower main girder



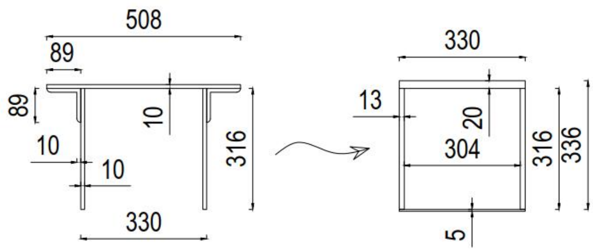

Picture 21, Cross girder




Picture 22, Longitudinal girder




Picture 23, Diagonals, 3 types




Picture 24, End frame

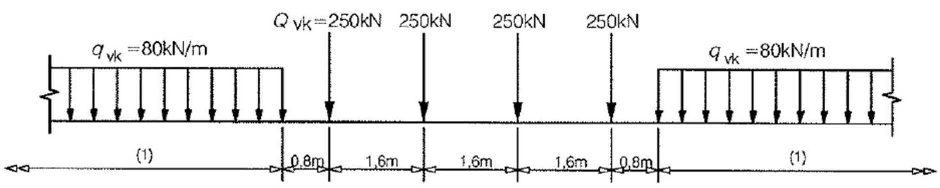
## 2.2 Loads

### 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

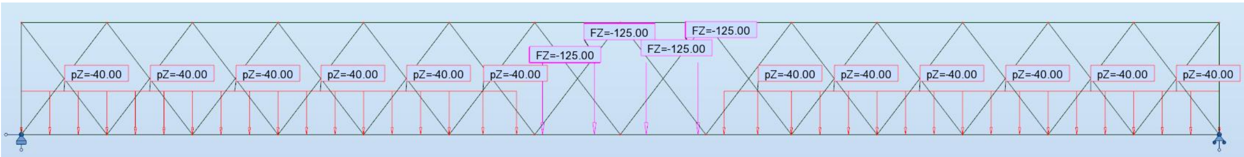
Train axle load is increased to 22,5 tons. Load is applied according to EN 1991-2, section 6.3.2, load model LM71.

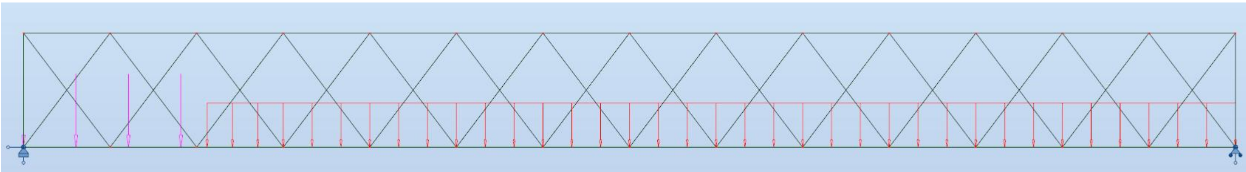



**Key**
(1) No limitation

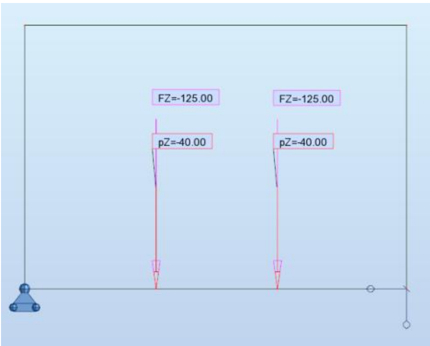
Picture 25, Train load model 71

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.



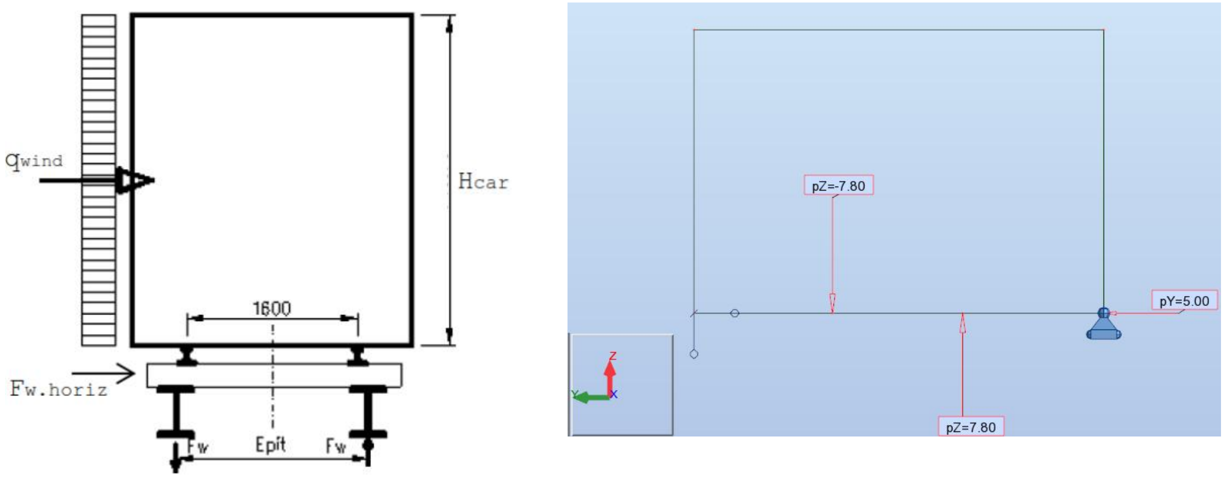
Picture 26, Load model 71 applications – side view (37m span bridge)




Picture 27, Load model 71 applications – 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 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 $\gamma$ 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 $\gamma$ 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).
$\gamma_P$ = 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 $\gamma$ 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 $\gamma$ values. The following values of $\gamma$ 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 $\psi$ factors for railway bridges

| Actions | | | $\psi_0$ | $\psi_1$ | $\psi_2$ [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 $\psi_0$ $F_{Wk}$ 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, $\psi_2$ 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 $\psi$ value should be applied to one group of loads as defined in EN 1991-2, and taken as equal to the $\psi$ value applicable to the leading component of the group.

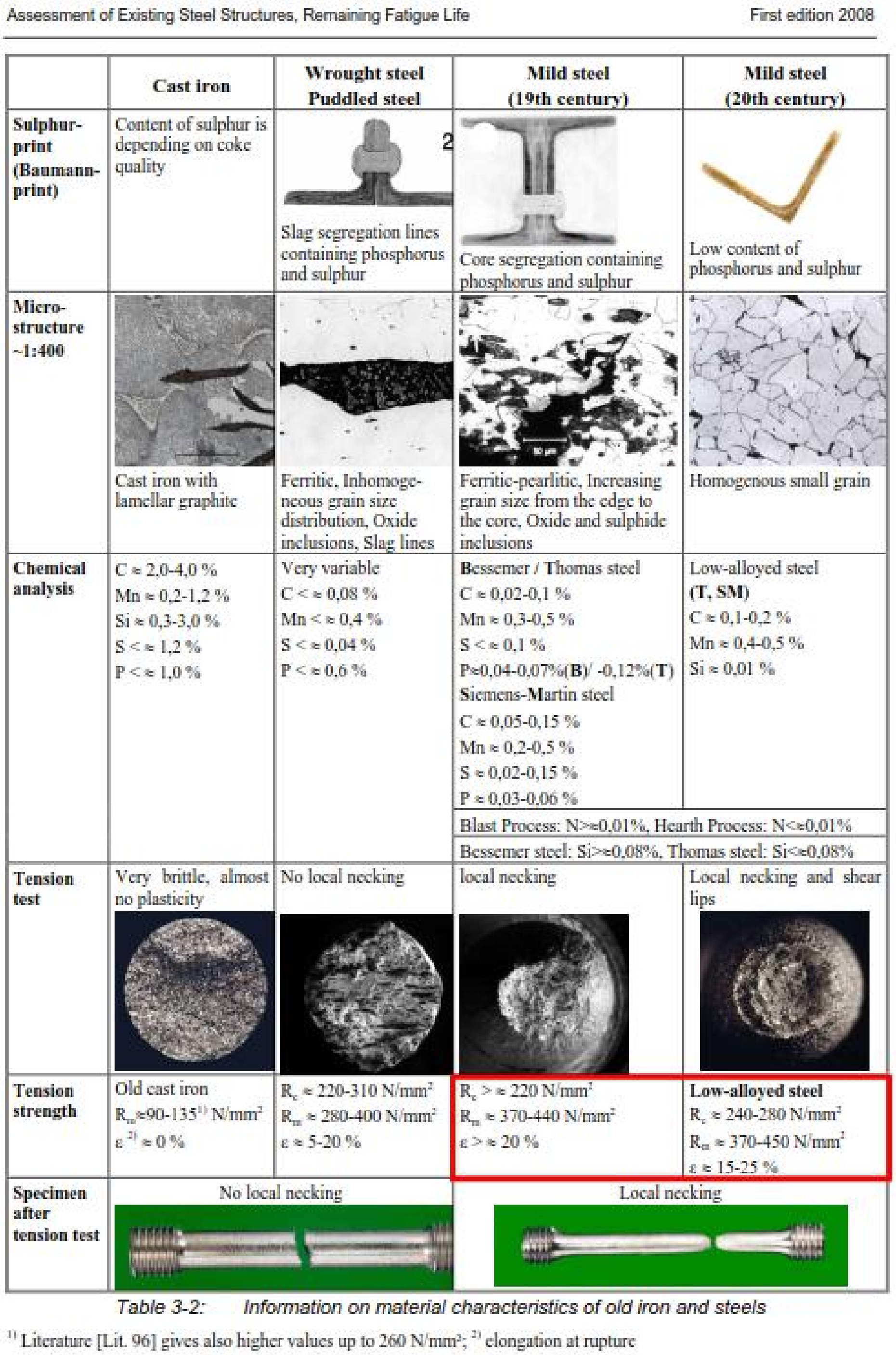
ULS Load combinations:

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<br>Puddled steel | Mild steel<br>(19th century) | Mild steel<br>(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 %<br>Mn ≈ 0,2-1,2 %<br>Si ≈ 0,3-3,0 %<br>S < ≈ 1,2 %<br>P < ≈ 1,0 % | Very variable<br>C < ≈ 0,08 %<br>Mn < ≈ 0,4 %<br>S < ≈ 0,04 %<br>P < ≈ 0,6 % | Bessemer / Thomas steel<br>C ≈ 0,02-0,1 %<br>Mn ≈ 0,3-0,5 %<br>S < ≈ 0,1 %<br>P≈0,04-0,07%(**B**)/ -0,12%(**T**)<br>Siemens-Martin steel<br>C ≈ 0,05-0,15 %<br>Mn ≈ 0,2-0,5 %<br>S ≈ 0,02-0,15 %<br>P ≈ 0,03-0,06 % | Low-alloyed steel<br>**(T, SM)**<br>C ≈ 0,1-0,2 %<br>Mn ≈ 0,4-0,5 %<br>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<br>$R_m$≈90-135[1)] N/mm²<br>ε [2)] ≈ 0 % | $R_e$ ≈ 220-310 N/mm²<br>$R_m$ ≈ 280-400 N/mm²<br>ε ≈ 5-20 % | $R_e$ > ≈ 220 N/mm²<br>$R_m$ ≈ 370-440 N/mm²<br>ε > ≈ 20 % | **Low-alloyed steel**<br>$R_e$ ≈ 240-280 N/mm²<br>$R_m$ ≈ 370-450 N/mm²<br>ε ≈ 15-25 % |
| **Specimen after tension test** | No local necking | | Local necking | |

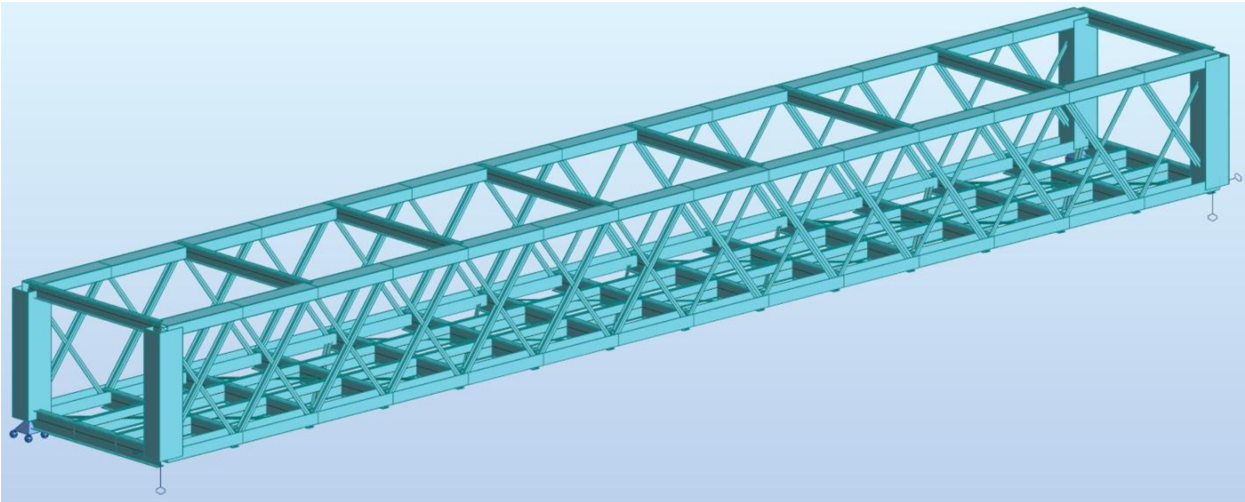
*Table 3-2: Information on material characteristics of old iron and steels*

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

Picture 29, steel material properties

# 3 RESULTS

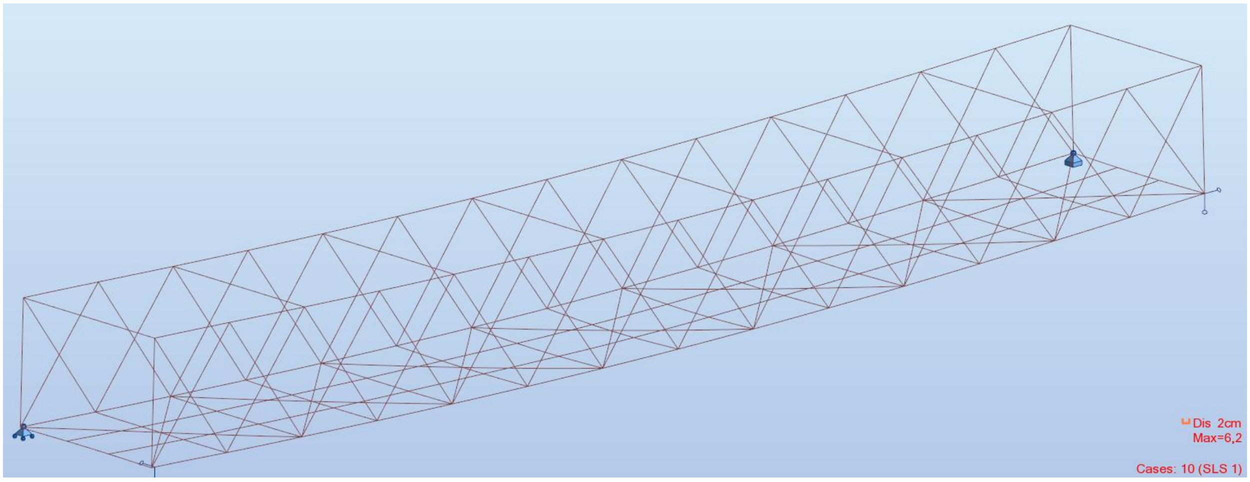
## 3.1 37 m span Lattice/Truss Bridge



Picture 30, View of FEM model

### 3.1.1 SLS results (Serviceability Limit State)

Total deflection of bridge is 6,2 cm = L/600 which is same as the limit.




Picture 31, Deflection

### 3.1.2 ULS results (Ultimate Limit State)

#### 3.1.2.1 Lower main girders' utilization

The highest utilization ratio for the members is 0,75.
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.

[27:10:2017] : [14:16:53] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Lower main girder

Results | Messages

| Member | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|
| 156 Simple bar_1 | Lower main girder | STEEL | 12.15 | 25.85 | 0.75 | 5 L2.Eq610b |
| 67 Simple bar_67 | Lower main girder | STEEL | 12.15 | 25.85 | 0.75 | 5 L2.Eq610b |
| 148 Simple bar_1 | Lower main girder | STEEL | 12.15 | 25.85 | 0.71 | 5 L2.Eq610b |
| 58 Simple bar_58 | Lower main girder | STEEL | 12.15 | 25.85 | 0.70 | 5 L2.Eq610b |
| 66 Simple bar_66 | Lower main girder | STEEL | 12.15 | 25.85 | 0.67 | 5 L2.Eq610b |
| 155 Simple bar_1 | Lower main girder | STEEL | 12.15 | 25.85 | 0.67 | 5 L2.Eq610b |
| 139 Simple bar_1 | Lower main girder | STEEL | 12.15 | 25.85 | 0.64 | 5 L2.Eq610b |
| 57 Simple bar_57 | Lower main girder | STEEL | 12.15 | 25.85 | 0.63 | 5 L2.Eq610b |
| 49 Simple bar_49 | Lower main girder | STEEL | 12.15 | 25.85 | 0.63 | 5 L2.Eq610b |
| 147 Simple bar_1 | Lower main girder | STEEL | 12.15 | 25.85 | 0.62 | 5 L2.Eq610b |
| 130 Simple bar_1 | Lower main girder | STEEL | 12.14 | 25.90 | 0.61 | 5 L2.Eq610b |
| 40 Simple bar_40 | Lower main girder | STEEL | 12.14 | 25.90 | 0.58 | 5 L2.Eq610b |
| 48 Simple bar_48 | Lower main girder | STEEL | 12.15 | 25.85 | 0.57 | 5 L2.Eq610b |
| 138 Simple bar_1 | Lower main girder | STEEL | 12.15 | 25.85 | 0.56 | 5 L2.Eq610b |
| 39 Simple bar_39 | Lower main girder | STEEL | 12.14 | 25.90 | 0.55 | 5 L2.Eq610b |
| 129 Simple bar_1 | Lower main girder | STEEL | 12.14 | 25.90 | 0.53 | 5 L2.Eq610b |
| 8 Simple bar_8 | Lower main girder | STEEL | 12.14 | 25.90 | 0.49 | 8 L4.Eq610b/at supp |
| 7 Simple bar_7 | Lower main girder | STEEL | 12.14 | 25.90 | 0.47 | 8 L4.Eq610b/at supp |
| 98 Simple bar_98 | Lower main girder | STEEL | 12.14 | 25.90 | 0.47 | 5 L2.Eq610b |
| 121 Simple bar_1 | Lower main girder | STEEL | 12.14 | 25.90 | 0.46 | 5 L2.Eq610b |
| 112 Simple bar_1 | Lower main girder | STEEL | 12.14 | 25.90 | 0.45 | 5 L2.Eq610b |
| 97 Simple bar_97 | Lower main girder | STEEL | 12.14 | 25.90 | 0.45 | 5 L2.Eq610b |
| 31 Simple bar_31 | Lower main girder | STEEL | 12.14 | 25.90 | 0.44 | 5 L2.Eq610b |
| 21 Simple bar_21 | Lower main girder | STEEL | 12.14 | 25.90 | 0.44 | 8 L4.Eq610b/at supp |
| 30 Simple bar_30 | Lower main girder | STEEL | 12.14 | 25.90 | 0.42 | 5 L2.Eq610b |
| 22 Simple bar_22 | Lower main girder | STEEL | 12.14 | 25.90 | 0.41 | 8 L4.Eq610b/at supp |
| 120 Simple bar_1 | Lower main girder | STEEL | 12.14 | 25.90 | 0.40 | 5 L2.Eq610b |
| 111 Simple bar_1 | Lower main girder | STEEL | 12.14 | 25.90 | 0.39 | 5 L2.Eq610b |

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 girders' utilization

The highest utilization ratio for the members is 0,84.
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.

[27:10:2017] : [14:17:58] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Upper main girder

Results | Messages

| Member | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|
| 86 Simple bar_86 | Upper main girder | STEEL | 19.78 | 12.82 | 0.84 | 5 L2.Eq610b |
| 175 Simple bar_1 | Upper main girder | STEEL | 19.78 | 12.82 | 0.84 | 5 L2.Eq610b |
| 174 Simple bar_1 | Upper main girder | STEEL | 19.78 | 12.82 | 0.78 | 5 L2.Eq610b |
| 85 Simple bar_85 | Upper main girder | STEEL | 19.78 | 12.82 | 0.78 | 5 L2.Eq610b |
| 173 Simple bar_1 | Upper main girder | STEEL | 19.78 | 12.82 | 0.77 | 5 L2.Eq610b |
| 84 Simple bar_84 | Upper main girder | STEEL | 19.78 | 12.82 | 0.77 | 5 L2.Eq610b |
| 172 Simple bar_1 | Upper main girder | STEEL | 19.78 | 12.82 | 0.72 | 5 L2.Eq610b |
| 83 Simple bar_83 | Upper main girder | STEEL | 19.78 | 12.82 | 0.72 | 5 L2.Eq610b |
| 171 Simple bar_1 | Upper main girder | STEEL | 19.78 | 12.82 | 0.69 | 5 L2.Eq610b |
| 82 Simple bar_82 | Upper main girder | STEEL | 19.78 | 12.82 | 0.69 | 5 L2.Eq610b |
| 170 Simple bar_1 | Upper main girder | STEEL | 19.78 | 12.82 | 0.64 | 5 L2.Eq610b |
| 81 Simple bar_81 | Upper main girder | STEEL | 19.78 | 12.82 | 0.64 | 5 L2.Eq610b |
| 169 Simple bar_1 | Upper main girder | STEEL | 19.73 | 12.98 | 0.55 | 5 L2.Eq610b |
| 80 Simple bar_80 | Upper main girder | STEEL | 19.73 | 12.98 | 0.55 | 5 L2.Eq610b |
| 168 Simple bar_1 | Upper main girder | STEEL | 19.73 | 12.98 | 0.51 | 5 L2.Eq610b |
| 79 Simple bar_79 | Upper main girder | STEEL | 19.73 | 12.98 | 0.51 | 5 L2.Eq610b |
| 167 Simple bar_1 | Upper main girder | STEEL | 19.73 | 12.98 | 0.45 | 5 L2.Eq610b |
| 78 Simple bar_78 | Upper main girder | STEEL | 19.73 | 12.98 | 0.44 | 5 L2.Eq610b |
| 166 Simple bar_1 | Upper main girder | STEEL | 19.73 | 12.98 | 0.41 | 5 L2.Eq610b |
| 77 Simple bar_77 | Upper main girder | STEEL | 19.73 | 12.98 | 0.41 | 5 L2.Eq610b |
| 100 Simple bar_1 | Upper main girder | STEEL | 19.73 | 12.98 | 0.37 | 5 L2.Eq610b |
| 10 Simple bar_10 | Upper main girder | STEEL | 19.73 | 12.98 | 0.37 | 5 L2.Eq610b |
| 9 Simple bar_9 | Upper main girder | STEEL | 19.73 | 12.98 | 0.34 | 5 L2.Eq610b |
| 99 Simple bar_99 | Upper main girder | STEEL | 19.73 | 12.98 | 0.34 | 5 L2.Eq610b |
| 76 Simple bar_76 | Upper main girder | STEEL | 19.73 | 12.98 | 0.33 | 8 L4.Eq610b/at supp |
| 165 Simple bar_1 | Upper main girder | STEEL | 19.73 | 12.98 | 0.32 | 5 L2.Eq610b |
| 75 Simple bar_75 | Upper main girder | STEEL | 19.73 | 12.98 | 0.30 | 8 L4.Eq610b/at supp |
| 164 Simple bar_1 | Upper main girder | STEEL | 19.73 | 12.98 | 0.29 | 5 L2.Eq610b |

Calc. Note | Close | Help

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 Diagonals' utilization

The highest utilization ratio for the members is 0,81.
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.

[27:10:2017] : [14:15:06] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Diagonals

Results | Messages

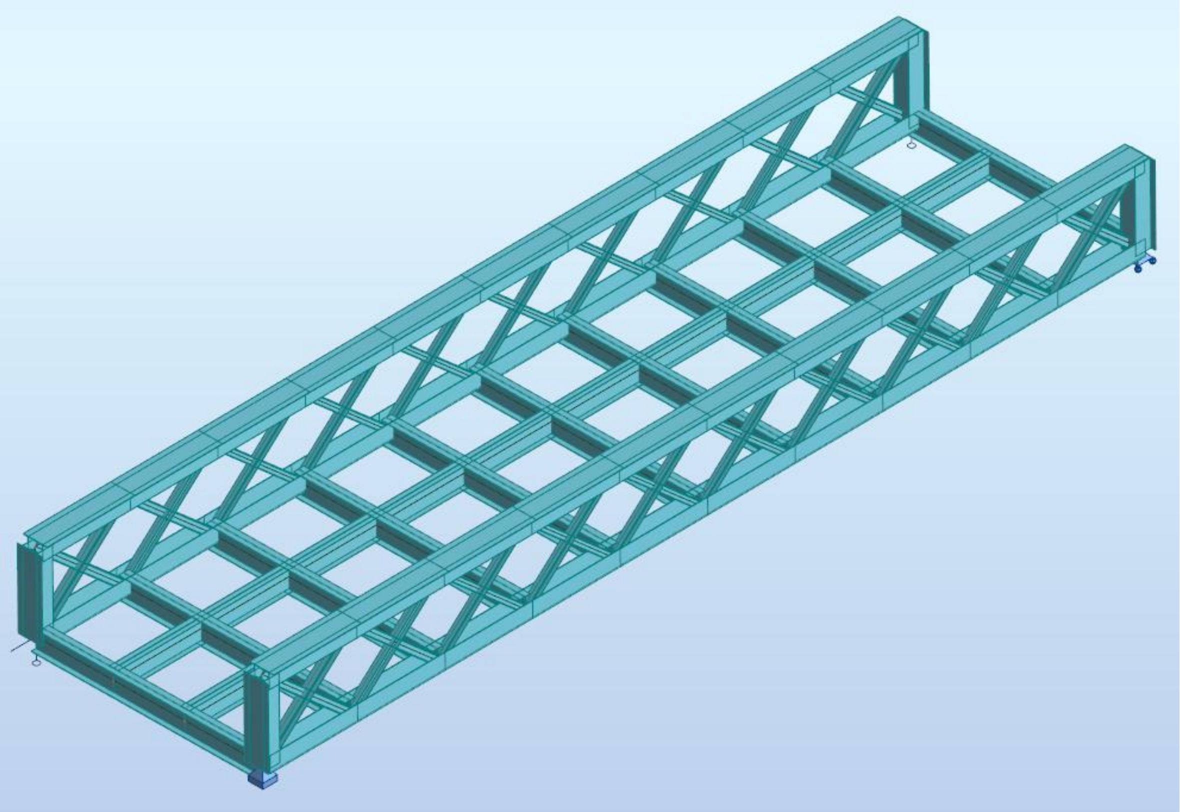
Calc. Note | Close | Help

| Member | | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|---|
| 16 Simple bar3_1 | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.81 | 5 L2.Eq610b |
| 106 Simple bar3_ | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.81 | 5 L2.Eq610b |
| 105 Simple bar3_ | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.75 | 5 L2.Eq610b |
| 15 Simple bar3_1 | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.75 | 5 L2.Eq610b |
| 116 Simple bar3_ | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.59 | 5 L2.Eq610b |
| 26 Simple bar3_2 | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.59 | 5 L2.Eq610b |
| 25 Simple bar3_2 | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.55 | 5 L2.Eq610b |
| 115 Simple bar3_ | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.55 | 5 L2.Eq610b |
| 125 Simple bar3_ | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.52 | 5 L2.Eq610b |
| 35 Simple bar3_3 | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.52 | 5 L2.Eq610b |
| 124 Simple bar3_ | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.49 | 5 L2.Eq610b |
| 34 Simple bar3_3 | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.48 | 5 L2.Eq610b |
| 133 Simple bar3_ | OK | Diagonals | STEEL | 6.85 | 11.92 | 0.39 | 5 L2.Eq610b |
| 44 Simple bar3_4 | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.36 | 5 L2.Eq610b |
| 134 Simple bar3_ | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.36 | 5 L2.Eq610b |
| 43 Simple bar3_4 | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.34 | 5 L2.Eq610b |
| 143 Simple bar3_ | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.27 | 5 L2.Eq610b |
| 53 Simple bar3_5 | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.27 | 5 L2.Eq610b |
| 142 Simple bar3_ | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.26 | 5 L2.Eq610b |
| 52 Simple bar3_5 | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.25 | 5 L2.Eq610b |
| 62 Simple bar3_6 | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.17 | 5 L2.Eq610b |
| 152 Simple bar3_ | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.17 | 5 L2.Eq610b |
| 61 Simple bar3_6 | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.16 | 5 L2.Eq610b |
| 151 Simple bar3_ | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.16 | 5 L2.Eq610b |
| 159 Simple bar3_ | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.01 | 8 L4.Eq610b/at supp |
| 160 Simple bar3_ | OK | Diagonals | STEEL | 7.03 | 11.83 | 0.01 | 8 L4.Eq610b/at supp |

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

Table 3, Utilization of diagonals in order of utilization ratio

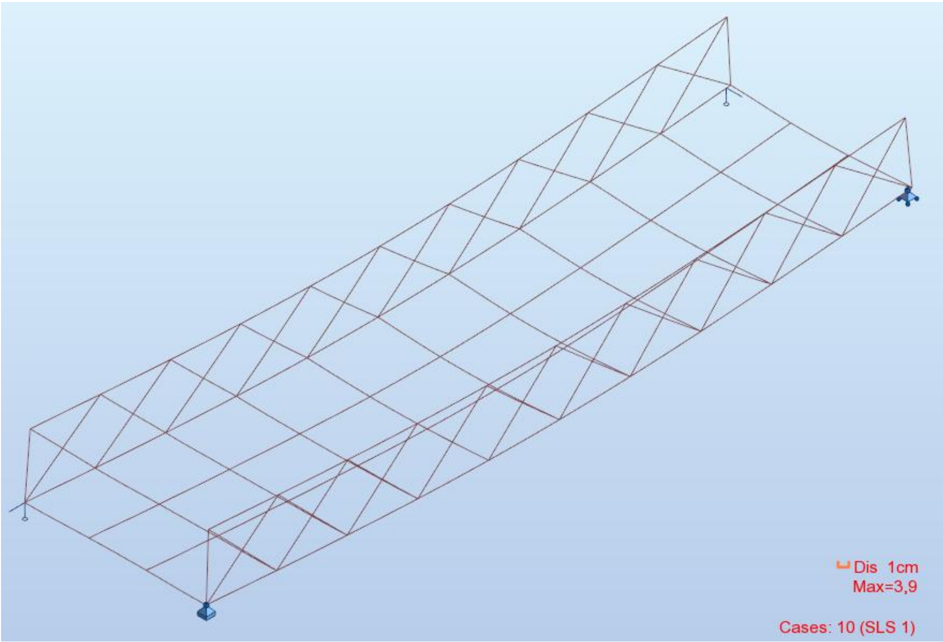
## 3.2 18,67 m span truss bridge



Picture 34, View of FEM model

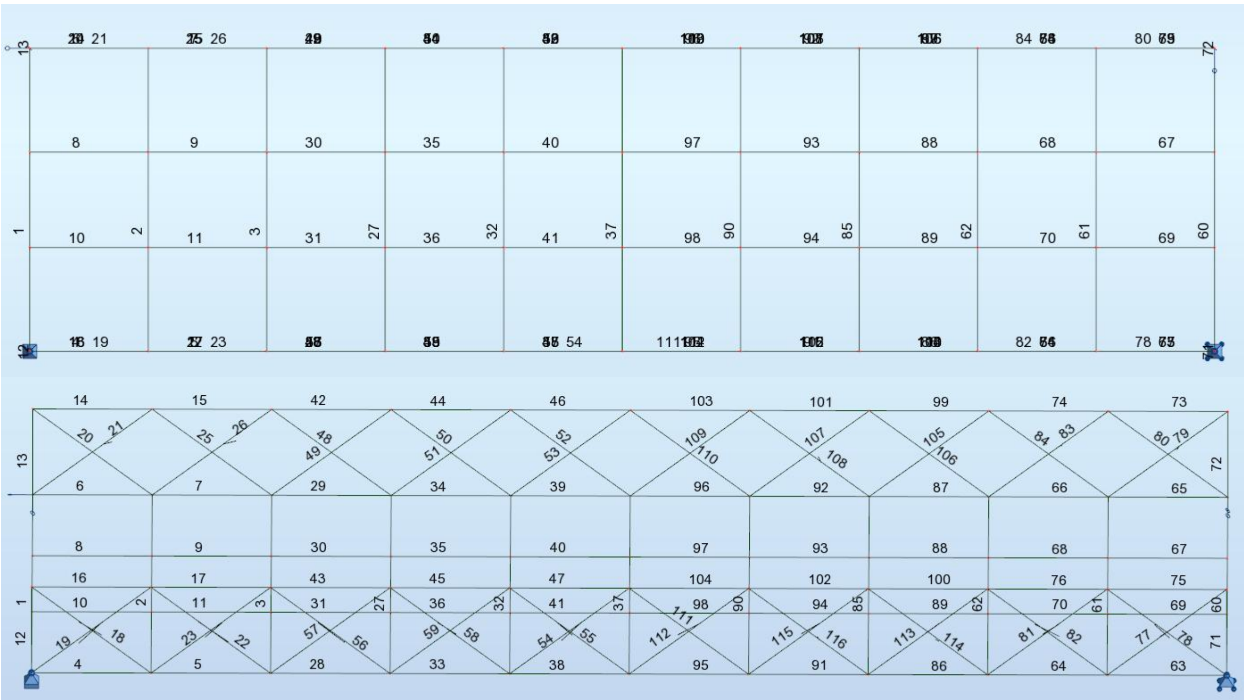
### 3.2.1 SLS results (Serviceability Limit State)

Total deflection of bridge is 3,9 cm = L/487




Picture 35, Deflection

### 3.2.2 ULS results (Ultimate Limit State)



Picture 36, member numbers for profiles that will be utilized – top view, side view

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

#### 3.2.2.1 Cross beams' utilization

The highest utilization ratio for the members is 0,88.
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.

[07:09:2017] : [14:44:39] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Cross beam

Results | Messages | Calc. Note | Close | Help

| Member | | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|---|
| 1 Simple bar_1 | OK | Cross beam | STEEL | 28.41 | 124.22 | 0.88 | 8 L4.Eq610b/at supp |
| 3 Simple bar_3 | OK | Cross beam | STEEL | 28.41 | 124.22 | 0.85 | 8 L4.Eq610b/at supp |
| 2 Simple bar_2 | OK | Cross beam | STEEL | 28.41 | 124.22 | 0.83 | 8 L4.Eq610b/at supp |
| 60 Simple bar_60 | OK | Cross beam | STEEL | 28.41 | 124.22 | 0.69 | 5 L2.Eq610b |
| 32 Simple bar_32 | OK | Cross beam | STEEL | 28.41 | 124.22 | 0.69 | 5 L2.Eq610b |
| 90 Simple bar_90 | OK | Cross beam | STEEL | 28.41 | 124.22 | 0.67 | 5 L2.Eq610b |
| 61 Simple bar_61 | OK | Cross beam | STEEL | 28.41 | 124.22 | 0.67 | 6 L3.Eq610b |
| 62 Simple bar_62 | OK | Cross beam | STEEL | 28.41 | 124.22 | 0.66 | 5 L2.Eq610b |
| 27 Simple bar_27 | OK | Cross beam | STEEL | 28.41 | 124.22 | 0.65 | 8 L4.Eq610b/at supp |
| 37 Simple bar_37 | OK | Cross beam | STEEL | 28.41 | 124.22 | 0.64 | 5 L2.Eq610b |
| 85 Simple bar_85 | OK | Cross beam | STEEL | 28.41 | 124.22 | 0.63 | 5 L2.Eq610b |

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

Table 4, Utilization of cross beams in order of utilization ratio

### 3.2.2.2 Longitudinal girders' utilization

The highest utilization ratio for the members is 0,94.
The critical load combination was L3: 1,25*Seflweight+1,50*Wind+1,45*0,8*TrafficLoad, and the traffic load's point loads were placed at midspan.

[07:09:2017] : [14:43:24] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Longitudinal girder

Results | Messages | Calc. Note | Close | Help

| Member | | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|---|
| 10 Simple bar_10 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.94 | 6 L3.Eq610b |
| 69 Simple bar_69 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.87 | 6 L3.Eq610b |
| 11 Simple bar_11 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.82 | 6 L3.Eq610b |
| 70 Simple bar_70 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.76 | 6 L3.Eq610b |
| 31 Simple bar_31 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.66 | 6 L3.Eq610b |
| 36 Simple bar_36 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.64 | 5 L2.Eq610b |
| 94 Simple bar_94 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.62 | 5 L2.Eq610b |
| 40 Simple bar_40 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.61 | 5 L2.Eq610b |
| 97 Simple bar_97 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.60 | 5 L2.Eq610b |
| 89 Simple bar_89 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.60 | 6 L3.Eq610b |
| 98 Simple bar_98 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.59 | 5 L2.Eq610b |
| 41 Simple bar_41 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.59 | 5 L2.Eq610b |
| 93 Simple bar_93 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.51 | 5 L2.Eq610b |
| 35 Simple bar_35 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.51 | 5 L2.Eq610b |
| 30 Simple bar_30 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.47 | 8 L4.Eq610b/at supp |
| 9 Simple bar_9 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.41 | 8 L4.Eq610b/at supp |
| 8 Simple bar_8 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.35 | 2 Wind |
| 67 Simple bar_67 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.30 | 2 Wind |
| 68 Simple bar_68 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.27 | 3 LM71-22,5 |
| 88 Simple bar_88 | OK | Longitudinal girder | STEEL | 11.43 | 57.03 | 0.24 | 8 L4.Eq610b/at supp |

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

Table 5, Utilization of longitudinal girders in order of utilization ratio

### 3.2.2.3 Trusses/diagonals' utilization

The highest utilization ratio for the members is 0,92.
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.

[07:09:2017] : [14:48:10] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Trusses

Results | Messages

Calc. Note | Close | Help

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

| Member | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|
| 84 Simple bar_84 | diagonals2 | STEEL | 40.08 | 83.78 | 0.92 | 5 L2.Eq610b |
| 26 Simple bar_26 | diagonals2 | STEEL | 40.08 | 83.78 | 0.92 | 5 L2.Eq610b |
| 23 Simple bar_23 | diagonals2 | STEEL | 40.08 | 83.78 | 0.88 | 5 L2.Eq610b |
| 82 Simple bar_82 | diagonals2 | STEEL | 40.08 | 83.78 | 0.88 | 5 L2.Eq610b |
| 106 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.86 | 5 L2.Eq610b |
| 49 Simple bar_49 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.86 | 5 L2.Eq610b |
| 114 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.83 | 5 L2.Eq610b |
| 57 Simple bar_57 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.83 | 5 L2.Eq610b |
| 80 Simple bar_80 | Diagonals1 | STEEL | 39.35 | 68.04 | 0.79 | 5 L2.Eq610b |
| 21 Simple bar_21 | Diagonals1 | STEEL | 39.35 | 68.04 | 0.79 | 5 L2.Eq610b |
| 78 Simple bar_78 | Diagonals1 | STEEL | 39.35 | 68.04 | 0.75 | 5 L2.Eq610b |
| 19 Simple bar_19 | Diagonals1 | STEEL | 39.35 | 68.04 | 0.75 | 5 L2.Eq610b |
| 25 Simple bar_25 | diagonals2 | STEEL | 40.08 | 83.78 | 0.63 | 5 L2.Eq610b |
| 83 Simple bar_83 | diagonals2 | STEEL | 40.08 | 83.78 | 0.63 | 5 L2.Eq610b |
| 105 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.62 | 5 L2.Eq610b |
| 48 Simple bar_48 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.62 | 5 L2.Eq610b |
| 81 Simple bar_81 | diagonals2 | STEEL | 40.08 | 83.78 | 0.59 | 5 L2.Eq610b |
| 22 Simple bar_22 | diagonals2 | STEEL | 40.08 | 83.78 | 0.59 | 5 L2.Eq610b |
| 56 Simple bar_56 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.59 | 5 L2.Eq610b |
| 113 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.59 | 5 L2.Eq610b |
| 20 Simple bar_20 | Diagonals1 | STEEL | 39.35 | 68.04 | 0.58 | 8 L4.Eq610b/at supp |
| 18 Simple bar_18 | Diagonals1 | STEEL | 39.35 | 68.04 | 0.55 | 8 L4.Eq610b/at supp |
| 116 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.55 | 5 L2.Eq610b |
| 59 Simple bar_59 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.55 | 5 L2.Eq610b |
| 79 Simple bar_79 | Diagonals1 | STEEL | 39.35 | 68.04 | 0.54 | 5 L2.Eq610b |
| 108 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.54 | 5 L2.Eq610b |
| 51 Simple bar_51 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.54 | 5 L2.Eq610b |
| 77 Simple bar_77 | Diagonals1 | STEEL | 39.35 | 68.04 | 0.51 | 5 L2.Eq610b |
| 107 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.48 | 5 L2.Eq610b |
| 50 Simple bar_50 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.48 | 5 L2.Eq610b |
| 58 Simple bar_58 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.46 | 5 L2.Eq610b |
| 115 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.46 | 5 L2.Eq610b |
| 109 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.26 | 5 L2.Eq610b |
| 52 Simple bar_52 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.26 | 5 L2.Eq610b |
| 55 Simple bar_55 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.25 | 5 L2.Eq610b |
| 112 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.25 | 5 L2.Eq610b |
| 110 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.19 | 8 L4.Eq610b/at supp |
| 111 Simple bar_1 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.15 | 8 L4.Eq610b/at supp |
| 54 Simple bar_54 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.15 | 5 L2.Eq610b |
| 53 Simple bar_53 | Diagonals3 | STEEL | 47.27 | 81.42 | 0.08 | 5 L2.Eq610b |

Table 6, Utilization of trusses in order of utilization ratio

### 3.2.2.4 Lower main girders' utilization

The highest utilization ratio for the members is 0,77.
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.

[07:09:2017] : [14:49:10] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Lower main girder

Results | Messages

| Member | | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|---|
| 96 Simple bar_96 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.77 | 5 L2.Eq610b |
| 39 Simple bar_39 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.77 | 5 L2.Eq610b |
| 38 Simple bar_38 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.70 | 5 L2.Eq610b |
| 95 Simple bar_95 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.70 | 5 L2.Eq610b |
| 92 Simple bar_92 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.69 | 5 L2.Eq610b |
| 34 Simple bar_34 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.69 | 5 L2.Eq610b |
| 33 Simple bar_33 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.63 | 5 L2.Eq610b |
| 91 Simple bar_91 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.63 | 5 L2.Eq610b |
| 87 Simple bar_87 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.56 | 5 L2.Eq610b |
| 29 Simple bar_29 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.56 | 5 L2.Eq610b |
| 28 Simple bar_28 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.51 | 5 L2.Eq610b |
| 86 Simple bar_86 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.51 | 5 L2.Eq610b |
| 66 Simple bar_66 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.36 | 5 L2.Eq610b |
| 7 Simple bar_7 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.36 | 5 L2.Eq610b |
| 4 Simple bar_4 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.33 | 8 L4.Eq610b/at supp |
| 5 Simple bar_5 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.33 | 5 L2.Eq610b |
| 64 Simple bar_64 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.32 | 5 L2.Eq610b |
| 63 Simple bar_63 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.30 | 5 L2.Eq610b |
| 6 Simple bar_6 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.28 | 8 L4.Eq610b/at supp |
| 65 Simple bar_65 | OK | Lower main gi | STEEL | 14.64 | 18.05 | 0.25 | 5 L2.Eq610b |

Calc. Note | Close | Help

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

Table 7, Utilization of lower main girder in order of utilization ratio

#### 3.2.2.5 Upper main girders' utilization

The highest utilization ratio for the members is 0,92.
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.

[07:09:2017] : [14:47:08] SFS-EN 1993-1:2005/NA:2007/A1:2014 - Upper main girder

Results | Messages

| Member | | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|---|
| 103 Simple bar_1 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.92 | 5 L2.Eq610b |
| 46 Simple bar_46 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.92 | 5 L2.Eq610b |
| 104 Simple bar_1 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.91 | 5 L2.Eq610b |
| 47 Simple bar_47 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.91 | 5 L2.Eq610b |
| 101 Simple bar_1 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.84 | 5 L2.Eq610b |
| 44 Simple bar_44 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.84 | 5 L2.Eq610b |
| 102 Simple bar_1 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.83 | 5 L2.Eq610b |
| 45 Simple bar_45 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.83 | 5 L2.Eq610b |
| 99 Simple bar_99 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.69 | 5 L2.Eq610b |
| 42 Simple bar_42 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.69 | 5 L2.Eq610b |
| 100 Simple bar_1 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.67 | 5 L2.Eq610b |
| 43 Simple bar_43 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.67 | 5 L2.Eq610b |
| 15 Simple bar_15 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.47 | 8 L4.Eq610b/at supp |
| 74 Simple bar_74 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.46 | 5 L2.Eq610b |
| 17 Simple bar_17 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.45 | 8 L4.Eq610b/at supp |
| 76 Simple bar_76 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.43 | 5 L2.Eq610b |
| 73 Simple bar_73 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.37 | 5 L2.Eq610b |
| 14 Simple bar_14 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.37 | 5 L2.Eq610b |
| 16 Simple bar_16 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.36 | 8 L4.Eq610b/at supp |
| 75 Simple bar_75 | OK | Upper main gir | STEEL | 15.66 | 12.86 | 0.35 | 5 L2.Eq610b |

Calc. Note | Close | Help

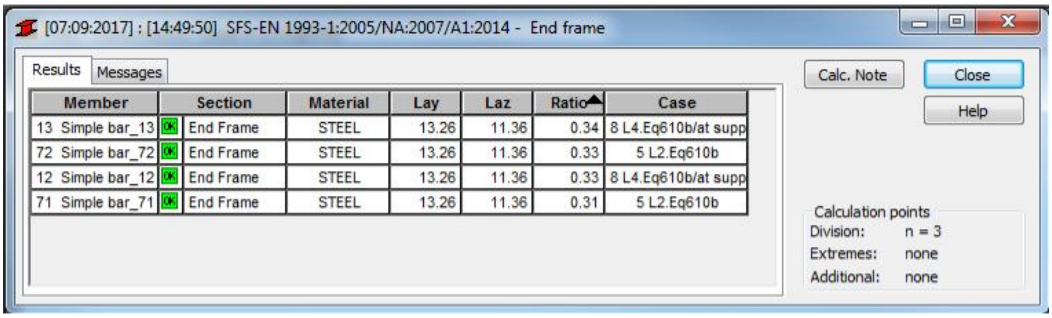
Calculation points
Division: n = 3
Extremes: none
Additional: none

Table 8, Utilization of upper main girder in order of utilization ratio

#### 3.2.2.6 End columns' utilization

The highest utilization ratio for the members is 0,34.

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.


[07:09:2017] : [14:49:50] SFS-EN 1993-1:2005/NA:2007/A1:2014 - End frame

Results | Messages

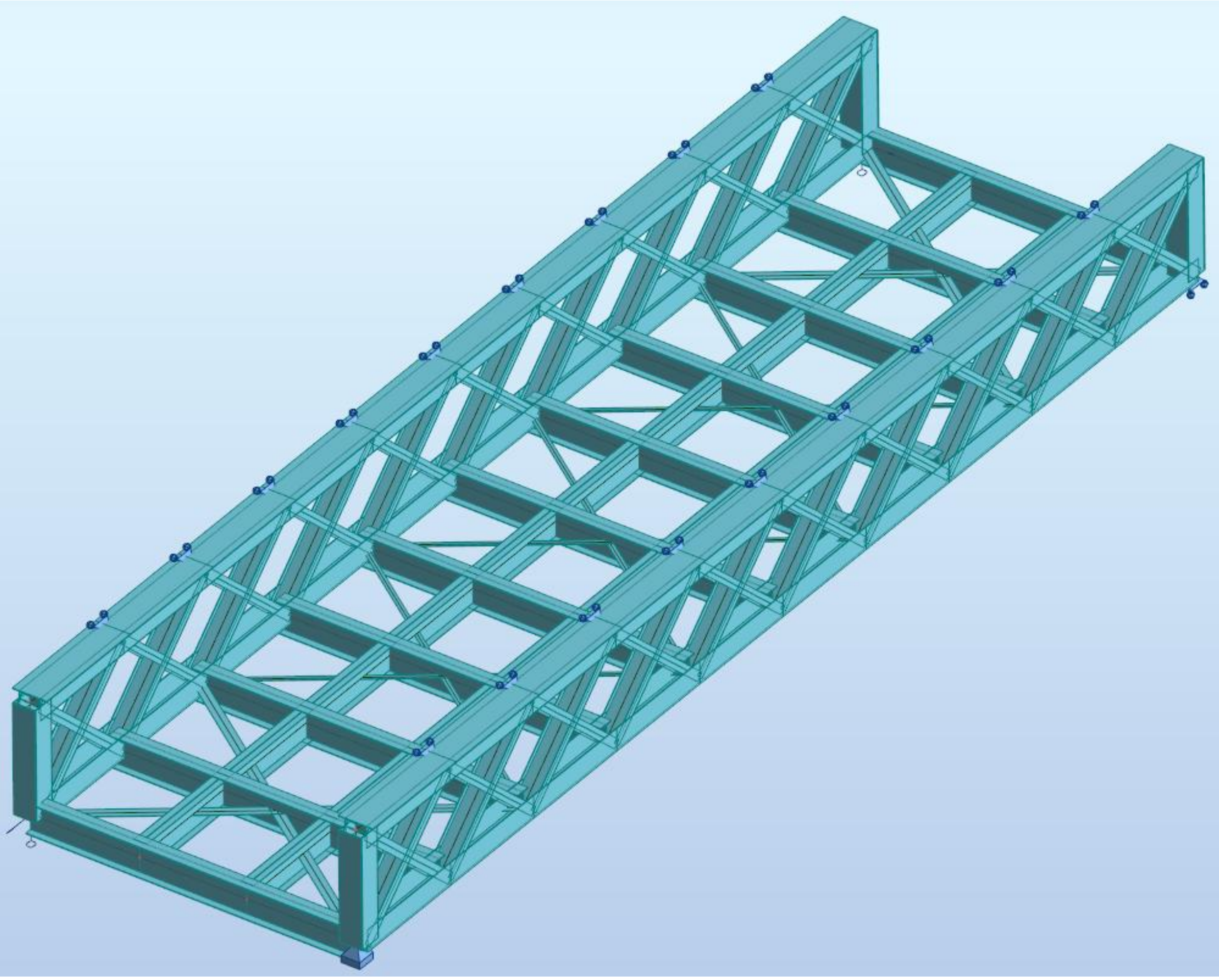
| Member | | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|---|
| 13 Simple bar_13 | OK | End Frame | STEEL | 13.26 | 11.36 | 0.34 | 8 L4.Eq610b/at supp |
| 72 Simple bar_72 | OK | End Frame | STEEL | 13.26 | 11.36 | 0.33 | 5 L2.Eq610b |
| 12 Simple bar_12 | OK | End Frame | STEEL | 13.26 | 11.36 | 0.33 | 8 L4.Eq610b/at supp |
| 71 Simple bar_71 | OK | End Frame | STEEL | 13.26 | 11.36 | 0.31 | 5 L2.Eq610b |

Calc. Note
Close
Help

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

Table 9, Utilization of end frame in order of utilization ratio

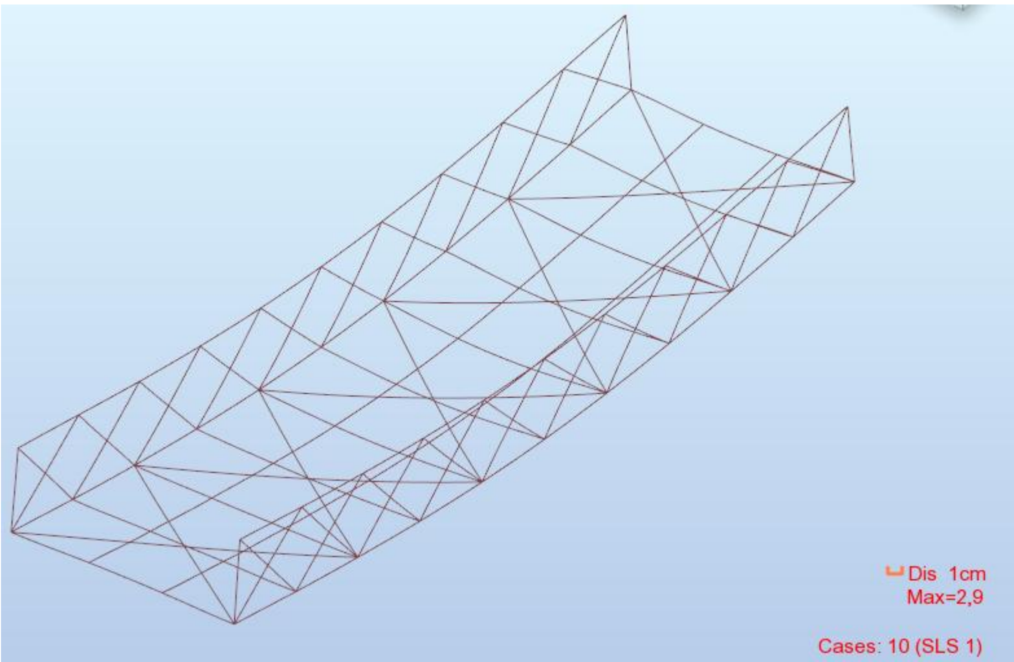
## 3.3 17 m span truss bridge



Picture 37, View of FEM model

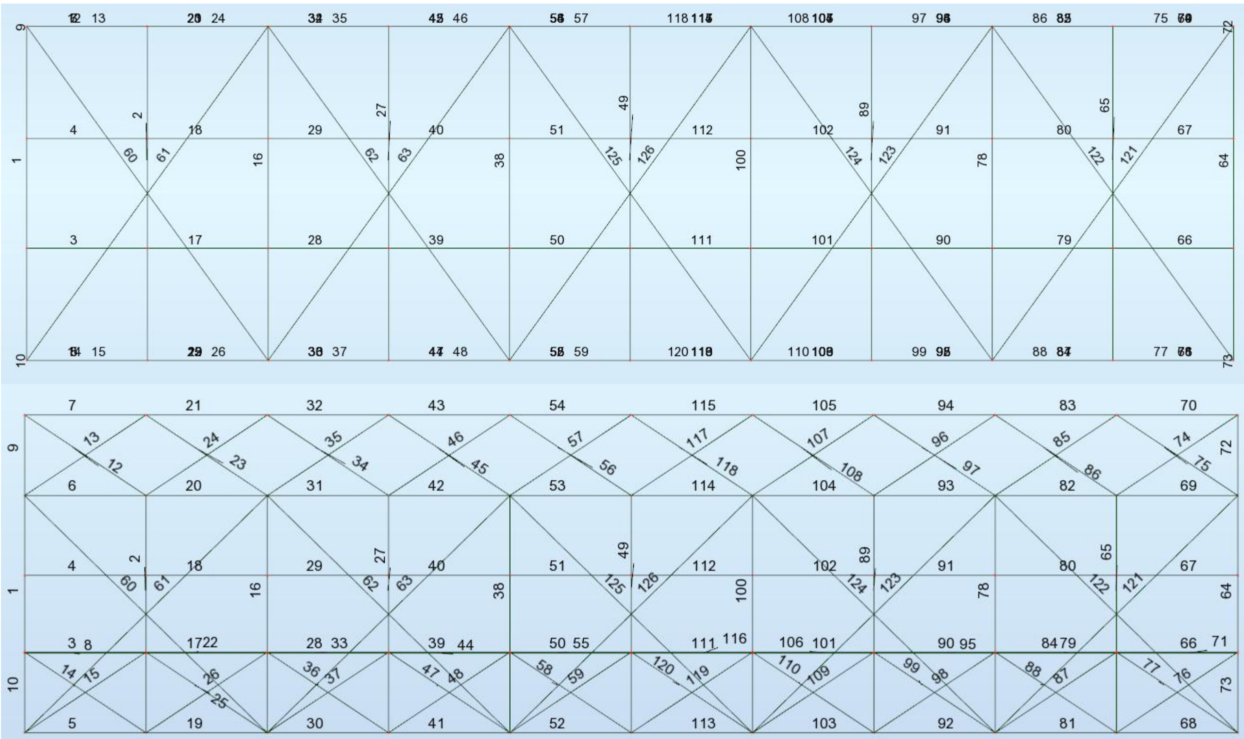
### 3.3.1 SLS results (Serviceability Limit State)

Total deflection of bridge is 2,9 cm = L/586



Picture 38, Deflection

### 3.3.2 ULS results (Ultimate Limit State)


Picture 39, member numbers for profiles that will be utilized – top view, side view

#### 3.3.2.1 Cross beams' utilization

The highest utilization ratio for the members is 0,90.
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.

SFS-EN 1993-1:2005/NA:2007/A1:2014 - Member Verification ( ULS ) 1 2 16to49By11 64 65 78 89 100

| Member | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|
| 49 Beam_49 | OK Cross beam | STEEL | 28.16 | 122.59 | 0.90 | 5 L2.Eq610b |
| 16 Beam_16 | OK Cross beam | STEEL | 28.16 | 122.59 | 0.88 | 8 L4.Eq610b/at supp |
| 100 Beam_100 | OK Cross beam | STEEL | 28.16 | 122.59 | 0.87 | 5 L2.Eq610b |
| 38 Beam_38 | OK Cross beam | STEEL | 28.16 | 122.59 | 0.85 | 5 L2.Eq610b |
| 2 Beam_2 | OK Cross beam | STEEL | 28.16 | 122.59 | 0.84 | 8 L4.Eq610b/at supp |
| 1 Column_1 | OK Cross beam | STEEL | 28.16 | 122.59 | 0.79 | 8 L4.Eq610b/at supp |
| 27 Beam_27 | OK Cross beam | STEEL | 28.16 | 122.59 | 0.75 | 8 L4.Eq610b/at supp |
| 89 Beam_89 | OK Cross beam | STEEL | 28.16 | 122.59 | 0.69 | 5 L2.Eq610b |
| 78 Beam_78 | OK Cross beam | STEEL | 28.16 | 122.59 | 0.65 | 5 L2.Eq610b |
| 64 Beam_64 | OK Cross beam | STEEL | 28.16 | 122.59 | 0.45 | 5 L2.Eq610b |
| 65 Beam_65 | OK Cross beam | STEEL | 28.16 | 122.59 | 0.44 | 8 L4.Eq610b/at supp |

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

Table 10, Utilization of cross beams in order of utilization ratio

#### 3.3.2.2 Longitudinal girders' utilization

The highest utilization ratio for the members is 0,54.
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.

SFS-EN 1993-1:2005/NA:2007/A1:2014 - Member Verification ( ULS ) 3 4 17to50By11 18to51By11 66 67 79to112By...

| Member | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|
| 112 Simple bar_1 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.54 | 5 L2.Eq610b |
| 111 Simple bar_1 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.52 | 5 L2.Eq610b |
| 51 Simple bar_51 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.52 | 5 L2.Eq610b |
| 102 Simple bar_1 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.51 | 5 L2.Eq610b |
| 17 Simple bar_17 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.50 | 8 L4.Eq610b/at supp |
| 3 Simple bar_3 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.50 | 5 L2.Eq610b |
| 50 Simple bar_50 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.50 | 5 L2.Eq610b |
| 101 Simple bar_1 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.50 | 5 L2.Eq610b |
| 28 Simple bar_28 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.50 | 8 L4.Eq610b/at supp |
| 40 Simple bar_40 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.49 | 5 L2.Eq610b |
| 39 Simple bar_39 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.49 | 5 L2.Eq610b |
| 66 Simple bar_66 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.49 | 5 L2.Eq610b |
| 29 Simple bar_29 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.48 | 8 L4.Eq610b/at supp |
| 18 Simple bar_18 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.48 | 8 L4.Eq610b/at supp |
| 4 Simple bar_4 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.45 | 8 L4.Eq610b/at supp |
| 67 Simple bar_67 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.45 | 5 L2.Eq610b |
| 79 Simple bar_79 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.44 | 5 L2.Eq610b |
| 80 Simple bar_80 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.43 | 5 L2.Eq610b |
| 90 Simple bar_90 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.33 | 5 L2.Eq610b |
| 91 Simple bar_91 | OK Longitudinal gi | STEEL | 10.75 | 52.45 | 0.31 | 8 L4.Eq610b/at supp |

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

Table 11, Utilization of longitudinal girders in order of utilization ratio

#### 3.3.2.3 Trusses/diagonals' utilization

The highest utilization ratio for the members is 0,93.
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.

SFS-EN 1993-1:2005/NA:2007/A1:2014 - Member Verification ( ULS ) 12to15 23to26 34to37 45to48 56to59 74to77...

Results | Messages

| Member | | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|---|
| 24 Simple bar_24 | OK | Diagonals2 | STEEL | 18.91 | 86.67 | 0.93 | 5 L2.Eq610b |
| 86 Simple bar_86 | OK | Diagonals2 | STEEL | 18.91 | 86.67 | 0.93 | 5 L2.Eq610b |
| 88 Simple bar_88 | OK | Diagonals2 | STEEL | 18.91 | 86.67 | 0.86 | 5 L2.Eq610b |
| 13 Simple bar_13 | OK | Diagonals1 | STEEL | 17.86 | 65.66 | 0.85 | 8 L4.Eq610b/at supp |
| 26 Simple bar_26 | OK | Diagonals2 | STEEL | 18.91 | 86.67 | 0.85 | 5 L2.Eq610b |
| 15 Simple bar_15 | OK | Diagonals1 | STEEL | 17.86 | 65.66 | 0.82 | 8 L4.Eq610b/at supp |
| 99 Simple bar_99 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.79 | 5 L2.Eq610b |
| 37 Simple bar_37 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.78 | 5 L2.Eq610b |
| 35 Simple bar_35 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.76 | 5 L2.Eq610b |
| 75 Simple bar_75 | OK | Diagonals1 | STEEL | 17.86 | 65.66 | 0.76 | 5 L2.Eq610b |
| 97 Simple bar_97 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.76 | 5 L2.Eq610b |
| 77 Simple bar_77 | OK | Diagonals1 | STEEL | 17.86 | 65.66 | 0.75 | 5 L2.Eq610b |
| 110 Simple bar_1 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.64 | 5 L2.Eq610b |
| 48 Simple bar_48 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.64 | 5 L2.Eq610b |
| 25 Simple bar_25 | OK | Diagonals2 | STEEL | 18.91 | 86.67 | 0.59 | 8 L4.Eq610b/at supp |
| 46 Simple bar_46 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.59 | 5 L2.Eq610b |
| 108 Simple bar_1 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.59 | 5 L2.Eq610b |
| 23 Simple bar_23 | OK | Diagonals2 | STEEL | 18.91 | 86.67 | 0.54 | 8 L4.Eq610b/at supp |
| 12 Simple bar_12 | OK | Diagonals1 | STEEL | 17.86 | 65.66 | 0.50 | 8 L4.Eq610b/at supp |
| 85 Simple bar_85 | OK | Diagonals2 | STEEL | 18.91 | 86.67 | 0.49 | 5 L2.Eq610b |
| 14 Simple bar_14 | OK | Diagonals1 | STEEL | 17.86 | 65.66 | 0.48 | 8 L4.Eq610b/at supp |
| 87 Simple bar_87 | OK | Diagonals2 | STEEL | 18.91 | 86.67 | 0.47 | 5 L2.Eq610b |
| 74 Simple bar_74 | OK | Diagonals1 | STEEL | 17.86 | 65.66 | 0.47 | 5 L2.Eq610b |
| 120 Simple bar_1 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.45 | 5 L2.Eq610b |
| 76 Simple bar_76 | OK | Diagonals1 | STEEL | 17.86 | 65.66 | 0.45 | 5 L2.Eq610b |
| 59 Simple bar_59 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.44 | 5 L2.Eq610b |
| 96 Simple bar_96 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.43 | 5 L2.Eq610b |
| 34 Simple bar_34 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.42 | 5 L2.Eq610b |
| 98 Simple bar_98 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.41 | 5 L2.Eq610b |
| 36 Simple bar_36 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.41 | 5 L2.Eq610b |
| 118 Simple bar_1 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.38 | 5 L2.Eq610b |
| 57 Simple bar_57 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.37 | 5 L2.Eq610b |
| 109 Simple bar_1 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.35 | 5 L2.Eq610b |
| 47 Simple bar_47 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.33 | 5 L2.Eq610b |
| 107 Simple bar_1 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.32 | 5 L2.Eq610b |
| 45 Simple bar_45 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.32 | 5 L2.Eq610b |
| 119 Simple bar_1 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.25 | 5 L2.Eq610b |
| 58 Simple bar_58 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.25 | 5 L2.Eq610b |
| 56 Simple bar_56 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.17 | 5 L2.Eq610b |
| 117 Simple bar_1 | OK | Diagonals3 | STEEL | 18.19 | 81.04 | 0.17 | 5 L2.Eq610b |

Calc. Note | Close | Help

Ratio: Analysis | Map

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

Table 12, Utilization of trusses in order of utilization ratio

#### 3.3.2.4 Lower main girders' 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.

SFS-EN 1993-1:2005/NA:2007/A1:2014 - Member Verification ( ULS ) 5 6 19to52By11 20to53By11 68 69 81to114B...

Results | Messages

| Member | | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|---|
| 114 Simple bar_1 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.66 | 5 L2.Eq610b |
| 53 Simple bar_53 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.66 | 5 L2.Eq610b |
| 113 Simple bar_1 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.64 | 5 L2.Eq610b |
| 52 Simple bar_52 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.64 | 5 L2.Eq610b |
| 104 Simple bar_1 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.60 | 5 L2.Eq610b |
| 42 Simple bar_42 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.60 | 5 L2.Eq610b |
| 103 Simple bar_1 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.58 | 5 L2.Eq610b |
| 41 Simple bar_41 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.58 | 5 L2.Eq610b |
| 93 Simple bar_93 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.48 | 5 L2.Eq610b |
| 31 Simple bar_31 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.48 | 5 L2.Eq610b |
| 92 Simple bar_92 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.46 | 5 L2.Eq610b |
| 30 Simple bar_30 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.46 | 5 L2.Eq610b |
| 6 Simple bar_6 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.35 | 8 L4.Eq610b/at supp |
| 5 Simple bar_5 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.35 | 8 L4.Eq610b/at supp |
| 82 Simple bar_82 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.31 | 5 L2.Eq610b |
| 20 Simple bar_20 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.30 | 5 L2.Eq610b |
| 19 Simple bar_19 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.30 | 5 L2.Eq610b |
| 81 Simple bar_81 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.29 | 5 L2.Eq610b |
| 69 Simple bar_69 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.28 | 5 L2.Eq610b |
| 68 Simple bar_68 | OK | Lower main gi | STEEL | 13.54 | 16.70 | 0.28 | 5 L2.Eq610b |

Calc. Note | Close | Help

Ratio: Analysis | Map

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

Table 13, Utilization of lower main girder in order of utilization ratio

### 3.3.2.5 Upper main girders' utilization

The highest utilization ratio for the members is 0,79.
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.

SFS-EN 1993-1:2005/NA:2007/A1:2014 - Member Verification ( ULS ) 7 8 21to54By11 22to55By11 70 71 83to116By...

Results | Messages

| Member | | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|---|
| 54 Simple bar_54 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.79 | 5 L2.Eq610b |
| 115 Simple bar_1 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.79 | 5 L2.Eq610b |
| 55 Simple bar_55 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.76 | 5 L2.Eq610b |
| 116 Simple bar_1 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.76 | 5 L2.Eq610b |
| 105 Simple bar_1 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.72 | 5 L2.Eq610b |
| 43 Simple bar_43 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.72 | 5 L2.Eq610b |
| 106 Simple bar_1 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.70 | 5 L2.Eq610b |
| 44 Simple bar_44 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.70 | 5 L2.Eq610b |
| 21 Simple bar_21 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.64 | 8 L4.Eq610b/at supp |
| 22 Simple bar_22 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.60 | 8 L4.Eq610b/at supp |
| 94 Simple bar_94 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.60 | 5 L2.Eq610b |
| 32 Simple bar_32 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.60 | 5 L2.Eq610b |
| 95 Simple bar_95 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.58 | 5 L2.Eq610b |
| 33 Simple bar_33 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.58 | 5 L2.Eq610b |
| 7 Simple bar_7 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.52 | 8 L4.Eq610b/at supp |
| 8 Simple bar_8 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.50 | 8 L4.Eq610b/at supp |
| 83 Simple bar_83 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.45 | 5 L2.Eq610b |
| 84 Simple bar_84 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.42 | 5 L2.Eq610b |
| 70 Simple bar_70 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.38 | 5 L2.Eq610b |
| 71 Simple bar_71 | OK | Upper main gir | STEEL | 14.49 | 11.90 | 0.37 | 5 L2.Eq610b |

Calc. Note | Close | Help

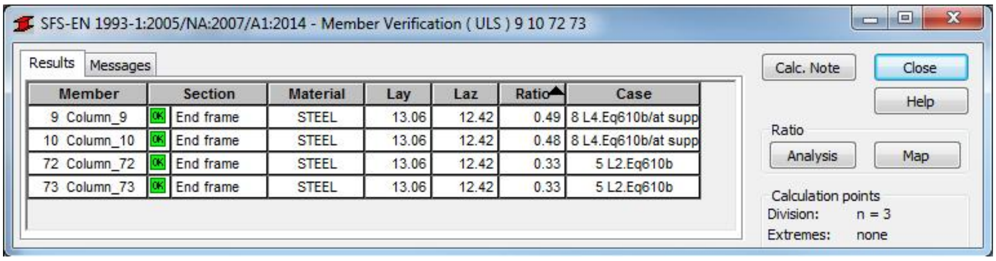
Ratio: Analysis | Map

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

Table 14, Utilization of upper main girder in order of utilization ratio

### 3.3.2.6 End columns' utilization

The highest utilization ratio for the members is 0,49.
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.

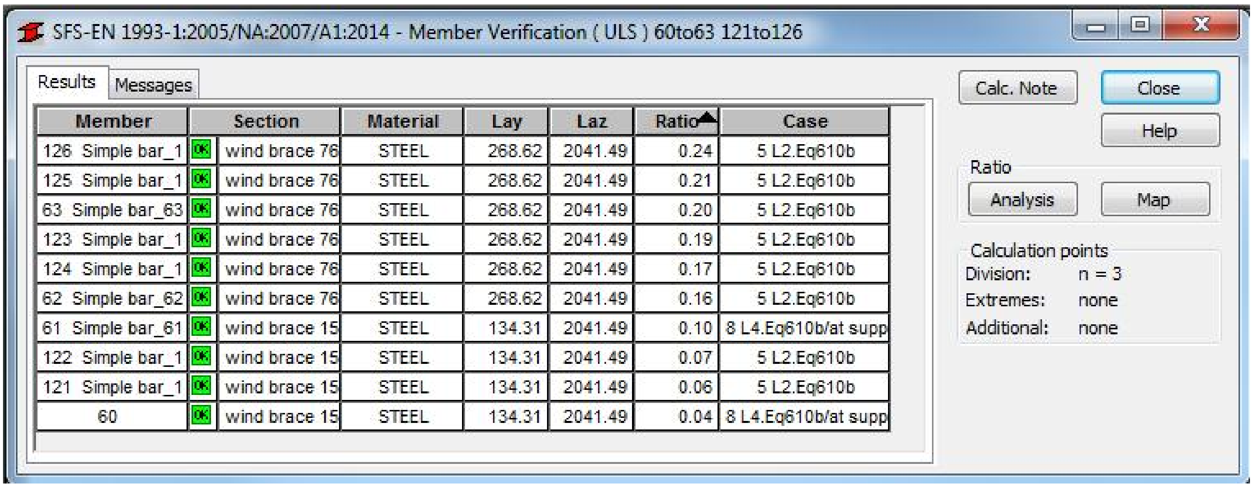



| Member | | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|---|
| 9 Column_9 | OK | End frame | STEEL | 13.06 | 12.42 | 0.49 | 8 L4.Eq610b/at supp |
| 10 Column_10 | OK | End frame | STEEL | 13.06 | 12.42 | 0.48 | 8 L4.Eq610b/at supp |
| 72 Column_72 | OK | End frame | STEEL | 13.06 | 12.42 | 0.33 | 5 L2.Eq610b |
| 73 Column_73 | OK | End frame | STEEL | 13.06 | 12.42 | 0.33 | 5 L2.Eq610b |

Table 15, Utilization of end frame in order of utilization ratio

#### 3.3.2.7 Wind bracings' utilization

The highest utilization ratio for the members is 0,24.
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.




| Member | | Section | Material | Lay | Laz | Ratio | Case |
|---|---|---|---|---|---|---|---|
| 126 Simple bar_1 | OK | wind brace 76 | STEEL | 268.62 | 2041.49 | 0.24 | 5 L2.Eq610b |
| 125 Simple bar_1 | OK | wind brace 76 | STEEL | 268.62 | 2041.49 | 0.21 | 5 L2.Eq610b |
| 63 Simple bar_63 | OK | wind brace 76 | STEEL | 268.62 | 2041.49 | 0.20 | 5 L2.Eq610b |
| 123 Simple bar_1 | OK | wind brace 76 | STEEL | 268.62 | 2041.49 | 0.19 | 5 L2.Eq610b |
| 124 Simple bar_1 | OK | wind brace 76 | STEEL | 268.62 | 2041.49 | 0.17 | 5 L2.Eq610b |
| 62 Simple bar_62 | OK | wind brace 76 | STEEL | 268.62 | 2041.49 | 0.16 | 5 L2.Eq610b |
| 61 Simple bar_61 | OK | wind brace 15 | STEEL | 134.31 | 2041.49 | 0.10 | 8 L4.Eq610b/at supp |
| 122 Simple bar_1 | OK | wind brace 15 | STEEL | 134.31 | 2041.49 | 0.07 | 5 L2.Eq610b |
| 121 Simple bar_1 | OK | wind brace 15 | STEEL | 134.31 | 2041.49 | 0.06 | 5 L2.Eq610b |
| 60 | OK | wind brace 15 | STEEL | 134.31 | 2041.49 | 0.04 | 8 L4.Eq610b/at supp |

Table 16, Utilization of wind bracing 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 37-m span bridge. The result shows (Appendix 3), 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 ($V_{Ed}/V_{Rd}$) = 1.284 > 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.*

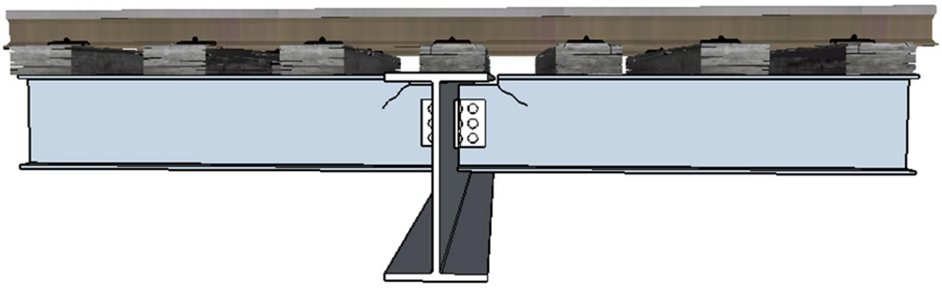
*Typically Fatigue Susceptible Details:*

- *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

17 and 18 m spans

For the 17 and 18m bridges the deflection limit L/600 (EN 1990-1, A2.4.4.2.3 (1), [1]) is exceeded. Also, the tension and stresses are very high, utilization rate is over 0,9.

It is recommended to replace those structures for the whole span. In 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.

37 m spans

Truss bridge calculations show that utilizations are feasible and within limits, but deflection is at the limit. It is recommended renewing cross girders and rail bearers. The new structural parts are going to be constructed with pre-cambering, to prevent deflection from dead load and strengthen the bridge.

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

APPENDIX 1: 17m girder bridge calculation
APPENDIX 2: 18m girder bridge calculation
APPENDIX 3: 37m span bridge connection calculation

# 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: $H := 570mm$

$t_w := 10mm$

$b_f := 250mm$

$t_f := 20mm$

$h_w := H - 2 \cdot t_f = 530 \cdot mm$

$A_{cross} := h_w \cdot t_w + 2b_f \cdot t_f = 15300 \cdot mm^2$

cross girder length: $l_{cross} := 4.775m$

### Dynamic factor

For track with standard maintenance:

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

Ref. EN1991-2 §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:<br>- as an element of a continuous grillage<br>- simply supported | <br>3 times cross girder spacing<br><br>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 $\Phi_3$

Ref. EN1991-2 §6.4.5.3 Table 6.2

Cross girder length: $l_{cross} = 4.775\,m$

*Dinamic factor for cross girder:*

$$L_{\phi.cross} := 2 \cdot l_{cross} = 9.55\,m$$

$$\phi_{cross} := \frac{2.16}{\sqrt{\frac{L_{\phi.cross}}{m}} - 0.2} + 0.73 = 1.477$$

Loads:

Self weight:

$$G_{cross} := A_{cross} \cdot \rho \cdot g = 1177 \cdot \frac{N}{m}$$

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 by rails and longitudinal girders to the cross girder....

distance between cross girders: $L_{cross} := 1727mm$

from distributed load - q.v: $F_{traffic.q} := q_v \cdot L_{cross} = 102 \cdot kN$

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

0 < $L_{cross} - n \cdot 1.6m = 0.127\,m$ < 1.6 m

from point load - Q.v: $F_{traffic.Q} := (n + 1) \cdot Q_v = 369 \cdot kN$

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)

$F_{traffic} := \max\left(F_{traffic.q}, F_{traffic.Q}\right) = 369.331 \cdot kN$

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 $\gamma_G \cdot G_{cross} = 1.589 \cdot \frac{kN}{m}$ distributed load along girder axis

$\gamma_G \cdot \left(G_{rail} \cdot L_{cross}\right) = 2.331 \cdot kN$ concentrated load at 'rail position'

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

L2.Eq610.b concentrated load at 'rail position'

$$F_1 := \gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot F_{traffic} + \gamma_{wind} \cdot \psi_{0i} \cdot (f_w \cdot L_{cross}) = 554 \cdot kN$$

distributed load along girder axis

$$f := \gamma_G \cdot G_{cross} = 1.471 \cdot \frac{kN}{m}$$

concentrated load at 'rail position'

$$F_2 := \gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot F_{traffic} - \gamma_{wind} \cdot \psi_{0i} \cdot (f_w \cdot L_{cross}) = 522 \cdot kN$$

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

L3.Eq610.b concentrated load at 'rail position'

$$\gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot \psi_{0i} \cdot F_{traffic} + \gamma_{wind} \cdot (f_w \cdot L_{cross}) = 452 \cdot kN$$

distributed load along girder axis

$$\gamma_G \cdot G_{cross} = 1.471 \cdot \frac{kN}{m}$$

concentrated load at 'rail position'

$$\gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot \psi_{0i} \cdot F_{traffic} - \gamma_{wind} \cdot (f_w \cdot L_{cross}) = 409 \cdot kN$$

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: $l_{cross} = 4.775\,m$

$$l_1 := \frac{(l_{cross} - E)}{2} = 1.632\,m$$

$$l_2 := l_1 = 1.632\,m$$

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

$$B_2 := \frac{f \cdot l_{cross}}{2} = 3.513 \cdot kN$$

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

$$A_1 := F_1 + F_2 - B_1 = 5.428 \times 10^5 N$$

$$A_2 := B_2 = 3.513 \times 10^3 N$$

$$A := A_1 + A_2 = 546 \cdot kN$$

bending moment at midsection:

$$M_{Ed.mid} := f \cdot \frac{l_{cross}^{2}}{8} + \left[A_1 \cdot \frac{l_{cross}}{2} - F_1 \cdot \left(\frac{l_{cross}}{2} - l_1\right)\right] = 881 \cdot kN \cdot m$$

bending moment at rail position 1:

$$M_{Ed.1} := \left(A_2 \cdot l_1 - f \cdot \frac{l_1^{2}}{2}\right) + \left(A_1 \cdot l_1\right) = 889 \cdot kN \cdot m$$

bending moment at rail position 2:

$$M_{Ed.2} := \left[A_2 \cdot \left(l_1 + E\right) - f \cdot \frac{\left(l_1 + E\right)^2}{2}\right] + \left[A_1 \cdot \left(l_1 + E\right) - F_1 \cdot E\right] = 873 \, m \cdot kN$$

check:

$$M_{Ed.3} := \left(B_2 \cdot l_2 - f \cdot \frac{l_2^{2}}{2}\right) + B_1 \cdot l_2 = 873 \, m \cdot kN$$

$$M_{Ed} := \max\left(M_{Ed.mid}, M_{Ed.1}, M_{Ed.2}\right) = 889 \cdot kN \cdot m$$

## Cross section resistance:

Cross section classification:

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

web: $\frac{h_w}{t_w} = 53$

$72 \cdot \varepsilon = 58.58$ Class 1

$83 \cdot \varepsilon = 67.53$ Class 2 => Class 1

$124 \cdot \varepsilon = 100.888$ Class 3 flange: $\frac{\frac{b_f - t_w}{2}}{t_f} = 6$

$9 \cdot \varepsilon = 7.323$ 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: $W_{pl} := b_f \cdot t_f \cdot (H - t_f) + \frac{t_w \cdot h_w^2}{4} = 3452250 \cdot mm^3$

elastic section modulus: $W_{el} := \frac{b_f \cdot H^2}{6} - \frac{(b_f - t_w) \cdot h_w^3}{6 \cdot H} = 3089991 \cdot mm^3$

Moment resistance of cross girder:

$\gamma_{M0} := 1$

$M_{pl.Rd} := \frac{W_{pl} \cdot f_y}{\gamma_{M0}} = 1226 \cdot kN \cdot m$

$\frac{M_{Ed}}{M_{pl.Rd}} = 0.726 \quad < \quad 1$ OK!

## Longitudinal girder/Railbearer:

Geometry:

I - profile:

$H_{long} := 374mm$

$t_w := 10mm$

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

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

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

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

$h_w := H_{long} - 2 \cdot t_f = 334 \cdot mm$

$A_{long} := h_w \cdot t_w + b_{f.1} \cdot t_{f.1} + b_{f.2} \cdot t_{f.2} = 13740 \cdot mm^2$

long. girder length: $L_{long} := L_{cross} = 1.727\,m$

## Dinamic factor for rail bearer:

$$L_{\phi.rail} := 3 \cdot L_{long} = 5.181\,m$$

$$\phi_{rail} := \frac{2.16}{\sqrt{\frac{L_{\phi.rail}}{m}} - 0.2} + 0.73 = 1.77$$

Loads:

Self weight:

$$G_{long} := A_{long} \cdot \rho \cdot g = 1057 \cdot \frac{N}{m}$$

Traffic Load: LM71-22,5

$$Q_v := 125kN$$

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

$$Q_v := \phi_{rail} \cdot Q_v = 221 \cdot kN$$

$$q_v := \phi_{rail} \cdot q_v = 71 \cdot \frac{kN}{m}$$

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

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

long. girder length: $L_{long} := L_{cross} = 1.727\,m$

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 $\gamma_G \cdot G_{long} + \gamma_G \cdot G_{rail} = 2.777 \cdot \frac{kN}{m}$

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

L2.Eq610.b $n := 1$

$0m < n \cdot 1.6m = 1.6\,m < L_{long} = 1.727\,m$

$F := \gamma_Q \cdot Q_v = 321 \cdot kN$

$f_1 := \gamma_G \cdot G_{long} + \gamma_{wind} \cdot \psi_{0i} \cdot f_w = 10.622 \cdot \frac{kN}{m}$ distributed load

$f_2 := \gamma_G \cdot G_{long} - \gamma_{wind} \cdot \psi_{0i} \cdot f_w = -7.979 \cdot \frac{kN}{m}$ distributed load

$\psi_{0i} := 0.8$

L3.Eq610.b $\gamma_Q \cdot \psi_{0i} \cdot Q_v = 257 \cdot kN$

$\gamma_G \cdot G_{long} + \gamma_{wind} \cdot f_w = 13.722 \cdot \frac{kN}{m}$ distributed load

$\gamma_G \cdot G_{long} - \gamma_{wind} \cdot f_w = -11.08 \cdot \frac{kN}{m}$ distributed load

Calc of reaction forces: $R_y := \frac{(n + 1) \cdot F + f_1 \cdot L_{long}}{2} = 330.052 \cdot kN$

Maximum moment at mid span:

long. girder length: $L_{long} = 1.727\,m$

$$l_1 := \frac{\left(L_{cross} - 1.6m\right)}{2} = 0.064\,m$$

$$l_2 := l_1 = 0.064\,m$$

$$M_{Ed} := R_y \cdot \frac{L_{long}}{2} - F \cdot \left(\frac{L_{long}}{2} - l_1\right) - f_1 \cdot \frac{L_{long}^2}{8} = 24 \cdot kN \cdot m$$

Cross section classification:

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

web: $\frac{h_w}{t_w} = 33.4$ $72 \cdot \varepsilon = 58.58$ Class 1

$83 \cdot \varepsilon = 67.53$ Class 2 => Class 1

$124 \cdot \varepsilon = 100.888$ Class 3

upper flange: $\dfrac{\dfrac{b_{f.1} - t_w}{2}}{t_{f.1}} = 9.75$

$9 \cdot \varepsilon = 7.323$ Class 1

$10\varepsilon = 8.136$ Class 2 => Class 3

$14 \cdot \varepsilon = 11.391$ Class 3

Class 4

lower flange: $\dfrac{\dfrac{b_{f.2} - t_w}{2}}{t_{f.2}} = 7.917$

$9 \cdot \varepsilon = 7.323$ 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

$\gamma_{M0} := 1$

plastic section modulus: $W_{pl} := b_f \cdot t_f \cdot \left(H_{long} - t_f\right) + \dfrac{t_w \cdot h_w^{\,2}}{4} = 2048890 \cdot mm^3$

elastic section modulus (from Robot Str. Analysis) : $W_{el} := 1282.6cm^3$

Moment resistance of cross girder:

$f_y = 355 \cdot \dfrac{N}{mm^2}$

$$M_{el.Rd} := \frac{W_{el} \cdot f_y}{\gamma_{M0}} = 455 \cdot kN \cdot m$$

$\dfrac{M_{Ed}}{M_{el.Rd}} = 0.053$ < $1$ OK!

## Main girder:

Geometry:

I - profile: $H := 1700mm$

$t_w := 20mm$

$b_f := 400\text{mm}$

$t_f := 30\text{mm}$

$h_w := H - 2 \cdot t_f = 1.64 \times 10^3 \cdot \text{mm}$

$A_{main} := h_w \cdot t_w + 2b_f \cdot t_f = 56800 \cdot \text{mm}^2$

Span: $L := 17.27\text{m}$

Dinamic factor for main girder

$L_{\phi.main} := L = 17.27\ \text{m}$

$$\phi_{main} := \frac{2.16}{\sqrt{\frac{L_{\phi.main}}{m}} - 0.2} + 0.73 = 1.276$$

| Case | Structural element | Determinant length $L_\Phi$ |
|---|---|---|
| **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:

$G_{main} := A_{main} \cdot \rho \cdot g = 4.37 \cdot \frac{kN}{m}$ self weight of main girder

$G_{cross} := \frac{G_{cross}}{2} = 0.589 \cdot \frac{kN}{m}$ self weight of cross girder

$G_{long} = 1.057 \cdot \frac{kN}{m}$ self weight of railbearer

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

Traffic Load: LM71-22,5

$Q_v := 125kN$

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

$Q_v := \phi_{main} \cdot Q_v = 160 \cdot kN$

$q_v := \phi_{main} \cdot q_v = 51 \cdot \frac{kN}{m}$

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

$$\gamma_G \cdot \left(G_{cross} + G_{main} + G_{long} + G_{rail}\right) = 9.47 \cdot \frac{kN}{m}$$

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

L2.Eq610.b

concentrated load at 'rail position'

$F_2 := \gamma_Q \cdot Q_v \cdot 4 = 925 \cdot kN$

distributed load along girder axis

$$f_2 := \gamma_G \cdot \left(G_{cross} + G_{main} + G_{long} + G_{rail}\right) + \gamma_Q \cdot q_v + \gamma_{wind} \cdot \psi_{0i} \cdot f_w = 92.08 \cdot \frac{kN}{m}$$

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

L3.Eq610.b

concentrated load at 'rail position'

$$F_3 := \gamma_Q \cdot \psi_{0i} \cdot Q_v \cdot 4 = 740 \cdot kN$$

distributed load along girder axis

$$f_3 := \gamma_G \cdot \left(G_{cross} + G_{main} + G_{long} + G_{rail}\right) + \gamma_Q \cdot \psi_{0i} \cdot q_v + \gamma_{wind} \cdot f_w = 80.378 \cdot \frac{kN}{m}$$

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.

$$M_{Ed} := \frac{f_2 \cdot L^2}{8} + \frac{F_2 \cdot L}{4} = 7427 \cdot kN \cdot m$$

Cross section resistance:

Cross section classification:

web: $\frac{h_w}{t_w} = 82$

| | | |
|---|---|---|
| $72 \cdot \varepsilon = 58.58$ | 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: $W_{pl} := b_f \cdot t_f \cdot \left(H - t_f\right) + \frac{t_w \cdot h_w^2}{4} = 33488000 \cdot mm^3$

elastic section modulus: $W_{el} := \frac{b_f \cdot H^2}{6} - \frac{(b_f - t_w) \cdot h_w^3}{6 \cdot H} = 28337380 \cdot mm^3$

Moment resistance of cross girder: $\gamma_{M0} := 1$

$$M_{el.Rd} := \frac{W_{el} \cdot f_y}{\gamma_{M0}} = 10060 \cdot kN \cdot m$$

$$\frac{M_{Ed}}{M_{el.Rd}} = 0.738 \quad < \quad 1 \quad \text{OK!}$$

Deflection of main girder:

second mometn of area: $I := \frac{h_w^3 \cdot t_w}{12} + \left(\frac{b_f}{12}\right) \cdot \left(H^3 - h_w^3\right) = 2.409 \times 10^{10} \cdot mm^4$

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

permanent characteristic: $g_k := G_{cross} + G_{main} + G_{long} + G_{rail} = 7.015 \cdot \frac{kN}{m}$

variable characteristic: $q_k := q_v + f_w = 59.309 \cdot \frac{kN}{m}$

$$Q_k := 4Q_v = 638.022 \cdot kN$$

the calculation of deflection is an approximation on the safe side

$$\delta_{max} := \frac{5 \cdot (g_k + q_k) \cdot L^4}{384 \cdot E_a \cdot I} + \frac{Q_k \cdot L^3}{48 \cdot E_a \cdot I} = 28.72 \cdot mm \quad < \quad \frac{L}{400} = 43.175 \cdot mm$$

## Total weight of structure:

$n_{cross} := 11$

$A_{cross} \cdot l_{cross} \cdot \rho \cdot n_{cross} = 6.309 \times 10^3 kg$

$n_{long} := 2$

$A_{long} \cdot L \cdot \rho \cdot n_{long} = 3.725 \times 10^{3}\, kg$

$n_{main} := 2$

$A_{main} \cdot L \cdot \rho \cdot n_{main} = 1.54 \times 10^{4}\, kg$

Buckiling support girder:

Geometry:

I - profile:

$h_b := 246mm$

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

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

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

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

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

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

$n_{buckl} := n_{cross} - 1 = 10$

Wind bracing:

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

$L_w := 5297mm$

$n_w := 10$

$$G_{TOT} := A_{main} \cdot L \cdot \rho \cdot n_{main} + A_{long} \cdot L \cdot \rho \cdot n_{long} + A_{cross} \cdot l_{cross} \cdot \rho \cdot n_{cross} \ldots + A_{buckl} \cdot l_{buckl} \cdot \rho \cdot n_{buckl} + A_w \cdot L_w \cdot \rho \cdot n_w = 26.617 \cdot tonne$$

$G_{TOT} \cdot g = 260.85 \cdot kN$

Side walk: $g_{sw} := 80 \dfrac{kg}{m}$

$L = 17.27\, m$

$G_{TOT} \cdot g + g_{sw} \cdot L \cdot g = 274.39 \cdot kN$

$G_{TOT} + g_{sw} \cdot L = 27999.07 \cdot kg$

# 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:

$$H := 490mm$$

$$t_w := 10mm$$

$$b_f := 250mm$$

$$t_f := 20mm$$

$$h_w := H - 2 \cdot t_f = 450 \cdot mm$$

$$A_{cross} := h_w \cdot t_w + 2b_f \cdot t_f = 14500 \cdot mm^2$$

cross girder length: $l_{cross} := 4.775m$

### Dynamic factor

For track with standard maintenance:

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

Ref. EN1991-2
§6.4.5.2 (2)

Detenninant length $L.\phi$:

| | | |
|---|---|---|
| **Steel grillage**: open deck without ballast bed [b] (for local and transverse stresses) | | |
| 3.1 | Rail bearers:<br>- as an element of a continuous grillage<br>- simply supported | <br>3 times cross girder spacing<br><br>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.<br>[b] It is recommended to apply $\Phi_3$ | | |

Ref. EN1991-2 §6.4.5.3 Table 6.2

Cross girder length: $l_{cross} = 4.775\,m$

*Dinamic factor for cross girder:*

$$L_{\phi.cross} := 2 \cdot l_{cross} = 9.55\,m$$

$$\phi_{cross} := \frac{2.16}{\sqrt{\frac{L_{\phi.cross}}{m}} - 0.2} + 0.73 = 1.477$$

Loads:

Self weight:

$$G_{cross} := A_{cross} \cdot \rho \cdot g = 1115 \cdot \frac{N}{m}$$

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 by rails and longitudinal girders to the cross girder.

distance between cross girders: $L_{cross} := 1867mm$

from distributed load - q.v: $F_{traffic.q} := q_v \cdot L_{cross} = 110 \cdot kN$

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

0 < $L_{cross} - n \cdot 1.6m = 0.267\,m$ < 1.6 m

from point load - Q.v: $F_{traffic.Q} := (n + 1) \cdot Q_v = 369 \cdot kN$

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)

$$F_{traffic} := \max\left(F_{traffic.q}, F_{traffic.Q}\right) = 369.331 \cdot kN$$

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 $\gamma_G \cdot G_{cross} = 1.506 \cdot \frac{kN}{m}$ distributed load along girder axis

$\gamma_G \cdot \left(G_{rail} \cdot L_{cross}\right) = 2.52 \cdot kN$ concentrated load at 'rail position'

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

L2.Eq610.b concentrated load at 'rail position'

$$F_1 := \gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot F_{traffic} + \gamma_{wind} \cdot \psi_{0i} \cdot (f_w \cdot L_{cross}) = 555 \cdot kN$$

distributed load along girder axis

$$f := \gamma_G \cdot G_{cross} = 1.394 \cdot \frac{kN}{m}$$

concentrated load at 'rail position'

$$F_2 := \gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot F_{traffic} - \gamma_{wind} \cdot \psi_{0i} \cdot (f_w \cdot L_{cross}) = 520 \cdot kN$$

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

L3.Eq610.b concentrated load at 'rail position'

$$\gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot \psi_{0i} \cdot F_{traffic} + \gamma_{wind} \cdot (f_w \cdot L_{cross}) = 454 \cdot kN$$

distributed load along girder axis

$$\gamma_G \cdot G_{cross} = 1.394 \cdot \frac{kN}{m}$$

concentrated load at 'rail position'

$$\gamma_G \cdot (G_{rail} \cdot L_{cross}) + \gamma_Q \cdot \psi_{0i} \cdot F_{traffic} - \gamma_{wind} \cdot (f_w \cdot L_{cross}) = 408 \cdot kN$$

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: $l_{cross} = 4.775\,m$

$$l_1 := \frac{(l_{cross} - E)}{2} = 1.632\,m$$

$$l_2 := l_1 = 1.632\,m$$

Calc of reaction forces:

$$B_1 := \frac{F_1 \cdot l_1 + F_2 \cdot (l_1 + E)}{l_{cross}} = 532 \cdot kN$$

$$B_2 := \frac{f \cdot l_{cross}}{2} = 3.329 \cdot kN$$

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

$$A_1 := F_1 + F_2 - B_1 = 5.434 \times 10^5 \, N$$

$$A_2 := B_2 = 3.329 \times 10^3 \, N$$

$$A := A_1 + A_2 = 547 \cdot kN$$

bending moment at midsection:

$$M_{Ed.mid} := f \cdot \frac{l_{cross}^{\;2}}{8} + \left[A_1 \cdot \frac{l_{cross}}{2} - F_1 \cdot \left(\frac{l_{cross}}{2} - l_1\right)\right] = 881 \cdot kN \cdot m$$

bending moment at rail position 1:

$$M_{Ed.1} := \left(A_2 \cdot l_1 - f \cdot \frac{l_1^{\;2}}{2}\right) + (A_1 \cdot l_1) = 890 \cdot kN \cdot m$$

bending moment at rail position 2:

$$M_{Ed.2} := \left[A_2 \cdot (l_1 + E) - f \cdot \frac{(l_1 + E)^2}{2}\right] + \left[A_1 \cdot (l_1 + E) - F_1 \cdot E\right] = 872 \, m \cdot kN$$

check:

$$M_{Ed.3} := \left(B_2 \cdot l_2 - f \cdot \frac{l_2^{\;2}}{2}\right) + B_1 \cdot l_2 = 872 \, m \cdot kN$$

$$M_{Ed} := \max(M_{Ed.mid}, M_{Ed.1}, M_{Ed.2}) = 890 \cdot kN \cdot m$$

## Cross section resistance:

Cross section classification:

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

web: $\frac{h_w}{t_w} = 45$ — $72 \cdot \varepsilon = 58.58$ Class 1

$83 \cdot \varepsilon = 67.53$ Class 2 => Class 1

$124 \cdot \varepsilon = 100.888$ Class 3

flange: $\dfrac{\frac{b_f - t_w}{2}}{t_f} = 6$

$9 \cdot \varepsilon = 7.323$ 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: $W_{pl} := b_f \cdot t_f \cdot (H - t_f) + \dfrac{t_w \cdot h_w^2}{4} = 2856250 \cdot mm^3$

elastic section modulus: $W_{el} := \dfrac{b_f \cdot H^2}{6} - \dfrac{(b_f - t_w) \cdot h_w^3}{6 \cdot H} = 2565391 \cdot mm^3$

Moment resistance of cross girder:

$\gamma_{M0} := 1$

$M_{pl.Rd} := \dfrac{W_{pl} \cdot f_y}{\gamma_{M0}} = 1014 \cdot kN \cdot m$

$\dfrac{M_{Ed}}{M_{pl.Rd}} = 0.878$ < $1$ OK!

## Longitudinal girder/Railbearer:

Geometry:

I - profile:

$H_{long} := 374mm$

$t_w := 10mm$

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

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

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

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

$h_w := H_{long} - 2 \cdot t_f = 334 \cdot mm$

$A_{long} := h_w \cdot t_w + b_{f.1} \cdot t_{f.1} + b_{f.2} \cdot t_{f.2} = 13740 \cdot mm^2$

long. girder length: $L_{long} := L_{cross} = 1.867\,m$

## Dinamic factor for rail bearer:

$L_{\phi.rail} := 3 \cdot L_{long} = 5.601\,m$

$$\phi_{rail} := \frac{2.16}{\sqrt{\frac{L_{\phi.rail}}{m}} - 0.2} + 0.73 = 1.727$$

Loads:

Self weight:

$$G_{long} := A_{long} \cdot \rho \cdot g = 1057 \cdot \frac{N}{m}$$

Traffic Load: LM71-22,5

$Q_v := 125kN$

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

$Q_v := \phi_{rail} \cdot Q_v = 216 \cdot kN$

$q_v := \phi_{rail} \cdot q_v = 69 \cdot \frac{kN}{m}$

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

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

long. girder length: $L_{long} := L_{cross} = 1.867\,m$

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 $\gamma_G \cdot G_{long} + \gamma_G \cdot G_{rail} = 2.777 \cdot \frac{kN}{m}$

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

L2.Eq610.b $n := 1$

$0m < n \cdot 1.6m = 1.6\,m < L_{long} = 1.867\,m$

$F := \gamma_Q \cdot Q_v = 313 \cdot kN$

$f_1 := \gamma_G \cdot G_{long} + \gamma_{wind} \cdot \psi_{0i} \cdot f_w = 10.622 \cdot \frac{kN}{m}$ distributed load

$f_2 := \gamma_G \cdot G_{long} - \gamma_{wind} \cdot \psi_{0i} \cdot f_w = -7.979 \cdot \frac{kN}{m}$ distributed load

$\psi_{0i} := 0.8$

L3.Eq610.b $\gamma_Q \cdot \psi_{0i} \cdot Q_v = 250 \cdot kN$

$\gamma_G \cdot G_{long} + \gamma_{wind} \cdot f_w = 13.722 \cdot \frac{kN}{m}$ distributed load

$\gamma_G \cdot G_{long} - \gamma_{wind} \cdot f_w = -11.08 \cdot \frac{kN}{m}$ distributed load

Calc of reaction forces: $R_y := \frac{(n+1) \cdot F + f_1 \cdot L_{long}}{2} = 322.922 \cdot kN$

Maximum moment at mid span:

long. girder length: $L_{long} = 1.867\,m$

$$l_1 := \frac{\left(L_{cross} - 1.6m\right)}{2} = 0.133\,m$$

$$l_2 := l_1 = 0.133\,m$$

$$M_{Ed} := R_y \cdot \frac{L_{long}}{2} - F \cdot \left(\frac{L_{long}}{2} - l_1\right) - f_1 \cdot \frac{L_{long}^2}{8} = 46 \cdot kN \cdot m$$

Cross section classification:

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

web: $\frac{h_w}{t_w} = 33.4$

$72 \cdot \varepsilon = 58.58$ 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

$\gamma_{M0} := 1$

plastic section modulus: $W_{pl} := b_f \cdot t_f \cdot (H_{long} - t_f) + \frac{t_w \cdot h_w^2}{4} = 2048890 \cdot mm^3$

elastic section modulus (from Robot Str. Analysis) : $W_{el} := 1282.6cm^3$

Moment resistance of cross girder:

$f_y = 355 \cdot \frac{N}{mm^2}$

$$M_{el.Rd} := \frac{W_{el} \cdot f_y}{\gamma_{M0}} = 455 \cdot kN \cdot m$$

$\frac{M_{Ed}}{M_{el.Rd}} = 0.102$ < 1 OK!

## Main girder:

Geometry:

I - profile: $H := 1700mm$

$t_w := 20mm$

$b_f := 400mm$

$t_f := 30mm$

$h_w := H - 2 \cdot t_f = 1.64 \times 10^3 \cdot mm$

$A_{main} := h_w \cdot t_w + 2b_f \cdot t_f = 56800 \cdot mm^2$

Span: $L := 18.67m$

Dinamic factor for main girder

$$L_{\phi.main} := L = 18.67\,m$$

$$\phi_{main} := \frac{2.16}{\sqrt{\frac{L_{\phi.main}}{m}} - 0.2} + 0.73 = 1.254$$

| Case | Structural element | Determinant length $L_\Phi$ |
|---|---|---|
| **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:

$G_{main} := A_{main} \cdot \rho \cdot g = 4.37 \cdot \frac{kN}{m}$ self weight of main girder

$G_{cross} := \frac{G_{cross}}{2} = 0.558 \cdot \frac{kN}{m}$ self weight of cross girder

$G_{long} = 1.057 \cdot \frac{kN}{m}$ self weight of railbearer

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

Traffic Load: LM71-22,5

$Q_v := 125kN$

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

$Q_v := \phi_{main} \cdot Q_v = 157 \cdot kN$

$q_v := \phi_{main} \cdot q_v = 50 \cdot \frac{kN}{m}$

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

$$\gamma_G \cdot \left(G_{cross} + G_{main} + G_{long} + G_{rail}\right) = 9.429 \cdot \frac{kN}{m}$$

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

L2.Eq610.b

concentrated load at 'rail position'

$F_2 := \gamma_Q \cdot Q_v \cdot 4 = 909 \cdot kN$

distributed load along girder axis

$$f_2 := \gamma_G \cdot (G_{cross} + G_{main} + G_{long} + G_{rail}) + \gamma_Q \cdot q_v + \gamma_{wind} \cdot \psi_{0i} \cdot f_w = 90.772 \cdot \frac{kN}{m}$$

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

L3.Eq610.b

concentrated load at 'rail position'

$$F_3 := \gamma_Q \cdot \psi_{0i} \cdot Q_v \cdot 4 = 727 \cdot kN$$

distributed load along girder axis

$$f_3 := \gamma_G \cdot (G_{cross} + G_{main} + G_{long} + G_{rail}) + \gamma_Q \cdot \psi_{0i} \cdot q_v + \gamma_{wind} \cdot f_w = 79.324 \cdot \frac{kN}{m}$$

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.

$$M_{Ed} := \frac{f_2 \cdot L^2}{8} + \frac{F_2 \cdot L}{4} = 8199 \cdot kN \cdot m$$

## Cross section resistance:

### Cross section classification:

| | | | | |
|---|---|---|---|---|
| web: | $\frac{h_w}{t_w} = 82$ | $72 \cdot \varepsilon = 58.58$ | 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: $W_{pl} := b_f \cdot t_f \cdot (H - t_f) + \frac{t_w \cdot h_w^2}{4} = 33488000 \cdot mm^3$

elastic section modulus: $W_{el} := \frac{b_f \cdot H^2}{6} - \frac{(b_f - t_w) \cdot h_w^{\ 3}}{6 \cdot H} = 28337380 \cdot mm^3$

Moment resistance of cross girder: $\gamma_{M0} := 1$

$$M_{el.Rd} := \frac{W_{el} \cdot f_y}{\gamma_{M0}} = 10060 \cdot kN \cdot m$$

$\frac{M_{Ed}}{M_{el.Rd}} = 0.815$ < $1$ OK!

Deflection of main girder:

second mometn of area: $I := \frac{h_w^{\ 3} \cdot t_w}{12} + \left(\frac{b_f}{12}\right) \cdot \left(H^3 - h_w^{\ 3}\right) = 2.409 \times 10^{10} \cdot mm^4$

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

permanent characteristic: $g_k := G_{cross} + G_{main} + G_{long} + G_{rail} = 6.984 \cdot \frac{kN}{m}$

variable characteristic: $q_k := q_v + f_w = 58.434 \cdot \frac{kN}{m}$

$$Q_k := 4Q_v = 627.08 \cdot kN$$

the calculation of deflection is an approximation on the safe side

$$\delta_{max} := \frac{5 \cdot (g_k + q_k) \cdot L^4}{384 \cdot E_a \cdot I} + \frac{Q_k \cdot L^3}{48 \cdot E_a \cdot I} = 37.27 \cdot mm \quad < \quad \frac{L}{400} = 46.675 \cdot mm$$

## Total weight of structure:

$$n_{cross} := \mathrm{ceil}\left(\frac{L}{L_{cross}}\right) = 11$$

$$A_{cross} \cdot l_{cross} \cdot \rho \cdot n_{cross} = 5.979 \times 10^3 \, kg$$

$$n_{long} := 2$$

$$A_{long} \cdot L \cdot \rho \cdot n_{long} = 4.027 \times 10^3 \, kg$$

$n_{main} := 2$

$A_{main} \cdot L \cdot \rho \cdot n_{main} = 1.665 \times 10^4\, kg$

Buckiling support girder:

Geometry:

I - profile:

$h_b := 246mm$

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

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

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

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

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

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

$n_{buckl} := n_{cross} - 1 = 10$

Wind bracing:

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

$L_w := 5466mm$

$n_w := 10$

$$G_{TOT} := A_{main} \cdot L \cdot \rho \cdot n_{main} + A_{long} \cdot L \cdot \rho \cdot n_{long} + A_{cross} \cdot l_{cross} \cdot \rho \cdot n_{cross} \ldots = 27.856 \cdot tonne$$
$$+ A_{buckl} \cdot l_{buckl} \cdot \rho \cdot n_{buckl} + A_w \cdot L_w \cdot \rho \cdot n_w$$

$G_{TOT} \cdot g = 272.99 \cdot kN$

Side walk: $g_{sw} := 80 \dfrac{kg}{m}$

$L = 18.67\, m$

$G_{TOT} \cdot g + g_{sw} \cdot L \cdot g = 287.62 \cdot kN$

$G_{TOT} + g_{sw} \cdot L = 29349.28 \cdot kg$

# APPENDIX 3

## Connection Calculation Report - Crossgirder-railbearer

### Material:

Bolt class: 4.6

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

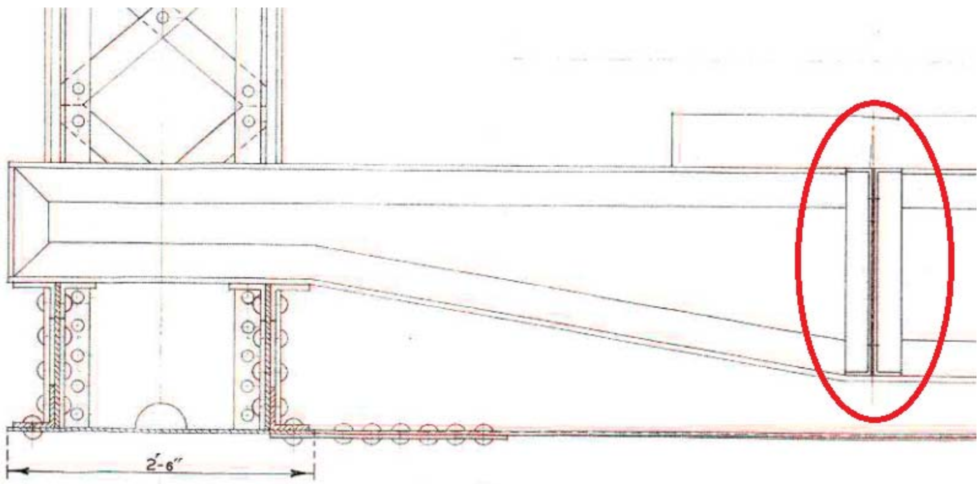
Ref. EN1993-1-8
§3.1.1
Table 3.1

Steel grade: S235

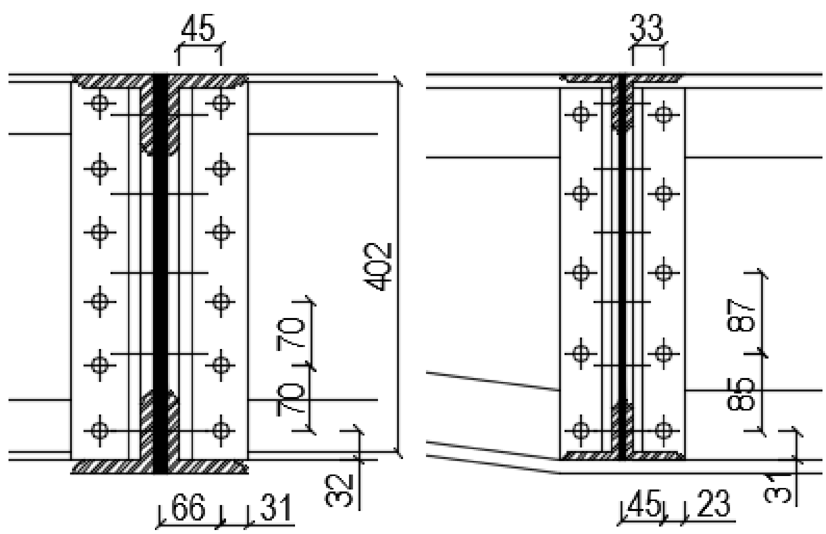
$f_y := 235 \frac{N}{mm^2}$ $\quad$ $f_u := 360 \frac{N}{mm^2}$

Ref. EN1993-1-1
§3.2.3
Table 3.1

**Geometry of joint:** The geometrical datas are assumptions, for achieving an accurate results, a site inspection is needed to gather the missing information.




*Connection between cross girder and railbearer*




*Supported beam side* *Supporting beam side*

$d := 20mm$

$A_b := \frac{d^2 \cdot \pi}{4} = 314 \cdot mm^2$ the gross cross section of the bolt

$d_0 := d + 2mm = 22 \cdot mm$

$p_1 := 70mm$

$z := 66mm$ is the transverse distance from the face of the supporting element to the centre of the bolt group

$n_b := 6$

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: $V_{Ed} \leq V_{Rd}$

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

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

$\alpha := 0$

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

$n_1 := n_b = 6$

$$\beta := \frac{6 \cdot z}{n_1 \cdot \left(n_1 + 1\right) \cdot p_1} = 0.135$$

$$F_{v.Rd} := \frac{\alpha_v \cdot f_{ub} \cdot A_b}{\gamma_{M2}} = 60.319 \cdot kN$$

$$V_{v.Rd} := \frac{2 \cdot n_b \cdot F_{v.Rd}}{\sqrt{\left(1 + \alpha \cdot n_b\right)^2 + \left(\beta \cdot n_b\right)^2}} = 562.962 \cdot kN$$

## 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

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

$e_2 := 31mm \quad e_1 := 32mm \qquad t_{ac} := 11mm$

$$k_{1.ver} := \min\left(2.8 \cdot \frac{e_2}{d_0} - 1.7, 2.5\right) = 2.245$$

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

$$\alpha_{b.ver} := \min\left(\frac{e_1}{3 \cdot d_0}, \frac{p_1}{3 \cdot d_0} - \frac{1}{4}, \frac{f_{ub}}{f_{u.ac}}, 1\right) = 0.485$$

$$F_{b.ver.Rd} := \frac{k_{1.ver} \cdot \alpha_{b.ver} \cdot f_{u.ac} \cdot d \cdot t_{ac}}{\gamma_{M2}} = 68.98 \cdot kN$$

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

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

$$k_{1.hor} := \min\left(2.8 \cdot \frac{e_1}{d_0} - 1.7, 1.4 \cdot \frac{p_1}{d_0} - 1.7, 2.5\right) = 2.373$$

$$\alpha_{b.hor} := \min\left(\frac{e_2}{3 \cdot d_0}, \frac{f_{ub}}{f_{u.ac}}, 1\right) = 0.47$$

$$F_{b.hor.Rd} := \frac{k_{1.hor} \cdot \alpha_{b.hor} \cdot f_{u.ac} \cdot d \cdot t_{ac}}{\gamma_{M2}} = 70.612 \cdot kN$$

$$V_{b.Rd} := \frac{2 \cdot n_b}{\sqrt{\left(\frac{1 + \alpha \cdot n_b}{F_{b.ver.Rd}}\right)^2 + \left(\frac{\beta \cdot n_b}{F_{b.hor.Rd}}\right)^2}} = 650 \cdot kN$$

## Bearing resistance of bolts on the beam web

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

$e_{2.w} := 45mm \qquad t_w := 14mm$

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

The vertical bearing resistance:

Ref. EN1993-1-8
§3.6.1
Table 3.4

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

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

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

$$\alpha_{b.ver.2} := \min\left(\frac{e_1}{3 \cdot d_0}, \frac{p_1}{3 \cdot d_0} - \frac{1}{4}, \frac{f_{ub}}{f_{u.w}}, 1\right) = 0.485$$

$$F_{b.ver.Rd.2} := \frac{k_{1.ver.2} \cdot \alpha_{b.ver.2} \cdot f_{u.w} \cdot d \cdot t_w}{\gamma_{M2}} = 97.745 \cdot kN$$

The horizontal bearing resistance:

Ref. EN1993-1-8
§3.6.1
Table 3.4

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

$$k_{1.hor.2} := \min\left(2.8\cdot\frac{e_1}{d_0} - 1.7, 1.4\cdot\frac{p_1}{d_0} - 1.7, 2.5\right) = 2.373$$

$$\alpha_{b.hor.2} := \min\left(\frac{e_{2.w}}{3\cdot d_0}, \frac{f_{ub}}{f_{u.w}}, 1\right) = 0.682$$

$$F_{b.hor.Rd.2} := \frac{k_{1.hor.2}\cdot\alpha_{b.hor.2}\cdot f_{u.w}\cdot d\cdot t_w}{\gamma_{M2}} = 130.457\cdot kN$$

$$V_{b.Rd.2} := 2\frac{n_b}{\sqrt{\left(\frac{1+\alpha\cdot n_b}{F_{b.ver.Rd.2}}\right)^2 + \left(\frac{\beta\cdot n_b}{F_{b.hor.Rd.2}}\right)^2}} = 1003\cdot kN$$

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:

$$F_{v.Rd} = 60\cdot kN$$

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

$e_2 := 23mm$ $\quad e_1 := 31mm$ $\quad p_1 := 85mm$

For edge bolts: $k_{1.ac} := \min\left(2.8\cdot\frac{e_2}{d_0} - 1.7, 2.5\right) = 1.227$

For end bolts: $\alpha_{b.ac.end} := \min\left(\frac{e_1}{3 \cdot d_0}, \frac{f_{ub}}{f_{u.ac}}, 1\right) = 0.47$

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

For end bolts: $F_{b.Rd.end} := \frac{k_{1.ac} \cdot \alpha_{b.ac.end} \cdot f_{u.ac} \cdot d \cdot t_{ac}}{\gamma_{M2}} = 36.524 \cdot kN$

For inner bolts: $F_{b.Rd.inn} := \frac{k_{1.ac} \cdot \alpha_{b.ac.inn} \cdot f_{u.ac} \cdot d \cdot t_{ac}}{\gamma_{M2}} = 77.76 \cdot kN$

$F_{b.Rd.min} := \min\left(F_{b.Rd.end}, F_{b.Rd.inn}\right) = 36.524 \cdot kN$

$F_{b.Rd.max} := \max\left(F_{b.Rd.end}, F_{b.Rd.inn}\right) = 77.76 \cdot kN$

$n_{b.2} := 5$ number of bolts on supporting beam side

$n_s := 2 \cdot n_{b.2} = 10$

$$F_{Rd} := \begin{array}{|l} F_{b.Rd.end} + F_{b.Rd.inn} \ \text{ if } \ F_{b.Rd.max} \le F_{v.Rd} \\ n_s \cdot F_{b.Rd.min} \ \text{ if } \ F_{b.Rd.min} \le F_{v.Rd} \le F_{b.Rd.max} \\ 0.8 \cdot n_s \cdot F_{v.Rd} \ \text{ if } \ F_{v.Rd} \le F_{b.Rd.min} \end{array} = 365 \cdot kN$$

## Supported beam side:

$e_2 := 31mm$ $e_1 := 32mm$ $p_1 := 70mm$

### 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

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

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

$h_{ac} := 402mm \qquad t_{ac} = 11 \cdot mm \qquad f_{y.ac} := f_y = 235 \cdot MPa$

$$V_{Rd.g} := 2 \cdot \frac{h_{ac} \cdot t_{ac}}{1.27} \cdot \frac{f_{y.ac}}{\sqrt{3} \cdot \gamma_{M0}} = 945 \cdot kN$$

Shear resistance of net section

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

$$A_{v.net} := t_{ac} \cdot \left(h_{ac} - n_1 \cdot d_0\right) = 2970 \cdot mm^2$$

$$V_{Rd.n} := 2 \cdot A_{v.net} \cdot \frac{f_{u.ac}}{\sqrt{3} \cdot \gamma_{M2}} = 988 \cdot kN$$

Block tearing resistance

Ref.
EN1993-1-8
§3.10.2 (2)

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

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

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

$$V_{Rd.b} := 2 \cdot \left(\frac{0.5 \cdot f_{u.ac} \cdot A_{nt}}{\gamma_{M2}} + \frac{f_{y.ac} \cdot A_{nv}}{\sqrt{3} \cdot \gamma_{M0}}\right) = 807 \cdot kN$$

$$V_{Rd.min} := \min\left(V_{Rd.g}, V_{Rd.n}, V_{Rd.b}\right) = 807 \cdot kN$$

## <u>Supporting beam side:</u>

### 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

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

$$V_{Rd.g.2} := 2 \cdot \frac{h_{ac} \cdot t_{ac}}{1.27} \cdot \frac{f_{y.ac}}{\sqrt{3} \cdot \gamma_{M0}} = 945 \cdot kN$$

Shear resistance of net section

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

$$A_{v.net.2} := t_{ac} \cdot \left(h_{ac} - n_{b.2} \cdot d_0\right) = 3212 \cdot mm^2$$

$$V_{Rd.n.2} := 2 \cdot A_{v.net.2} \cdot \frac{f_{u.ac}}{\sqrt{3} \cdot \gamma_{M2}} = 1068 \cdot kN$$

Block tearing resistance

Ref.
EN1993-1-8
§3.10.2 (2)

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

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

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

$$V_{Rd.b.2} := 2 \cdot \left( \frac{0.5 \cdot f_{u.ac} \cdot A_{nt.2}}{\gamma_{M2}} + \frac{f_{y.ac} \cdot A_{nv.2}}{\sqrt{3} \cdot \gamma_{M0}} \right) = 872 \cdot kN$$

$$V_{Rd.min.2} := \min\left(V_{Rd.g.2}, V_{Rd.n.2}, V_{Rd.b.2}\right) = 872 \cdot kN$$

# 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

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

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

$h_w := h_{ac} = 402 \cdot mm \quad t_w = 14 \cdot mm$

$A_{v.wb} := h_w \cdot t_w = 5628 \cdot mm^2$

$$V_{Rd.g.wb} := A_{v.wb} \cdot \frac{f_{y.b}}{\sqrt{3} \cdot \gamma_{M0}} = 763.592 \cdot kN$$

Shear resistance of net section

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

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

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

$$V_{Rd.n.wb} := A_{v.wb.net} \cdot \frac{f_{u.b}}{\sqrt{3} \cdot \gamma_{M0}} = 785.658 \cdot kN$$

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)

$A_{nt.wb} := t_w \cdot (e_{2.w} - 0.5 \cdot d_0) = 476 \cdot mm^2$

$A_{nv.wb} := t_w \cdot \left[ e_1 + (n_1 - 1) \cdot p_1 - (n_1 - 0.5) \cdot d_0 \right] = 3.654 \times 10^3 \cdot mm^2$

$$V_{Rd.b.wb} := 2 \cdot \left( \frac{0.5 \cdot f_{u.b} \cdot A_{nt.wb}}{\gamma_{M2}} + \frac{f_{y.b} \cdot A_{nv.wb}}{\sqrt{3} \cdot \gamma_{M0}} \right) = 1129 \cdot kN$$

$V_{Rd.min.wb} := \min(V_{Rd.g.wb}, V_{Rd.n.wb}, V_{Rd.b.wb}) = 764 \cdot kN$

## Summary of design checks:

Shear resistance:

Bolt group design

Supported beam side

Shear resistance of bolts: $V_{v.Rd} = 563 \cdot kN$

Bearing resistance of bolts on angle cleats: $V_{b.Rd} = 650 \cdot kN$

Bearing resistance of bolts on the beam web: $V_{b.Rd.2} = 1003 \cdot kN$

Supporting beam side

Resistance: $F_{Rd} = 365 \cdot kN$

Shear resistance of the angle cleats

Supported beam side

Shear resistance: $V_{Rd.min} = 807 \cdot kN$

Supporting beam side

Shear resistance: $V_{Rd.min.2} = 872 \cdot kN$

Shear resistance of the beam web

Shear and block tearing resistance

Shear resistance: $V_{Rd.min.wb} = 764 \cdot kN$

$$V_{Rd} := \min\left(V_{v.Rd}, V_{b.Rd}, V_{b.Rd.2}, V_{Rd.min}, V_{Rd.min.2}, V_{Rd.min.wb}, F_{Rd}\right) = 365 \cdot kN$$

$V_{Ed} := 468.8kN$ From Robot Stractural Analysis

$\frac{V_{Ed}}{V_{Rd}} = 1.284$ > 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 on supporting beam side.

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