Optimisation of the transfer carriage structure

Thesis Report

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Summary

This work has been carried out in collaboration with DREVER International S.A. located in Liège. Drever International is a market leader for continuous annealing furnaces and galvanizing plants for steel and stainless steel strip. In the heat treatment of strips (automotive qualities), it is necessary to use a transfer carriage in order to translate an induction furnace (with a weight of about 80 tons). Currently, this carriage has several overhangs. It consists of girder beams and comprises a system to compensate torsion efforts. The design of this equipment is guided by the respect of deflection criteria.

The aim of this thesis is to improve the structural system and to optimize the transfer carriage structure. After studying the current solution, eight new solutions are proposed at the pre-design stage. All of them are studied parametrically analyzing their advantages and disadvantages what leads to the selection of the solution for the detailed design. The selected solution is composed of planar trusses made of hollow sections instead of built-up box girders used in the initial solution. Two variants of the selected solution are studied in detail. As a result, both variants of the selected solution ensure considerable material savings compared to the current solution, as well as some simplifications related to the manufacturing of the structure.

List of symbols

Latin letters:

A - Area of a cross-section
A_{net} - Net area of a cross-section
A_{nt} - Net area subjected to tension (figuring in the block tearing resistance formula)
A_{nv} - Net area subjected to shear (figuring in the block tearing resistance formula)
A_v - Shear area of a cross-section
a - Throat thickness of a fillet weld
b_{eq} - Equivalent single bracing width
e - Eccentricity between the bolt row and the axis of the angle
E - Modulus of elasticity
$F_{b,Rd}$ - Design bearing resistances of a plate per bolt
F_{Rd} - Splice joint factored resistance
$F_{t,Rd}$ - Design tensile resistance of a bolt
f_u - Ultimate tensile strength of steel material
$F_{v,Rd}$ - Design resistance of a single bolt per shear plane
f_y - Yield strength of steel material
G - Shear modulus
h_{eq}- Equivalent single bracing length
I - second moment of area
L - System length of a member
L_{cr} - Critical buckling length of a member
l_{eff} - Whitmore effective width
M_{Ed} - Design bending moment
$M_{ip,1,Rd}$ - Design in-plane moment resistance of a plate connected to the RHS chord
M_{Rd} - Design value of the resistance to bending moments
n - Number of bolts in a joint
$N_{b,Rd}$ - Design buckling resistance of a member
N_{Ed} - Design axial force
$N_{Ed,eff}$ - Hypothetical effective axial load (splice joints)
$N_{i,Rd}$ - Design resistance of a joint in lattice girder made of hollow sections
N_{Rd} - Design values of the resistance to axial forces
T - Applied torque
T_f - Actual total bolt tension (splice joints)
t_p - Gusset plate thickness
t_{sp} - Stiffening plate thickness
$V_{b,Rd}$ - Bearing resistance of a bolt group
V_{Ed} - Design shear force
$V_{eff,Rd}$ - Block tearing resistance of a bolt group
V_L - Shear resistance of an overlap joint in the longitudinal direction
V_{Rd} - Design value of the resistance to shear forces
$V_{v,Rd}$ - Bolt group resistance to shear
W_{el} - Elastic section modulus of a cross-section
W_{pl} - Plastic section modulus of a cross-section
z - Vertical displacement

Greek letters:

α - Imperfection factor for the relevant buckling curve
β - Ratio of the width of the brace members, to that of the chord
β_w - Correlation facto for fillet welds
γ_c - Amplifying coefficient
γ_{M0} - Partial factor for resistance of cross-section whatever the class is
γ_{M1} - Partial factor for resistance of members to instability assessed by member checks
γ_{M2} - Partial factor for resistance of cross-sections in tension to fracture
γ_{M5} - Partial factor for resistance of joints in hollow section lattice girder
θ_i - Angle between brace member and the chord (i=1,2 or 3)
λ - Slenderness of a member
λ_{ov} - Overlap ratio, expressed as a percentage (joints in hollow section lattice girder)
$\bar{\lambda}$ - Relative slenderness of a member
$\bar{\lambda}_{eff}$ - Effective slenderness ratio for buckling of angles
ν_E - Overall safety factor
σ_a - Allowable normal stress
$\sigma_{\parallel}$ - Normal stress parallel to the weld throat
$\sigma_{\perp}$ - Normal stress perpendicular to the weld throat
τ_a - Allowable shear stress
$\tau_{\parallel}$ - Shear stress (in plane of the throat) parallel to the axis of the weld
$\tau_{\perp}$ - Shear stress (in plane of the throat) perpendicular to the axis of the weld
Φ - Value to determine the reduction factor χ
χ - Reduction factor for relevant buckling mode
ψ - Dynamic coefficient

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1. Introduction

This thesis is dedicated to the carriage structure intended to support the induction furnace in the heat treatment of steel strips and it is conducted in collaboration with Drever International. The current solution was developed by the company and consists of a system of mutually perpendicular beams/girders, loaded normal to its plane.

The main objective of this thesis is to improve the structural system and to optimize the transfer carriage structure. Generally speaking, the optimization can be carried out in terms of the weight reduction, simplification of joints and details, manufacturing, transportation, assembling, etc. There is no perfect solution which can satisfy all these criteria, but the parametric study is necessary in order to select a solution that fits the most all these requirements, what will be conducted in this work as well.

The content of this thesis is organized in five chapters, as follows:

Chapter 2 analyzes the so-called initial solution of the carriage structure which is subject to the optimization. General layout of the structure is presented, methodology and assumptions for the design and results. The results are used as a basis for comparisons with all proposed solutions in the subsequent chapter. Apart from this, standards applicable to cranes are reviewed, as well as the classification of cranes.

Chapter 3 deals with proposed solutions at the pre-design stage, showing their layout, parametric studies and results in terms of internal forces, displacements and the estimated weight as well. All these solutions are compared mutually and with the initial solution. Benefits and drawbacks of each solution are reviewed and selection of the solution for the detailed design is conducted here as well. The selected solution is composed of planar trusses made of hollow sections and further will be designed in two versions:

- Solution 6-1: Welded solution made completely of hollow sections
- Solution 6-2: Bolted solution made of hollow section chords and angles as braces

Chapter 4 presents the main methodology used in the design process. Design is conducted through the serviceability limit states and ultimate limit states for both variants of the selected solution. A special care is taken for design of joints, since some cases that occur in this structure are not covered by the codes. After the detailed design stage the weight of the structure is estimated precisely. The estimation shows significant material savings and the fact that the savings were underestimated in the pre-design stage. The computation details are provided in Annex A (for Solution 6-1) and Annex B (for Solution 6-2).

Chapter 5 is devoted for the conclusions. It summarizes the material savings and shows the importance of the optimization process.

2. Study of the initial solution

2.1. General layout and dimensions of the structure

The initial solution can be described as a grillage structure, in terms of its layout and dimensions. It consists of a system of mutually perpendicular beams/girders, loaded normal to its plane. The only part of the structure that is out of the horizontal plane are brackets, with the purpose to compensate torsion effects. For clear understanding, designation of all parts of the structure is given in *Figure 2.1*.

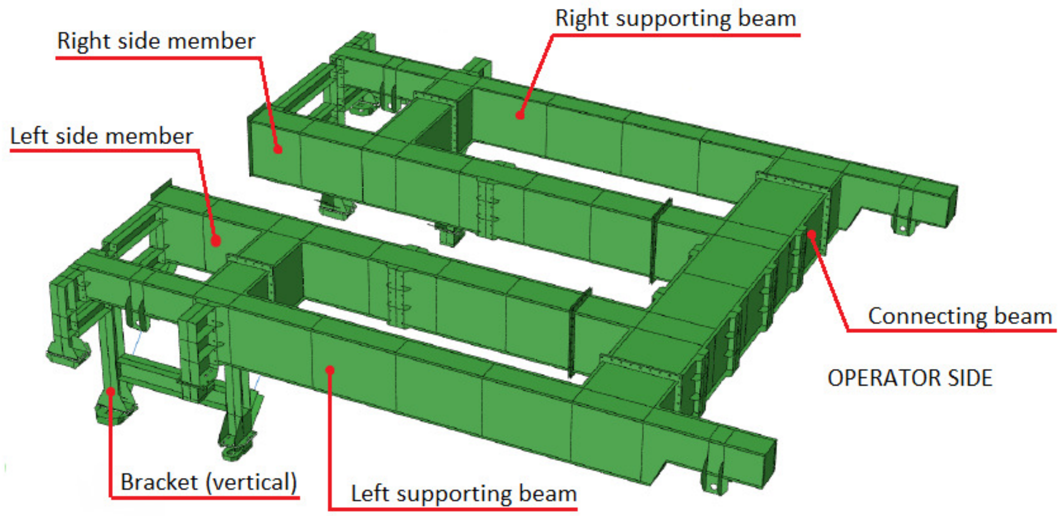


Figure 2.1: General layout and designation (source: Drever calculation sheets)

Sides of the structure (right/left) are assigned in relation to the view from the operator side, and coincides to the project of the equipment that is supported by the structure. For the equipment, this designation is important, for instance the fact that steel strip during the production process (galvanizing/annealing) travels from the right side towards the structure.

The carriage structure is supported by a pair of rails placed on top of the crane runway beams. The runway beam forms a frame together with columns. The distance between the rails (axis-to-axis) is 9 m, what matches with the distance between the columns (axis-to-axis as well). Since the craneway structure is beyond the scope of this Master thesis, it will be described briefly, showing only the data that is in relation with the carriage structure. Cross-section of the runway beam is a welded box section and its shape and dimensions are shown in *Figure 2.2*. The crane carriage assembly is placed on level +9920 what is the position of the top edge of the rail, while the horizontal reactions are transferred to the runway beam at levels +9715 and +8065 what means that the lever arm is 9715-8065=1830 mm. Due to the fact that the horizontal reactions act in opposite directions, on the distance of 1830 mm, the runway beam has to resist significant torsion and that is the reason for selecting a built-up box section, while for the columns circular hollow sections are selected and their height is 7630 mm.

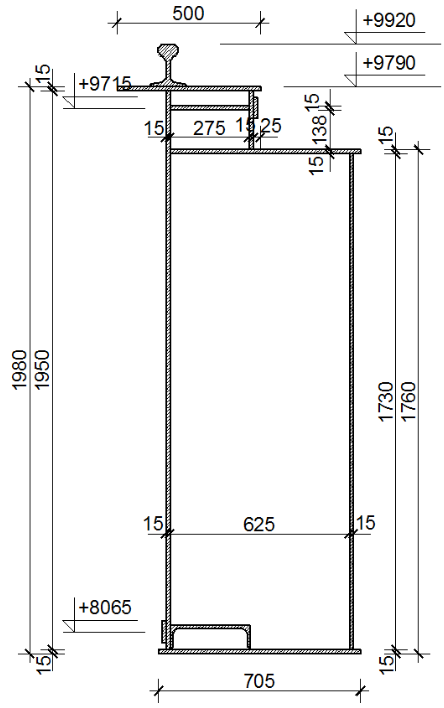


Figure 2.2: Runway beam (cross-section)

The crane carriage assembly is composed of the carriage structure itself plus secondary structural assemblies that are not part of the detailed study, however their weight and loads acting on them have to be taken into account for the carriage structure analysis. The secondary assemblies are: three floor assemblies (upper floor assembly, carriage floor assembly and lower floor assembly) including their columns, four bogie platform assemblies and several ladders. As an illustration of the structure including all sub-assemblies, *Figure 2.3* is given. The crane carriage assembly is intended to translate from the park position to the operating position for a distance of 10855 mm.

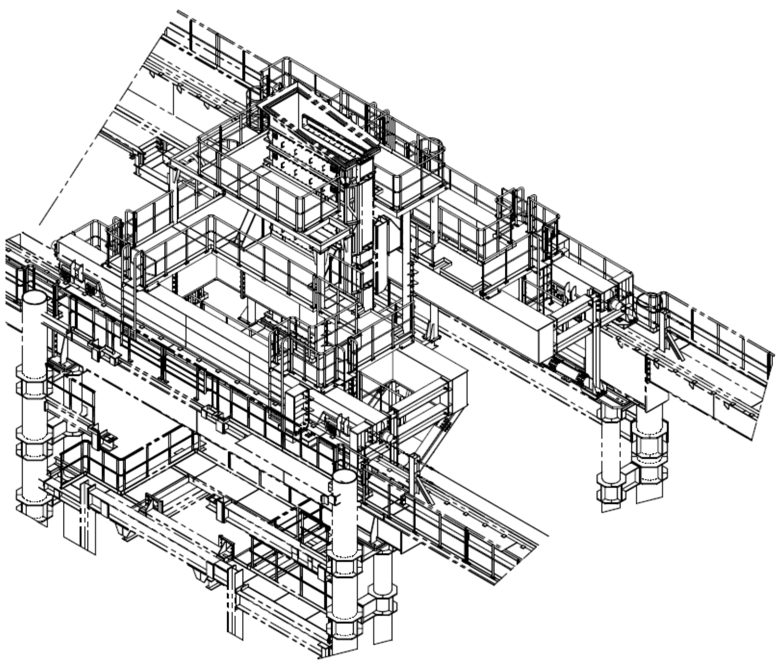


Figure 2.3: Carriage assembly-3D view (source: Drever calculation sheets)

A scheme of the structure showing axes of the structural members is given in *Figure 2.5* and the elevation is given in *Figure 2.4*. The carriage structure is placed on four roller bogies that transfer vertical loading to the craneway structure and allow translation of the carriage. In the longitudinal direction the distance between two bogies is 10465 mm, while the distance between two horizontal bogies in the same direction is 2535 mm, what is illustrated in *Figure 2.4* as well.

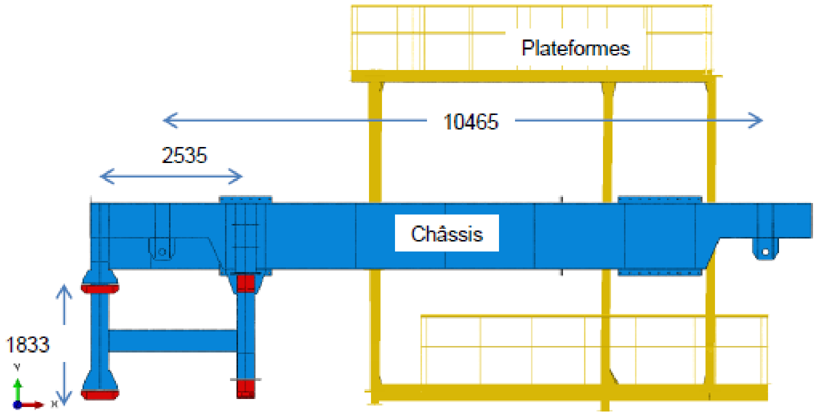


Figure 2.4: Carriage structure-elevation (source: Drever calculation sheets)

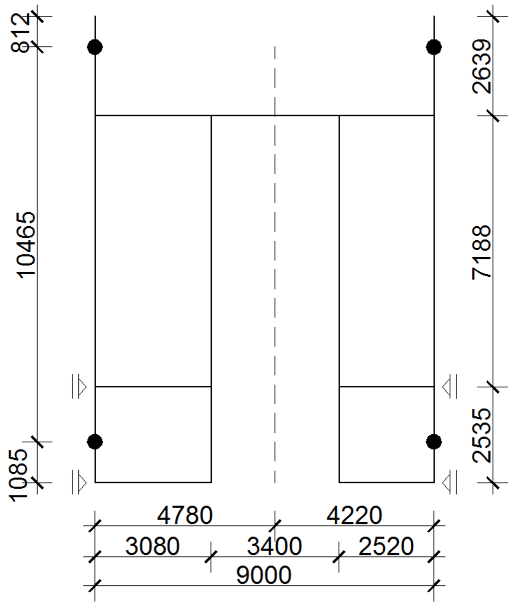
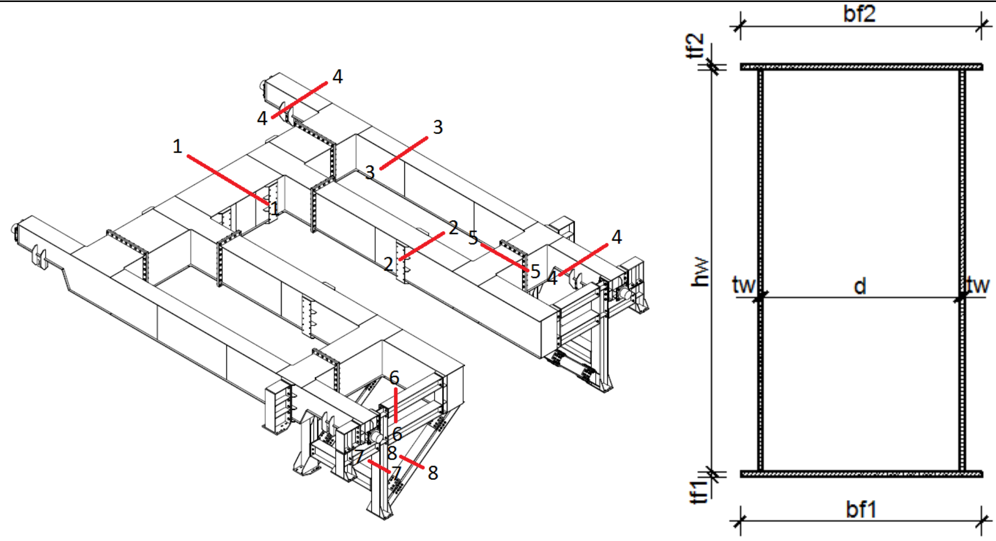


Figure 2.5:Scheme - axes of the structural members

The height of the carriage structure is 1165 mm and in other words, the highest point of the carriage is on level +11210, while the brackets extend on the bottom side for a value of 1980 mm (to level +8065, what is the position of the bottom rail for the horizontal reactions).

Cross-sections selected for the supporting beams, side members and connecting beam are welded built-up box sections with constant height, except for the supporting beams that are tapered, to allow placing of the roller bogies. All above mentioned beams are made of plates with constant thickness along the longitudinal axis. The cross-section sizes of the main structural members are given in *Table 2.1*. It should be mentioned that the beams are stiffened by means of transverse stiffeners (diaphragms), in order to prevent distortion of the box, shear buckling of the web etc.

Table 2.1: Dimensions of cross-sections



1 to 5: Box sections
6 to 8: I sections

Beam/cut	b_{f1} [mm]	t_{f1} [mm]	b_{f2} [mm]	t_{f2} [mm]	h_w [mm]	t_w [mm]	d [mm]
Connecting beam (1-1)	1320	15	1320	15	1135	15	1190
Right and left side member (2-2)	680	15	680	15	1135	12	556
Right and left supporting beam (3-3)	524	15	524	15	1135	12	400
Right and left supporting beam (4-4)	524	25	524	15	600	12	400
Transverse beam (5-5)	800	15	800	15	1135	12	676
Outer transverse beam (6-6)	300	20	300	20	260	12	/
Bracket-vertical (7-7)	300	25	300	25	250	20	/
Bracket-diagonal (8-8)	300	20	300	20	260	12	/

2.2. Standards applicable to cranes and classification of the structure

Since the cranes are specific structures, and their difference compared to ordinary structures is in terms of nature of the acting loads (dynamic and repetitive character), a special attention has to be paid in order to reach safe, serviceable and reliable structure. Due to the mentioned reason, special codes and standards have been developed for design of cranes. In Europe, those standards are FEM (fr. Fédération Européene de la Manutention, eng. European Materials Handling Federation) and EN (European Committee for Standardization). Apart from FEM and EN, on the worldwide level, other standards are present as well, like: International (ISO), American (ASME), Chinese, Australian, Canadian, etc.

For the crane that is subject of this study, the relevant European standards are:
- FEM 1.001: Rules for the Design of Hoisting Appliances (Booklets 1 to 8)
- EN 13001-1: Cranes - General design - Part 1, General principles and requirements
- EN 13001-2: Cranes - General design - Part 2, Load actions
- EN 13001-3-1: Cranes - General design - Part 3, Limit states and proof of competence of steel structures

The initial solution was designed according to FEM 1.001. In order to obtain comparable results of all other solutions that will be analyzed later in this work to the initial solution and upon the suggestion of the DREVER representatives, FEM 1.001 will be used further for the classification, loads and combination of loads, while the resistance of cross-sections, members and joints will be calculated according to the relevant parts of Eurocode 3.

Since the standard covers a variety of hoisting appliances, which can be utilized in different ways, the code requires that a structure has to be classified, to take into account these differences. Consequently, when a structure is classified the code suggests which design checks should be performed. They are mainly related to the fatigue life and wear of parts of a structure. For instance, the significant difference is present between a crane moving on very high speeds and with a high number of working cycles and a crane of a light utilization. In the first case, the structure is susceptible to the fatigue, while the second structure is not and its design will be governed by the ultimate limit states and serviceability limit states. The amplifying coefficient γ_c takes into account a probability of exceeding the calculated stress, which results from imperfect methods of calculation and unpredicted events. According to the code, actions on the structure should be multiplied by the coefficient γ_c and the coefficient itself is dependent on the classification while its value varies from 1.0 to 1.2.

According to FEM 1.001, a crane should be classified on three levels:

- the appliance as a whole
- the individual mechanisms as a whole
- the structural and mechanical components

To classify a structure the code uses two criteria, namely:

- the total duration of use of the item considered
- the hook load, loading or stress spectra to which the item is subjected

The total duration of use is divided into ten classes of utilization, designated by the symbols U0 to U9 and dependent on the number of cycles n_{max}. U0 corresponds to $n_{max} \leq 16000$ cycles, while U9 corresponds to $n_{max} \leq 4000000$ cycles and they are summarized in Table T.2.1.2.2. of FEM 1.001, Booklet 2.

There are four spectrum classes, Q1 to Q4, which are dependent on the spectrum factor k_p that is indeed a distribution function with possible values from 0 to 1. The relation between k_p and the spectrum classes is given in Table T.2.1.2.3. of FEM 1.001, Booklet 2.

Finally, the appliance as a whole is classified by combining the class of utilization and load spectrum class, resulting in eight possible classes, A1 to A8. The scheme for combining U and Q values is provided in Table T.2.1.2.4. of FEM 1.001, Booklet 2.

On the other hand, EN 13001-1 does not combine the total number of cycles and the load spectra in an unique class of the appliance as a whole. Four criteria are specified by EN13001-1 for the classification and accordingly, for each criteria a different class is assigned, namely:

- the total number of working cycles during the specified design life (class U0 to U9)
- the average distances (class Dh0 to Dh9 for hoisting, Dt0 to Dt9 for traversing, Dc0 to Dc9 for travelling and Da0 to Da5 for angular displacement)
- the relative frequencies of loads to be handled-load spectra (class Q0 to Q5)
- the average number of accelerations per movement (class P0 to P3)

It should be mentioned that EN 13001-1 omits the classes of mechanisms (in FEM 1.001 designated as M-classes) and instead uses criteria for average distances and average number of accelerations per movement.

The carriage structure that is subject of the study is intended to support two caissons. The caissons are imposed on the structure by an external device such a mobile crane. The structure itself does not have any hoisting device or traverse trolley, what means that there is no need for the classification of mechanisms, while the classification of the appliance as a whole is already described in detail.

For the proof of competence of a structure, generally, there are two well-known methods: the limit state method and the allowable stress method. At the time when FEM 1.001 was developed (the first edition in 1962, the second in 1970 and the third in 1998) the allowable stress method was commonly used by engineers and suggested by codes. FEM 1.001 uses the allowable stress method for the proof of competence, however the limit state method is mentioned in Booklet 9 (included in the 3rd revision in 1998) when EN 13001 still had been under the development. According to EN 13001 the limit state method is applicable without any restriction though the allowable stress method is allowed for cranes or portions of cranes where all masses act only unfavorable and with a linear relationship between load actions and load effect. Here, the allowable stress method is considered as a special case of the limit state method, where the same numerical value is assigned for all partial safety factors. For instance, in the case of overhead crane and portal crane without cantilevers or in other words, for all cranes which lift/support loads inside the area bounded by the supporting substructure. This is the case for the carriage analyzed in this Master Thesis as well. The allowable stress method is not allowed for tower cranes, because a counterweight equilibrates the imposed loading, and acts favorable, what means that they should be multiplied by different partial safety factors.

The proof of competence, using the limit state method, according to EN 13001 should be checked as follows:

$$\gamma_n \cdot \Sigma(\gamma_p \cdot f_i) \leq \frac{R_k}{\gamma_m}$$

where:
γ_n is the risk coefficient, where applicable
γ_p is the partial safety factor applied to individual load according to the load combination
f_i is the characteristic load i on the element including dynamic factors
R_k is the characteristic resistance of the material, particular element or connection
γ_m is the resistance coefficient

while for the allowable stress method, the proof of competence according to EN 13001 should be checked as:

$$\Sigma\, f_i \leq \sigma_a = \frac{R_k}{\gamma_f \cdot \gamma_n}$$

where:
f_i is the characteristic load i on the element including dynamic factors
R_k is the characteristic resistance of the material, particular element or connection
γ_f is the overall safety factor applied to the specified strength according to the load combination under consideration

γ_n is the risk coefficient, where applicable
σ_a is the allowable stress

FEM 1.001 uses the allowable stress method as it was already mentioned and only the designation is different compared to EN 13001. The allowable stress and overall safety factor (designated in FEM 1.001 as σ_a and ν_E respectively) are defined by the code depending on the stress state in a member and the applied load case. The code takes into account the mechanical characteristics of used steel. For case of steel where the ratio between the yield strength and the ultimate tensile strength is lower than 0.7 and for members subjected to simple tension or compression, the values of ν_E are provided in *Table 2.2*. Load cases I-III given in the code will be discussed later.

Table 2.2: Overall safety factor according to FEM 1.001

Load case	Case I	Case II	Case III
ν_E	1.5	1.33	1.1

Steel commonly in use, like S235, S275 and S355 produced in accordance with EN 10025-2 (non-alloy structural steels), EN 10021-1 (hot finished structural hollow sections of non-alloy and fine grain steels) and EN 10219-1 (cold formed welded structural hollow sections of non-alloy and fine grain steels) fulfill the condition $f_y/f_u<0.7$. As an illustration, *Table 2.3* is given.

Table 2.3: Steel commonly in use - yield and ultimate strength

Steel grade	f_y [N/mm^2]	f_u [N/mm^2]	f_y/f_u
S235	235	360	0.653
S275	275	430	0.640
S355	355	510	0.696

In the case of steels with high elastic limit ($f_y/f_u>0.7$), the use of the coefficient ν_E does not provide a sufficient level of safety, and a guidance for the calculation of the allowable stress is given in FEM 1.001 paragraph 3.2.1.1. 2).

For shear, the allowable stress is $\tau_a=\sigma_a/3^{0.5}$ (σ_a is the allowable tensile stress), while for a member subjected to combined loads, Von-Misses criterion is used and it is defined by the following formula: $\sigma_x^2+\sigma_y^2-\sigma_x\,\sigma_y+3\,\tau_{xy}^2<\sigma_a$.

Since the load cases are in relation with the proof of competence and the fact that this sub-chapter is dedicated to design codes, loads and load cases that have to be taken into account according to FEM 1.001 will be summarized here, while the application of the rules on the carriage structure is provided in sub-chapter 2.4.

Loads entering into the design of structures, specified by the code are:

- The principal loads exerted on the structure of the appliance, assumed to be stationary, in the most unfavorable state of loading
- Loads due to vertical motions
- Loads due to horizontal motions
- Loads due to climatic effects

The principal loads include the loads due to the dead weight of the components and the loads due to the working load (designated as S_G and S_L respectively). As regards loads due to vertical motions, they include loads due to hoisting of the working load and loads due to acceleration of the hoisting motion. Loads due to vertical motions are not relevant for the carriage structure under the study since it is not equipped by any hoisting device. Loads due to horizontal motion are the inertia effects due to acceleration, the effects of centrifugal force, transverse horizontal reactions resulting from rolling action and buffer effects. Wind load, snow load and temperature variations belong to the climatic effects and they are not relevant for the carriage structure due to the fact that the structure is located inside a building.

The code specifies three different load cases to be considered in the calculations, namely:
- Case I: Appliance working without wind
- Case II: Appliance working with wind
- Case III: Appliance subjected to exceptional loadings

A relation could be made now between the safety factor ν_E and the load cases. As long as the probability of occurrence is lower, the safety factor is lower. For instance, for Case III the value is 1.1. Similar philosophy is used in Eurocode 0 through the limit state method and partial safety factors for loading, where the combination of loads in persistent and transient design situations for ULS is $1.35G_{kj,sup}+1.5Q_{kj,inf}+1.5\psi_{0,i}5Q_{k,i}$ while the combination in accidental and seismic design situations for ULS is $G_{kj,sup}+A_d+(\psi_{1,1}$ or $\psi_{2,1})Q_{k,1}+\psi_{2,i}Q_{k,i}$.

2.3. Calculation assumptions

This sub-chapter is devoted to the calculation assumptions used by DREVER International in design of the initial solution, what is the basis for all further considerations.
- Actions on the carriage are calculated according to FEM 1.001.
- Combinations of actions are calculated according to FEM 1.001.
- Cross-section design checks are performed in accordance with FEM 1.001, while the joints are designed in accordance with Eurocode 3. In design of the optimized solution (chapter 4), Eurocode 3 is used for cross-section/member checks as well as for design of joints.
- The structure is exposed to the maximum environmental temperature of 80°C and it is free to expand.
- At the temperature of 80°C the mechanical properties are not altered. Hence, together with the previous assumption leads that the effect of the temperature can be neglected.
- The corrosion effect is not taken into account.
- The structure is located inside a building what means that the climate effects are not relevant.
- The seismic action is not considered.
- The number of stress cycles during the design working life of the structure is 20000, thus the fatigue assessment is not necessary.
- The maximum transfer speed is 4 m/min (an elevation and the lateral translation are not present).
- Four roller bogies transfer the vertical loading to the runway structure and allow the translational movement. Each roller bogie consists of two rollers and each is equipped by an engine.
- The caissons cannot swing since they are fixed to the structure by the supporting plates, while the forces coming from the accelerations/decelerations are taken into account, acting in the center of gravity of the caissons
- The deflection limit is l/1250 and the value is 7.17 mm
- Steel grade S235 is used for the structure and the mechanical properties are given in *Table 2.4.*

Table 2.4: Mechanical properties for steel grade S235

Yield strength	f_y=235 N/mm^2
Ultimate tensile strength	f_u=360 N/mm^2
Modulus of elasticity	E=210 000 N/mm^2
Shear modulus	G=81 000 N/mm^2
Poisson's coefficient	ν=0.3

Classification of the structure was discussed in sub-chapter 2.2 and here it will be applied to the carriage structure in accordance with FEM 1.001.

The number of stress cycles during the design working life of the structure is 20000 what leads to the utilization class **U1**. The structure is subjected to 100% of the imposed load during its use what results in the spectrum factor k_p equal to 1 and consequently the spectrum class is **Q4**. By combining these values, the appliance as a whole is classified as **A3**.

Respecting the classification, the amplifying coefficient is γ_c=1.05. Even though the structure does not have any hoisting device, the dynamic coefficient has to be taken into account with is minimum value, hence ψ=1.15.

2.4. Loads and combinations of loads

2.4.1. Dead weight (S_G)

It includes weight of the carriage structure itself and weight of the secondary structural assemblies, for example the platforms.

For the standard acceleration due to gravity a value of 9.81 m/s^2 is taken while the density of steel is ρ=7850 kg/m^2. In analysis of the initial solution, weight of the whole carriage assembly was taken automatically by the software in the model created by DREVER International. The platform assemblies are not within the scope of the Thesis, therefore their weight will be applied to the carriage structure at positions of their columns in further models following the scheme given in *Figure 2.6.*

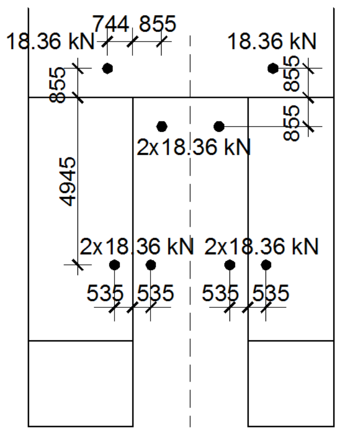


Figure 2.6: Loading from the platforms applied on the carriage structure

2.4.2. Working loads (S_L)

The structure is loaded by two caissons with the weight of 60t and 20t (see *Figure 2.7* for 3D view), two ducts with the weight of 5t each and the inductor. Loads from the inductor are transferred to the carriage through the platform structure (see *Figure* 2.8 for 3D view). Schemes (plan view) are given in *Figure 2.9* and *Figure 2.10*.

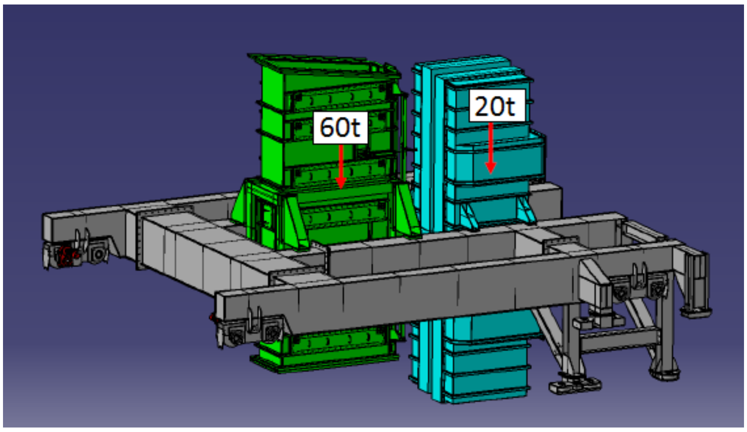


Figure 2.7: Loading from the caissons-3D view (source: Drever calculation sheets)

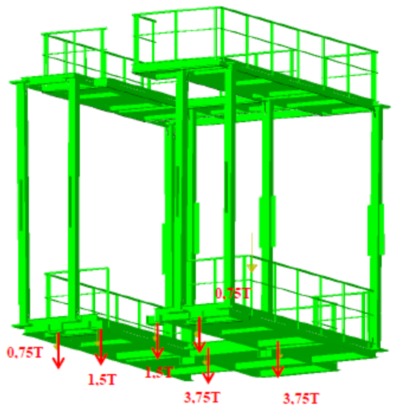


Figure 2.8: Loading from the inductor-3D view (source: Drever calculation sheets)

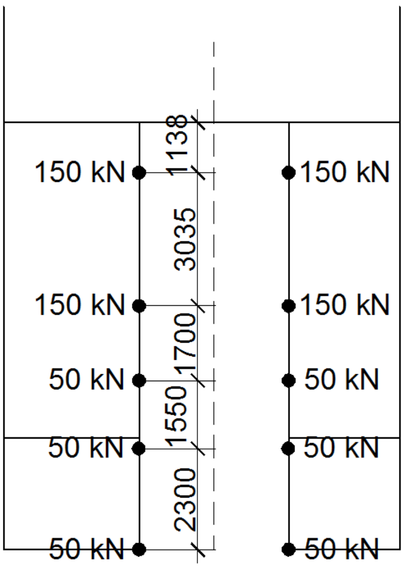


Figure 2.9: Loading (caissons and ducts)

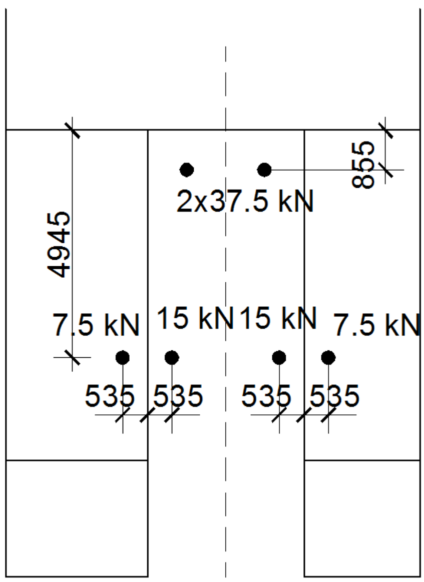


Figure 2.10: Loading (inductor)

2.4.3. Loads due to horizontal motions

The structure is intended to move with a speed of 4 m/min. According to FEM 1.001, for moderate and high speed appliances (normal application) the corresponding acceleration of 0.5 m/s^2 should be taken into account. The acceleration is applied on the whole carriage, on the caissons at their center of gravity, on the ducts, platforms and inductor.

On the caisson, the inertial force I acts at its center of gravity and when we move the force on the top edge of the beam, we have an additional moment M, that is further resolved to a couple of forces V. Finally, the horizontal force I and the vertical forces V act on the structure. Step-by-step procedure is given in *Figure 2.11*, and the values in *Table 2.5*. For the platforms, ducts and inductor see *Table 2.6*.

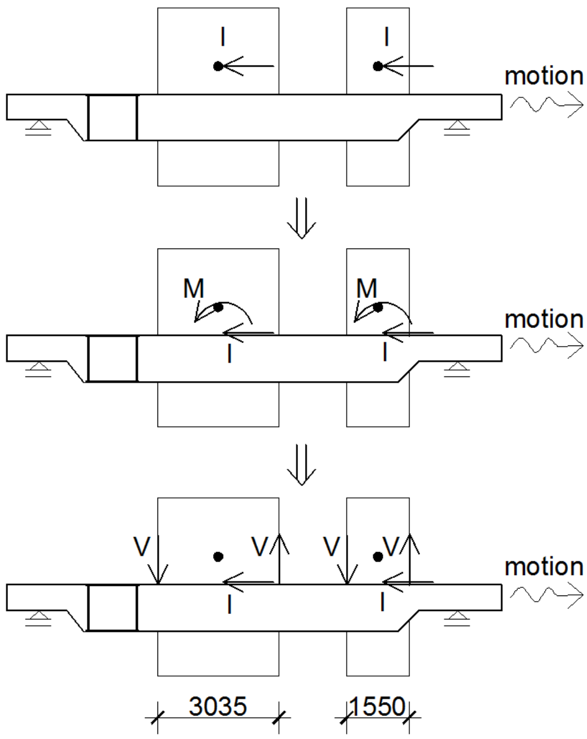


Figure 2.11: Inertial forces due to horizontal motions: step-by-step procedure

Table 2.5: Caissons - inertial forces

	m [kg]	I [N]	e [m]	M [Nm]	d [m]	V [N]	V_1 [N]	V_1 [kN]
Caisson 60t	60000	30000	0.361	10830	3.035	3568.4	1784.2	1.78
Caisson 20t	20000	10000	0.4	4000	1.55	2580.6	1290.3	1.29

where
I is the inertial force ($I=m\cdot a$)
e is the distance between center of gravity of the caisson and the top edge of the beam
d is the distance between the supporting plates
V_1 is the vertical force acting on one beam ($V_1=V/2$)

Table 2.6: Inertial forces

	m [kg]	I [N]	I [kN]
Duct 5t (per duct)	5000	2500	2.5
Platform (per column)	1836	918	0.92
Carriage structure	28062	14031	14.03
Inductor	12000	6000	6

The inertial forces of the carriage structure itself are applied as uniformly distributed loading along the structural members.

Buffer effects on the structure are neglected according to FEM 1.001 because the horizontal speed of the appliance is lower than 0.7 m/s.

The structure does not slew, hence the effects of centrifugal force does not have to be taken into account. These effects are relevant only for jib cranes.

Transverse reactions due to rolling action are only important for the runway structure which is not within the scope of this Thesis.

2.4.4. Other actions

Loads due to vertical motions are not present as the structure does not have any hoisting device. Loads due to climatic effects are not relevant, because the structure is located inside a building. According to FEM 1.001, for gangways and platforms intended only for access of personnel the load (S_P) of 1.5 kN/m^2 should be taken into account, but these loads are not to be used in the calculations for girders (only for design of platform structures which are out of the scope of the Thesis).

2.4.5. Combinations of loads

As it was already mentioned in sub-chapter 2.2, the code specifies three load cases to be considered in the calculations. Case II and Case III do not have to be considered as long as the structure is not exposed to wind actions neither to exceptional actions. Possible combinations of loads that are taken into account (corresponding to Case I) are indicated in *Table 2.7*.

Table 2.7: SLS and ULS load combinations

SLS	ULS
$X_{1,SLS}=\gamma_c\cdot S_G$	$X_{1,ULS}=1.5\cdot X_1$
$X_{2,SLS}=\gamma_c\cdot(S_G+\psi\cdot S_L)$	$X_{2,ULS}=1.5\cdot X_2$
$X_{3,SLS}=\gamma_c\cdot(S_G+\psi\cdot S_L+S_{Hx+})$	$X_{3,ULS}=1.5\cdot X_3$
$X_{4,SLS}=\gamma_c\cdot(S_G+\psi\cdot S_L+S_{Hx-})$	$X_{4,ULS}=1.5\cdot X_4$
$X_{5,SLS}=\gamma_c\cdot(S_G+\psi\cdot S_P)$	$X_{5,ULS}=1.5\cdot X_5$

As it is obvious from *Table 2.7*, the unique safety coefficient with a value of 1.5 was used for all loads in the calculations of the initial model performed by DREVER International and the same will be used for the all further solutions that will be analyzed upon the agreement with the DREVER representatives. This approach is given in FEM 1.001 and allowed for the use by EN 13001 for the case of bridge cranes, what is explained in detail in sub-chapter 2.2.

Load combinations X_3 and X_4 are basically the same and the only difference between them is that the inertial force S_{Hx} acts in opposite directions, dependent on the direction of motion. Load combination X_5 is relevant only for the platforms which will not be analyzed in the further proposed solutions.

2.5. Results

Table 2.8 is an overview of the results taken from the calculation sheets done by DREVER International.

Table 2.8: Initial model - summary of the results

Load combination	Local stress [MPa]	Nominal stress [MPa]	Deflections [mm]
X_1	69.17	<100	2.11
X_2	155.9	<100	7.33
X_3 and X_4	170.8	<100	7.33

It should be mentioned that the stresses given in *Table 2.8* are Von-Misses equivalent stresses and they are based on the characteristic values of acting loads, hence the safety coefficient is included through the allowable stress value, which is in this case 235/1.5=156.66 MPa. The nominal stresses were limited to 100 MPa (resulting from the characteristic values of acting loads) by the request of the customer, while the local stresses were limited to 156.66 MPa.

The local stress resulting from load combinations X_3 and X_4 has a peak value of 170.8 MPa, what means that the limiting value was exceeded in a point. The above-mentioned stress is located at the position where the cross-section is changing its height (right/left supporting beam, close to the space reserved for the roller bogie). It may appear as a result of singularities of the finite elements model. Since the stress is very localized and all other values in this zone are less than 156.66 MPa, this was considered as acceptable by the designer. A graphical interpretation of that zone is given in *Figure 2.12*.

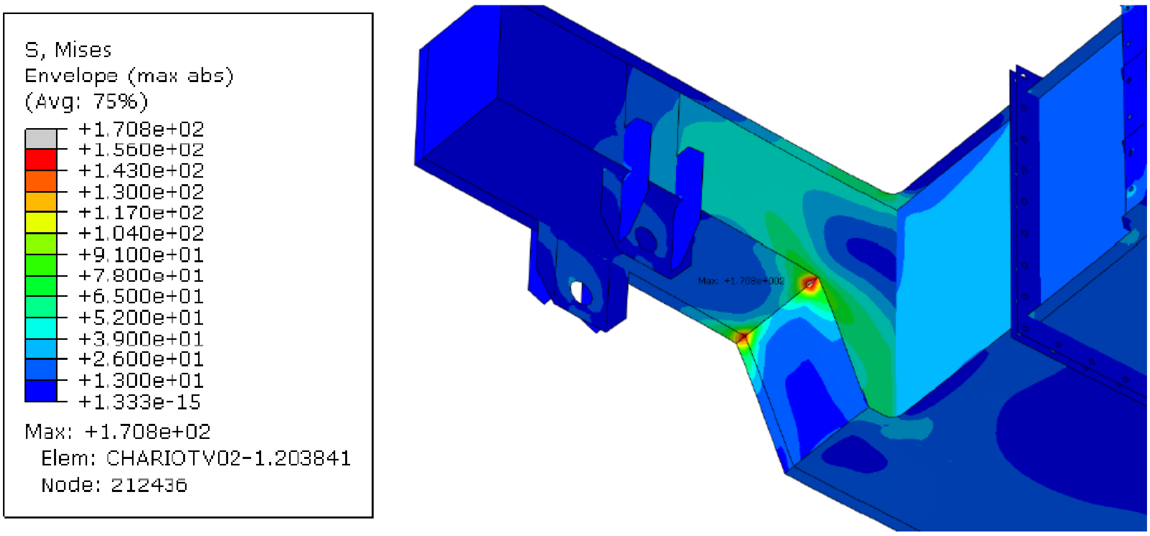


Figure 2.12: Local stresses (Von Mises) for load combination X_3/X_4 (source: Drever caculation sheets)

The overall distribution of stresses in the carriage structure for load combinations X_1-X_4 and for the characteristic values of acting loads, taken from the calculations report is represented in *Figure 2.13*, *Figure 2.14* and *Figure 2.15.*

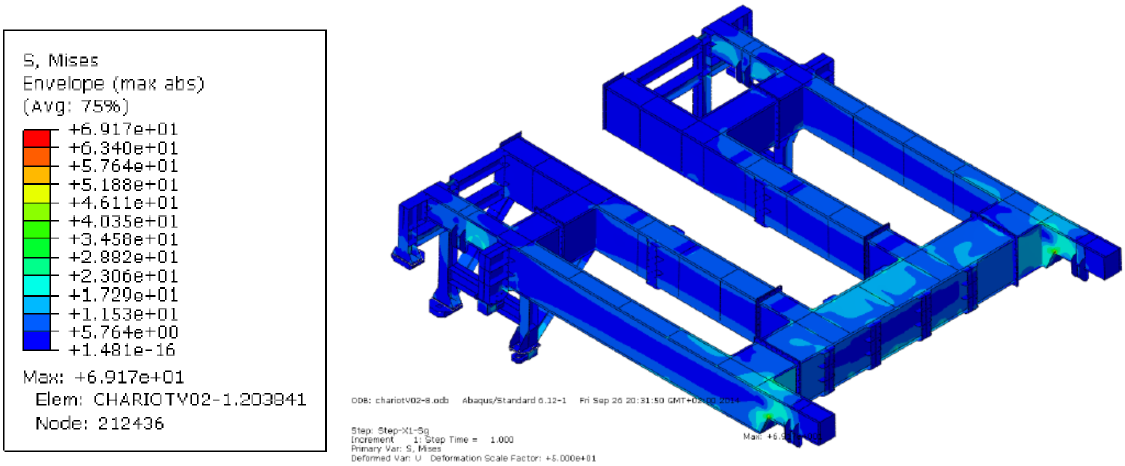


Figure 2.13: Overall distribution of stresses for load combination X_1 (source: Drever calculation sheets)

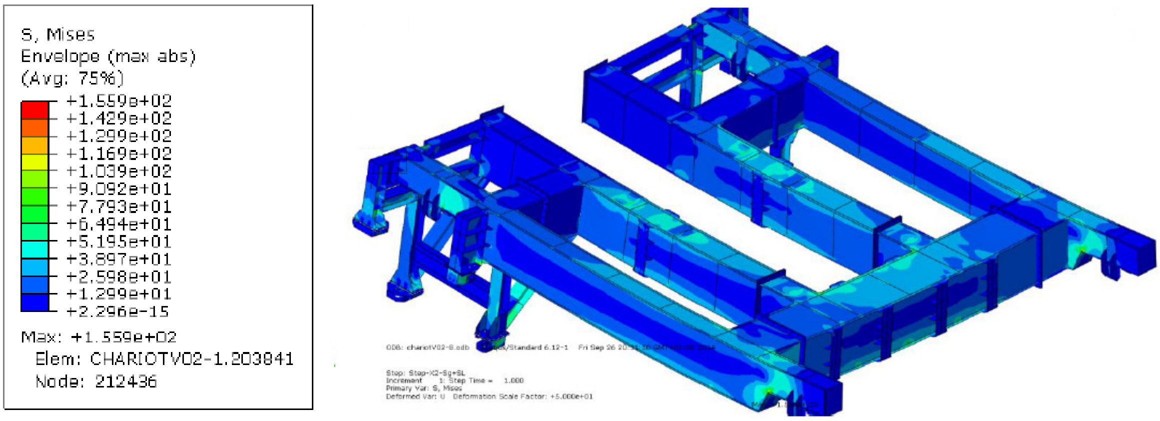


Figure 2.14: Overall distribution of stresses for load combination X_2 (source: Drever calculation sheets)

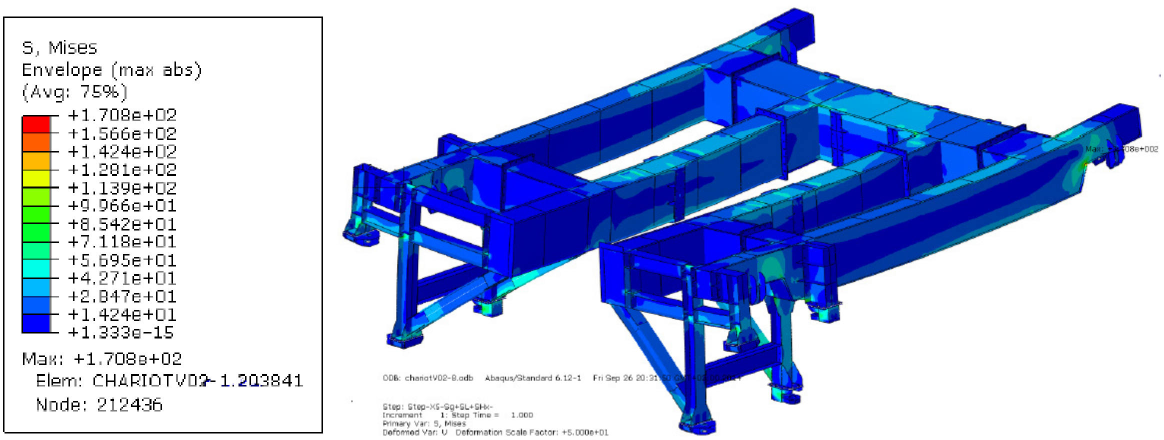


Figure 2.15: Overall distribution of stresses for load combination X_3/X_4 (source: Drever calculation sheets)

As it is obvious from the given values, defections are governing for design of the structure. The deflection limit is 7.17 mm (see sub-chapter 2.3) and even though the limit was slightly exceeded, it was considered as acceptable in the calculations report from DREVER

International. A picture of the deformed model (load combination X3) can be seen in *Figure 2.16*.

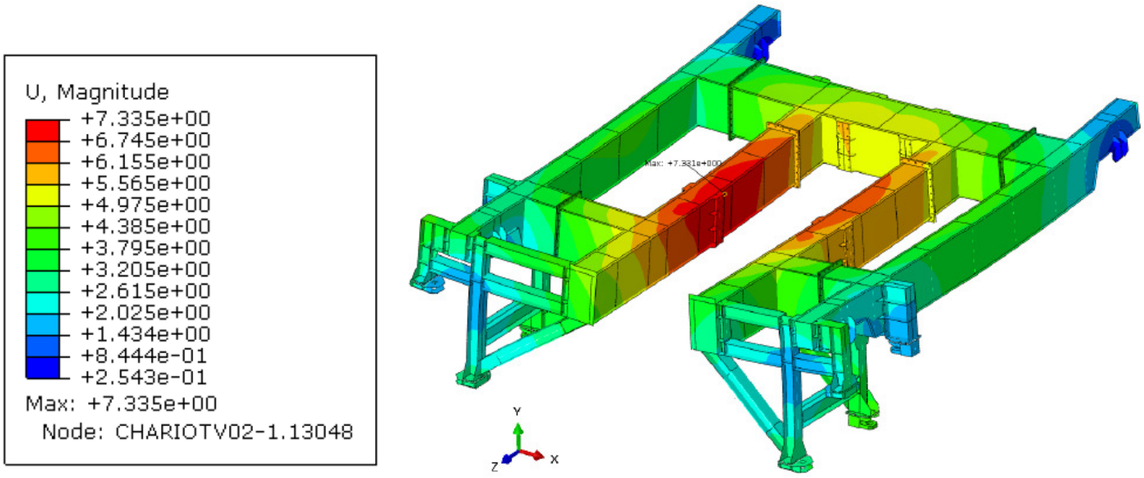


Figure 2.16: Deformed model of the structure for load combination X_3 (source: Drever calculation sheets)

For the producer, weight of the structure is one of the most important aspects and together with the complexity of structural details and joints governs the final price. Thus, the weight of the carriage structure, taken from the drawings provided by DREVER International, is summarized in *Table 2.9* and presented separately for each part of the structure that is transported to the construction site as an assembly. The weight of each assembly consists of the weight of a structural member itself plus the weight of all additional parts welded to a member, for instance end plates, transverse diaphragms, sub-assemblies intended to fix roller bogies to the structure, sub-assemblies for fixing the platform columns, etc. *Table 2.9* will be used later as a basis for the development of a new concept.

Table 2.9: Initial structure - material specification

N	Member	Weight each [kg]	Quantity	Weight total [kg]
1	Cross-back	6649.84	1	6649.84
2	Right side member	5026.03	1	5026.03
3	Left side member	4811.48	1	4811.48
4	Right supporting beam	6339.74	1	6339.74
5	Left supporting beam	6339.74	1	6339.74
6	Left connection beam	187.47	2	374.94
7	Right connection beam	253.52	2	507.04
8	Outer connection beam	143.26	2	286.52
9	Transversal connection beam	255.46	2	510.92
10	Right inner tensioner bracket	636.22	1	636.22
11	Left inner tensioner bracket	632.40	1	632.40
12	Right tensioner bracket reinforcement	228.12	1	228.12
13	Right carrying wheel bracket reinforcement	219.86	1	219.86
14	Left tensioner bracket reinforcement	174.93	1	174.93
15	Left carrying wheel bracket reinforcement	165.38	1	165.38
16	Splint inner reinforcement	8.57	16	137.12
17	Splint inner reinforcement	12.94	16	207.04
18	Right inner carrying wheel bracket	604.41	1	604.41

19	Left inner carrying wheel bracket	604.41	1	604.41
20	Outer carrying wheel bracket	418.93	2	837.86
21	Outer tensioner bracket	190.57	2	381.14
22	Outer tensioner bracket support	216.64	2	433.28
23	Bridle	79.86	2	159.72
			Total	36268.14

2.6. Model for the calibration and development of a new concept

For structural analysis of the carriage structure, a finite element model was created by DREVER International and all above-mentioned results are related to that model. The model was composed of shell finite elements with reduced integration (S4R and S3R) and the average size of elements is 30 mm. The calculations were carried out in Abaqus Standard v6.12.

As it is obvious, the model is complex. The structural analysis in the pre-design stage of future solutions has to remain simple, due to the facts that many uncertainties are present before the detailed design stage and a numerical model is constantly subjected to corrections. On the other hand, a numerical model in pre-design has to be accurate enough in order to supply the designer with reliable data and allow him to make proper assumptions for the detailed design.

With the aim to obtain the relation between the initial model (done by DREVER) and the pre-design models for new (optimized) solutions of the carriage structure, a finite element model has been created that resembles the model used by DREVER International. It is named Model for the calibration. The model is composed of linear (beam) finite elements, which will be used for all further models as well.

A comparison between these two models has to be made here. Deflections are the governing design criterion for this structure, as it can be seen in sub-chapter 2.5 and it is used for the calibration. Consequently the serviceability limit states check will be used for the selection of feasible solutions. As a simplification, the load combinations that will be considered here are only X_1 and X_2.

The following simplifications have been introduced in Model for the calibration:

- Instead of the haunched beam, the right/left side member has been modeled using two different cross-sections with a constant height, one corresponding to the cross-section in the middle of the beam and another one corresponding to the reduced cross-section. Their axes are placed in the same plane, and a sudden change of the cross-section properties are present in a node.
- Outer carrying wheel brackets have been omitted, because they are very stiff and short, and the corresponding node on the side member has been rigidly connected to the horizontal support.
- Working loads (weight of the caissons) have been modeled as point loads, while in the reality and in the initial model, they are introduced through plates of certain dimensions (uniformly distributed load on the area).

Model for the calibration is graphically presented in *Figure 2.17*.

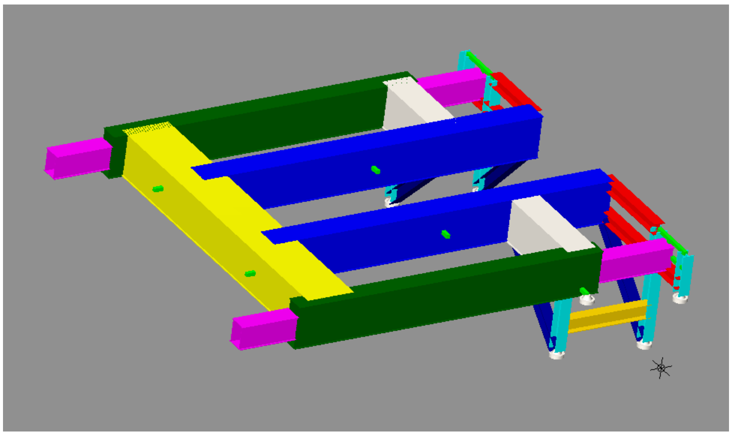


Figure 2.17: Model for the calibration - 3D view

A comparison should be made between these two models in terms of the vertical displacements. The values are given in *Table 2.10*, while a graphical representation of the deformed model under loading is given in *Figure 2.18*. The maximum displacement is reached in the middle of the right side member.

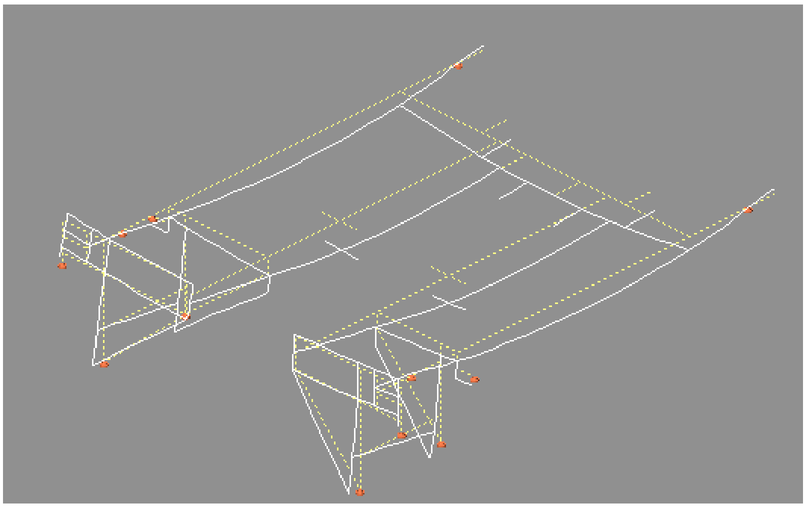


Figure 2.18: Deformed model of the structure

Table 2.10: Displacements

Load case/combination	Initial model z [mm]	Model for the calibration z [mm]
S_G	2.01	1.32
S_L	4.33	4.25
$X_1=\gamma_c \cdot S_G$	2.11	1.39
$X_2=\gamma_c \cdot (S_G+\psi \cdot S_L)$	7.335	6.52

From the results it is obvious that the displacements resulting from the working loads are almost identical (4.33 mm and 4.25 mm), what means that Model for the calibration is reliable approximation of the behavior of the structure, although it is simpler and modeled using liner(beam) finite elements instead of shell finite elements. The difference in the displacements resulting from the self-weight of the structure is due to the fact that the initial model includes end plates, transverse diaphragms, sub-assemblies intended to fix roller bogies to the structure, sub-assemblies for fixing the platform columns what increases the weight, while Model for the calibration takes into account only the weight of structural members itself.

Due to the above-mentioned facts, the deflection limit that will be used for the estimation of all future models will be **6.85 mm** instead of 7.17 mm, in order to compensate simplifications that have been used in computational models.

The vertical reactions resulting from the dead weight $R(S_G)$ will be used in further models for the estimation of the weight of the structure, while the horizontal reactions for the estimation of shear forces and consequently torque imposed on the runway beams. The values are given in *Table 2.11*, *Table 2.12* and the designation in *Figure 2.19*.

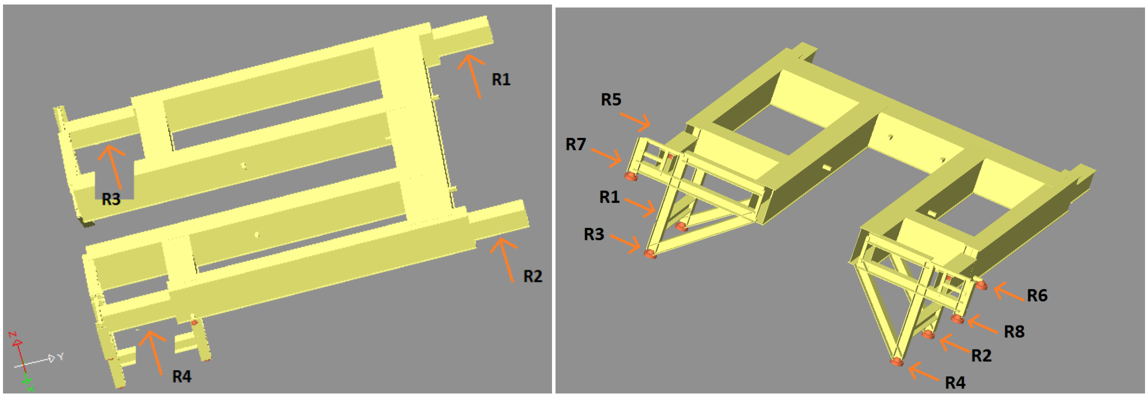


Figure 2.19: Vertical and horizontal reactions - designation

Table 2.11: Vertical reactions [kN]

$R_1(S_G)$	93.93
$R_2(S_G)$	100.92
$R_3(S_G)$	117.51
$R_4(S_G)$	114.09
Σ	426.45

The weight of the platforms is 146.88 kN and when it is subtracted from the sum of reactions the resulting value is the weight of the structure itself in the computational model.

426.88-146.88=279.57 kN

Table 2.12: Horizontal reactions $R(X_{2,ULS})$ [kN]

R_1	R_2	R_3	R_4	R_5	R_6	R_7	R_8
420.65	349.21	188.84	157.72	-452.59	-373.79	-158.00	-134.24

The brackets have strong importance on the overall behavior of the structure since they provide a vertical support to the right/left side member on the motor side of the structure. Their importance can be seen when they are removed and the finite element model is calculated again. Obtained results are given in *Table 2.13*, while a graphical representation of the model in *Figure 2.20*.

Table 2.13: Displacements [mm]

$z(S_G)$	4.33
$z(S_L)$	18.97
$z(X_1)$	4.54
$z(X_2)$	27.5

Without the brackets, the maximum displacement is 27.5 mm, what is almost four times more than the allowed value.

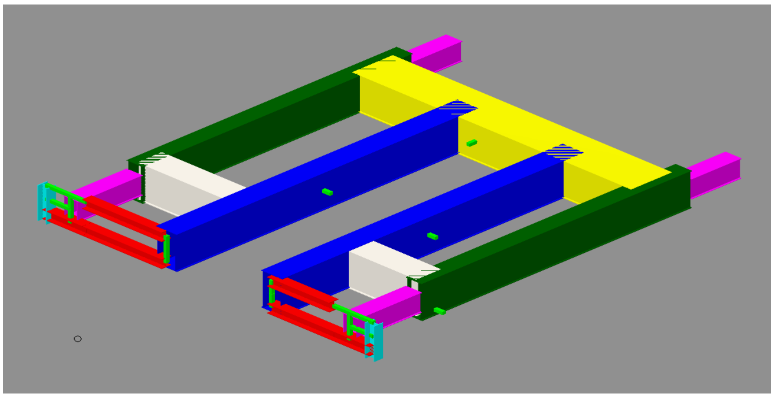


Figure 2.20: Model without the brackets - 3D view

3. Pre-design of proposed solutions

The aim of this chapter is to propose solutions for the improvement of the initial solution on the pre-design level. Proposed solutions will be studied parametrically in order to select the best solution to be studied in detail.

Generally speaking, a structure can be optimized in such a way to:

- Reduce weight of the structure,
- Simplify structural joints and details,
- Reduce the number of specific parts/assemblies (rollers, bogies, etc.),
- Increase the amount of work to be done in a workshop and therefore to reduce the amount of work on a construction site,
- Intend to perform welding in a workshop rather than on a site,
- Increase the size of parts to be transported up to the limits specified by the traffic regulations,
- Use commercial open or hollow sections rather than built-up sections,
- Intend to have as much as possible uniform members in a structure in order to speed up the production and to reduce the cost,
- Limit the thickness of steel plates on a value within the limitations of producer's equipment for cutting, drilling, etc.,
- Reduce reactions on a substructure.

All these factors will be taken into account for the development of new solutions, although neither of solutions is able to suit all of them together. Through the parametric studies benefits and drawbacks of each solution will be studied as well as the extent of fulfilling the above-mentioned factors, what will provide essential data to select the best solution.

As a simplification, the load combinations that will be considered in the pre-design are only X_1 and X_2.

3.1. Solution 1

3.1.1. General layout

In this solution, the right/left supporting beam and connecting beam remain the same as they are in the initial solution. The right and left supporting members are in form of truss girders with parallel chords. As an illustration *Figure 3.1* is given. Considering the positions of applied loads (working loads, see *Figure 2.9*), and their spacing, it is obvious that the bending moments in the upper chord are inevitable. The brace members are considered as pin-connected to the chords and the chords as continuous beams. The vertical distance between the chords (axis-to-axis) is 1265 mm and the top edge of the upper chord is aligned with the top edge of the connecting beam. The bottom chord extends below the connecting beam and it can be easily fixed to its bottom flange (for instance using angles welded to the chord and bolted to the upper flange) what is obvious from the bottom view, given in *Figure 3.2*.

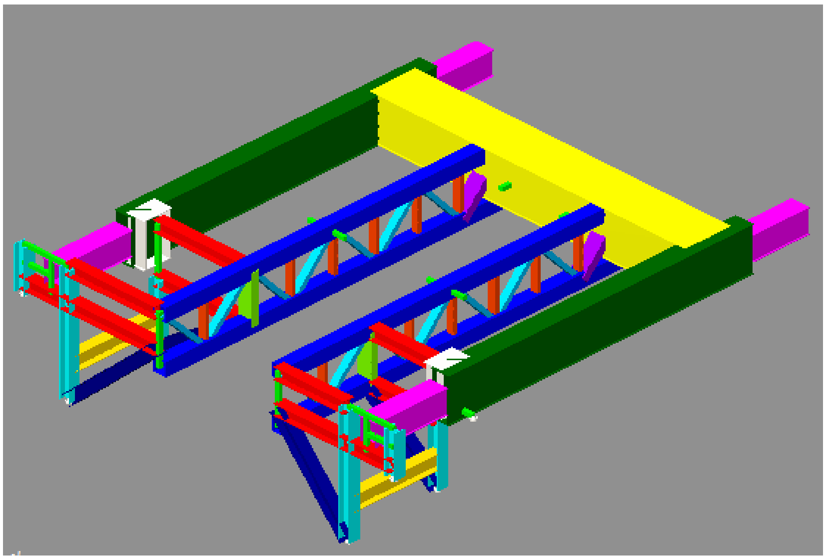


Figure 3.1: Solution 1- 3D view, top side

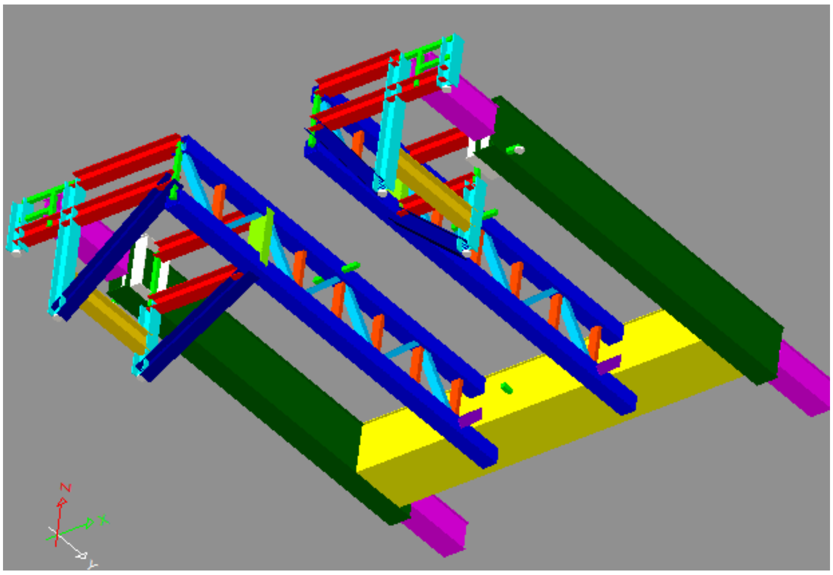


Figure 3.2: Solution 1 - 3D view, bottom side

A lighter cross-section is used to connect the side members and the supporting beams, where instead of box cross-sections welded I shape profiles are used (red colored beams in *Figure 3.1*). Considering the fact that members are modeled by their axes, and the size of the connecting beam, the diagonal next to connecting beam (purple colored in *Figure 3.2*) can be avoided by a small re-arrangement of other braces. This does not have importance on the overall behavior and it will not be contemplated more in the pre-design phase. Cross-sections that are used for the truss girder are given in *Table 3.1*.

Table 3.1: Cross-sections, right/left side member

Chords	SHS 350x350x12.5
Diagonal braces	RHS 200x120x6
Vertical braces	RHS 200x200x4

3.1.2. Results, parametric studies and remarks

As it was already explained on Model for the calibration, the vertical displacements are the governing criterion for the design. The values for Solution 1 are indicated in *Table 3.2* and the maximum vertical displacement is equal to the limiting value (6.85 mm).

Table 3.2: Solution 1 - vertical displacements [mm]

$z(S_G)$	1.27
$z(S_L)$	4.57
$z(X_1)$	1.33
$z(X_2)$	6.85

The vertical reactions are indicated in *Table 3.3*. Compared to Model for the calibration the difference is 18.57 kN what results in the saving of steel for 1821.7 kg and expressed in percentages, around 6.5%. The final saving could be slightly higher, because Model for the calibration does not take into account the weight of transverse diaphragms of built-up box which are not structural parts of the truss girder. The weight reduction could be higher if a higher truss is applicable, because the stiffness increases as a power function with an increment of the height, while it increases linearly with an enlargement of the cross-section area.

Table 3.3: Solution 1 - vertical reactions [kN]

$R_1(S_G)$	92.42
$R_2(S_G)$	98.94
$R_3(S_G)$	109.65
$R_4(S_G)$	106.87
Σ	407.88

The horizontal reactions, given in *Table 3.4* have almost the same value as they are in the initial solution, hence there is no change in applied shear forces and torque to the runway beam.

Table 3.4: Solution 1 - horizontal reactions $R(X_{2,ULS})$[kN]

R_1	R_2	R_3	R_4	R_5	R_6	R_7	R_8
428.49	361.25	181.65	146.29	-447.26	-371.70	-166.60	-139.57

In order to prove that the ultimate limit states are not the governing criterion, *Table 3.5* shows the maximum values of elastic stresses (ULS) and *Table 3.6* the buckling reduction factor for structural members under compression. For instance, the maximum normal stress in the upper chord is 115.1 MPa what is only 49% of the yield strength. Reduction due to buckling is slight, less than 10%. Bending of the upper chord does not have important influence on the design, because the serviceability limit state is governing.

Table 3.5: Solution 1 - stress $\sigma(X_{2,ULS})$, absolute values [MPa]

Upper chord	115.10
Bottom chord	79.43
Diagonal braces	101.63
Vertical braces	53.36

Table 3.6: Solution 1 - Buckling reduction factor χ

	Diagonal brace	Vertical brace
L_{cr} [mm]	1750	1265
λ	35.50	29.91
χ	0.91	0.94

The main advantages are the weight reduction and a possibility for more savings by increasing the height of the truss. A fabricator can take the advantage of Solution 1 if he prefers to do more cutting and to weld braces rather than doing the longitudinal welds of a built-up box (where is sometimes difficult to satisfy the tolerances and to reach acceptable level of quality).

3.2. Solution 2

3.2.1. General layout

Solution 2 comprises of built-up girders where the right/left supporting beam and connecting beam remain the same as they are in the initial solution. The right/left side member form a bowstring structure together with a pillar and ties. This structure allows the cross-section of the right/left side member to be reduced. To obtain a certain level of uniformity and to speed up the production, the same cross-section is adopted as it is used for the right/left supporting beam. The column and the ties can be easily connected to the beam on a site because they are considered as pin-connected, hence simple bolted connections working in shear can be used. Due to the fact that columns for the platforms already exist on the structure, they might be utilized as an integral part of the bowstring structure. The influence of the pillar and ties on the overall behavior will be studied parametrically. Compared to the initial solution, this structure has less roller bogies for the horizontal reactions (four instead of eight) and less brackets which are intended to support the structure and transfer loads to the horizontal roller bogies. This reduction has influence on the final cost since these parts are expensive. An overview of the structural layout is given in *Figure 3.3*.

In the finite element model cross-sections that have been used for the pillar and the ties are given in *Table 3.7* while all other are inherited from the initial solution.

Table 3.7: Cross-sections (pillar and tie)

Pillar	CHS 273x10
Tie	CHS 273x12.5

The height of the pillar is 4060 mm measured from the axis of the right/left side member to the axis of the tie, what means that the pillar reaches the top edge of the handrail on the upper platform (level +14700). A scheme is given in *Figure 3.4* (red color-existing columns, blue color-ties).

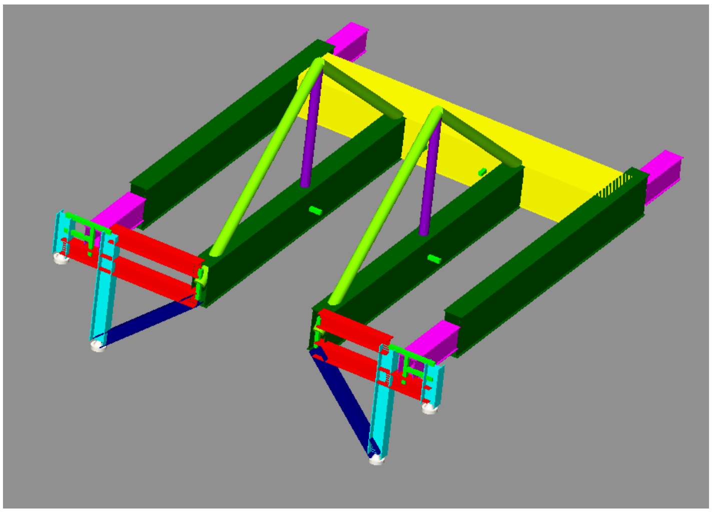


Figure 3.3: Solution 2 - 3D view

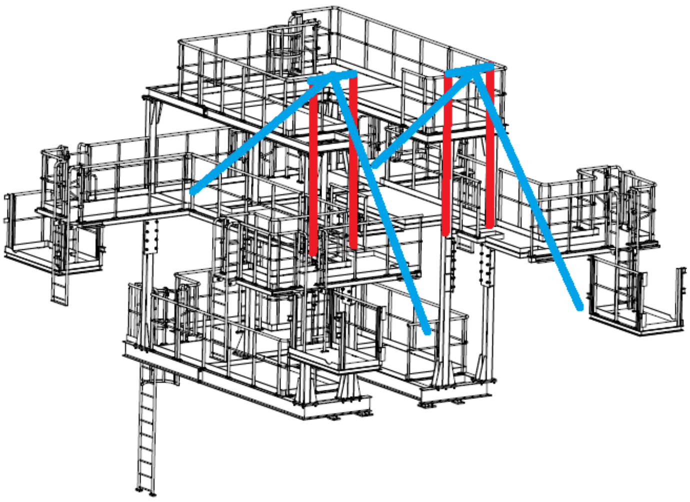


Figure 3.4: Existing columns and ties (source: Drever calculation sheets)

3.2.2. Results, parametric studies and remarks

The maximum values of the vertical displacements are given in *Table 3.8* and they are within the limit (6.85 mm).

Table 3.8: Solution 2 - vertical displacements [mm]

$z(S_G)$	1.31
$z(S_L)$	4.52
$z(X_1)$	1.38
$z(X_2)$	6.83

The vertical reactions are represented in *Table 3.9*. Compared to Model for the calibration the difference is 23.19 kN what results in the saving of steel for 2274.94 kg and expressed in percentages, around 8%. Additionally, a certain amount of material could be saved by the utilization of the existing platform columns (in the finite element model the pillar has been modeled using CHS 273x10). The fact that this solution has smaller number of roller bogies has to be accounted for the final cost.

Table 3.9: Solution 2 - vertical reactions [kN]

$R_1(S_G)$	93.56
$R_2(S_G)$	102.30
$R_3(S_G)$	103.96
$R_4(S_G)$	103.44
Σ	403.26

The horizontal reactions are given in *Table 3.10*. The increased value of the horizontal reactions can be regarded as a drawback of this solution. They act on the runway beams, for example on one beam by maximum shear action of R_3=475.17 kN, R_7=-469.12 kN and torque of $T(R_3,R_7)$ =864.26 kNm compared to the meximum reactions from the initial solution R_1=420.65 kN, R_5=-452.59 kN and torque of $T(R_1,R_5)$ =797.77 kNm what means that the acting torque is 8% higher and it can influence the dimensions of the runway beams, although the runway beams itself are beyond the scope of this Thesis.

Table 3.10: Solution 2 - horizontal reactions $R(X_{2,ULS})$[kN]

R_1	R_2	R_3	R_4	R_5	R_6	R_7	R_8
/	/	475.17	397.36	/	/	-469.12	-391.31

Regarding the ultimate limit states, *Table 3.11* shows the maximum values of elastic stresses (ULS) and it is obvious that the ultimate limit states are not governing for the design. For instance, the maximum value in *Table 3.11* is 73.88 MPa, what is only 31% of the yield strength.

Table 3.11: Solution 2 - stress $\sigma(X_{2,ULS})$, absolute values [MPa]

Right side member	42.78
Left side member	44.63
Connecting beam	73.88
Pillar	52.74
Tie	41.42

It is worthy to be mentioned that the design buckling resistance of the pillar, which is loaded in compression, is only slightly lower than the design ultimate resistance of its cross-section. This is proved by calculating the reduction factor, what is given in *Table 3.12*.

Table 3.12: Solution 2 - Buckling reduction factor χ

	Pillar
L_{cr} [mm]	4060
λ	43.61
χ	0.86

The ties are placed in the vertical plane that coincides with the axis of the side member and they pass through the platforms area and may limit the accessibility for maintenance workers.

A case study analyzing the influence of the stiffness of parts of the bowstring structure on the overall behavior has been conducted. Variable parameters are the area of the pillar and tie. For the study, their cross-sectional areas are multiplied by 0.25, 0.5, 0.75, 1, 2, 4 and 10 and lastly selected to be infinitely stiff. The influence is analyzed through the resulting deflections (load combination X_2). The initial values of cross-sectional area are indicated in *Table 3.13*.

Table 3.13: Cross-sectional area (pillar and tie)

Pillar	CHS 273x10	A_p=8262 mm^2
Tie	CHS 273x12.5	A_t=10230 mm^2

First, the cross-sectional area of the tie is set as a constant and the cross-section of the pillar is subjected to the variation. The resulting values are illustrated in *Table 3.14*.

Table 3.14: Vertical displacements (cross-section of the pillar is variable)

Multiplier	A_p [mm^2]	$z(X_2)$ [mm]
0.25	2065.5	7.65
0.50	4131	7.16
0.75	6196.5	6.95
1	8262	6.83
2	16524	6.65
4	33048	6.59
10	82620	6.66
Infinite	/	6.32

Second, the cross-sectional area of the pillar is set as a constant and the cross-section of the tie is subjected to the variation. The resulting values are illustrated in *Table 3.15*.

Table 3.15: Vertical displacements (cross-section of the tie is variable)

Multiplier	A_t [mm^2]	$z(X_2)$ [mm]
0.25	2557.5	7.89
0.50	5115	7.30
0.75	7672.5	7
1	10230	6.83
2	20460	6.57
4	40920	6.51
10	102300	6.78
Infinite	/	6.02

If both members are selected as infinite stiff, the corresponding deflection is 5.28 mm what is theoretically the lowest value and it can be further influenced only by changing the stiffness of the beam.

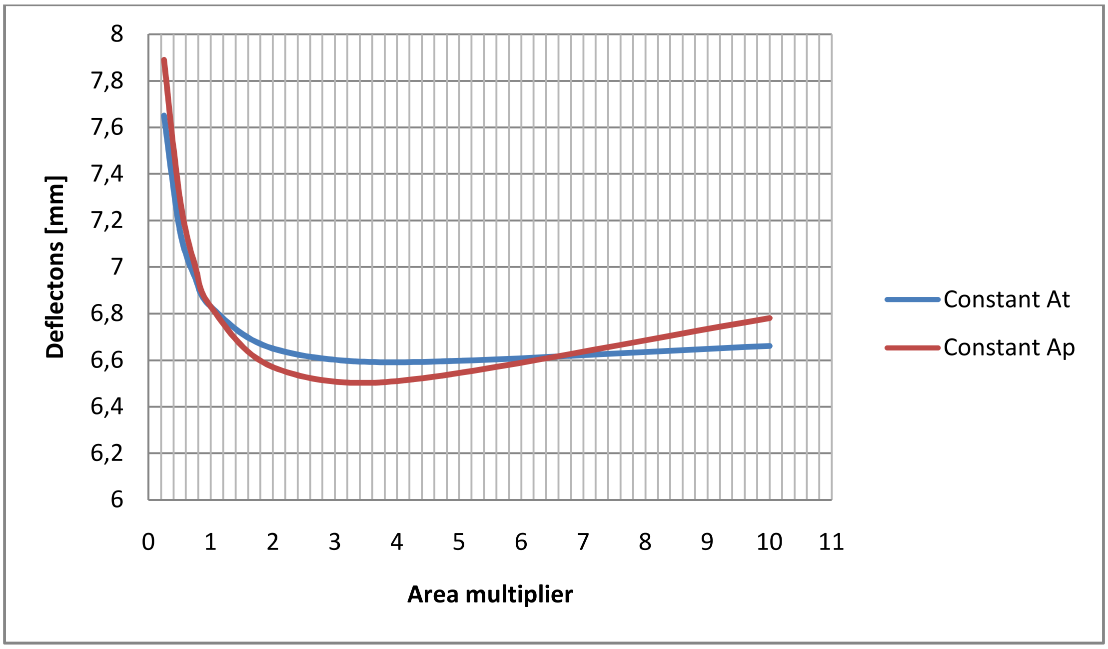


Figure 3.5: Diagram area multiplier vs deflections [mm]

Figure 3.5 represents a graphical interpretation of *Table 3.14* and *Table 3.15*. As it can be seen from the graph, any significant enhancement of the selected cross-section properties cannot provide a benefit in terms of the overall behavior of the structure. For instance, if the tie is two times stiffer, the deflections are lower for only 0.18 mm while the cost is considerably higher. If the cross-sectional area of the tie is multiplied more than three times, the displacements will be even higher (red line on the graph) due to the fact that the weight of the tie is increased at the same time (the analysis has been conducted for load combination X_2 where figures the dead weight). It can be concluded that the appropriate cross-section properties were selected.

A case study related to the position of the bowstring members has been conducted as well. The position of three nodes have been shifted according to *Figure 3.6*. In each iteration only one node is shifted while other two remain on their initial positions.

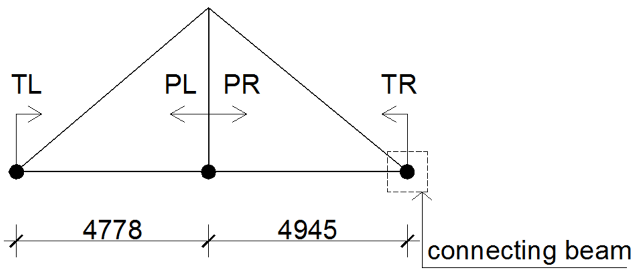


Figure 3.6: Horizontal shifting of nodes - designation

Table 3.16: Node shift [m] vs vertical deflections [mm]

Node shift [m]	Deflections - max. values [mm]			
	TL	TR	PL	PR
0.5	6.95	7.04	6.97	6.76
1	7.14	7.28	7.14	6.84
1.5	7.38	7.55	7.36	7.03
2	7.68	7.87	7.61	7.29

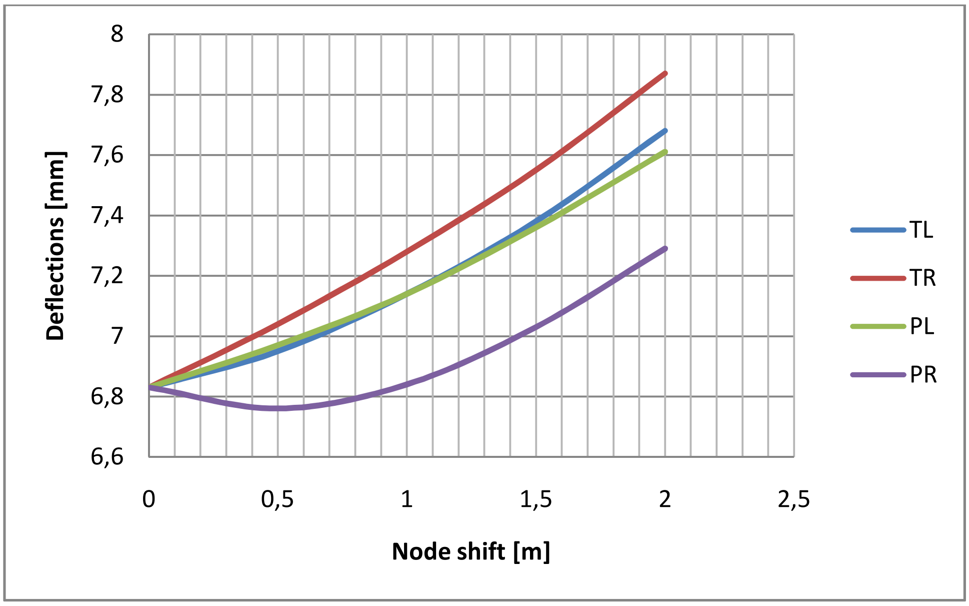


Figure 3.7: Diagram node shift [m] vs deflections [mm]

Figure 3.7 shows a graphical representation of *Table 3.16*. As it is obvious, the bowstring structure provides the highest rigidity to the system if the ties are anchored at the ends of the beam (a value of the node shift in this case is 0 m). By shifting the pillar to the right (PR=0.5 m) a slightly smaller values of the deflections can be obtained, what results mainly because of the positions of applied loads. This shift has not been included in the proposed solution because the initial position (PR=0 m) corresponds to the position of the platform column that can be utilized as a part of the bowstring structure.

The influence of the column height has been analyzed by shifting its top node downwards according to *Figure 3.8*.

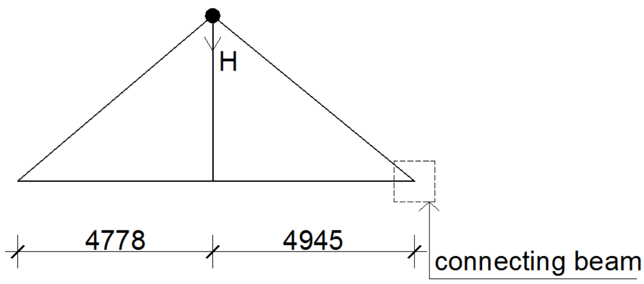


Figure 3.8: Vertical shifting of nodes - designation

Table 3.17: Node shift [m] vs vertical deflections [mm]

Node shift [m]	Deflections [mm]
0.5	6.87
1	6.95
1.5	7.12
2	7.41
2.5	7.87

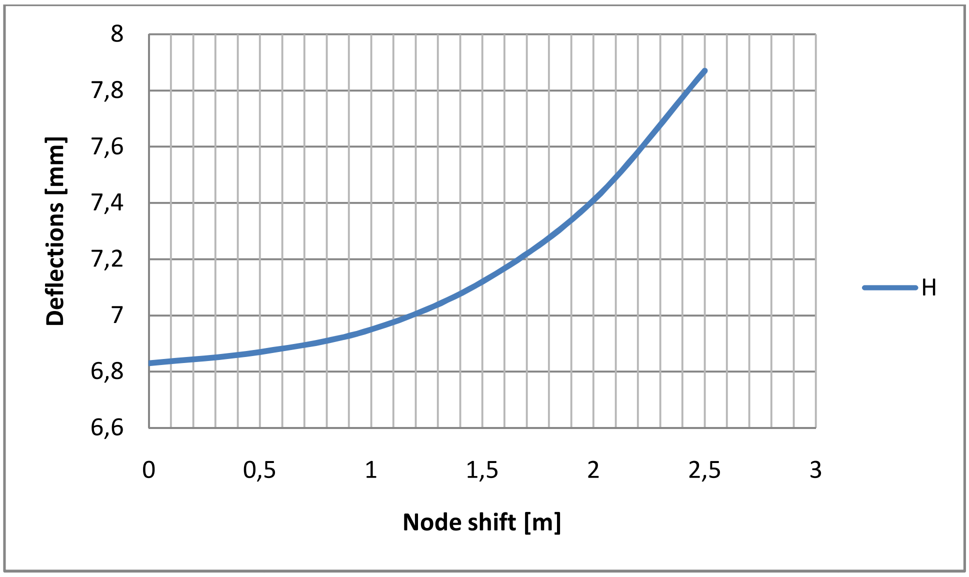


Figure 3.9: : Diagram node shift [m] vs deflections [mm]

Figure 3.9 shows a graphical representation of *Table 3.17*. By shortening the pillar the structure loses its rigidity. It can be explained on the level of internal forces in the tie.

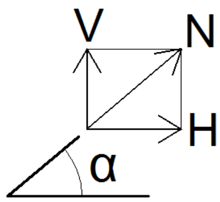


Figure 3.10: Resolving force N to the horizontal and vertical component

When the tensile force N in the tie, shown in *Figure 3.10*, is resolved to its horizontal and vertical component as follows:

$$H = N \cdot \cos \alpha$$
$$V = N \cdot \sin \alpha$$

As the angle α decreases, the sinus function gives a smaller value and consequently a smaller value of the vertical component. At the same time by lowering the angle the horizontal component gets a higher value. Since the overhanging side of the structure needs to be supported in the vertical direction in order to decrease the deflections, the vertical component of the tensile force should be utilized as much as possible what means that a higher value of the angle α has the positive effect on the overall behavior of the structure. In addition, a higher value of the horizontal component acts adversely to the beam because it introduces higher compressive forces to the beam, what can cause stability problems. In the proposed solution the angle has been selected as the highest possible since the pillar reaches the top edge of the handrail on the upper platform. Generally, the angle may be increased by shifting the pillar towards the overhanging side of the structure, but the shift does not affect the overall behavior as it was already explained, because the pillar is farther from the connecting beam which acts as a support.

A possible modification of the right/left side member that satisfies the deflection criterion ($z(X_2)$=6.88mm and may be regarded as satisfactory) could be obtained by using a hot rolled profile HEB1000 instead of the built-up box girder (right and left side member) and by using CHS 273x16 instead of CHS 273x12.5 for the tie. The vertical reactions in total are 404.08 kN, what is only 0.82 kN more compared to the basic Solution 2. In this case, the beam may be subjected to stability problems, because I sections are more prone to lateral-torsional buckling, axial (compression) forces are introduced to the beam from the tie and the buckling length is 9723 mm (λ_z=152).

3.3. Solution 3

3.3.1. General layout

Solution 3 is a variant of Solution 2 and comprises of built-up girders and a bowstring structure. The bowstring structure has a different configuration compared to Solution 2. It consists of two pillars and a polygonal tie (slope-horizontal-slope). The bowstring structure in this case is more stiff what allows the beam cross-sections to be reduced, mainly by means of reducing the thickness of plates. An overview of the structural layout is given in *Figure 3.11* and *Figure 3.12*. Beside the different cross-sections used for the side members and the supporting beams, all other considerations are remaining from Solution 2, like the use of four roller bogies for the horizontal reactions (instead of eight used in the initial solution) and a corresponding number of brackets. The height of the pillars is 4060 mm measured from the axis of the right/left side member to the axis of the tie. This solution cannot utilize the existing platform columns because their positions does not coincide to the positions of the pillars. The horizontal part of the tie passes through the platforms area and may limit the accessibility for maintenance workers what means that the platforms might be rearranged.

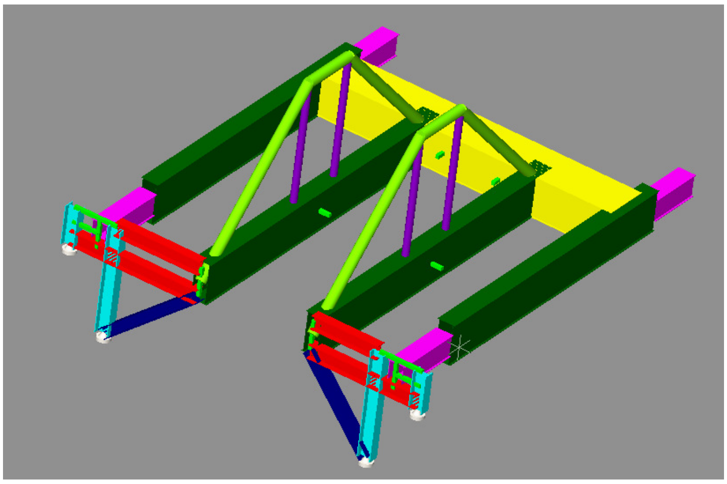


Figure 3.11: Solution 3 - 3D view

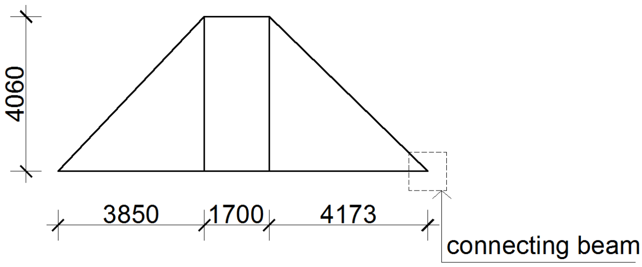
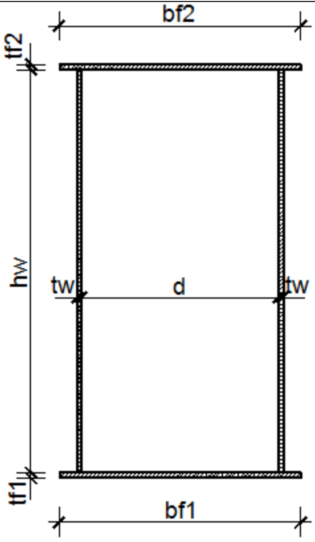


Figure 3.12: Solution 3 - 2D view (elevation)

In the finite element model cross-sections that have been used for the structural members are given in *Table 3.18* and *Table 3.19*.

Table 3.18: Solution 3 - cross-sections



	Beam	b_{f1} [mm]	t_{f1} [mm]	b_{f2} [mm]	t_{f2} [mm]	h_w [mm]	t_w [mm]	d [mm]
Initial solution	Right and left side member	680	15	680	15	1135	12	556
	Right and left supporting beam	524	15	524	15	1135	12	400
	Right and left supporting beam (on the supports)	524	25	524	15	600	12	400
Solution 3	Right and left side member	524	12	524	12	1135	10	400
	Right and left supporting beam	524	12	524	12	1135	10	400
	Right and left supporting beam (on the supports)	524	15	524	15	600	12	400

Table 3.19: Solution 3 - cross-sections (pillar and tie)

Pillar	CHS 219.1x10
Tie	CHS 273x12.5

3.3.2. Results, parametric studies and remarks

The maximum values of the vertical displacements reached in the structure are provided in *Table 3.20* and they are not beyond the limiting value (6.85 mm)

Table 3.20: Solution 3 - vertical displacements [mm]

$z(S_G)$	1.31
$z(S_L)$	4.55
$z(X_1)$	1.37
$z(X_2)$	6.85

The vertical reactions are represented in *Table 3.21*. Compared to Model for the calibration the difference is 41.25 kN what results in the saving of steel for 4046.63 kg and expressed in terms of percentages, approximately 15%.

Table 3.21: Solution 3 - vertical reactions [kN]

$R_1(S_G)$	89.60
$R_2(S_G)$	98.08
$R_3(S_G)$	99.10
$R_4(S_G)$	98.42
Σ	385.20

Table 3.22 presents the horizontal reactions. The values are almost identical to the values obtained for Solution 2. They are higher than the values in the initial solution what can be considered as a drawback of this solution. The difference of torque acting on the runway beam is approximately 9% (871.76 kNm compared to 797.77 kNm).

Table 3.22: Solution 3 - horizontal reactions $R(X_{2,ULS})$[kN]

R_1	R_2	R_3	R_4	R_5	R_6	R_7	R_8
/	/	479.10	399.97	/	/	-473.40	-394.26

Regarding the ultimate limit states, *Table 3.23* shows the maximum values of elastic stresses (ULS) and similarly to Solution 2 the ultimate limit states are not governing for the design. For instance, the maximum value in *Table 3.24* is 73.02 MPa, what is only 31% of the yield strength.

Table 3.23: Solution 3 - stress $\sigma(X_{2,ULS})$, absolute values [MPa]

Right side member	48.35
Left side member	53.16
Connecting beam	73.02
Pillar	41.97
Tie	43.08

Similarly to Solution 2, the pillars are not influenced much by stability problems what confirms the buckling reduction factor calculated in *Table 3.24*.

Table 3.24: Solution 3 - Buckling reduction factor χ

	Pillar
L_{cr} [mm]	4060
λ	54.86
χ	0.79

Generally, the bowstring configuration used for Solution 3 can be regarded as more efficient than the configuration used for Solution 2 what is obvious from the weight of the structure. This can be explained on a simple example. An illustration is provided in *Figure 3.13*. If we assume the whole bowstring structure working as a sole cross-section with variable properties along its axis, the moment of inertia varies from I_{beam} to $I_{max.}$ For Solution 2, the maximum value (I_{max}) is obtained only in one cross-section, above the pillar, while the configuration that uses two pillars has the maximum moment of inertia on complete length between the pillars, what is the reason of Solution 3 being more rigid. On the other hand, Solution 3 requires more joints to connect the members. The bowstring structure might limit the accessibility for maintenance workers and overlap with some equipment.

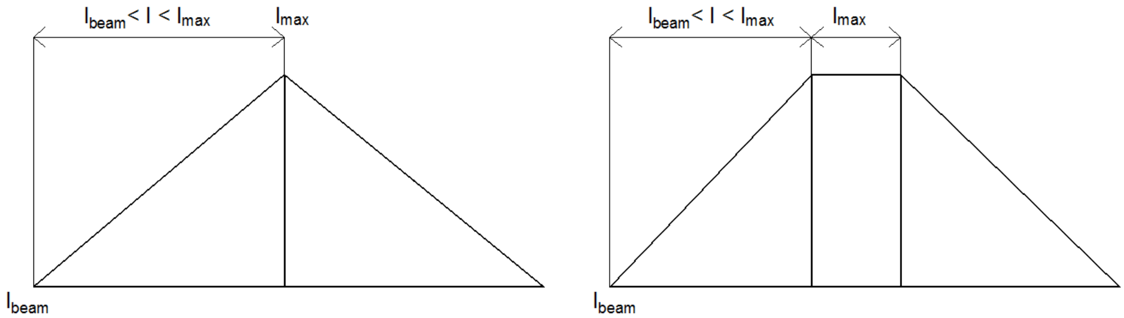


Figure 3.13: Moment of inertia - bowstring structure

3.4. Solution 4

3.4.1. General layout

This solution shows a completely different idea compared to previous three solutions. It is composed of two box trusses mutually connected by a planar truss perpendicular to them. The box truss has trapezoidal cross-section with four chords. Its upper chords are considerably stiffer than the bottom chords because the loading is imposed between nodes. As a simplification compared to previous solutions, this structure does not contain any welded built-up member, but on contrary it requires more labor and equipment to construct the joints. The structure consists of square hollow sections used as chords and circular hollow sections used as braces. Circular hollow sections have been selected due to the reason that the braces are spatial members and it is easier in this case to connect CHS to the chord than SHS (see *Table 3.25*). Three different cross-section sizes are selected for braces in order to save the material. Generally, according to the structural mechanics, a trapezoidal shape composed of simple connected bars is not in-plane stiff. To obtain the in-plane stiffness, an additional diagonal might be used, what is applied here as well, rather than obtaining the frame effect what would be quite difficult in this case. The vertical distance between the chords (axis-to-axis) is 2750 mm. A graphical overview of the structure is given in *Figure 3.14* and the dimensions in *Figure 3.15*. The positions of the vertical supports are the same as they are from previously proposed solutions. The horizontal roller bogies are placed at the edge nodes of the truss, what is illustrated in *Figure 3.16* with the designation. The structure could be almost entirely prepared in a workshop. The size of one box-truss complies with the traffic limitations, they could be transported to the site one by one and there interconnected by the connecting truss. As a result, the majority of the necessary work is moved to a workshop, what increases the quality and speeds-up the construction.

In the finite element model, the chords are regarded as continuous and the braces as pin-connected.

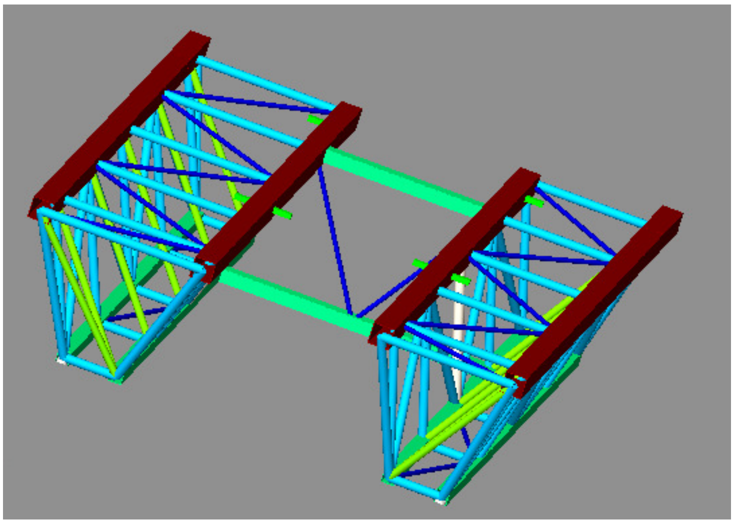


Figure 3.14: Solution 4 - 3D view

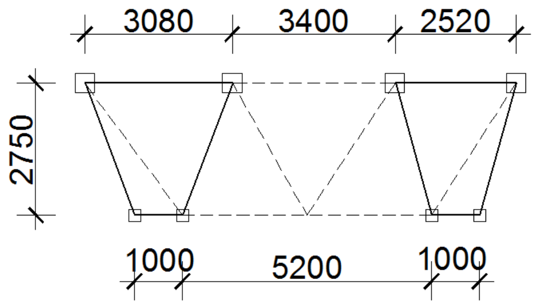


Figure 3.15: Solution 4 - dimensions

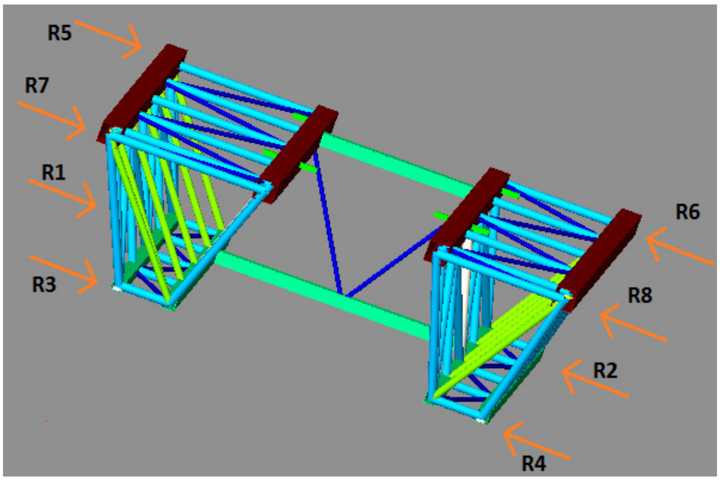


Figure 3.16: Solution 4 - horizontal reactions (designation)

Table 3.25: Solution 4 - cross-sections

Upper chord	SHS 400x400x12.5
Bottom chord	SHS 250x250x10
Brace (type 1)	CHS 88.9x6.3
Brace (type 2)	CHS 168.3x6
Brace (type 3)	CHS 168.3x8

3.4.2. Results, parametric studies and remarks

As it was done previously, the deflection criterion is the first to be mentioned. The maximum obtained deflection for Solution 4 is 6.77 mm, what is within the limits and the values for all load cases/combinations are given in *Table 3.26*. The vertical displacements resulting from the dead weight are lower here than for the initial solution, what is the first sign that the structure has higher stiffness-to-weight ratio.

Table 3.26: Solution 4 - vertical displacements [mm]

$z(S_G)$	1.07
$z(S_L)$	4.68
$z(X_1)$	1.12
$z(X_2)$	6.77

The structure is significantly lighter than the initial and all other previously proposed solutions. In numbers, the sum of the vertical reactions resulting from the dead weight is 330.06 kN (see *Table 3.27*), what is 96.39 kN lower in comparison to model for the calibration. This means that Solution 4 allows approximately 10 t of steel to be saved. On the other hand, the production of this structure is more expensive since the joints are complex. A compromise between the labor cost and the saving of steel should be assessed by the manufacturer.

Table 3.27: Solution 4 - vertical reactions [kN]

$R_1(S_G)$	85.87
$R_2(S_G)$	88.51
$R_3(S_G)$	78.13
$R_4(S_G)$	78.09
Σ	330.06

Table 3.28 presents the horizontal reactions resulting from load combination X_2. The values are lower than the values calculated in Model for the calibration, what is one more advantage of this solution. The maximum horizontal reaction calculated here is for 55.12 kN lesser than the maximum from model for the calibration. This may result in the reduction of cross-section properties used for the runway beams, what leads to an additional saving.

Table 3.28: Solution 4 - horizontal reactions $R(X_{2,ULS})$[kN]

R_1	R_2	R_3	R_4	R_5	R_6	R_7	R_8
217.37	185.40	397.47	328.47	-211.17	-178.97	-395.22	-326.46

Table 3.29 is an overview of the maximum values of elastic stresses (ULS). As it is obvious, the elastic stress reaches 53% of the yield strength what means that the ultimate limit states are not governing for the design. It is worthy to be mentioned that the range of stresses is higher than it was in previous solutions (usually it was 30-40% of the yield strength), what means that this structure utilizes better its structural members and provide enough stiffness at the same time. The buckling reduction factor for some members is given in *Table 3.30*. The buckling length here is taken equal to the system length although a reduction is possible. Compared to other proposed solutions, this is more susceptible to stability problems because its structural members are more slender, thus some of them may need to be strengthened

during the detailed design (for example, the brace in the horizontal plane next to the horizontal support).

Table 3.29: Solution 4 - stress σ($X_{2,ULS}$), absolute values [MPa]

Upper chord	103.82
Bottom chord	57.60
Internal braces (in-plane stiffeners)	124.50
Horizontal plane braces	99.66
Inclined plane braces	115.47

The direction of the internal braces (in-plane stiffeners) has been selected in such a way that they are loaded only in tension.

Table 3.30: Solution 4 - Buckling reduction factor χ

	Horizontal plane brace	Inclined plane brace
Cross-section	CHS 88.9x6.3	CHS 168.3x8
L_{cr} [mm]	4041	3871
λ	29.91	68.27
χ	0.32	0.71

A feasibility of this solution is mainly dependent on design and construction of joints. Some of them are very complex due to the fact that the structure is spatial and in certain nodes 4-6 braces are connected (an example given in *Figure 3.17*). As it is presented, three braces in the horizontal plane are connected to the chord, three braces in the inclined plane and one internal brace as well. This requires a precise production and a special equipment to cut the members properly, for example a laser cutter. Joints should be designed according to EN 1993-1-8, which provides a limited amount of rules for design of multiplanar joints (only for the most common configurations). For design of joints that are not covered by EN 1993-1-8, a finite element analysis may be used what is sometimes time consuming and requires an advanced knowledge.

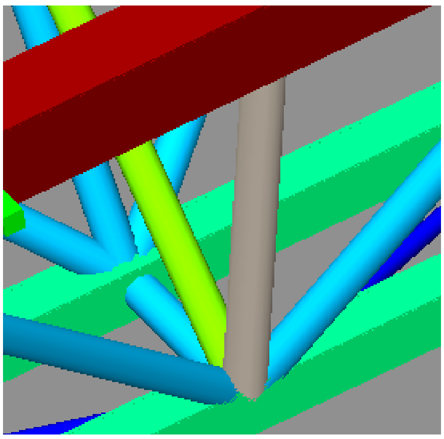


Figure 3.17: Solution 4 - example of a complex joint

The structure itself does not overlap with any secondary assembly, what is illustrated in *Figure 3.18*. The drag chain assembly, which was just below the carriage and supported by the runway frame, should be moved downwards, below the bottom chords of the carriage assembly. As an advantage of Solution 4 is also the fact that the structure is accessible from all sides and it allows some equipment to pass through the structure if it is necessary, what is not the case when box girders are used.

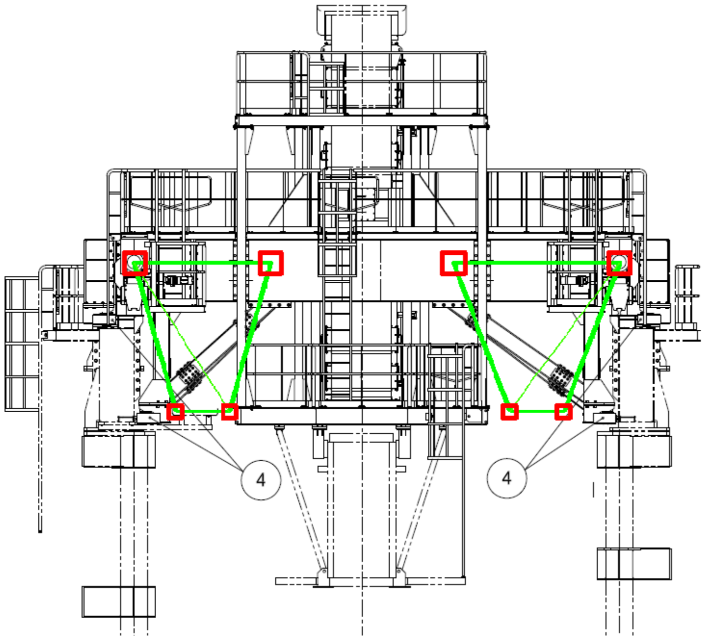


Figure 3.18: Solution 4 - complete carriage assembly (source: Drever calculation sheets)

3.5. Solution 5

3.5.1. General layout

Solution 5 has been developed using a similar concept as it was used for Solution 4. The structure consists of two triangular box trusses mutually connected by a planar truss perpendicular to them. A triangular structure composed of three members mutually connected by means of pinned joints is in-plane stiff, what was not the case for a trapezoidal shape. This allows Solution 5 to have lesser number of brace members than Solution 4. For the upper chords square hollow sections are selected because they are exposed to the applied loads between nodes.. The bottom chord has to support the horizontal roller bogies at its ends and to provide the stiffness in the horizontal plane, therefore a rectangular hollow section is used, oriented with its higher dimension in the horizontal plane. Braces are composed of circular hollow sections in order to simplify the construction of spatial joints. Two different sizes of cross-sections are used for the braces. The selected profiles are previewed in *Table 3.31*. The vertical distance between the chords (axis-to-axis) is 2750 mm. A graphical illustration of the structural layout is given in *Figure 3.19* and *Figure 3.20*. Supports, both the horizontal and vertical are on the same positions as they are in Solution 4 and the same designation will be used here (see *Figure 3.16*). Similarly to Solution 4, the structure can be prepared in a workshop, transported to the site truss-by-truss and there interconnected by the connecting truss, what increases the quality and speeds-up the construction as it was already mentioned.

In the finite element model, the chords are considered as continuous and the braces as pin-connected.

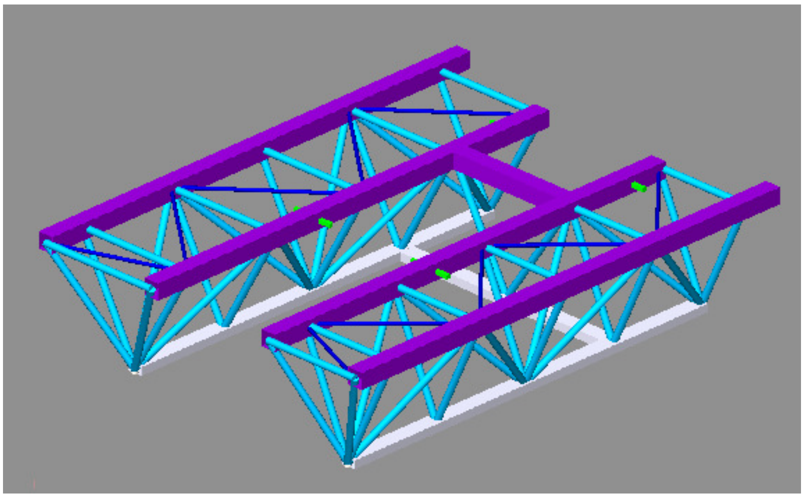


Figure 3.19: : Solution 5 - 3D view

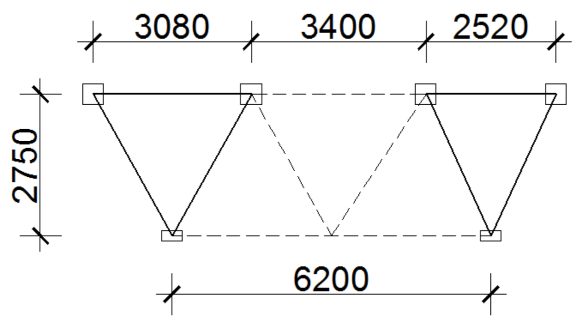


Figure 3.20: Solution 5 - dimensions

Table 3.31: Solution 5 - cross-sections

Upper chord	SHS 400x400x12.5
Bottom chord	RHS 400x200x10
Brace (type 1)	CHS 88.9x5
Brace (type 2)	CHS 168.3x10

3.5.2. Results, parametric studies and remarks

The maximum values of the vertical displacements are given in *Table 3.32* and they are within the limit (6.85 mm).

Table 3.32: Solution 5 - vertical displacements [mm]

$z(S_G)$	1.07
$z(S_L)$	4.73
$z(X_1)$	1.12
$z(X_2)$	6.84

The sum of the vertical reactions is 320 kN as it can be seen in *Table 3.33*. Compared to model for the calibration, the difference in the vertical reactions resulting from the dead weight is 106.45 kN, what means that Solution 5 is around 38% lighter than the initial solution. This leads to the saving of 10.5 t of steel. The amount could be higher but that can be estimated only after the detailed design, since Model for the calibration does not take into account the weight of some secondary parts of the structure (for example diaphragms) that are not structural part of a truss girder. On the contrary, Solution 5 requires more labor for the production compared to solutions composed of built-up box girders and the joints are more complex as well.

Table 3.33: Solution 5 - vertical reactions [kN]

$R_1(S_G)$	83.63
$R_2(S_G)$	86.00
$R_3(S_G)$	75.31
$R_4(S_G)$	75.06
Σ	320.00

The horizontal reactions resulting from load combination X_2 are given in *Table 3.34*. They are slightly higher than the values calculated in Solution 4, but still are lower than the values obtained in Model for the calibration, what can be regarded as an advantage of Solution 5, because it can lead to the reduction of cross-section properties of the runway structure.

Table 3.34: Solution 5 - horizontal reactions $R(X_{2,ULS})$[kN]

R_1	R_2	R_3	R_4	R_5	R_6	R_7	R_8
241.20	212.23	405.95	336.08	-183.44	152.88	-384.54	-316.27

The elastic normal stresses (ULS), resulting from load combination X_2 calculated for the most loaded structural members are given in *Table 3.35*. Their values are of the same range as the values calculated for Solution 4 and the same remarks are applicable here. The maximum utilization ratio is 0.49, what means that only 49% of the yield strength is exploited. To show the influence of stability problems on the design resistance of a member, the buckling reduction factors are given in *Table 3.36*. The buckling length here is taken equal to the system length despite a reduction is possible. The diagonal brace next to the horizontal support might need to be strengthened during the detailed design due to the buckling.

Table 3.35: Solution 5 - stress $\sigma(X_{2,ULS})$, absolute values [MPa]

Upper chord	115.52
Bottom chord	64.66
Horizontal plane braces	82.64
Inclined plane braces	114.89

Table 3.36: Solution 5 - Buckling reduction factor χ

	Horizontal plane brace	Inclined plane brace
Cross-section	CHS 88.9x5	CHS 168.3x10
L_{cr} [mm]	4041	4096
λ	136.06	73.01
χ	0.33	0.68

Solution 5 has a lower number of braces compared to Solution 4, what is an advantage from the manufacturing point of view and may make this structure more feasible. Some joints became simpler, like the joint given in *Figure 3.21*, where two braces are connected to the bottom chord. On the contrary, some of them are still complex, what is illustrated in *Figure 3.22*, where six brace members (two diagonals and one vertical in each of the inclined planes) are connected to the bottom chord in a node. A re-arrangement of the braces may simplify the complexity by decreasing number of members connected together in a node.

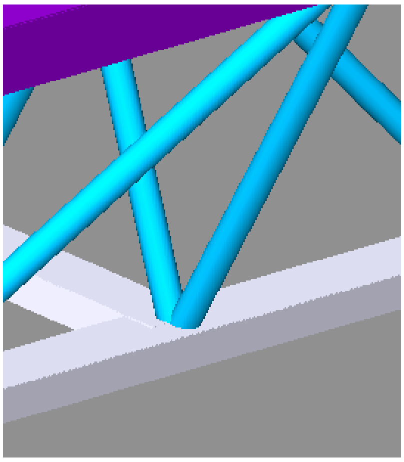


Figure 3.21: Solution 5 - example of a simple joint

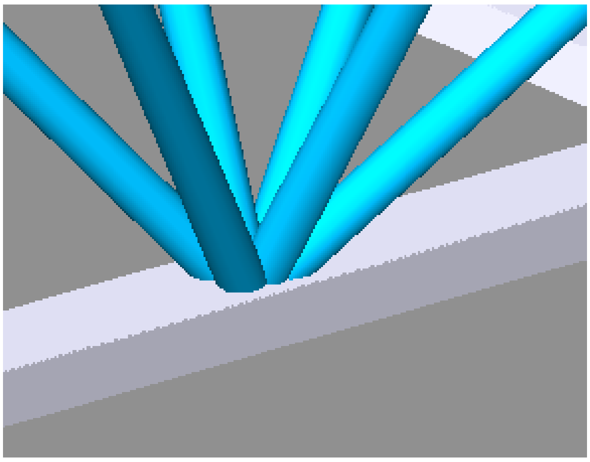


Figure 3.22: Solution 5 - example of a complex joint

Similarly to Solution 4, in Solution 5 the structure itself does not overlap with any sub-assembly of the carriage, what is displayed in *Figure 3.23* and can be considered as an advantage. The drag chain assembly, supported by the runway frame should be moved downwards as it was mentioned for Solution 4 as well.

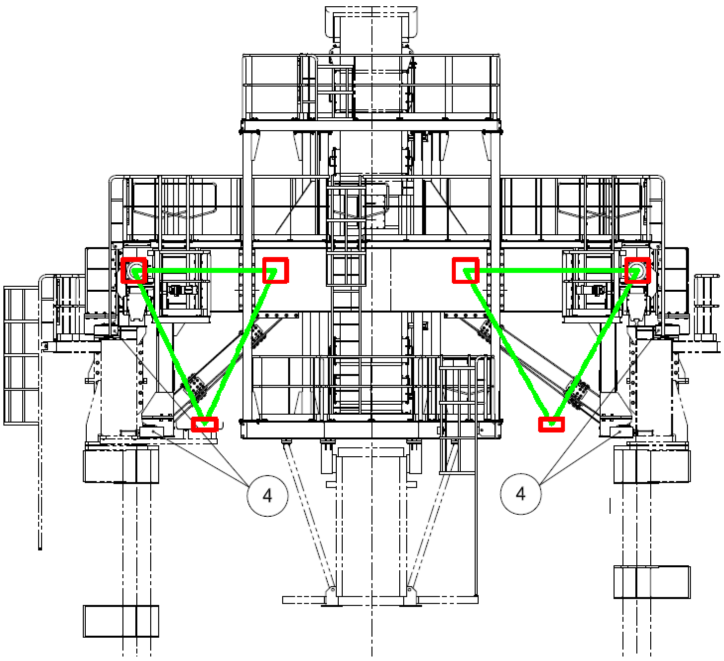


Figure 3.23: Solution 5 - complete carriage assembly (source: Drever calculation sheets)

Since the distance between the bottom chord and the rail for the horizontal reactions is approximately 1000 mm (measured from the outer face of the chord, see *Figure 3.24*), an additional structural member is necessary to support the roller bogie and connect it to the chord. As a simplification, this member was not modeled in the finite element model, although it should be stiff enough to avoid deformations and to allow a proper contact between the bogie and the rail.

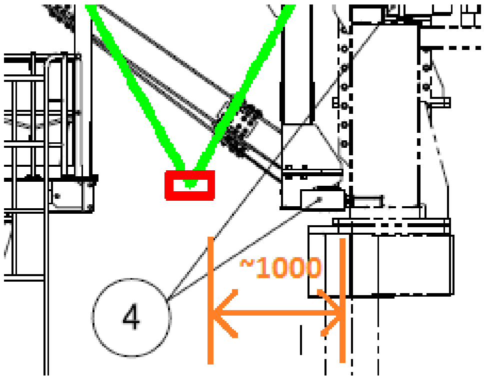


Figure 3.24: Solution 5 - bottom chord (source: Drever calculation sheets)

3.6. Solution 6

3.6.1. General layout

The structure is composed of planar trusses (side members and supporting beams) and the connecting beam that can be described the most closely as a box truss, although it does not have diagonal braces in the horizontal planes (see *Figure 3.25*). The height of the structure is increased compared to the initial solution. Here, the distance (axis-to-axis) between the chords is 1500 mm and the roller bogies supporting the structure vertically should be placed below the bottom chord. The main aim of this solution is to avoid the production of built-up box girders and to utilize the commercial hollow sections. The horizontal distance between the chords of the connecting beam is 1020 mm (axis-to-axis). For the chords, square hollow sections are used, with two different cross-sectional thicknesses in order to save the material and the dimensions are designated in *Figure 3.25* as well. Although the upper chord is subjected to the applied loads between nodes, the same cross-section size is used for both upper and lower chords in order to enhance the overall stiffness of the structure and to satisfy the deflection criterion. For geometry, a Warren truss with verticals is selected to increase the stiffness of the structure. Braces are composed of rectangular and square hollow sections, depending on their position in the structure. Compared to the initial solution, the vertical support between the brackets on the motor side of the structure has been moved for 182.5 mm in order to suit the geometry of the truss (its position now is on the half distance between the brackets). The horizontal roller bogies remain at the same position as they are in the initial solution. The chords are intended to be continuous, welded mutually by means of butt welds, while the braces are welded to the chords. Considering their sizes and consequently their second moment of area, the best approximation for this situation is to model the chords as continuous and the braces as pin-connected to the chords what has been done in the finite element model.

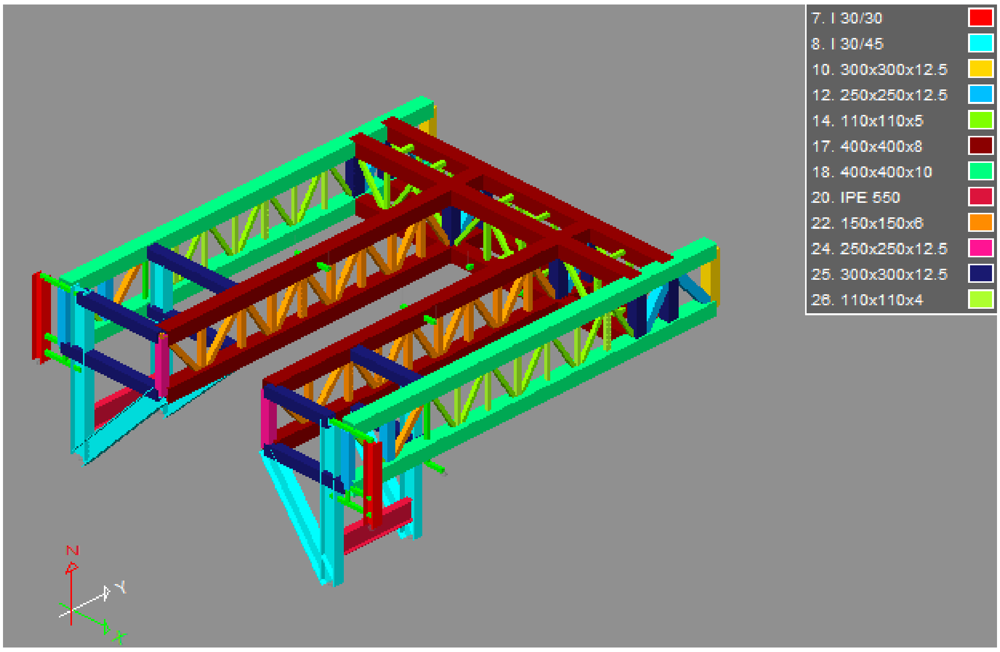


Figure 3.25: Solution 6 - 3D view and cross-sectional dimensions

3.6.2. Results, parametric studies and remarks

The maximum vertical displacements calculated on the finite element model are given in *Table 3.37*. They are governing for the design and the maximum obtained value is just below the limit of 6.85 mm.

Table 3.37: Solution 6 - vertical displacements [mm]

$z(S_G)$	1.19
$z(S_L)$	4.60
$z(X_1)$	1.25
$z(X_2)$	6.82

The weight of the structure is estimated on the basis of the vertical reactions, which are presented in *Table 3.38*. The sum of the vertical reactions resulting from the dead weight is 1.05 kN higher compared to model for the calibration. The final weight could be slightly lower compared to the initial solution, what cannot be evaluated in pre-design due to the fact that some secondary structural parts are not modeled in Model for the calibration.

Table 3.38: Solution 6 - vertical reactions [kN]

$R_1(S_G)$	91.26
$R_2(S_G)$	98.02
$R_3(S_G)$	120.42
$R_4(S_G)$	117.80
Σ	427.50

A benefit of this solution, instead of the weight, is related to its production, as the construction of built-up sections is avoided and the structure composed entirely of commercial hollow sections.

Table 3.39 shows the horizontal reactions resulting from load combination X_2 and the values are in compliance with the designation given in *Figure 2.19*. Generally, the values are considerably lower than the values calculated on model for the calibration. Comparing the maximum horizontal reaction calculated here and the maximum value from model for the calibration, the difference is 98.86 kN what can positively influence design of the runway beams and reduce their cross-section properties.

Table 3.39: Solution 6 - horizontal reactions $R(X_{2,ULS})$[kN]

R_1	R_2	R_3	R_4	R_5	R_6	R_7	R_8
353.72	289.51	222.10	187.26	-318.76	-235.35	-243.44	-209.79

In order to indicate the behavior at the ultimate limit states, *Table 3.40* presents the maximum values of elastic stresses (ULS) calculated in structural members and it is obvious that the ultimate limit states are not governing for the design. All stresses are in the range of 40-50% of the yield stress. Buckling resistance of members in compression is assessed by calculating the buckling reduction factors in *Table 3.41*. As a buckling length the system length is used in the pre-design stage, although EN 1993-1-1 allows a certain reduction for hollow sections. The resistance is only slightly influenced by the instability, for instance, out-of-plane buckling of the chord reduces the resistance for 6%.

Table 3.40: Solution 6 - stress $\sigma(X_{2,ULS})$, absolute values [MPa]

Right/left side member	Chords	108.23
	Braces	73.02
Right/left supporting beam	Chords	79.26
	Braces	74.51
Connecting beam	Chords	98.30
	Braces	125.63

Table 3.41: Solution 6 - Buckling reduction factor χ

	Chord	Diagonal brace
Cross-section	SHS 400x400x8	CHS 110x110x4
L_{cr} [mm]	6678	1778
λ	42.00	41.34
χ	0.94	0.94

Joints in this solution are mainly planar, except the joint at the intersection between the perpendicular beams (see *Figure 3.26*) where four diagonals and one vertical are connected to the chord. Planar joints can be regarded as an advantage from the manufacturing point of view and the design as well.

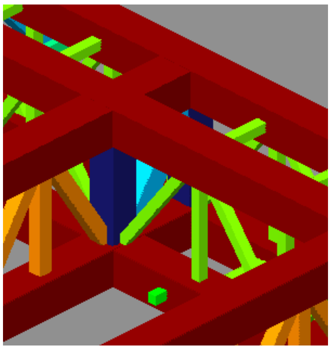


Figure 3.26: Joint at the intersection of the perpendicular beams

A case study related to the modeling has been made. Instead of pin-connected ends of the braces, all connections in the finite element model are considered as rigidly connected.

Table 3.42: Solution 6 - influence of the modeling approach on the displacements

	Displacements [mm]		Relative change [%]
	Solution 6	Solution 6 (rigid conn.)	
$z(S_G)$	1.19	1.16	2.6
$z(S_L)$	4.60	4.47	2.9
$z(X_1)$	1.25	1.22	2.5
$z(X_2)$	6.82	6.62	3

Table 3.42 indicates the difference between these two approaches in modeling. As it is obvious from the values, the difference is rather slight, resulting from the fact that the chords have considerably higher second moment of area and the braces behave closely to pin-connected in relation to the chords.

To assess the possibility of utilizing the spatial behavior of the structure by coupling the trusses on the right side, as well as on the left side, a case study has been conducted. The chords are interconnected using RHS 400x200x6, which has significant cross-section properties, and using the Chevron braces configuration to obtain an acceptable angle of inclination of the brace members. An illustration is given in *Figure 3.27*. The influence is estimated by comparing the displacements (*Table 3.43*) since they are governing for the design.

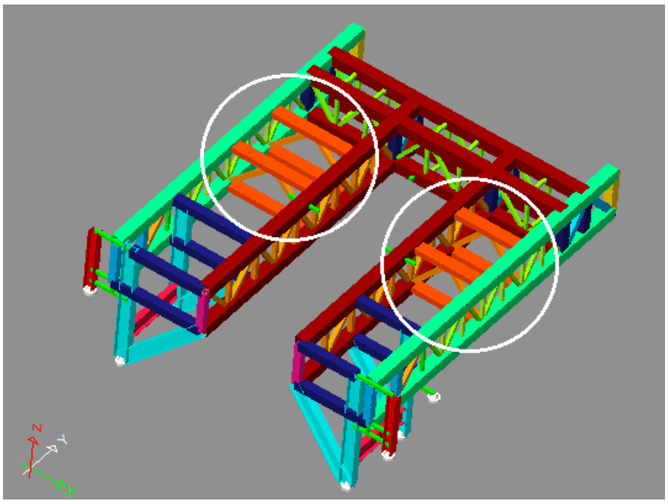


Figure 3.27: Solution 6 - coupled trusses - 3D view

Table 3.43 Solution 6 - influence of the coupling on the displacements

	Displacements [mm]		Relative change [%]
	Solution 6	Solution 6 (coupled)	
$z(S_G)$	1.19	1.25	+4.80
$z(S_L)$	4.60	4.52	-1.77
$z(X_1)$	1.25	1.32	+5.30
$z(X_2)$	6.82	6.78	-0.59

Comparing the values resulting from load case X_2, the displacements are lower only for 0.04 mm what means that the coupling does not have any important influence on the overall behavior of the structure. On contrary, it increases the weight of the structure for 1.63 t (6% of the total weight). This inefficiency results mainly from the geometry. On the right side of the carriage structure, the span between the coupled trusses is 3080 mm, the height is 1500 mm and the coupling truss is not stiff enough to redistribute the forces spatially for the given span/height ratio.

3.7. Solution 7

3.7.1. General layout

The structure is a variant of Solution 6 with the main aim to simplify the production. The layout and cross-sections of the connecting beam remain the same in this solution as they were proposed in Solution 6. Instead of the Warren trusses with verticals that are used in Solution 6, Vierendeel trusses are applied here for the side members and supporting beams. The application of Vierendeel trusses simplifies the production since the truss consists only from verticals. To produce a vertical brace, a construction worker has to cut the section straightly, perpendicularly to its axis and to weld it to the chord by means of a filled weld all around the member. On the other hand, to produce a diagonal brace, which is part of a Warren truss, a worker has to perform bevelled cuts (one or double sided) and to weld it to the chord in an inclined position. A Vierendeel truss behaves as a frame, what means that all structural members are subjected to the combined effects of axial forces, shear forces and bending moments and consequently it is more deformable. In a Warren truss with continuous chords, the chords are subjected to the combined effects of axial forces, shear forces and bending moments, while the braces are subjected to axial forces only. This may result in the heavier structure, but the increase of the weight can be compensated by simplification of the production and less labor costs. A graphical representation of the structure proposed by this solution is given in *Figure 3.28*. The braces are composed of rectangular hollow sections (see *Figure 3.28* for the dimensions as well). All other considerations related to the general layout are inherited from Solution 6 and will not be repeated here. The structure has been modeled assuming rigid connections between members.

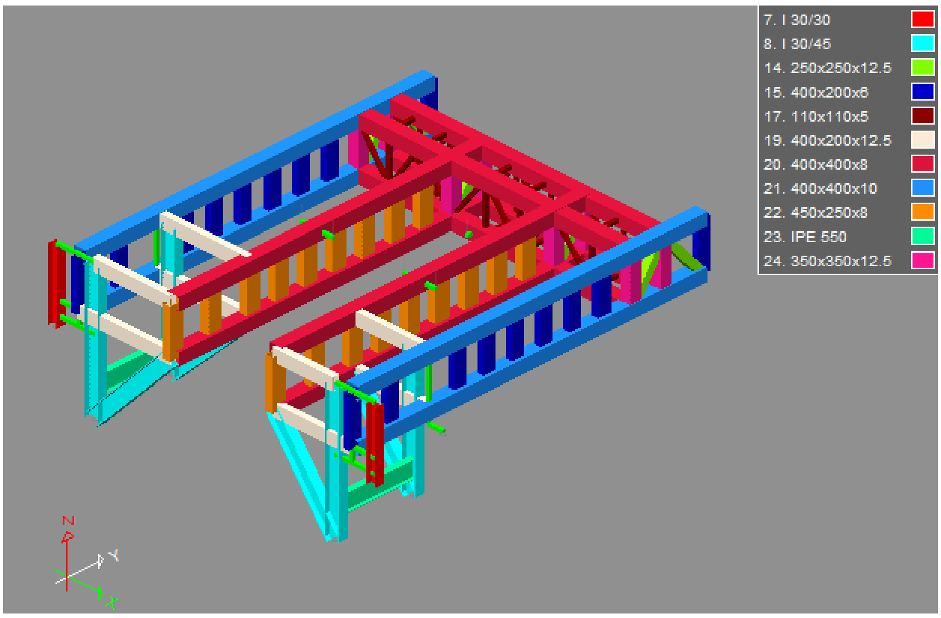


Figure 3.28: Solution 7 - 3D view and cross-sectional dimensions

3.7.2. Results, parametric studies and remarks

The deflection criterion is governing for the design, thus the vertical displacements are the first task that has to be checked. The values are given in *Table 3.44* and the maximum vertical displacement is lower than the limiting value (6.85 mm).

Table 3.44: Solution 7 - vertical displacements [mm]

$z(S_G)$	1.22
$z(S_L)$	4.52
$z(X_1)$	1.28
$z(X_2)$	6.74

The vertical reactions resulting from the dead weight are displayed in *Table 3.45*. Compared to Solution 6, the sum is 5.37 kN higher, what is slight increase and can be compensated by the simpler manufacturing.

Table 3.45: Solution 7 - vertical reactions [kN]

$R_1(S_G)$	93.17
$R_2(S_G)$	100.11
$R_3(S_G)$	120.93
$R_4(S_G)$	118.66
Σ	432.87

The values of the horizontal reactions are given in *Table 3.46*. Their range is similar to the range of values calculated in Solution 6 what is considerably lower than the values obtained in Model for the calibration and it can lead to the reduction of cross-section properties of the runway structure, which is not part of the detailed study.

Table 3.46: Solution 7 - horizontal reactions $R(X_{2,ULS})$[kN]

R_1	R_2	R_3	R_4	R_5	R_6	R_7	R_8
367.05	302.52	208.92	173.63	-342.95	-277.07	-222.37	-188.42

Table 3.47 is an overview of the maximum values of elastic stresses (ULS). As it is obvious, the elastic normal stress reaches 55% of the yield strength what means that the ultimate limit states are not governing for the design. Generally, braces in a Vierendeel truss should be designed as beam-columns because they are subjected to the combined action of axial forces and bending moments. In this case, their cross-sections were selected by the serviceability criterion, what results in quite stiff and stocky members. For the selected cross-sections of brace members and calculated buckling lengths (see *Table 3.48*) the buckling resistance check can be avoided according to EN 1993-1-1 because the relative slenderness is lower than 0.2.

Table 3.47: Solution 7 - stress $\sigma(X_{2,ULS})$, absolute values [MPa]

Right/left side member	Chords	98.00
	Braces	88.41
Right/left supporting beam	Chords	113.48
	Braces	105.89
Connecting beam	Chords	98.15
	Braces	128.55

Table 3.48: Solution 7 - Buckling reduction factor χ

	Brace (type 1) RHS 400x200x6		Brace (type 2) RHS 450x250x8	
	In-plane	Out-of-plane	In-plane	Out-of-plane
L_{cr} [mm]	1500	1500	1500	1500
λ	10.27	17.54	9.09	14.29
λ_{rel}	0.11	0.19	0.10	0.15

A feasibility of this solution is strongly dependent on the assumption that all members are connected rigidly, which was used in the finite element model. In the case of hollow section joints, the above-mentioned assumption may not be appropriate and it has to be proven through a detailed study, for example using the finite element approach. Rectangular hollow section 450x250x8 mm is not considered by some producers as standard size and it may be difficult for the procurement.

A case study related to the stiffness of certain openings and their influence on the overall behavior of the structure has been conducted. As it can be seen in *Figure 3.28*, the diagonals are present in some openings (at the intersections and next to the supports). Without them, the structure becomes notably softer. The structure without the diagonal next to the vertical support is given in *Figure 3.29* and the displacements are calculated in *Table 7.49*. For the structure without the diagonals at the intersections and the diagonal next to the vertical support, an illustration is given in *Figure 3.30* and the displacements are given in *Table 3.50*.

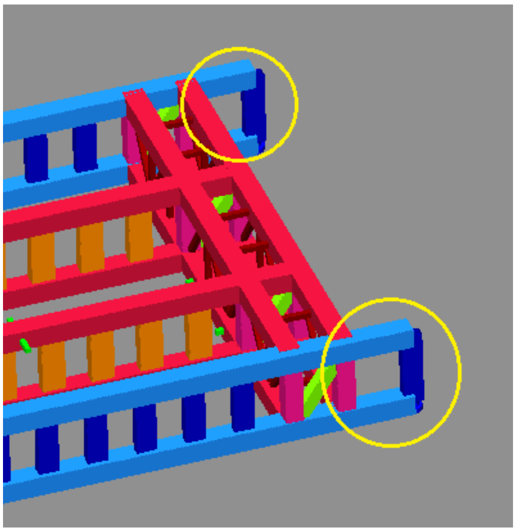


Figure 3.29: Solution 7 - structure without the diagonal next to the vertical support

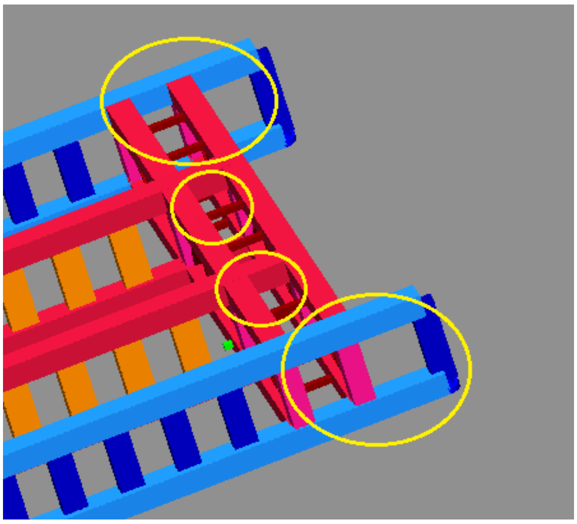


Figure 3.30: Solution 7 - structure without the diagonals at the intersections

Table 3.49: Solution 7 - influence of the diagonal next to the support on the displacements

	Displacements [mm]		Relative change [%]
	Solution 7	Solution 7 (no diagonal at the support)	
$z(S_G)$	1.22	1.40	14.75
$z(S_L)$	4.52	4.86	7.52
$z(X_1)$	1.28	1.47	14.84
$z(X_2)$	6.74	7.32	8.61

Table 3.50: Solution 7 - influence of the diagonals next to the support and at the intersections on the displacements

	Displacements [mm]		Relative change [%]
	Solution 7	Solution 7 (no diagonals at the intersections)	
$z(S_G)$	1.22	1.45	18.85
$z(S_L)$	4.52	5.09	12.61
$z(X_1)$	1.28	1.53	19.53
$z(X_2)$	6.74	7.65	13.50

The self-weight of these members is negligible compared to the gain of their presence on the overall behavior of the structure. It can be proved by comparing the vertical displacements resulting from load case X_2. The displacements can increase for 13.5% and to compensate it by stiffening other structural members is considerably expensive.

3.8. Solution 8

3.8.1. General layout

The layout of Solution 8 is in a considerable amount inherited from Solution 7. The side members and supporting beams are identical to Solution 7, as well as the height of the structure and the positions of the roller bogies. The aim here is to simplify the production more by avoiding diagonal braces in the connecting beam. This leads the connecting beam to be in a form of Vierendeel truss as well. The diagonal braces at the intersections and next to the vertical support are still present on the structure because of their importance what is demonstrated in the previous sub-chapter. A graphical representation of the structure proposed by this solution is given in *Figure 3.31* together with the cross-sectional dimensions.

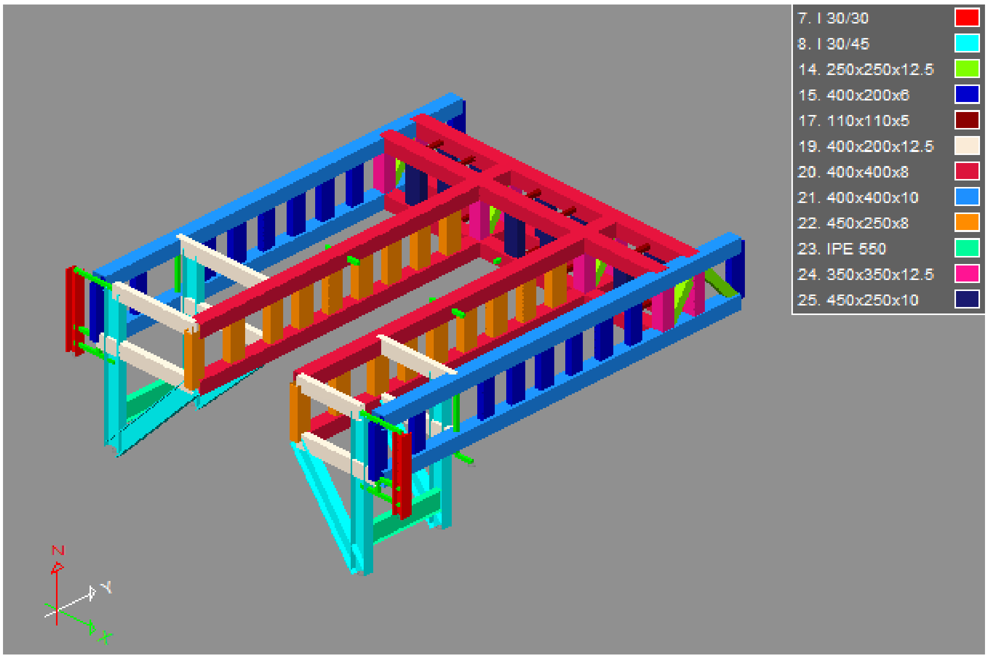


Figure 3.31: Solution 8 - 3D view and cross-sectional dimensions

3.8.2. Results, parametric studies and remarks

The deflections calculated for this solution are slightly higher than the resulting deflections in Solution 7 what is reasonable because the connecting beam has smaller stiffness here (a Vierendeel truss instead of a Warren truss with verticals). The values are given in *Table 3.51* and they are below the limiting value of 6.85 mm.

Table 3.51: Solution 8 - vertical displacements [mm]

$z(S_G)$	1.27
$z(S_L)$	4.55
$z(X_1)$	1.33
$z(X_2)$	6.82

The vertical reactions resulting from the dead weight are given in *Table 3.52*. Compared to Model for the calibration, the sum of the vertical reactions is 14.9 kN higher and a compromise should be made by the manufacturer of the structure, between the enhanced weight and the simplified production.

Table 3.52: Solution 8 - vertical reactions [kN]

$R_1(S_G)$	97.20
$R_2(S_G)$	103.05
$R_3(S_G)$	121.74
$R_4(S_G)$	119.36
Σ	441.35

The horizontal reactions resulting from the load combination X_2 are presented in *Table 3.53*. Their values are almost identical to the values calculated for Solution 7, and the same benefit is applicable here (possible reduction of cross-section properties of the runway structure).

Table 3.53: Solution 8 - horizontal reactions $R(X_{2,ULS})$[kN]

R_1	R_2	R_3	R_4	R_5	R_6	R_7	R_8
369.71	305.80	209.09	173.41	-344.75	-279.04	-223.88	-190.00

In order to indicate the behavior at the ultimate limit states, the maximum values of elastic stresses (ULS) calculated in structural members are given in *Table 3.54*. All stresses are in the range of 40-50% of the yield stress and as a result, the ultimate limit states are not governing for the design. The brace members are stocky, hence they are not susceptible to stability problems (the relative slenderness is lower than 0.2).

Table 3.54: Solution 8 - stress $\sigma(X_{2,ULS})$, absolute values [MPa]

Right/left side member	Chords	95.33
	Braces	90.92
Right/left supporting beam	Chords	114.24
	Braces	107.09
Connecting beam	Chords	107.46
	Braces	93.69

A feasibility of Solution 8 is mostly dependent on the assumption that all members are connected rigidly, what may be questionable in the case of hollow section joints. The stiffness of joints has to be proven through a detailed study in order to verify the assumption.

A case study showing the influence of coupling the trusses on the right and left side has been conducted. The layout is given in *Figure 3.32* and the maximum values of the vertical displacements are presented in *Table 3.55*. The beams are connected using RHS 400x200x12.5 mm.

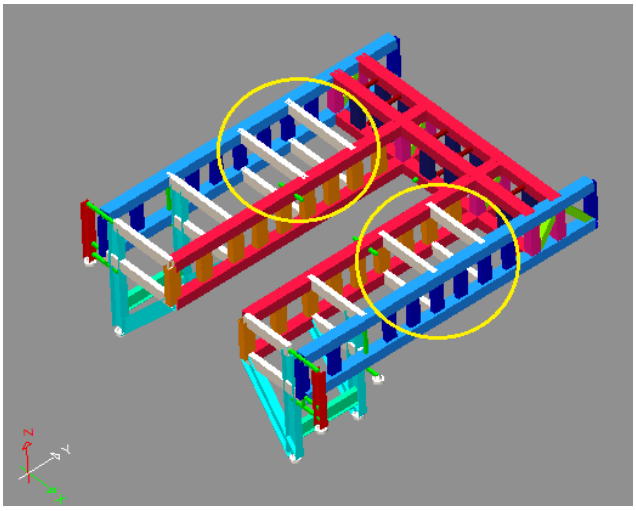


Figure 3.32: Solution 8 - coupled Vierendeel trusses - 3D view

Table 3.55: Solution 8 - influence of the coupling on the displacements

	Displacements [mm]		Relative change [%]
	Solution 8	Solution 8 (coupled)	
$z(S_G)$	1.27	1.35	+5.93
$z(S_L)$	4.55	4.40	-3.40
$z(X_1)$	1.33	1.41	+5.67
$z(X_2)$	6.82	6.72	-1.49

The influence of the coupling is negligible, as it can be seen from the displacements. The vertical displacements resulting from load case X_2 are lower only for 0.1 mm while the weight is increased for 2.36 t (8% of the total weight). The non-efficiency results mainly from the span/height ratio as it was described for Solution 6.

3.9. Overall comparison and selection of the solution for the detailed design

This sub-chapter will summarize all considerations that were explained in previous sub-chapters. A comparison will be made regarding the deflections (*Table 3.56*), vertical reactions (*Table 3.57*), weight of the structure (*Table 3.57*) and horizontal reactions (*Table 3.58*).

All proposed solutions were arranged in terms of the structural system, dimensions and selected cross-sections to satisfy the deflection criterion of 6.85 mm, therefore the vertical displacements in each solution are either equal to the limit or slightly below it. As it is given in *Table 3.56*, the values are between 6.74 mm and 6.85 mm (load combination X_2) . All other values occur in the structure as a consequence of this criterion.

Table 3.56: Vertical displacements [mm] - overall comparison

	$z(S_G)$	$z(S_L)$	$z(X_1)$	$z(X_2)$	
Model for the calibration	1.32	4.25	1.39	6.52	
Solution 1	1.27	4.57	1.33	6.85	
Solution 2	1.31	4.52	1.38	6.83	
Solution 3	1.31	4.55	1.37	6.85	
Solution 4	1.07	4.68	1.12	6.77	<6.85
Solution 5	1.07	4.73	1.12	6.84	
Solution 6	1.19	4.60	1.25	6.82	
Solution 7	1.22	4.52	1.28	6.74	
Solution 8	1.27	4.55	1.33	6.82	

As it was mentioned previously, to estimate the weight of the structure, the sum of the vertical reactions resulting from the dead weight is used ($\Sigma R(S_G)$). Since the dead weight includes the self-weight of the platforms (G_P=146.88 kN), their weight is subtracted from the sum of the vertical reactions to evaluate the weight of the carriage structure itself. The weight of the structure of each solution is compared to Model for the calibration (column ΔR in *Table 3.57*, the minus sign means that a solution is lighter than model for the calibration). The column ΔG is equal to the column ΔR, only the units are different. A precise assessment is possible only after the detailed design since at that step all cross-sections are definitely set, as well as joints, what results in known amount of supplementary material necessary for stiffeners, diaphragms, end plates, etc. The weight given in *Table 3.57* is more reliable estimation for solutions 4-8 as long as they are trusses composed of hollow sections what means that they do not need diaphragms/stiffeners like a box girder. In addition, joints in trusses are lighter compared to joints in built-up box girders. Taking all mentioned here into account, solutions 6-8 can be expected to provide the material saving after the detailed design as well, although on this level they have slightly higher estimated weight.

Table 3.57: Vertical reactions and weight of the structure - overall comparison

	$\Sigma R(S_G)$ [kN]	$\Sigma R(S_G)-G_P$ [kN]	ΔR [kN]	ΔG [t]
Model for the calibration	426.45	279.57	/	/
Solution 1	407.88	261.00	-18.57	-1.82
Solution 2	403.26	256.38	-23.19	-2.27
Solution 3	385.20	238.32	-41.25	-4.05
Solution 4	330.06	183.18	-96.39	-9.46
Solution 5	320.00	173.12	-106.45	-10.44
Solution 6	427.50	280.62	+1.05	+0.10
Solution 7	432.87	285.99	+6.42	+0.63
Solution 8	441.35	294.47	+14.9	+1.46

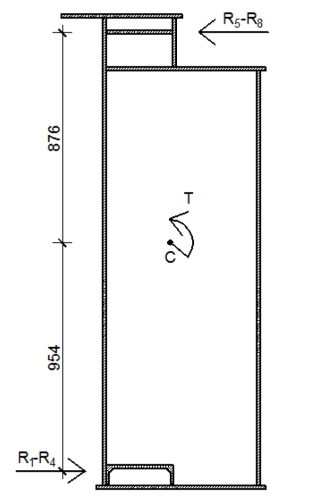
The horizontal reactions are important from the point of view of the runway structure. The overhanging part of the carriage is held by the brackets that are supported in the horizontal direction by the runway beam, hence the horizontal reactions act on the runway beam by shear action (*Table 3.58*) and when reduced on the centroid of the beam, by torque (*Table 3.59*). As a result of the analysis, it can be concluded that the maximum horizontal action on the runway beam for all proposed solutions, except solutions 1-3, is lower compared to the initial solution. Solution 1 has almost the same values to Model for the calibration. High values of the horizontal actions obtained in Solution 2 and 3 result from the fact that these two solutions have lesser number of the horizontal roller bogies (4 instead of 8).

Table 3.58: Horizontal reactions $R(X_{2,ULS})$[kN] - overall comparison

	R_1	R_2	R_3	R_4	R_5	R_6	R_7	R_8
Model for the calibration	420.65	349.21	188.84	157.72	-452.59	-373.79	-158.00	-134.24
Solution 1	428.49	361.25	181.65	146.29	-447.26	-371.70	-166.60	-139.57
Solution 2	/	/	475.17	397.36	/	/	-469.12	-391.31
Solution 3	/	/	479.10	399.97	/	/	-473.40	-394.26
Solution 4	217.37	185.40	397.47	328.47	-211.17	-178.97	-395.22	-326.46
Solution 5	241.20	212.23	405.95	336.08	-183.44	152.88	-384.54	-316.27
Solution 6	353.72	289.51	222.10	187.26	-318.76	-235.35	-243.44	-209.79
Solution 7	367.05	302.52	208.92	173.63	-342.95	-277.07	-222.37	-188.42
Solution 8	369.71	305.80	209.09	173.41	-344.75	-279.04	-223.88	-190.00

Table 3.59: Torque acting on the runway beam $T(X_{2,ULS})$[kNm] - overall comparison

	Torque			
	T_1 (R_1,R_5)	T_2 (R_2,R_6)	T_3 (R_3,R_7)	T_4 (R_4,R_8)
Model for the calibration	797.77	660.59	318.56	268.06
Solution 1	800.58	670.24	319.24	261.82
Solution 2	/	/	864.26	721.87
Solution 3	/	/	871.76	726.94
Solution 4	392.36	333.65	725.40	599.34
Solution 5	390.80	336.39	724.13	597.67
Solution 6	616.68	482.36	425.14	362.42
Solution 7	650.59	531.32	394.11	330.70
Solution 8	654.70	536.17	395.59	331.87



Generally, the ultimate limit states are not governing for the design and cross-sections in all solutions are utilized around 50% at the ultimate limit state what was explained and illustrated in sub-chapters dedicated for the results of each proposed solution.

Benefits and drawbacks for each proposed solution are summarized in *Table 3.60*.

Table 3.60: Benefits and drawbacks - summary

	Benefits	Drawbacks
Solution 1	Slight reduction of the weight; If a higher truss is applicable, the weight reduction could be larger;	More labor for the production; Bending in the chords inevitable; Utilization of built-up sections;
Solution 2	Slight reduction of the weight; 4 horizontal roller bogies less; Tie and pillar connected to the structure by pinned connections; Utilization of the existing platform column as a pillar;	Horizontal reactions higher; Utilization of built-up sections; The structure may limit the mobility of workers on the platforms;

Solution 3	Reduction of the weight around 10%; 4 horizontal roller bogies less; Tie and pillar connected to the structure by pinned connections;	Horizontal reactions higher; Utilization of the existing platform column as a pillar is not possible; Utilization of built-up sections; The structure may limit the mobility of workers on the platforms;
Solution 4	Significant reduction of the weight; Lower horizontal reactions; Utilization of commercial hollow sections;	More labor for the production; Very complex details and joints;
Solution 5	The lightest structure among proposed solutions; Lower horizontal reactions; Smaller number of members compared to Solution 4; Utilization of commercial hollow sections;	More labor for the production; Very complex details and joints; An additional member (stiff) necessary to support the horizontal roller bogie;
Solution 6	Utilization of commercial hollow sections; Smaller horizontal reactions Pinned joints;	More labor for the production;
Solution 7	Utilization of commercial hollow sections; Smaller horizontal reactions; Diagonal braces partly avoided;	Stiffness of joints is questionable;
Solution 8	Utilization of commercial hollow sections; Smaller horizontal reactions; Diagonal braces completely avoided;	Increase of the weight of the structure; Stiffness of joints is questionable;

To summarize, the two main aspects figuring in these pre-design studies and defining the final cost are the weight of the structure and the amount of labor needed to fabricate it. All these solutions can be classified in groups based on the similarities among them (see *Table 3.61*, *Table 3.62* and *Table 3.63*).

Table 3.61: Group A of the pre-design solutions

Group	Criterion	Solution	Fabrication complexity	Material saving
A	Layout similar to the initial solution with some improvements, built-up box sections.	Solution 1 Solution 2 Solution 3	Medium	Low to medium

Table 3.62: Group B of the pre-design solutions

Group	Criterion	Solution	Fabrication complexity	Material saving
B	Box trusses composed of hollow sections.	Solution 4 Solution 5	Very high	High

Table 3.63: Group C of the pre-design solutions

Group	Criterion	Solution	Fabrication complexity	Material saving
C	Planar trusses composed of hollow sections.	Solution 6 Solution 7 Solution 8	Medium	Low to medium

After presenting all above-mentioned facts related to the proposed solutions, the representatives from DREVER International selected **Solution 6** to be studied in detail. The main reason for selecting Solution 6 is related to the fabrication. The company strongly prefers to use commercial sections instead of built-up sections mainly because of the fabrication tolerances what is sometimes rather difficult to be satisfied in the workshop where this structure is intended to be manufactured, therefore Group A was discarded. Solutions from Group B were not selected because of their fabrication complexity. Within Group C, Solution 6 was preferred as long as it provides higher material saving compared to Solution 7 and Solution 8. An additional reason is that the assumption related to joints (joints considered as fully rigid) in Solution 7 and Solution 8 may be questionable.

Upon a suggestion of the representatives from DREVER International, two variants of Solution 6 will be studied in detail:

- Solution 6-1: Welded solution made completely of hollow sections
- Solution 6-2: Bolted solution made of hollow section chords and angles as braces

4. Detailed design of the selected solutions

4.1. Welded solution made completely of hollow sections (Solution 6-1)

4.1.1. Improvements and final layout of the structure

This chapter will present the final layout of the structure that is improved after the detailed study of the pre-design proposal. The methodology used during the design will be explained in the following sub-chapters and the computation details can be found in Annex A.

The main goals for the improvements were to:

- Decrease the number of braces
- Avoid vertical brace members as much as possible
- Simplify joints

Firstly, the finite element model has been improved by introducing load cases X_3 and X_4 and all possible load combinations. The pre-design stage included only X_1 and X_2 for the sake of simplicity. In the design stage, the region around the brackets and horizontal supports is modeled more precisely.

The improved structural layout with adopted cross-sections is given in *Figure 4.1*.

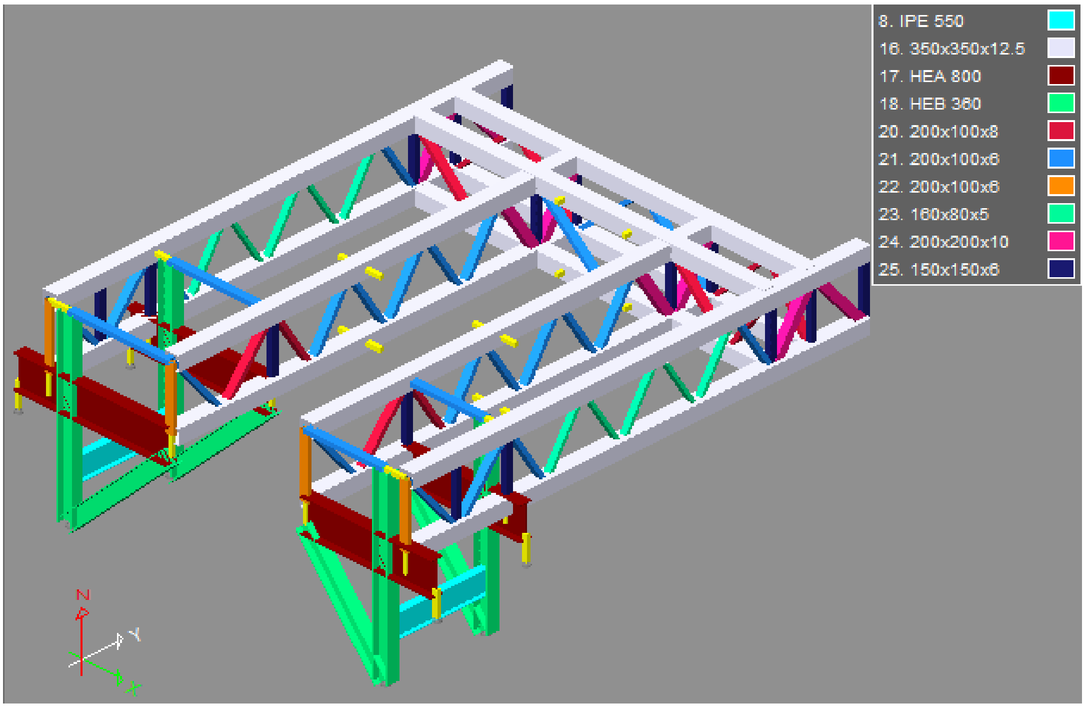


Figure 4.1: Solution 6-1, improved structural layout - 3D view

The height of the trusses remains 1500 mm, measured between the chord axes. The selection of cross-sections is governed strongly by the range of validity for the application of the rules for design of joints (Chapter 7 of EN 1993-1-8) in terms of width-to-height ratios, width-to-thickens ratios, width of braces to width of chord ratios, gap size, overlap size, cross-section class, allowed eccentricity, etc. This will be explained in detail in sub-chapter 4.1.4. Generally, the cross-sections have similar sizes to the pre-design proposal and the adjustments have been done mainly in such a way to change their shape (for example a rectangular hollow section instead of a square hollow section) or to reduce the size at the expense of the increased thickness.

Compared to the pre-design proposal, where SHS 400x400x8 mm and SHS 400x400x10 mm were selected for the chord cross-sections, the design stage replaces them with SHS 350x350x12.5 mm. A more compact cross-section was required by the rules for design of joints, what is also an advantage for the procurement to use the same cross-section for all chords as well as the fact that SHS 350x350x12.5 mm is more available on the market.

In order to simplify the production, a special care was taken about the joints in order to obtain gap joints rather than overlap. From the manufacturing point of view, K-type joints are more desirable than KT-type joints, hence an intention has been made to avoid vertical braces. As it can be seen (*Figure 4.1*) the vertical braces are present now only at the intersections between the perpendicular trusses and around the supports. This is conducted mainly at the expense of the increased thickness of diagonal braces. In overlap joints, the vertical brace has smaller size, therefore it overlaps the diagonal braces. Some joints in the pre-design proposal are spatial (see *Figure 3.26* in sub-chapter 3.6.2.) what is not desirable. With the aim to avoid this complex detail, brace members in the connecting beam have been rearranged, by changing their directions in one of the vertical parallel planes. An illustration is given in

Figure 4.2 and *Figure 4.3*. This leads to the fact that all joints in the structure can be considered as planar joints, what simplifies the production and the design as well.

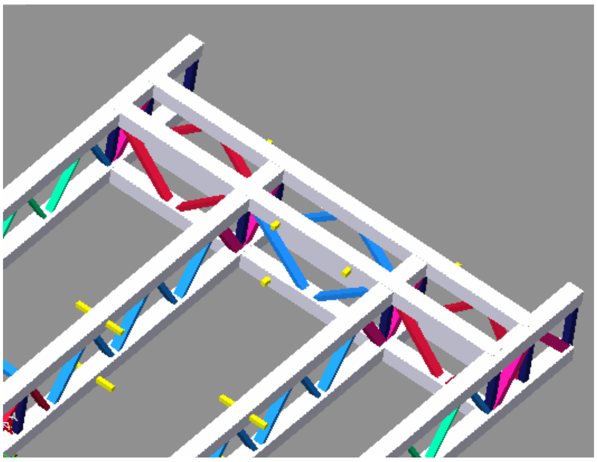


Figure 4.2: Solution 6-1, connecting beam - 3D view

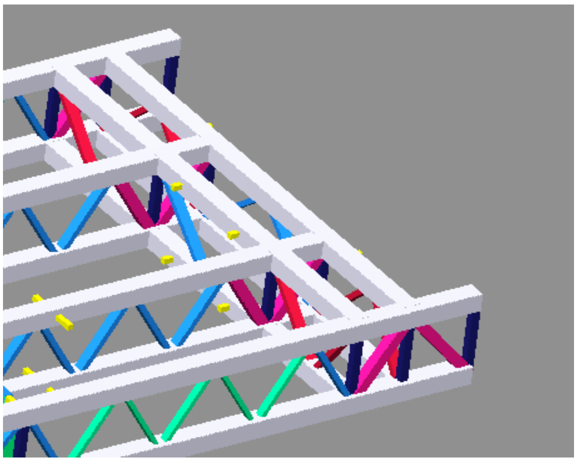


Figure 4.3: Solution 6-1, connecting beam - 3D view

The study shows that the stiffness of the transversal beam in the bracket structure has an important influence on the overall behavior of the carriage structure, hence a beam with high second moment of area is used (HEA 800). Its top edge should fit with the top edge of the bottom chord since the space below the chord is limited by the height of the roller bogie assembly. In order to obtain this, the top part of the beam can easily be cut off at the intersection with the chord. The detail is given in *Figure 4.4* and *Figure 4.5*.

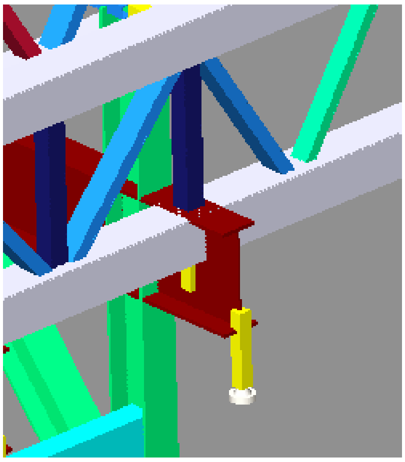


Figure 4.4: Solution 6-1, brackets - detail

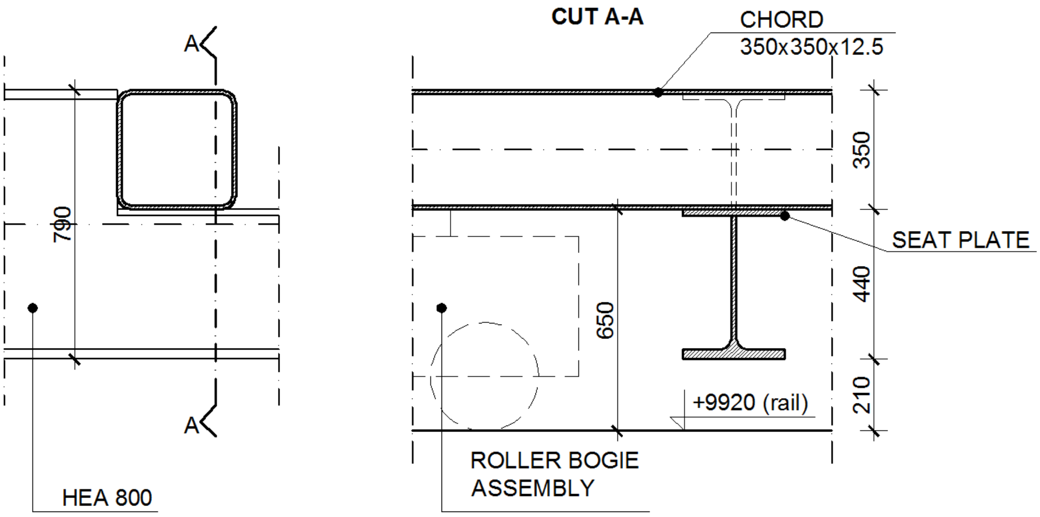


Figure 4.5: Solution 6-1, brackets - detail

4.1.2. Serviceability limit states

Although the computation details can be found in Annex A, the serviceability limit state checks are presented here because they are governing for the design in this case. The vertical displacements calculated in the finite element model are compared with the limiting value of 6.85 mm as it was already used in pre-design (see sub-chapter 2.6). The maximum values are presented in *Table 4.1* and an illustration of the deformed shape for load combination X_3 is given in *Figure 4.6*.

Table 4.1: Solution 6-1, vertical displacements [mm]

$z(X_1)$	1.28	≤6.85
$z(X_2)$	6.83	
$z(X_3)$	6.85	
$z(X_4)$	6.81	

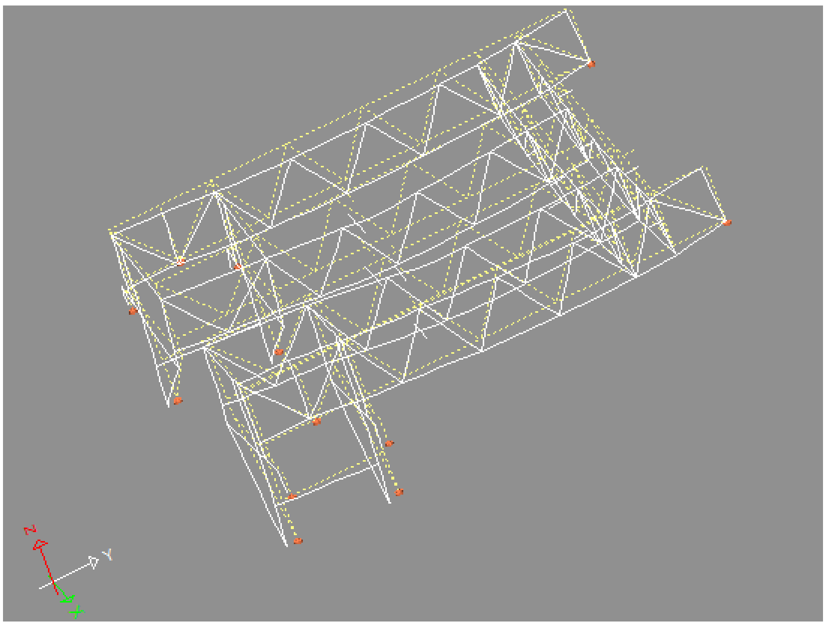


Figure 4.6: Solution 6-1, deformed model for load combination X_3

4.1.3. Ultimate limit states

4.1.3.1. Design of cross-sections

The cross-section design checks are performed according to EN 1993-1-1. As it was already mentioned the finite element model consists of continuous chords and pin-ended braces. As a result the brace members are loaded in axial compression/tension while the chords are subjected to the combined effects of axial forces, bending moments and shear forces.

All cross-sections in the structure are adjusted in such a way to satisfy class 1 or class 2 condition since this criterion has to be fulfilled for design of joints according to EN 1993-1-8, Chapter 7. As a simplification, in the classification of cross-sections, the upper chords are classified for the condition of pure compression, although there is a certain amount of bending moments along the chords. Similarly, the bottom chords are classified assuming pure bending, despite the fact that the tensile force decrease the compression part resulting from bending.

The interaction between shear forces and bending moments does not have to be performed because in all cross-sections is satisfied the condition that the shear force is less than half the plastic resistance (EN 1993-1-1 clause 6-2.8(2)). In the structure there are no fastener holes in tension thus the net section resistance does not have to be checked.

Formulas used for the design checks are summarized in *Table 4.2*.

Table 4.2: Design of cross-sections - formulas summary

Situation	Design resistance	Design check
Axial tension	$N_{pl,Rd}=\frac{A\cdot f_y}{\gamma_{M0}}$ EN 1993-1-1 (6.6)	$\frac{N_{Ed}}{N_{t,Rd}}\leq 1$ EN 1993-1-1 (6.5)
Axial compression	$N_{c,Rd}=\frac{A\cdot f_y}{\gamma_{M0}}$ EN 1993-1-1 (6.10)	$\frac{N_{Ed}}{N_{c,Rd}}\leq 1$ EN 1993-1-1 (6.9)
Bending moment	$M_{pl,Rd}=\frac{W_{pl}\cdot f_y}{\gamma_{M0}}$ EN 1993-1-1 (6.13)	$\frac{M_{Ed}}{M_{c,Rd}}\leq 1$ EN 1993-1-1 (6.12)
Axial force and bending	$N_{t,Rd}$ or $N_{c,Rd}$ $M_{c,Rd}$	$\frac{N_{Ed}}{N_{Rd}}+\frac{M_{y,Ed}}{M_{y,Rd}}+\frac{M_{z,Ed}}{M_{z,Rd}}\leq 1$ EN 1993-1-1 (6.2)
Shear	$V_{pl,Rd}=\frac{A_v\cdot(f_y/\sqrt{3})}{\gamma_{M0}}$ EN 1993-1-1 (6.18)	$\frac{V_{Ed}}{V_{c,Rd}}\leq 1$ EN 1993-1-1 (6.17)

The partial safety factor γ_{M0} is taken equal to 1.00 as it is recommended by Eurocode. Details of the calculations are given in Annex A.

4.1.3.2. Stability of structural members

Structural members subjected to axial compression or axial compression and bending may buckle and the code specifies stability checks to be performed in these cases.

First of all, appropriate buckling lengths have to be quantified what is done here following the recommendations given in Annex BB.1 of EN 1993-1-1. For a hollow section chord the buckling length L_{cr} may be taken as 0.9L for both in-plane and out-of-plane buckling, where L is the system length for the relevant plane. The in-plane system length is the distance between the joints. The out-of-plane system length is the distance between the lateral supports. For a hollow section brace member without cropping or flattening, welded around its perimeter to hollow section chords, the buckling length L_{cr} may be taken as 0.75L for both in-plane and out-of-plane buckling provided that a girder has parallel chords and the brace-to-chord width ratio β is less than 0.6. The ratio β is lower than 0.6 considering that the widest diagonal is 200 mm, the chord has the width of 350 mm, what gives β=0.571.

Hollow sections are not susceptible to torsion and as a result torsional buckling, torsional-flexural buckling and lateral-torsional buckling checks are not relevant for this structure. This is not valid for the brackets, because they are made of open I sections and lateral-torsional buckling has to be checked.

The calculation steps for the verification of a compression member against flexural buckling according to EN 1993-1-1 are summarized in *Table 4.3*.

Table 4.3: Flexural buckling design check- formulas summary

Step	Formula
Non-dimensional slenderness	$\bar{\lambda}=\frac{L_{cr}}{i}\frac{1}{\lambda_1}$ EN 1993-1-1 (6.50)
Φ coefficient	$\Phi=0.5\left[1+\alpha\left(\bar{\lambda}-0.2\right)+\bar{\lambda}^2\right]$ EN 1993-1-1 (6.49)
Buckling reduction factor	$\chi=\frac{1}{\Phi+\sqrt{\Phi^2-\bar{\lambda}^2}}$ EN 1993-1-1 (6.49)
Buckling resistance	$N_{b,Rd}=\frac{\chi\cdot A\cdot f_y}{\gamma_{M1}}$ EN 1993-1-1 (6.47)
Buckling verification	$\frac{N_{Ed}}{N_{b,Rd}}\leq 1$ EN 1993-1-1 (6.46)

The partial safety factor γ_{M1} is taken equal to 1.00 as it is recommended by Eurocode.
Details of the calculations are given in Annex A.

For a hollow section, the buckling curve depends on its production, and the buckling curve "a" should be taken for hot finished tubes (S235) while the buckling curve "c" should be taken for cold formed tubes (S235). Since on the market a section with a certain dimensions can be found both as hot finished or cold formed, the buckling curve "c" is selected because the imperfection factor α is higher for cold formed (assumption on the safe side).

It should also be stated here that the equations for the design buckling resistance of a member given in EN 1993-1-1 were derived with the assumption that a member is loaded by constant axial compression force along its length. For the structure under the study, this complies for the case of in-plane buckling of the chord. In the case of out-of-plane buckling, the axial force varies between the lateral supports along the chord of the structure. As a simplification, taking into account the fact that the ultimate limit states are not governing criterion for the design, it is assumed that the axial force is constant between the lateral supports with its maximum value.

Moreover, in-plane buckling of the chord has to be calculated only for the connecting beam, coming from the reason that the non-dimensional slenderness for chords of the side members and supporting beams is less than 0.2 and the buckling effects may be ignored according to EN 1993-1-1 clause 6.3.1.2(4).

Chords in this structure are subjected to the combined effects of bending (around the major axis) and axial forces, thus they have to satisfy the equations presented in *Table 4.4*.

Table 4.4: Stability of members loaded in bending and axial compression- formulas summary

Design resistance	Design check
$N_{Rk}=A \cdot f_y$ $M_{Rk}=W_{pl,y} \cdot f_y$ EN 1993-1-1 (Table 6.7)	$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}}+k_{yy}\frac{M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}}\leq 1$ EN 1993-1-1 (6.61)
$N_{Rk}=A \cdot f_y$ $M_{Rk}=W_{pl,y} \cdot f_y$ EN 1993-1-1 (Table 6.7)	$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}}+k_{zy}\frac{M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}}\leq 1$ EN 1993-1-1 (6.62)

Hollow sections are not susceptible to torsional deformation, hence $\chi_{LT}=1$.

Generally, the interaction factors k_{yy} and k_{zy} may be obtained using two methods given in EN 1993-1-1. For this structure, the factors are obtained from Annex B of EN 1993-1-1 (alternative method 2) since the method is applicable for hollow sections and at the same time faster to apply. According to Annex B of EN 1993-1-1, the coefficient k_{zy} may be taken equal to zero for hollow sections under axial compression and uniaxial bending $M_{y,Ed}$.

Considering all above-mentioned facts, the design checks for a beam-column (*Table 4.4*) become simplified and they are given in *Table 4.5*. As it can be seen, according to the applied method, for rectangular hollow sections there is no need for combining out-of-plane buckling and in-plane bending moments (uniaxial bending).

Table 4.5: Stability of members loaded in bending and axial compression- formulas summary (simplified)

Design resistance	Design check
$N_{Rk}=A\cdot f_y$ $M_{Rk}=W_{pl,y}\cdot f_y$ EN 1993-1-1 (Table 6.7)	$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}}+k_{yy}\frac{M_{y,Ed}}{1\cdot\frac{M_{y,Rk}}{\gamma_{M1}}}\leq 1$ EN 1993-1-1 (6.61)
$N_{Rk}=A\cdot f_y$ EN 1993-1-1 (Table 6.7)	$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}}\leq 1$ EN 1993-1-1 (6.62)

Details of the calculations are given in Annex A.

4.1.4. Design of joints

4.1.4.1. Generalities related to joints in hollow section lattice structures

This chapter is dedicated to design of joints and the methodology used for joints in the carriage structure will be presented here. Generally, the rules given in EN 1993-1-8 Chapter 7 are used for the design and supplemented with the recommendations from available literature for some particular cases which are not covered by EN 1993-1-8. On contrary to the design rules for joints between open sections, developed by ECCS and based on the component method, the rules for hollow section joints given in Eurocode are based on semi-empirical investigations and approved with test results. The rules for hollow section joints were developed by CIDECT (International Committee for Research and Technical Support for Hollow Section Structures). As a result, the range of validity given in EN 1993-1-8 for each particular case has to be fulfilled. The design of the structure is governed by the serviceability criterion which governs the selection of cross-sections in terms of area and second moment of area. The exact shape of cross-sections and the thickness as well is governed by the range of validity for the application of the rules given in EN 1993-1-8.

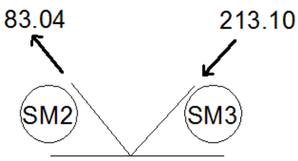
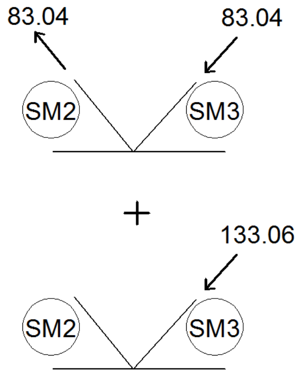
EN 1993-1-8 in Figure 7.1 gives the overview of the types of joints in hollow section lattice girders covered by the code, with their designation (K joint, KT joint, T joint, etc.). The code does not clearly state whether the classification is determined by the geometry or by the loading. According to [Wardenier et al., 2010] a joint has to be classified based on the method of force transfer, not on the physical appearance, as follows:

- A joint can be considered as a K joint if the force component perpendicular to the chord is equilibrated mutually by the adjacent braces joined in a node (with a tolerance of 20%).
- If the force component normal to the chord is equilibrated by beam shear in the chord, a joint has to be classified as either a T joint or a Y joint, depending on its geometry.
- If the force component normal to the chord is transferred through the chord from one side to another, a joint has to be classified as an X joint.

As it is recommended, for cases between above-mentioned the forces should be resolved to the components acting in the patterns of the basic cases, to be checked separately and as a result their utilization ratios to be summed. For example, a joint with a K type geometry, but with different internal forces in the braces, should be resolved to two cases, a part of it where the forces in the braces are equilibrated to be calculated as a K joint and the remainder as an

Y joint and finally their utilization ratios to be summed up. This approach is applied on the joints calculated in Annex A and as an overview the final values for joint 2 (right/left side member) are given in *Table 4.6* with comparison to the case where the joint is calculated only as a K type. The axial force in brace member SM3 is 2.5 times higher than the force in brace member SM2.

Table 4.6: Influence of the classification on the design checks of joints

Calculated according to the geometry	Calculated according to the forces
	
Applied equations for the resistance of a K type joint	Applied equations for the resistance of a K type and an Y type joint
$\frac{N_{SM2,Ed}}{N_{SM2,Rd}}=\frac{83.04}{706.9}=0.1175$ $\frac{N_{SM3,Ed}}{N_{SM3,Rd}}=\frac{213.10}{677.40}=0.3145$	$\frac{N_{SM2,Ed}}{N_{SM2,Rd}}=\frac{83.04}{706.9}=0.1175$ $\frac{N_{SM3,Ed}}{N_{SM3,Rd}}=\frac{83.04}{677.40}+\frac{133.06}{391.80}=0.4621$

Although the utilization ratios are small in this case, the difference in the results is obvious. For the joint calculated without considering the ratio between the axial forces the utilization ratio is 31.45% and with considering the method of force transfer the utilization ratio is 46.21%.

It should be mentioned that the design resistance of the weld connecting the braces to the chord, stated by clause 7.3.1(4) of EN 1993-1-8, should not be less that the design resistance of the cross-sections of those braces in order to allow for non-uniform stress-distributions and sufficient deformation capacity to allow for redistribution of bending moments. Following the concept of full strength fillet weld and the derivation of full strength throat thickness for double fillet end welds given in [Jaspart and Weynand, 2016], a derivation will be done here for the throat thickness of a single fillet weld (see *Figure 4.7*) which has to be applied in the case of hollow section joints.
The design resistance of a fillet weld, given in EN 1993-1-8 (equation 4.1) is:

$$\sqrt{\sigma_{\perp}^2+3\left(\tau_{\perp}^2+\tau_{\parallel}^2\right)}\leq\frac{f_u}{\beta_w\gamma_{M2}} \text{ and } \sigma_{\perp}\leq\frac{0.9f_u}{\gamma_{M2}}$$

where:
f_u is the nominal ultimate tensile strength of the weaker part joined
β_w is the appropriate correlation factor (β_w=0.8 for steel S235)

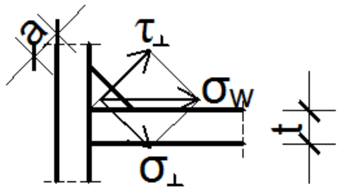


Figure 4.7: Fillet weld - stresses

$$N_{Ed,max}=\frac{tf_y}{\gamma_{M0}}$$
$$\sigma_w=\frac{N_{Ed,max}}{a}=\frac{t\cdot f_y}{a\cdot\gamma_{M0}}$$
$$\sigma_\perp=\tau_\perp=\frac{\sigma_w}{\sqrt{2}}$$
$$\tau_\parallel=0$$
$$\sigma_{eq}=\sqrt{\left(\frac{\sigma_w}{\sqrt{2}}\right)^2+3\left(\frac{\sigma_w}{\sqrt{2}}\right)^2}\leq\frac{f_u}{\beta_w\gamma_{M2}}$$
$$\frac{2\sigma_w}{\sqrt{2}}\leq\frac{f_u}{\beta_w\gamma_{M2}}$$
$$\frac{2\cdot t\cdot f_y}{\sqrt{2}\cdot a\cdot\gamma_{M0}}\leq\frac{f_u}{\beta_w\gamma_{M2}}$$
$$a\geq\frac{\sqrt{2}\cdot\beta_w\cdot f_y\cdot\gamma_{M2}}{f_u\cdot\gamma_{M0}}t$$

For steel grade S235 and the partial safety factors recommended by Eurocode (γ_{M0}=1; γ_{M2}=1) the minimum throat thickness of a single sided fillet weld is a≥0.923t. Since the majority of brace members in the structure has thickness 6 mm or less, this means that the welding can be performed in one pass in order to produce the requested full strength welds.

Further will be mentioned some particular cases that are located in the carriage structure but not covered by Eurocode, such as unidirectional K type joints and overlapped KT type joints.

4.1.4.2. Unidirectional K joints

The so-called unidirectional K joint is a joint with the geometry of a K joint but the axial forces in both braces act in the same direction, either both in compression or both in tension. The standard formulas for K type joints, given in Table 7.10 and Table 7.12 of EN 1993-1-8 are not valid in this case. The guidance is given in [Tata Steel, 2013] and recommends checking the resistance as a T joint using the equivalent bracing size as long as the failure mode is similar to the failure mode of a T joint. The equivalent single bracing width and length to be used in the standard T joint resistance formulas for rectangular braces on rectangular chords are:

$$b_{eq}=\frac{b_1+b_2}{2}$$

$$h_{eq}=\frac{h_1}{\sin\theta_1}+g+\frac{h_2}{\sin\theta_2}$$

where: g=gap (+) or orverlap (-).

After calculating the equivalent single bracing resistance it should be converted to the resistance of two actual brace members. The proportioning is suggested to be done in proportion to their internal forces $N_{i,Ed}$ as follows:

$$N_{1,Rd}=N_{eq,Rd}\frac{N_{1,Ed}}{N_{1,Ed}\sin\theta_1+N_{2,Ed}\sin\theta_2}$$

$$N_{2,Rd}=N_{eq,Rd}\frac{N_{2,Ed}}{N_{1,Ed}\sin\theta_1+N_{2,Ed}\sin\theta_2}$$

An additional recommendation is that each individual brace member should be checked in relation to the chord using the standard T or Y joint formula.

4.1.4.3. Overlap KT joints

In the carriage structure some joints have the geometry of an overlap KT joint. According to [Tata Steel, 2013] the resistance of each overlapping brace member should be calculated using the formula for brace failure from EN 1993-1-8 Table 7.10 but with the modification (see *Table 4.7*). The modification shown here is for the case where the vertical brace member overlaps two diagonal braces, illustrated in *Figure 4.8*.

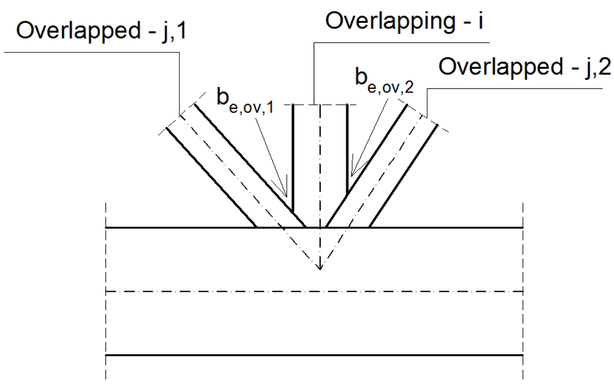


Figure 4.8: Overlap KT joint

Table 4.7: Brace failure formulas applicable for overlap KT joints

Standard formula	Modified formula
$25\%\leq\lambda_{ov}<50\%$ $N_{i,Rd}=f_{yi}t_i\left(b_{eff}+b_{e,ov}+2h_i\frac{\lambda_{ov}}{50}-4t_i\right)/\gamma_{M5}$	$25\%\leq\lambda_{ov}<50\%$ $N_{i,Rd}=f_{yi}t_i\left(b_{e,ov,1}+b_{e,ov,2}+2h_i\frac{\lambda_{ov}}{50}-4t_i\right)/\gamma_{M5}$
$50\%\leq\lambda_{ov}<80\%$ $N_{i,Rd}=f_{yi}t_i\left(b_{eff}+b_{e,ov}+2h_i-4t_i\right)/\gamma_{M5}$	$50\%\leq\lambda_{ov}<80\%$ $N_{i,Rd}=f_{yi}t_i\left(b_{e,ov,1}+b_{e,ov,2}+2h_i-4t_i\right)/\gamma_{M5}$

According to the recommendation from EN 1993-1-8 the design of the overlapped brace member j can be based on the efficiency ratio of the overlapping brace member to the overlapped brace member as follows:

$$N_{j,Rd}=N_{i,Rd}\frac{A_j f_{yj}}{A_i f_{yi}}$$

In the case of an unidirectional KT joint, additionally to the brace failure check using the modified formula, the chord side failure, chord side wall buckling or punching shear check (depending on the coefficient β) should be performed using the similar approach to the one presented in sub-chapter 4.1.4.1. for unidirectional K joints. The only difference is related to the equivalent single bracing width and length, which are calculated in this case as follows:

$$b_{eq}=\frac{b_1+b_2+b_3}{3}$$

$$h_{eq}=\frac{h_1}{\sin\theta_1}+\frac{h_2}{\sin\theta_2}+\frac{h_3}{\sin\theta_3}+g$$

where: g=gap (+) or orverlap (-).

If the chord side failure check is not fulfilled, what occurred for joint 2 (right/left supporting beam), the cord face can be reinforced by means of the flange plate. For compression loading, EN 1993-1-8 recommends calculating the resistance using the standard formula for T,X or Y joint, where the chord thickness t_0 is replaced with the thickness of the plate t_p and k_n=1.

EN 1993-1-8 states in Table 7.8 that the connection between the braces and chord face has to be checked for shear is the overlap exceeds $\lambda_{ov,lim}$ or if the braces are rectangular sections with $h_i<b_i$ and/or $h_j<b_j$, but the formula is not given. [Tata Steel, 2013] suggests checking the shear resistance by applying the following formula:

$$V_L\leq\left[\frac{f_{ui}}{\sqrt{3}}\frac{\left[\left(\frac{100-\lambda_{ov}}{100}\right)2h_i+b_{eff,i}\right]t_i}{\sin\theta_i}+\frac{f_{uj}}{\sqrt{3}}\frac{\left(2h_j+c_s b_{eff,j}\right)t_j}{\sin\theta_j}\right]/\gamma_{M5}$$

when $\lambda_{ov}\geq100\%$ the formula is modified to:

$$V_L\leq\frac{f_{uj}}{\sqrt{3}}\frac{\left(2h_j+b_j+b_{eff,j}\right)t_j}{\sin\theta_j}/\gamma_{M5}$$

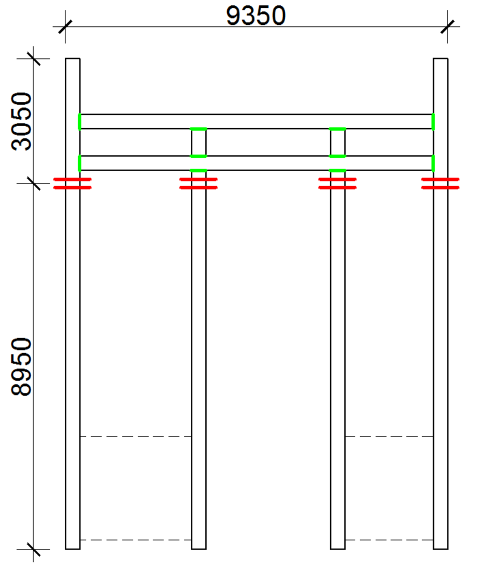
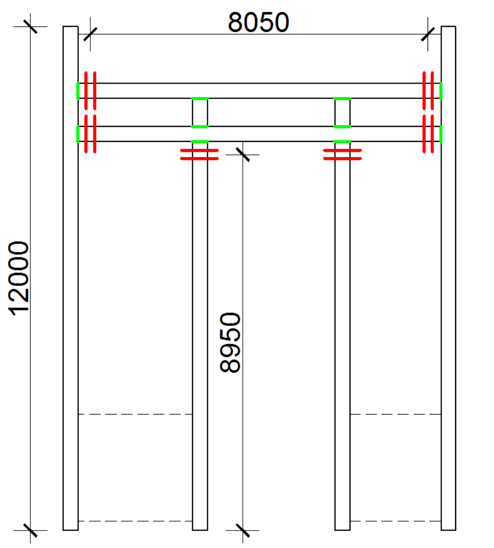
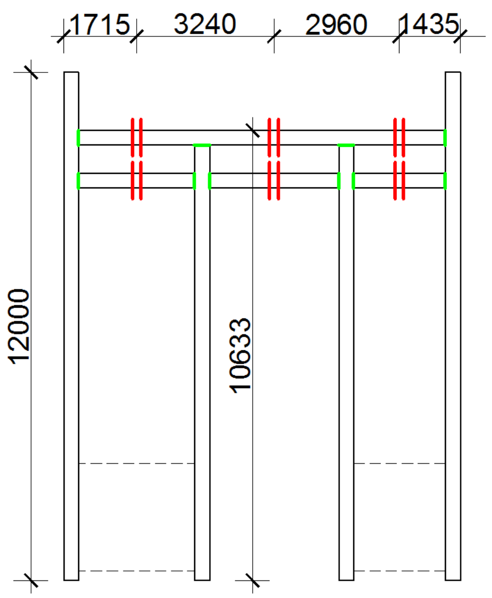
where:
c_s=1 when hidden toe is not welded
c_s=2 when hidden toe is welded

These formulas can be used for KT joints as well by applying the appropriate effective width.

4.1.4.4. Site joints

The carriage structure is intended to be produced in a workshop and later transported to the construction site and erected. The size of pieces coming from the workshop should be as large as possible and the maximum size depends mainly on the traffic regulations. For the so-called site joints, bolted splice joints with end plates are selected. In order to chose the best position, a parametric study is presented in *Table 4.8*.

Table 4.8: Parametric study - site joints positions

Option	Plan view	Remarks
A		Chords: 8 splice joints in total (red); Braces: 8 splice joints in total; 20 but welds to be done in a workshop to connect the chords (green); Max. tensile forces: Right/left side member Chord: N_{Ed}=407.75 kN Braces are in compression Right/left supporting beam Chord: N_{Ed}=588.52 kN Braces: $N_{Ed}\approx$0 kN
B		Chords: 12 splice joints in total (red); Braces: 12 splice joints in total; 20 but welds to be done in a workshop to connect the chords (green); Max. tensile forces: Right/left side member Chord: N_{Ed}=407.75 kN Braces are in compression Connecting beam Chord: N_{Ed}=303.88 kN Braces: N_{Ed}=365.29 kN
C		Chords: 12 splice joints in total (red); Braces: 0 splice joints; 20 but welds to be done in a workshop to connect the chords (green); Max. tensile forces: Connecting beam Chord: N_{Ed}=293.36 kN

Generally, all three possible options comply with the traffic regulations. Although the tensile forces in the chords for Option B at the positions of splices are lower compared to Option A, and the number of butt welds is the same, Option A is more favorable since it has less joints in total to be produced and the braces are in compression. Since joints between the braces and chords for Option C are gap K type and the gap length has a quite big value because of the small angle between the brace and chord, the site joint could be placed in the gap. This results in the fact that the site joints are necessary only for the chords. On the other hand, this may cause some problems in the construction phase, since certain chords in the connecting beam work as a cantilever before the final stage (see *Figure 4.9*).

Taking into account all above-mentioned, Option A is selected for the carriage structure and the calculation details can be found in Annex A.

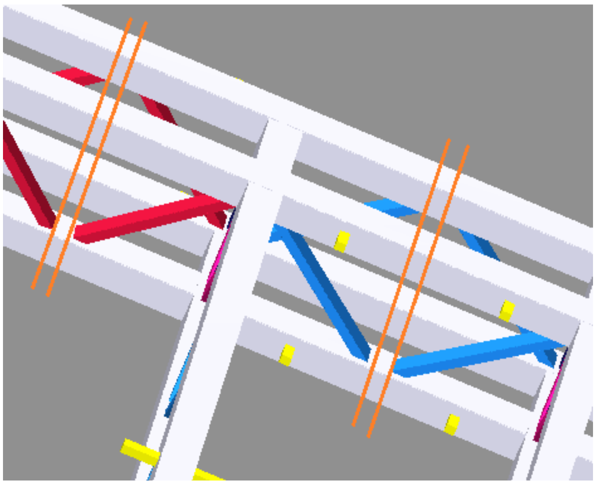


Figure 4.9: Site joints - Option C

Joints in compression zones should be designed to transmit a certain amount of tensile forces. According to EN 1993-1-8 clause 6.2.7.1(14), splice material should transmit at least 25% of the maximum compressive force in the column, provided that the members are prepared for full contact in bearing. This clause is related to column splices connecting H or I section. Furthermore, [Kurobane et al., 2004] recommends designing the column for a tensile load equal to 20% of the column capacity. Although this approach is dedicated to columns, it is applied here as well since the lack of the design recommendations for hollow section trusses.

For the case of the carriage structure under the study and selected option A, and for the brace in the right/left supporting beam, where the axial force is $N_{Ed} \approx 0$ kN, the splice is designed to transmit a load equal to 20% of the brace capacity. The brace in the right/left side member is designed to transmit 25% of the compressive force since the brace is in compression. Besides axial tension loading the chords are loaded by bending moments as well. In order to account this, [Packer et al., 2009] suggests designing a joint using a hypothetical effective axial load, as follows:

$$N_{Ed,eff} = \left(\frac{N_{Ed}}{A} + \frac{M_{Ed}}{W}\right) A$$

where:
A is the cross-sectional area
W is the elastic or plastic section modulus

It should be mentioned that this method is conservative as long as it applies the maximum tensile stress (which occurs only in the edge fiber of the cross-section) to the whole cross-section.

Eurocode does not provide the design procedure for splice joints in hollow section structures. For the design, the method given by [Packer et al., 2009] can be used and it will be shortly presented here. The design method is based on a modified T stub design procedure. Among two possibilities of the so-called rectangular flange plate joint, which are with bolts along two sides of RHS and with bolts along four sides of RHS, two sided bolts are selected for splice joints in the carriage structure. The method is valid for the case where the bolts are placed inside the space that is limited by the width of the cross-section.

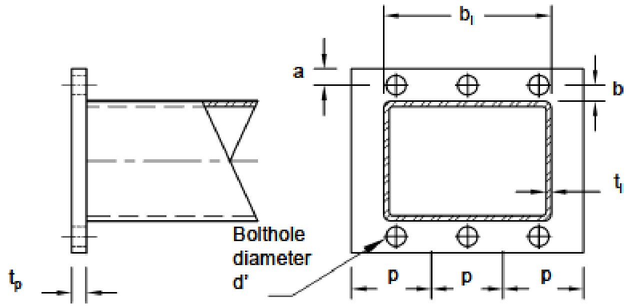


Figure 4.10: Distances in a splice joint (source: Packer et al., 2009)

First, a trial end plate thickness (t_p) can be calculated from the following condition:

$$\sqrt{\frac{KP_f}{1+\delta}} \leq t_p \leq \sqrt{KP_f}$$

by substituting:

$$P_f = \frac{N_{Ed}}{n}$$

$$K = \frac{4b' 10^3}{f_{yp} p}$$

$$\delta = 1 - \frac{d'}{p}$$

where:
n is the number of bolts
a and b are given in *Figure 4.10*

Second, the ratio α represents the reletion of the bending moment per unit plate width at the bolt line, to the bending moment per unit plate width at the inner plastic hinge. The value is calculated as follows:

$$\alpha = \left(\frac{KF_{t,Rd}}{t_p^2} - 1\right)\left(\frac{a+d/2}{\delta(a+b+t_i)}\right) \geq 0$$

where:
$F_{t,Rd}$ is the tensile resistance of a bolt, calculated according to Table 3.4 of EN 1993-1-8

Third, the joint factored resistance F_{Rd} can be obtained from:

$$F_{Rd} = \frac{t_p^2(1+\delta\alpha)}{K} \geq N_{Ed}$$

In addition, the actual total bolt tension, including prying can be calculated from:

$$T_f \approx P_f \left(1+\frac{b'}{a'}\left(\frac{\delta\alpha}{1+\delta\alpha}\right)\right) \leq N_{t,Rd}$$

by using the modified ratio α from the equation:

$$\alpha=\left(\frac{KP_f}{t_p^2}-1\right)\frac{1}{\delta} \geq 0$$

where:
$a'=a+d/2$
$b'=b-d/2+t_i$
$a \leq 1.25b$

4.1.5. Material specification

After the detailed design stage the weight of the structure can be estimated precisely, considering a real length of each structural member and multiplying it by its unit weight (per m^1). The summary is given in *Table 4.9* for each cross-sectional size used in the structure, as well as for additional plates used in the structure in *Table 4.10*. Detailed tables, with the exact length of each member and its weight can be found in Annex A.

Table 4.9: Solution 6-1, summary of the weight (structural members)

Section	Total length [m]	Weight [kg/m^1]	Total weight [kg]
SHS 350x350x12.5	122.33	127	15536.16
RHS 200x100x6	49.26	26.4	1300.47
RHS 200x100x8	19.45	33.9	659.22
RHS 160x80x5	14.18	17.5	248.15
SHS 150x150x6	18.40	26.4	485.76
SHS 200x200x10	12.41	57	707.48
HEB 360	30.74	142	4365.65
HEA 800	13.20	224	2956.80
IPE 550	5.07	105	537.42
		Σ	**26797.11**

Table 4.10: Solution 6-1, summary of the weight (additional items)

Item	Dimensions [mm]	Weight [kg]	Quantity	Total weight [kg]
End plate (chord)	520x400x22	35.92	16	574.75
End plate (brace SB12)	220x210x12	4.35	8	34.82
End plate (brace SM11)	310x220x12	6.42	8	51.40
Chord face stiffener	500x350x15	20.06	2	41.21
			Σ	**702.17**

This Thesis does not analyze in detail joints between the brackets and trusses, sub-assemblies intended to fix roller bogies to the structure, sub-assemblies for fixing the platform columns and caissons, etc. In order to compensate this and estimate the final weight of the structure, all these parts will be accounted as 10% of the structural weight (see *Table 4.11*).

Table 4.11: Solution 6-1, summary of the weight (total)

Item	Weight [kg]
Structural members	26797.11
Plates	702.17
+10% of the structural weight	2679.71
Total	**30178.99**

As an overview of the contribution of parts of the structure to the total weight *Table 4.12* is given. The braces contribute only with 11.93% to the total weight, but on contrary they require more labor for the production than other parts of the structure.

Table 4.12: Solution 6-1, contribution of parts of the structure to the total weight

Part of the structure	Weight [kg]	Contribution [%]
Chords	15536.16	57.98
Braces	3197.64	11.93
Brackets	8063.31	30.09
Total	26797.11	100.00

4.2. Bolted solution made of hollow section chords and angles as braces (Solution 6-2)

4.2.1. Improvements and final layout of the structure

This chapter will present the final layout of Solution 6-2 and the methodology used during the design will be given in the following sub-chapters. The computation details can be found in Annex B.

Generally, Solution 6-2 is a variant of Solution 6-1, developed upon a suggestion of the DREVER International representatives. This solution has been conceived with the aim to analyze the feasibility of the carriage structure in the case where the fabrication facilities are limited. This means that a part of the fabrication process is moved from the workshop to the site. In addition, the structure can be transported easily because all structural parts can fit in one shipping container and later connected on a site. The layout, shown in *Figure 4.11* with its cross-sections, is almost the same as the layout of Solution 6-1. In terms of the geometry, both solutions are completely the same. The only difference is related to the brace members, which are in Solution 6-2 composed of hot rolled L profiles (single and double configuration), connected to the chords by means of bolted connections. To obtain a bolted connection between the brace and the chord, a gusset plate welded to the chord should be used. The thickness of all gusset plates in the structure is 12 mm. The chords remain unchanged compared to Solution 6-1. Only three sizes of L profiles are used for the whole structure to simplify the procurement and on the other hand the variation among single and double configuration is performed (depending on the necessary stiffness) with the aim to save the material (see *Figure 4.12*) The double configuration is mainly in the form of back-to-back oriented angles. A star-battened configuration is applied for the vertical braces at the intersection of the mutually perpendicular trusses, where the gusset plate on the bottom chord has perpendicular direction to the gusset plate on the upper chord (see *Figure 4.13*).

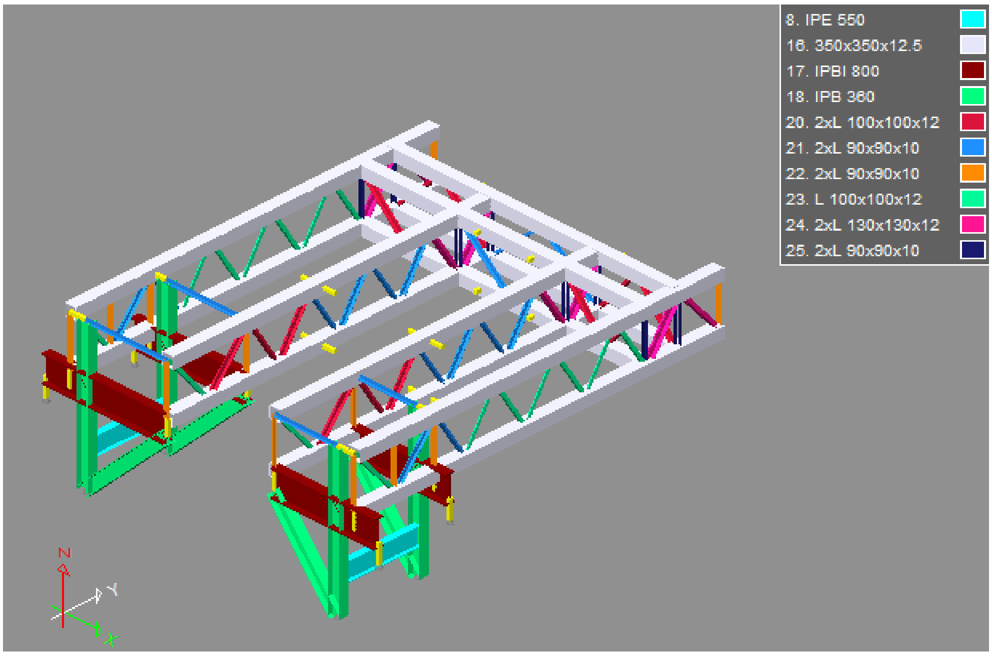


Figure 4.11: Solution 6-2, improved structural layout - 3D view

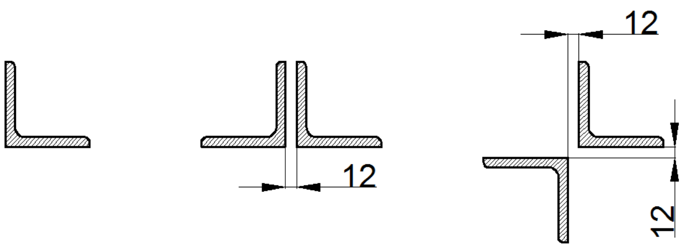


Figure 4.12: Solution 6-2, angle configurations

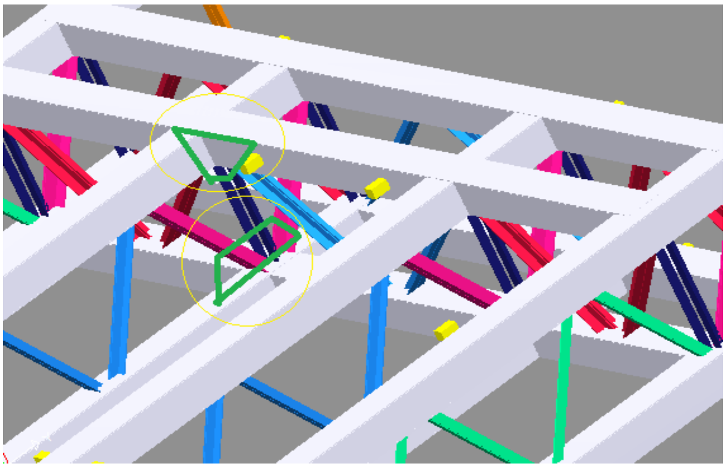


Figure 4.13: Solution 6-2, braces and gusset plates at the intersections

In order to reach a higher level of the uniformity in the fabrication, all bolt holes on the braces are placed at the same positions, keeping the values of the end distance e_1=50 mm and spacing p_1=70 mm (see Figure 4.14). For the bolts used (M12 and M16) the distances are in the compliance with Table 3.3 of EN 1993-1-8 where are defined minimum and maximum spacing, end and edge distances. An exception is the brace member SB16 (next to the vertical support) where the distances are higher and M20 bolts are used. All bolts in the structure are grade 8.8.

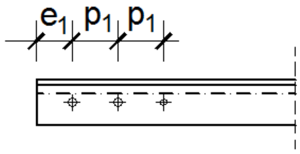


Figure 4.14: End distance and spacing of bolts for an angle

4.2.2. Serviceability limit states

The serviceability limit state checks are presented here as long as they are governing for the design of the carriage structure. The maximum values of the vertical displacements are presented in *Table 4.13* and an illustration of the deformed shape for load combination X_3 is given in *Figure 4.15*.

Table 4.13: Solution 6-2, vertical displacements [mm]

$z(X_1)$	1.28	≤6.85
$z(X_2)$	6.79	
$z(X_3)$	6.81	
$z(X_4)$	6.78	

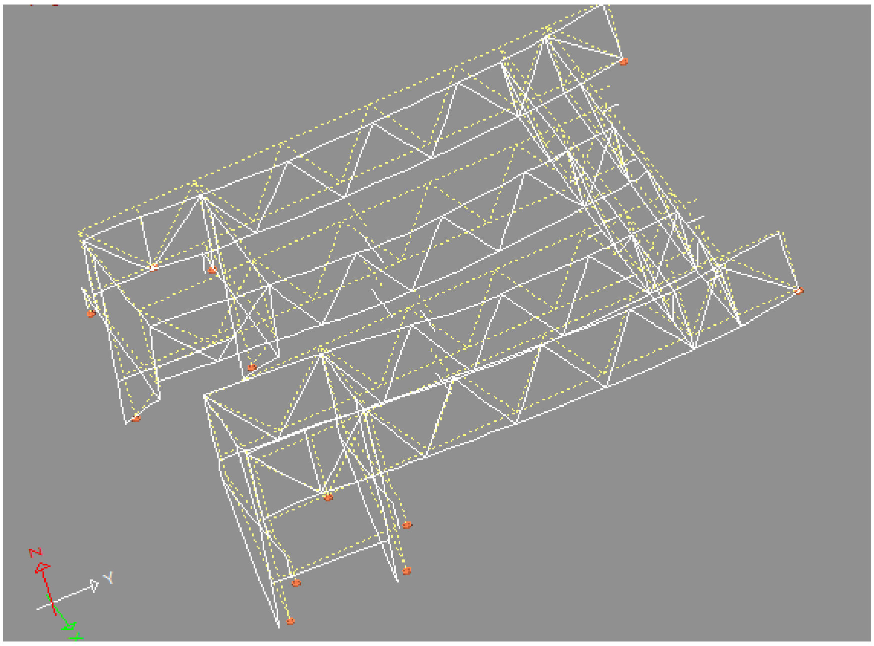


Figure 4.15: Solution 6-2, deformed model for load combination X_3

4.2.3. Ultimate limit states

4.2.3.1. Design of cross-sections

Generally, the majority of the methodology used in the design of Solution 6-1 applies for Solution 6-2 as well. In this sub-chapter will be presented only the facts that differ from the statements given in sub-chapter 4.1.3.1 and they are related to the brace members.

According to clause 3.10.3(2) of EN 1993-1-8, a single angle in tension connected by a single row of bolts in one leg may be treated as concentrically loaded over an effective net section.

The design formulas are given in *Table 4.14*. In the carriage structure, all braces are connected by either two or three bolts in a single row to the gusset plate.

Table 4.14: Design of cross-sections (angle in tension) - formulas summary

Situation	Design resistance	Design check
Axial tension	$N_{pl,Rd}=\frac{A\cdot f_y}{\gamma_{M0}}$ EN 1993-1-1 (6.6) $N_{u,Rd}=\frac{\beta_2 A_{net}\cdot f_u}{\gamma_{M2}}$ EN 1993-1-8 (3.12) $N_{u,Rd}=\frac{\beta_3 A_{net}\cdot f_u}{\gamma_{M2}}$ EN 1993-1-8 (3.13)	$\frac{N_{Ed}}{N_{t,Rd}}\leq 1$ EN 1993-1-1 (6.5) where: $N_{t,Rd}=\min(N_{pl,Rd}, N_{u,Rd})$

The reduction factors β_2 (2 bolts) and β_3 (3 bolts) are dependent on the pitch p_1 and the bolt hole diameter d_0. Their values can be found in Table 3.8 of EN 1993-1-8 and they vary between 0.4 and 0.7.
The partial safety factors are γ_{M0}=1.00 and γ_{M2}=1.25 as it is recommended by Eurocode.
Details of the calculations are given in Annex B.

4.2.3.2. Stability of structural members

The design methodology for stability checks of chords, described in 4.1.3.2. applies here as well. Angles as web members are the specific case, since they are connected to the gusset plate by bolts in one leg. According to Annex BB1 of EN 1993-1-1, if an angle is fixed appropriately (at least two bolts if bolted) the eccentricities may be neglected and end fixities allowed for in the design of angles as web members in compression. The effective relative slenderness should be calculated in this case as:

$$\bar{\lambda}_{eff,v}=0.35+0.7\bar{\lambda}_v$$
$$\bar{\lambda}_{eff,y}=0.50+0.7\bar{\lambda}_y$$
$$\bar{\lambda}_{eff,z}=0.50+0.7\bar{\lambda}_z$$

The buckling length should be taken as equal to the system length for angles designed using the effective relative slenderness.

Resulting from the fact that the design of the carriage structure is governed by the serviceability limit states, the braces composed of back-to-back oriented angles satisfy the stability checks without a need to interconnect them, what simplifies the fabrication. This means that the buckling resistance is calculated for one single angle and multiplied by 2 in order to obtain the final resistance which has to be compared with the acting force N_{Ed}.

The vertical braces at the intersection of mutually perpendicular trusses are composed as a star-battened configuration. According to clause 6.6.4 of EN 1993-1-1, they can be designed as a single integral member provided that the maximum distance between the battens is $70i_{min}$, where i_{min} is the minimum radius of gyration of one angle. For L 90x90x10 mm, that is used in this case, 70 i_{min}=1225 mm, what means that the battens will be placed at the ends of the brace. The connection between the angle and batten is bolted to keep the consistency with all other connections. The shear force that has to be transferred by the battens is negligible compared to the resistance of a bolt M12 and the values can be found in Annex B.

4.2.4. Design of joints

4.2.4.1. Brace members connected by bolts

Generally, the connection between the brace member and gusset plate is bearing type (type A) according to Table 3.2 of EN 1993-1-8, thus the design ultimate shear resistance and the design bearing resistance should be checked, as well as the design block tearing resistance for members loaded in tension. Besides the axial forces, the bolts are loaded by bending moments due to the fact that the bolt row does not coincide with the axis of the angle. The analogy could be made to the resistance of the bolt group for a fin plate connection, given in [Jaspart and Weynand, 2016]. The bolt group resistance to shear forces should be calculated as follows:

$$V_{v,Rd}=\frac{nF_{v,Rd}}{\sqrt{1+\left(\frac{6e}{(n+1)p_1}\right)^2}}$$

where:
$F_{v,Rd}$ is the resistance of a single bolt per shear plane (EN 1993-1-8, Table 3.4)
n is the number of bolts
e is the eccentricity between the bolt row and the axis of the angle
p_1 is the spacing between bolts

For the bolts used in the carriage structure (grade 8.8), the design shear resistance $F_{v,Rd}$ is given in *Table 4.15*.

Table 4.15: Bolt shear resistance per shear plane

Bolt size	$F_{v,Rd}$ [kN]
M12	43.43
M16	77.21
M20	120.60

Using the same analogy, the design bearing resistance of the bolt group should be calculated as:

$$V_{b,Rd}=\frac{n}{\sqrt{\left(\frac{1}{F_{b,lg,Rd}}\right)^2+\left(\frac{\beta_0}{F_{b,tr,Rd}}\right)^2}}$$

where:

$$\beta_0=\frac{6e}{(n+1)p_1}$$

$F_{b,lg,Rd}$ and $F_{b,tr,Rd}$ are the design bearing resistances of a plate per bolt, in the longitudinal and transversal direction, respectively (EN 1993-1-8, Table 3.4)

In addition, for members in tension, according to clause 3.10.2 of EN 1993-1-8 the block tearing resistance should be checked as well, and for a bolt group subject to eccentric loading the following formula applies:

$$V_{eff,Rd}=0.5f_u\frac{A_{nt}}{\gamma_{M2}}+\frac{1}{\sqrt{3}}\frac{f_yA_{nv}}{\gamma_{M0}}$$

where:
A_{nt} is net area subjected to tension
A_{nv} is net area subjected to shear
(see *Figure 4.16*)

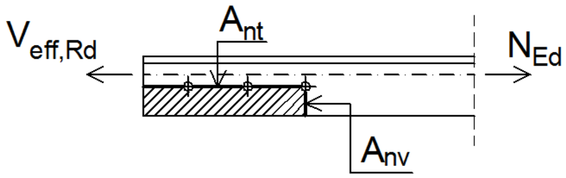


Figure 4.16: Areas subjected to tension and shear for the block tearing resistance check

4.2.4.2. Gusset plates

Generally, a gusset plate provides a simple way to connect the axially loaded brace members to the chord by the means of bolts in shear. Local failure checks should be conducted for the bearing resistance of the gusset plate, and the block tearing resistance where is relevant using the methodology explained in the previous sub-chapter. Resulting from the fact that the gusset plate connects the braces oriented in different directions to the chord, a complex stress state occurs in the gusset plate. In order to check the tension/compression resistance of a gusset plate, as well as its stability, the peak tensile/compressive stress should be calculated. According to many authors [Jaspart and Weynand, 2016], [Thornton et al. 2011] etc., the peak stress occurs on the Whitmore section (Whitmore, 1952). The Whitmore section is placed at the last row of fasteners, and the so-called Whitmore effective width (l_{eff}) is equal to the distance between two lines starting at the first bolt row and radiating outward at 30° (see *Figure 4.17*).

$$l_{eff}=2(n-1)p_1\tan 30^{\circ}$$

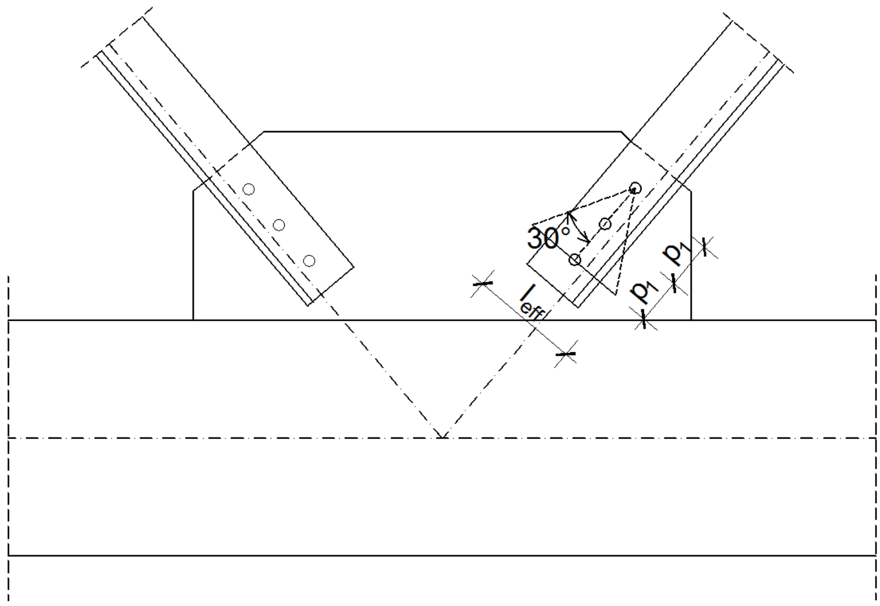


Figure 4.17: Whitmore section

On the Whitmore effective width, for supported members in tension the following should be checked:

- Design plastic resistance
- Net section resistance

while members in compression should be checked for:

- Design plastic resistance
- Stability (flexural buckling)

Regarding the flexural buckling of a gusset plate [Jaspart and Weynand, 2016] suggests checking a gusset plate as a column following the rules given in EN 1993-1-1, with the buckling length equal to:

$$L_{cr}=K\frac{l_1+l_2+l_3}{3}$$

The distances l_1 to l_3 are presented graphically in *Figure 4.18* and the buckling length coefficient K should be taken equal to 0.65.

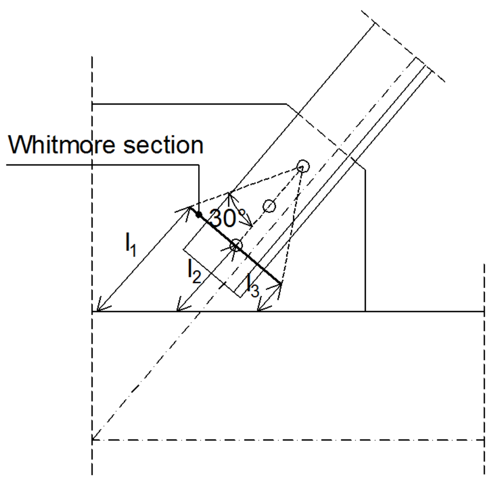


Figure 4.18: Whitmore section and buckling lengths

Besides the local checks, according to [Jaspart and Weynand, 2016] a gusset plate should be checked for the global cross section failure under the resultant of forces transferred by more than one of the supported members. The critical section is given in *Figure 4.19* and it should be checked for normal and shear stress according to the theory of elasticity. The members are designed with the centre lines noding, what means that the critical section is subjected to axial forces, shear forces and bending moments.

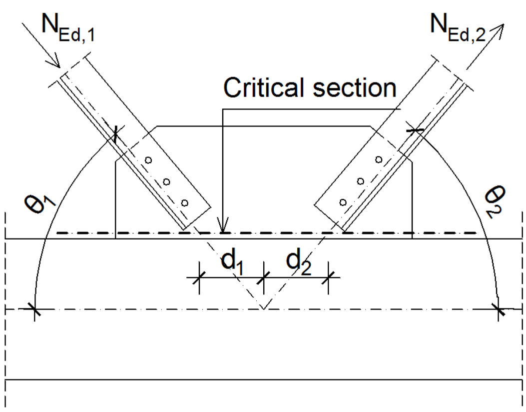


Figure 4.19: Gusset plate - critical cross-section

Forces acting on the critical section are:

- Axial force $N_{Ed}=N_{Ed,1}\sin\theta_1-N_{Ed,2}\sin\theta_2$
- Shear force $V_{Ed}=N_{Ed,1}\cos\theta_1+N_{Ed,2}\cos\theta_2$
- Bending moment $M_{Ed}= N_{Ed,1}d_1\sin\theta_1+ N_{Ed,2}d_2\sin\theta_2$

The design check is in terms of the elastic stresses, as follows:

$$\frac{N_{Ed}}{A}\pm\frac{M_{Ed}}{W}\leq f_y$$

$$\frac{1.5V_{Ed}}{A}\leq\frac{f_y}{\sqrt{3}}$$

In the calculations the critical section is assumed to be adjacent to the chord, neglecting the fillet weld thickness.

The gusset plate is welded to the chord by the means of double sided fillet welds. In order to allow the predicted failure mode to occur before the weld failure, the full strength double fillet welds are applied, what is stated in Chapter 7 of EN 1993-1-8 and explained in this Thesis in the chapter dedicated for hollow section joints (see 4.1.4.1.). The minimum throat thickness for a full strength double sided fillet weld, for steel grade S235 should be $a\geq0.46t$ as it is stated in [Jaspart and Weynand, 2016]. As long as the thickness of the gusset plate is 12 mm, the throat thickness of the weld should be 6 mm, what is an advantage from the fabrication point of view because welds up to 6 mm can be produced in one pass. A guidance is given in Table 7.13 of EN 1993-1-8 for the so-called longitudinal plate. The relevant failure mode is the chord face failure, what results from low β ratios (the gusset plate thickness to the chord width). Due to the fact that the gusset plate transfers axial forces, shear forces and bending moments to the chord face (see forces acting on the critical section in 4.2.4.3. and *Figure 4.19*) an additional guidance is necessary, because EN 1993-1-8 provides a formula only for the axially loaded longitudinal plate. To avoid the punching shear failure, a criterion is given in [Kurobane et al., 2004] for simple shear joints to hollow section columns and according to [Packer et al., 2009] it is applicable for longitudinal gusset plates as well. The criterion is to ensure that the tension resistance of the tab under axial load is less than the shear resistance of the RHS wall along two planes. To satisfy this criterion, the following formula applies:

$$t_p<1.16\frac{f_{y0}}{f_{yp}}t_0$$

where:

t_p is the thickness of the gusset plate
t_0 is the thickness of the chord
f_{y0} is the yield strength of the chord
f_{yp} is the yield strength of the gusset plate

For the carriage structure, where the same steel grade is used for all parts of the structure, and t_0=12.5 mm, the criterion becomes:

$$t_p<1.16\cdot\frac{235}{235}\cdot12.5$$

$$t_p<14.5\text{ mm}$$

As it was mentioned previously, the selected thickness of the gusset plate is t_p=12 mm what is in compliance with the criterion above.

EN 1993-1-8 does not provide a formula to calculate the in-plane moment resistance of the longitudinal plate. A recommendation is given in [Tata Steel, 2013] as follows:

$$M_{ip,1,Rd}=0.5N_{1,Rd}h_1$$

where:

$N_{1,Rd}$ as the axial resistance of the longitudinal plate connected to the RHS chord (calculated according to Table 7.13 of EN 1993-1-8)

h_1 is the length of the plate

Although Eurocode does not provide the resistance formula in this case it gives the requirement for the design check for connections subjected to combined bending and axial force as follows:

$$\frac{N_{i,Ed}}{N_{i,Rd}}+\frac{M_{ip,i,Ed}}{M_{ip,i,Rd}}+\frac{M_{op,i,Ed}}{M_{op,i,Rd}}\leq 1$$

where the terms are the utilization ratios for axial forces, in-plane bending moments and out-of-plane bending moments, respectively. The out-of-plane resistance is not relevant because there are no out-of-plane acting forces. For the carriage structure, generally, the length of the gusset plate has been selected as the minimum necessary to connect the members properly and to satisfy the requested geometry. For some joints, the length is slightly extended, mainly because of the bending action imposed to the joint.

For joints loaded by high compressive or tensile forces, the design criterion is rather difficult to be satisfied, resulting from the thin RHS face. This situation can be found in the carriage structure as well, for joints at the vertical supports (right/left supporting beam, joint 2 and joint 8). Theoretically, in order to satisfy the design criterion for joint 8 using the gusset plate only, it should have the length of 1800 mm, what is not feasible in practice. The easiest way to stiffen this joint is by an additional pate, perpendicular to the gusset and placed on the chord face, forming a T stub. The plate increases the footprint of the gusset and consequently increases the design resistance. According to [Packer et al., 2009], the stiffened joint can be calculated using the standard formula for a T joint (RHS-to-RHS) given in EN 1993-1-8, provided that the stiffening plate is rigid enough. In the T joint design formula, the brace member width (b_1) should be replaced by the stiffening plate width (b_{sp}). The plate is rigid enough if the following criterion is satisfied:

$$t_{sp}\geq 0.5t_0e^{3\beta^*}$$

where:

t_{sp} is the thickness of the stiffening plate

t_0 is the thickness of the chord

b_0 is the width of the chord

$\beta^*=\dfrac{b_{sp}-t_1}{b_0-t_0}$

The geometry of a stiffened joint is presented graphically in *Figure 4.20.*

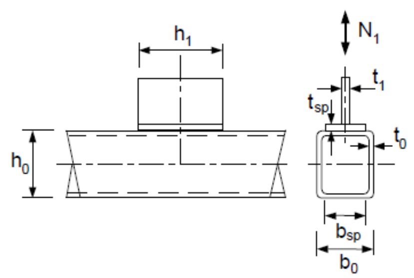


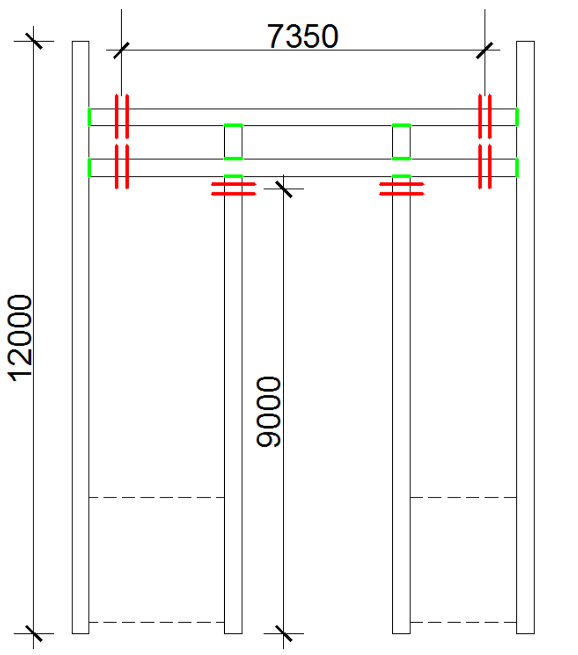
Figure 4.20: Stiffened joint geometry (source: Packer et al., 2009)

For the carriage structure, the two critical joints are stiffened using plates 190 mm wide, what is the minimum width that satisfies the design checks. In order to consider this plate rigid enough, the thickness should be $t_{sp} \geq 30.41$ mm. It should be mentioned that the necessary thickness of the stiffening plate has an exponential relation to its width according to the given formula, what means that the width should be applied as smaller as possible in order to avoid disproportionate thicknesses.

4.2.4.3. Site joints

Solution 6-2 is intended to be assembled on a site by connecting the braces to the chords by the means of bolted connections. As a result, splice joints are placed at the chords only and they should be outside the gusset plates in order to avoid complex details. Considering this in addition to the facts from the case study given in 4.1.4.3. related to Solution 6-1, the scheme given in *Table 4.16* could be regarded the most reasonable for Solution 6-2.

Table 4.16: Site joints (Solution 6-2)

	Chords: 12 splice joints in total (red); 20 but welds to be done in a workshop to connect the chords (green); Max. tensile forces: Right/left side member Chord: N_{Ed}=403.55 kN Connecting beam Chord: N_{Ed}=303.10 kN

The splice joint positions are analogous to Option B for Solution 6-1 what is advantageous in this case as long as the right/left supporting beam is 12 m long what is equal to the standard mill length of a section. It is worthy to be mentioned that the selection of the splice joint positions in Solution 6-1 was influenced by the internal forces in brace members as well, what is not relevant for Solution 6-1. The top chords of the connecting beam will come from the workshop as an assembly, as well as the bottom chords.

The design methodology for splice joints is already explained in 4.1.4.3. and the computation details for Solution 6-2 are given in Annex B.

4.2.5. Material specification

Similarly to sub-chapter 4.1.5. dedicated for Solution 6-1, the summary of the calculated weight is given in *Table 4.17* for each cross-sectional size used in the structure, as well as for additional plates used in the structure in *Table 4.18*. Detailed tables, with the exact length of each member and its weight can be found in Annex B.

Table 4.17: Solution 6-2, summary of the weight (structural members)

Section	Total length [m]	Weight [kg/m^1]	Total weight [kg]
SHS 350x350x12.5	122.33	127	15536.16
L 90x90x10	100.48	13.4	1346.38
L 100x100x12	52.89	17.8	941.51
L 120x120x13	20.40	23.6	481.44
HEB 360	30.74	142	4365.65
HEA 800	13.20	224	2956.80
IPE 550	5.07	105	537.42
		Σ	**26271.88**

Table 4.18: Solution 6-2, summary of the weight (additional items)

Item	Dimensions [mm]	Weight [kg]	Quantity	Total weight [kg]
End plate (chord)	520x400x22	35.92	8	287.38
End plate (chord)	520x400x18	29.39	8	235.12
Chord face stiffener	790x190x32	37.71	2	75.41
Chord face stiffener	750x190x32	35.80	2	71.59
			Σ	**669.50**

The structure consists of 72 gusset plates, where the size of each gusset plate is selected to be as smaller as possible. The total weight of the gusset plates is 1074.44 kg (see Annex B for more details).

This Thesis does not analyze in detail joints between the brackets and trusses, sub-assemblies intended to fix roller bogies to the structure, sub-assemblies for fixing the platform columns and caissons, etc. In order to compensate this and estimate the final weight of the structure, all these parts will be accounted as 10% of the structural weight (see *Table 4.19*).

Table 4.19: Solution 6-2, summary of the weight (total)

Item	Weight [kg]
Structural members	26271.88
Plates	669.50
Gusset plates	1074.44
+10% of the structural weight	2627.19
Total	**30643.01**

Table 4.20 is an overview of the contribution of each part of the structure to the final weight. Similarly to Solution 6-1, for Solution 6-2 the braces participate only with 10.17% to the final weight.

Table 4.20: Solution 6-2, contribution of parts of the structure to the total weight

Part of the structure	Weight [kg]	Contribution [%]
Chords	15536.16	59.14
Braces	2672.41	10.17
Brackets	8063.31	30.69
Total	26271.88	100.00

4.3. Final comparison between the solutions

Although some of the benefits or drawbacks have already been mentioned in the text, the main aim of this sub-chapter is to summarize the results from the detailed design stage for Solution 6-1 and Solution 6-2 and to compare them with the initial one. All these solutions were designed in such a way to satisfy the serviceability limit state criterion, as well as the ultimate limit state design checks, what is comprehensively explained in the previous chapters. To compare the estimated weight between the solutions *Table 4.21* is given.

Table 4.21: Estimated final weight - comparison

Solution	Weight [kg]	ΔG [kg]
Initial solution	36268.14	/
Solution 6-1	30178.99	-6089.15
Solution 6-2	30643.01	-5625.13

Both solutions, developed from Solution 6 proposed in the pre-design stage, have similar weight, what is expected due to the fact that the brace members are the only difference among them. Comparing them to the initial solution, the material saving is significant and given in numbers, approximately 6.1 t of steel for Solution 6-1 and 5.6 t for Solution 6-2. It is worthy to be mentioned that the rough estimation given in the pre-design stage underestimated the material savings, mainly because model for the calibration underestimated the dead weight (comparison between the proposed solutions was based on the vertical reactions). For instance, Solution 6 was selected mainly because of the fabrication (with the aim to replace built-up box sections with commercial hollow sections) what resulted at the end in a considerably lighter structure. Generally, there are two reasons in this case that allowed the weight reduction. The height of the trusses is 1500+350=1850 mm while the height of the box girders in the initial solution is 1165 mm, what means that Solutions 6-1 and 6-2 have higher stiffness-to-weight ratio compared to the initial solution. Basically, this height was selected in order to compose the truss with a proper geometry (approved by the DREVER International representatives as well). The second reason is related to the type of beams, since a truss girder provides more possibilities for the optimization than a built-up box. For instance, a built-up box girder is made of plates, and its cross-sectional dimensions are often constant along the axis, or changed in a certain node. On contrary, each member of a truss can be adopted with different cross-sectional size between the nodes.

Generally, a comparison between Solution 6-1 and Solution 6-2 is mainly related to the fabrication process, as long as the weight of the structure is similar for both solutions (Solution 6-2 is 464.02 kg heavier, what is negligible compared to the total weight of the structure). If a workshop is well equipped, Solution 6-1 may be advantageous. The structure can be produced almost completely in the workshop, transported to the site on a standard truck's trailer and assembled by the means of bolted connections (site joints).

If the fabrication facilities of a workshop are limited, Solution 6-2 may be applied, where a part of the fabrication process is moved from the workshop to the site. In the workshop, the following is done: cutting of the sections, drilling of the holes and welding the gusset plates to the chords. On the site the braces are connected to the chords by the means of bolted connections. In addition, the structure can be transported easily because all structural parts can fit in one shipping container and later connected on a site.

Regarding the corrosive protection, Solution 6-1 is advantageous, because hollow sections have smaller surface exposed to the environment compared to open sections.

5. Conclusions

The main aim of this thesis was to optimize the transfer carriage structure through the improvements of the structural system, where the design is guided by the serviceability limit states criteria.

On the basis of the results and their interpretation given in the previous chapters, the following conclusions can be drawn:

- Solution 6 (composed of planar trusses made of hollow sections) was selected at the pre-design stage among eight proposed solutions as the best compromise between the fabrication complexity and material savings.
- Solution 6 was improved during the detailed design and studied in two variants, namely: Solution 6-1 (Welded solution made completely of hollow sections) and Solution 6-2 (Bolted solution made of hollow section chords and angles as braces).
- The final reduction of the weight compared to the initial solution is: 6089.15 kg for Solution 6-1 and 5625.13 kg for Solution 6-2, what means that a truss girder provides more possibilities for the material savings than a built-up box.
- Design of cross-sections and stability of structural members are not governing for the design what is obvious from the design checks, where the utilization ratio is mainly below 0.5.
- Design of joints had important influence on the design, in terms of the layout and dimensions, in order to satisfy the validity limits given in EN 1993-1-8.
- The selected solutions impose smaller loads to the crane runway beams compared to the initial solution. Optimization of the runway structure could be a subject of future studies.

References

CEN, EN 13001-1: Cranes - General design - Part 1: General principles and requirements, European Committee for Standardization, 2015

CEN, EN 13001-2: Cranes - General design - Part 2: Load actions, European Committee for Standardization, 2014

CEN, EN 13001-3-1: Cranes - General design - Part 3: Limit states and proof of competence of steel structures, European Committee for Standardization, 2012

CEN, EN 1993-1-1: Eurocode 3: Design of steel structures. Part 1-1: General rules and rules for buildings, European Committee for Standardization, 2005

CEN, EN 1993-1-8: Eurocode 3: Design of steel structures. Part 1-8: Design of joints, European Committee for Standardization, 2005

DREVER International "Note de calcul pour justification du chariot pre-cooler, des poutres de roulement et des colonnes", 2014

FEM 1.001: Rules for the design of hoisting appliances, 3rd edition, European Materials Handling Federation, 1998

J. Wardenier, J.A. Packer, X.-L. Zhao, van der Vegte "Hollow sections in structural applications", ISBN 978-90-72830-86-9, 2010

J.A. Packer, J. Wardenier, X.-L. Zhao, G.J. van der Vegte, Y. Kurobane "Design guide for rectangular hollow section (RHS) joints under predominantly static loading", ISBN 978-3-938817-04-9, 2009

J.-P. Jaspart, K. Weynand "Design of joints in steel and composite structures", ISBN 978-3-433-02985-5, 2016

Tata Steel Europe Limited "Design of welded joints", 2013

Thornton W., Lini C. "The Whitmore Section", Modern steel construction, 2011

Y. Kurobane, J.A. Packer, J. Wardenier, N. Yeomans "Design guide for structural hollow section column connections", ISBN 3-8249-0802-6, 2004

A. Annex A

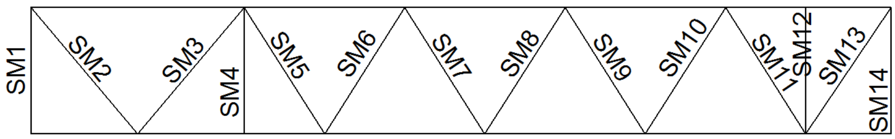
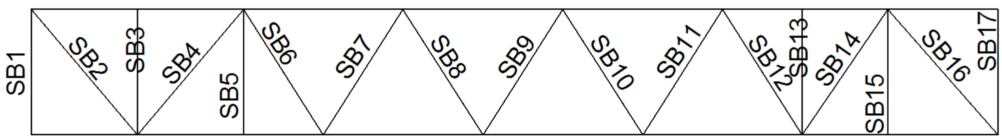
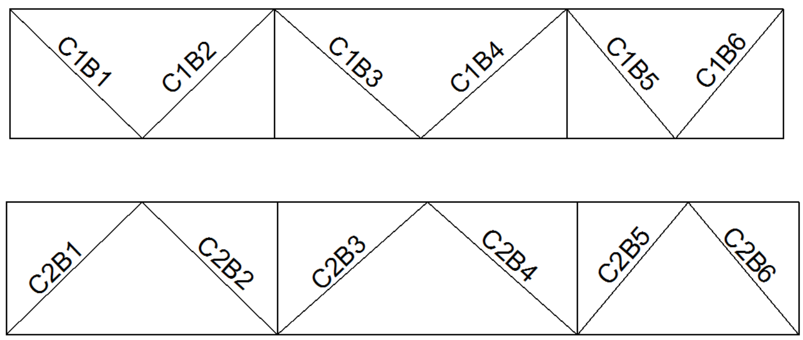
List of tables

Note: Drawings are provided at the end of the report.

Table A.1: Material properties and partial safety factors

Coeff.	Value
f_y [N/mm^2]	235
f_u [N/mm^2]	360
ε	1
γ_{M0}	1
γ_{M1}	1
γ_{M2}	1.25

Table A.2: Designation - brace members

Right/left side member	
Right/left supporting beam	
Connecting beam	

Braces loaded in tension

Table A.3: Tension forces [kN] - right/left side member (braces)

	Right side member				Left side member				
Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
SM2	17.39	78.22	83.04	73.51	16.97	72.6	78.13	67.18	83.04
SM5	45.07	249.66	253.28	246.34	46.16	252.29	256.15	248.72	256.15
SM7	13.61	71.21	76	66.52	15.49	78.73	83.81	73.75	83.81
SM10	31.51	168.3	166.12	170.67	29.89	161.65	159.18	164.31	170.67
SM12	6.96	15.55	13.81	17.34	5.31	10.7	8.91	12.52	17.34
SM13	42.71	328.4	325.31	331.76	45.6	336.21	333.18	339.53	339.53

Table A.4: Design checks (tension) - right/left side member (braces)

Member	Section	A [mm^2]	N_{Rd} [kN]	N_{Ed}/N_{Rd}
SM2	RHS 200x100x6	3360	789.6	0.105
SM5	RHS 200x100x8	4320	1015.2	0.252
SM7	RHS 200x100x6	3360	789.6	0.106
SM10	RHS 200x100x6	3360	789.6	0.216
SM12	SHS 150x150x6	3360	789.6	0.022
SM13	SHS 200x200x10	7260	1706.1	0.199

Table A.5: Tension forces [kN] - right/left supporting beam (braces)

	Right supporting beam				Left supporting beam				
Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
SB1	25.06	101.75	104.29	99.36	23.3	96.98	99.71	94.39	104.29
SB5	13.96	65.95	63.73	68.26	14.18	70.11	67.6	72.72	72.72
SB6	56.16	192.62	196.42	189.18	57	204.21	208.33	200.45	208.33
SB8	27.04	89.86	92.4	87.5	27.79	97.47	100.15	94.96	100.15
SB10	17.81	78.16	80.5	75.93	18.49	85.47	87.92	83.13	87.92
SB14	63.72	249.02	243.32	255.13	66.11	272.03	265.56	278.92	278.92
SB15	44.38	161.61	159.82	163.69	49.06	182.01	180.09	184.23	184.23

Table A.6: Design checks (tension) - right/left supporting beam (braces)

Member	Section	A [mm^2]	N_{Rd} [kN]	N_{Ed}/N_{Rd}
SB1	RHS 200x100x6	3360	789.6	0.132
SB5	SHS 150x150x6	3360	789.6	0.092
SB6	RHS 200x100x6	3360	789.6	0.264
SB8	RHS 160x80x5	2240	526.4	0.190
SB10	RHS 160x80x5	2240	526.4	0.167
SB14	SHS 200x200x10	7260	1706.1	0.163
SB15	SHS 150x150x6	3360	789.6	0.233

Table A.7: Tension forces [kN] - connecting beam (braces)

Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
C1B1	71.16	361.52	358.05	365.44	365.44
C1B3	11.96	40.11	41.41	38.88	41.41
C1B4	22.66	90.72	91.53	90.06	91.53
C1B6	63.88	333.43	329.79	337.47	337.47
C2B2	77.91	322.18	318.17	326.66	326.66
C2B5	91.87	375.19	371.09	379.88	379.88

Table A.8: Design checks (tension) - connecting beam (braces)

Member	Section	A [mm^2]	N_{Rd} [kN]	N_{Ed}/N_{Rd}
C1B1	RHS 200x100x8	4320	1015.2	0.360
C1B3	RHS 200x100x6	3360	789.6	0.052
C1B4	RHS 200x100x6	3360	789.6	0.116
C1B6	RHS 200x100x8	4320	1015.2	0.332
C2B2	RHS 200x100x8	4320	1015.2	0.322
C2B5	RHS 200x100x8	4320	1015.2	0.374

Braces loaded in compression

Table A.9: Cross-section classification for the braces loaded in compression

Cross-section	c [mm]	t [mm]	c/t	Class
RHS 200x100x6	182	6	30.33	1
RHS 200x100x8	176	8	22	1
RHS 160x80x5	145	5	29	1
SHS 150x150x6	132	6	22	1
SHS 200x200x10	170	10	17	1
c≈b-3t Limiting values (S235): Class 1: c/t=33 Class 2: c/t=38 Class 3: c/t=42				

Table A.10: Compression forces [kN] - right/left side member (braces)

	Right side member				Left side member				
Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
SM1	15.79	150.01	154.04	146.08	15.27	145.13	149.71	140.64	154.04
SM3	31.83	206.48	213.06	200.1	31.93	203.29	210.7	196.08	213.06
SM4	25.04	193.28	192.85	193.85	25.93	198.77	197.88	199.83	199.83
SM6	42.23	329.06	333.96	324.44	44.07	336.3	341.55	331.33	341.55
SM8	tension	46.36	51.43	41.22	tension	53.74	59.15	48.27	59.15
SM9	22.13	150.67	148.83	152.64	20.21	143.17	140.99	145.48	152.64
SM11	40.84	364.42	359.36	369.75	40.42	361.31	356.04	366.84	369.75
SM14	29.34	167.39	164.44	170.53	31.57	172.78	169.98	175.77	175.77

Table A.11: Design checks (compression + stability) - right/left side member (braces)

Mem.	Section	A [mm^2]	$N_{pl,Rd}$ [kN]	L_{sys} [mm]	L_{cr} [mm]	i_y [mm]	i_z [mm]	min i [mm]	$\bar{\lambda}$	Φ	χ	$N_{b,Rd}$ [kN]	$N_{Ed}/N_{b,Rd}$
SM1	200x100x6	3360	789.6	1500	1125.0	71.2	41.4	41.4	0.289	0.564	0.955	753.72	0.20
SM3	200x100x8	4320	1015.2	1963.8	1472.9	69.5	40.4	40.4	0.388	0.621	0.904	917.25	0.23
SM4	150x150x6	3360	789.6	1500	1125.0	59.3	59.3	59.3	0.202	0.521	0.999	788.78	0.25
SM6	200x100x6	3360	789.6	1777.7	1333.3	71.2	41.4	41.4	0.343	0.594	0.927	732.04	0.47
SM8	200x100x6	3360	789.6	1777.7	1333.3	71.2	41.4	41.4	0.343	0.594	0.927	732.04	0.08
SM9	200x100x6	3360	789.6	1777.7	1333.3	71.2	41.4	41.4	0.343	0.594	0.927	732.04	0.21
SM11	200x200x10	7260	1706.1	1777.7	1333.3	76.5	76.5	76.5	0.186		1	1706.1	0.22
SM14	150x150x6	3360	789.6	1500	1125.0	59.3	59.3	59.3	0.202	0.521	0.999	788.78	0.22
$L_{cr}=0.75L_{sys}$ α=0.49													

Table A.12: Compression forces [kN] - right/left supporting beam (braces)

	Right supporting beam				Left supporting beam				
Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
SB2	28.55	97.71	99.93	95.67	25.41	87.02	89.71	84.48	99.93
SB3	33.85	127.03	128.9	125.37	33.22	128.36	130.28	126.65	130.28
SB4	89.69	352.59	356.35	349.4	90.29	367.49	370.95	364.61	370.95
SB7	32.08	100.49	103.27	97.92	32.73	107.84	110.72	105.17	110.72
SB9	22.8	84.92	87.4	82.58	23.54	92.59	95.2	90.13	95.2
SB11	15.57	78.09	80.33	75.95	16.5	85.94	88.3	83.68	88.3
SB12	3.92	4.83	1.52	8.17	3.17	2.21	tens.	5.89	8.17
SB13	25.34	124.31	123.47	125.3	25.47	131.42	130.37	132.63	132.63
SB16	155.05	576.36	565	588.71	166.28	640.66	627.76	654.62	654.62
SB17	7.06	15.31	15.99	14.67	8.05	18.83	19.62	18.09	19.62

Table A.13: Design checks (compression + stability) - right/left supporting beam (braces)

Mem.	Section	A [mm^2]	$N_{pl,Rd}$ [kN]	L_{sys} [mm]	L_{cr} [mm]	i_y [mm]	i_z [mm]	min i [mm]	$\bar{\lambda}$	Φ	χ	$N_{b,Rd}$ [kN]	$N_{Ed}/N_{b,Rd}$
SB2	200x100x6	3360	789.6	1963.8	1472.9	71.2	41.4	41.4	0.379	0.616	0.908	717.30	0.14
SB3	150x150x6	3360	789.6	1500.0	1125.0	59.3	59.3	59.3	0.202	0.521	0.999	788.78	0.17
SB4	200x100x6	3360	789.6	1963.8	1472.9	71.2	41.4	41.4	0.379	0.616	0.908	717.30	0.52
SB7	160x80x5	2240	526.4	1777.7	1333.3	56.8	33.0	33.0	0.430	0.649	0.881	463.86	0.24
SB9	160x80x5	2240	526.4	1777.7	1333.3	56.8	33.0	33.0	0.430	0.649	0.881	463.86	0.21
SB11	160x80x5	2240	526.4	1777.7	1333.3	56.8	33.0	33.0	0.430	0.649	0.881	463.86	0.19
SB12	200x100x6	3360	789.6	1777.7	1333.3	71.2	41.4	41.4	0.343	0.594	0.927	732.04	0.01
SB13	150x150x6	3360	789.6	1500.0	1125.0	59.3	59.3	59.3	0.202	0.521	0.999	788.78	0.17
SB16	200x200x10	7260	1706.1	1996.1	1497.1	76.5	76.5	76.5	0.208	0.524	0.996	1698.78	0.39
SB17	150x150x6	3360	789.6	1500.0	1125.0	59.3	59.3	59.3	0.202	0.521	0.999	788.78	0.02
L_{cr}=0.75L_{sys} α=0.49													

Table A.14: Compression forces [kN] - connecting beam (braces)

Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
C1B2	61.87	341.2	336.34	346.45	346.45
C1B5	68.51	379.46	373.97	385.38	385.38
C2B1	96.71	384.19	381.48	387.51	387.51
C2B3	8.73	47.01	48.08	46.01	48.08
C2B4	23.52	102.17	102.93	101.55	102.93
C2B6	92.00	369.77	367.36	372.76	372.76

Table A.15: Design checks (compression + stability) - connecting beam (braces)

Mem.	Section	A [mm^2]	$N_{pl,Rd}$ [kN]	L_{sys} [mm]	L_{cr} [mm]	i_y [mm]	i_z [mm]	min i [mm]	$\bar{\lambda}$	Φ	χ	$N_{b,Rd}$ [kN]	$N_{Ed}/N_{b,Rd}$
C1B2	200x100x8	4320	1015.2	2149.8	1612.4	71.2	41.4	41.4	0.415	0.639	0.889	903.01	0.38
C1B5	200x100x8	4320	1015.2	2108.8	1581.6	59.3	59.3	59.3	0.284	0.561	0.957	971.83	0.40
C2B1	200x100x8	4320	1015.2	2149.8	1612.4	71.2	41.4	41.4	0.415	0.639	0.889	903.01	0.43
C2B3	200x100x6	2240	526.4	2267.2	1700.4	56.8	33.0	33.0	0.549	0.736	0.815	429.19	0.11
C2B4	200x100x6	3360	789.6	2267.2	1700.4	56.8	33.0	33.0	0.549	0.736	0.815	643.79	0.16
C2B6	200x100x8	4320	1015.2	1824.0	1368.0	56.8	33.0	33.0	0.441	0.657	0.875	888.46	0.42
L_{cr}=0.75L_{sys}; α=0.49													

Bottom chords

Table A.16: Cross-section classification (bottom chords)

Cross-section	c [mm]	t [mm]	c/t	Class
SHS 350x350x12.5	312.5	12.5	25	1
c≈b-3t Bottom chord is classified for pure bending (safe-side assumption) Limiting values for the compression flange(S235): Class 1: c/t=33 Class 2: c/t=38 Class 3: c/t=42				

Table A.17: Chords - design resistances

SHS 350x350x12.5	
N_{Rd} [kN]	3807
M_{Rd} [kNm]	474.7
V_{Rd} [kN]	1187.18

Table A.18: Internal forces and design checks (bending and axial forces) - right/left side member (bottom ch.)

		max. N (tension), corr. M					max. M, corr. N (tension)				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
Right side member	$X_{1,ULS}$	79.08	3.38	0.02	0.01	0.03	31.38	10.27	0.01	0.02	0.03
	$X_{2,ULS}$	555.13	37.19	0.15	0.08	0.22	187.40	68.18	0.05	0.14	0.19
	$X_{3,ULS}$	564.69	37.47	0.15	0.08	0.23	178.04	69.93	0.05	0.15	0.19
	$X_{4,ULS}$	546.39	37.04	0.14	0.08	0.22	185.10	66.50	0.05	0.14	0.19
Left side member	$X_{1,ULS}$	82.04	3.46	0.02	0.01	0.03	31.40	10.94	0.01	0.02	0.03
	$X_{2,ULS}$	560.67	37.12	0.15	0.08	0.23	178.25	72.12	0.05	0.15	0.20
	$X_{3,ULS}$	568.54	37.35	0.15	0.08	0.23	180.51	74.06	0.05	0.16	0.20
	$X_{4,ULS}$	553.66	36.82	0.15	0.08	0.22	176.22	70.25	0.05	0.15	0.19

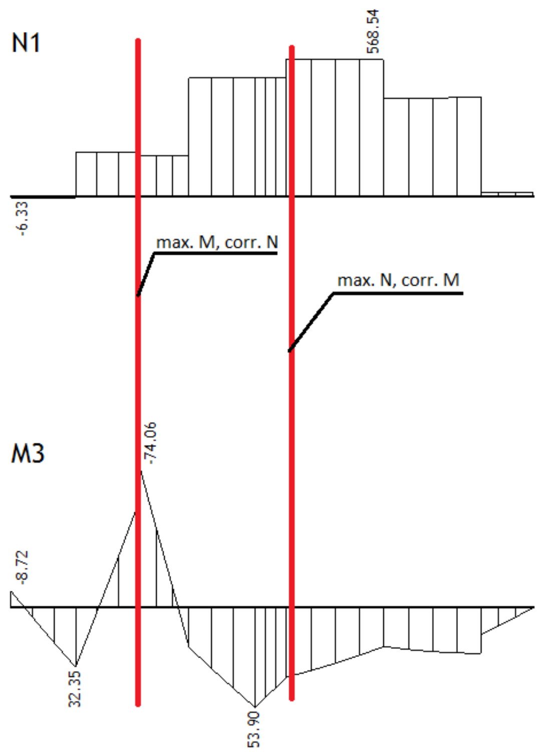


Diagram is given for $X_{3,ULS}$, left side member

Table A.19: Internal forces and design checks (shear) - right/left side member (bottom chord)

Member	Shear forces [kN]					
	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
Right side member	20.24	95.65	97.09	94.24	97.09	0.08
Left side member	20.21	99.63	101.20	98.15	101.20	0.09

Table A.20: Internal forces and design checks (bending and axial forces) - right/left suppor. beam (bottom ch.)

		max. N (tension), corr. M					max. M, corr. N (tension)				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
Right supporting beam	$X_{1,ULS}$	135.52	2.48	0.04	0.01	0.04	43.33	29.27	0.01	0.06	0.07
	$X_{2,ULS}$	517.46	19.05	0.14	0.04	0.18	180.34	114.00	0.05	0.24	0.29
	$X_{3,ULS}$	538.47	19.57	0.14	0.04	0.18	185.76	115.29	0.05	0.24	0.29
	$X_{4,ULS}$	497.64	18.58	0.13	0.04	0.17	175.17	112.90	0.05	0.24	0.28
Left supporting beam	$X_{1,ULS}$	141.12	2.33	0.04	0.00	0.04	45.86	28.55	0.01	0.06	0.07
	$X_{2,ULS}$	564.12	20.50	0.15	0.04	0.19	197.89	114.77	0.05	0.24	0.29
	$X_{3,ULS}$	588.58	21.01	0.15	0.04	0.20	204.26	116.11	0.05	0.24	0.30
	$X_{4,ULS}$	541.50	20.02	0.14	0.04	0.18	191.83	113.61	0.05	0.24	0.29

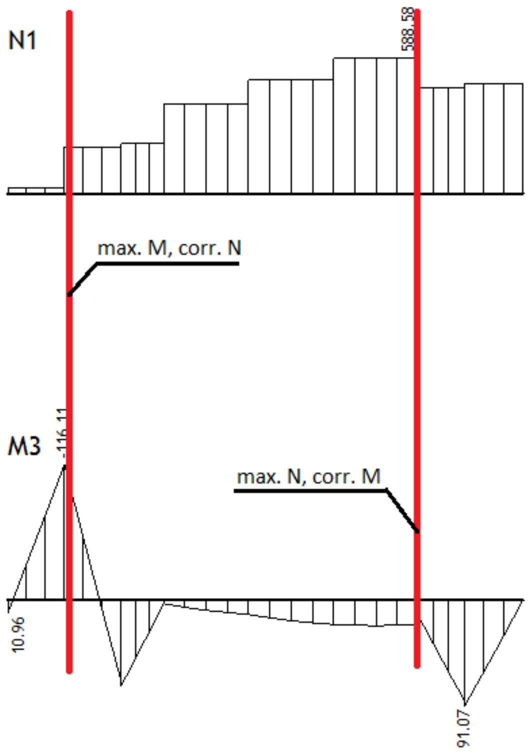


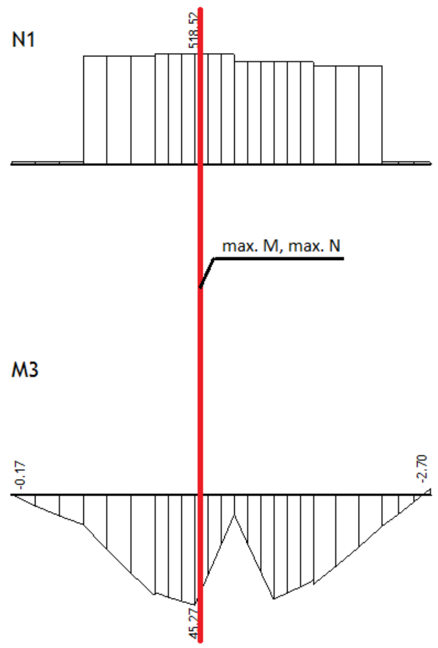
Diagram is given for $X_{3,ULS}$, left supporting beam

Table A.21: Internal forces and design checks (shear) - right/left supporting beam (bottom chord)

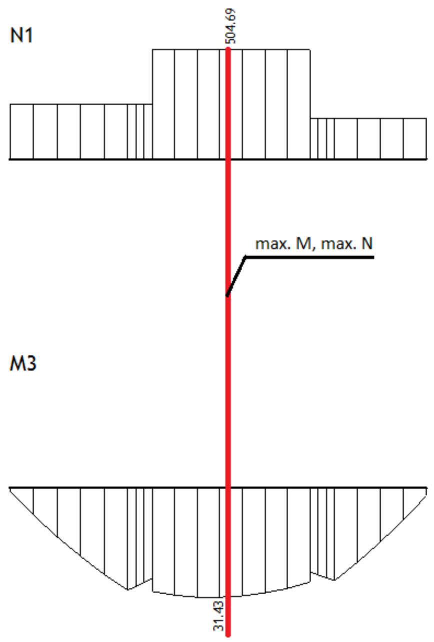
Member	Shear forces [kN]					
	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
Right supporting beam	36.32	145.87	145.78	146.19	146.19	0.12
Left supporting beam	35.63	147.82	147.37	148.49	148.49	0.13

Table A.22: Internal forces and design checks (bending and axial forces) - connecting beam (bottom chord)

		max. N (tension), max. M				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
C1	$X_{1,ULS}$	102.85	10.58	0.03	0.02	0.05
	$X_{2,ULS}$	494.57	45.67	0.13	0.10	0.23
	$X_{3,ULS}$	518.52	45.27	0.14	0.10	0.23
	$X_{4,ULS}$	480.72	46.14	0.13	0.10	0.22
C2	$X_{1,ULS}$	122.52	7.13	0.03	0.02	0.05
	$X_{2,ULS}$	548.63	31.73	0.14	0.07	0.21
	$X_{3,ULS}$	504.69	31.43	0.13	0.07	0.20
	$X_{4,ULS}$	593.36	32.07	0.16	0.07	0.22



C1, $X_{3,ULS}$



C2, $X_{3,ULS}$

Table A.23: Internal forces and design checks (shear) - connecting beam (bottom chord)

Member	Shear forces [kN]					
	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
C1	11.33	44.28	44.99	43.64	44.99	0.04
C2	13.28	14.19	14.23	14.19	14.19	0.01

Upper chords

Table A.24: Cross-section classification (upper chords)

Cross-section	c [mm]	t [mm]	c/t	Class
SHS 350x350x12.5	312.5	12.5	25	1
c≈b-3t Bottom chord is classified for pure compression (safe-side assumption) Limiting values in compression(S235): Class 1: c/t=33 Class 2: c/t=38 Class 3: c/t=42				

Table A.25: Chords - design resistances

SHS 350x350x12.5	
N_{Rd} [kN]	3807
M_{Rd} [kNm]	474.7
V_{Rd} [kN]	1187.18

Table A.26: Internal forces and design checks (bending and axial forces) - right/left side member (upper ch.)

	Comb.	max. N (compression), max. M				
		N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
Right side member	$X_{1,ULS}$	85.93	12.22	0.02	0.03	0.05
	$X_{2,ULS}$	532.55	129.74	0.14	0.27	0.41
	$X_{3,ULS}$	534.42	131.50	0.14	0.28	0.42
	$X_{4,ULS}$	545.76	128.03	0.14	0.27	0.41
Left side member	$X_{1,ULS}$	88.29	12.23	0.02	0.03	0.05
	$X_{2,ULS}$	536.19	129.76	0.14	0.27	0.41
	$X_{3,ULS}$	539.53	131.51	0.14	0.28	0.42
	$X_{4,ULS}$	547.94	128.06	0.14	0.27	0.41

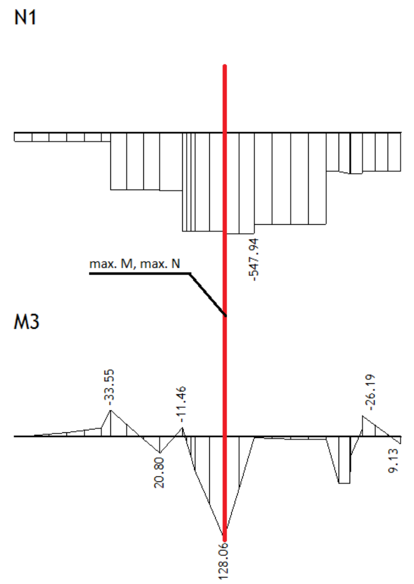


Diagram is given for $X_{4,ULS}$, left side member

Table A.27: Internal forces and design checks (shear) - right/left side member (upper chord)

Member	Shear forces [kN]					
	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
Right side member	25.24	163.78	165.13	166.06	166.06	0.14
Left side member	25.31	166.92	165.40	169.12	169.12	0.14

Table A.28: Stability checks - right/left side member (upper chord)

Buckling plane	Section	L_{sys} [mm]	L_{cr} [mm]	i [mm]	$\bar{\lambda}$	Φ	χ	$N_{b,Rd}$ [kN]	$N_{Ed,max}$ [kN]	$N_{Ed}/N_{b,Rd}$
In-plane	350x350x12.5	1908.0	1717.2	136	0.134		1	3807.0	547.94	0.14
Out-of-plane	350x350x12.5	6678.0	6010.2	136	0.471	0.677	0.859	3270.2	547.94	0.17

$L_{cr}=0.9L_{sys}$
$\alpha=0.49$
For $\bar{\lambda} \leq 0.2$ only cross-sectional checks apply.
For out-of-plane buckling is assumed that the axial force is constant between the lateral supports with its maximum value.
Interaction (M+N) is not relevant since there is no in-plane buckling (see 4.1.3.2. for the explanations)

Table A.29: Internal forces and design checks (bending and axial forces) - right/left suppor. beam (upper ch.)

	Comb.	max. N (compression), max. M				
		N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
Right supporting beam	$X_{1,ULS}$	134.91	18.26	0.04	0.04	0.08
	$X_{2,ULS}$	512.95	79.21	0.13	0.17	0.30
	$X_{3,ULS}$	521.05	76.06	0.14	0.16	0.30
	$X_{4,ULS}$	515.36	79.47	0.14	0.17	0.31
Left supporting beam	$X_{1,ULS}$	141.86	19.10	0.04	0.04	0.08
	$X_{2,ULS}$	526.53	85.29	0.14	0.18	0.32
	$X_{3,ULS}$	567.35	85.03	0.15	0.18	0.33
	$X_{4,ULS}$	565.87	85.66	0.15	0.18	0.33

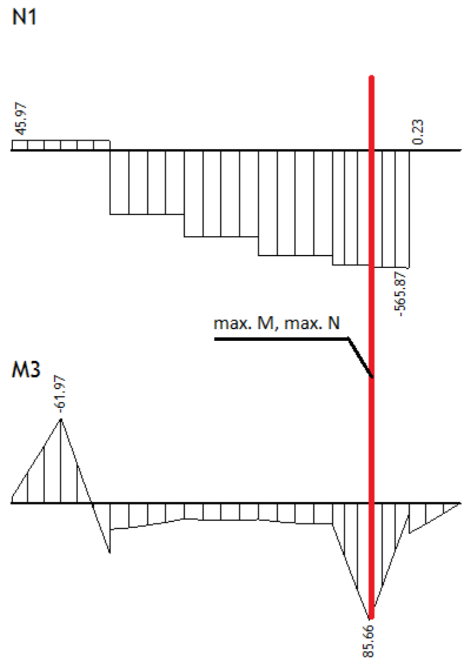


Diagram is given for $X_{4,ULS}$, left supporting beam

Table A.30: Internal forces and design checks (shear) - right/left supporting beam (upper chord)

Member	Shear forces [kN]					
	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
Right supporting beam	19.52	77.78	77.98	77.69	77.98	0.07
Left supporting beam	19.31	79.33	79.47	79.33	79.47	0.07

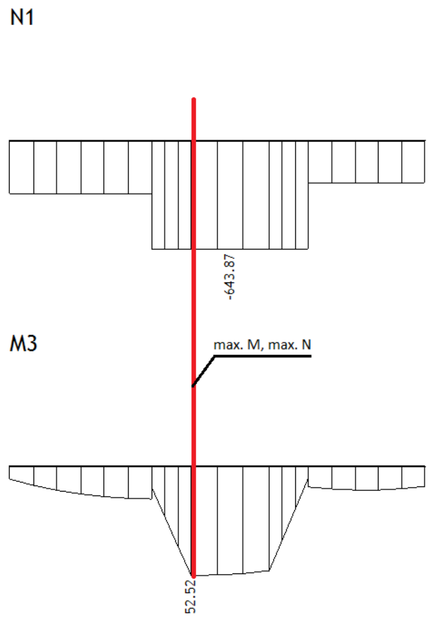
Table A.31: Stability checks - right/left supporting beam (upper chord)

Buckling plane	Section	L_{sys} [mm]	L_{cr} [mm]	i [mm]	$\bar{\lambda}$	Φ	χ	$N_{b,Rd}$ [kN]	$N_{Ed,max}$ [kN]	$N_{Ed}/N_{b,Rd}$
In-plane	350x350x12.5	1908.0	1717.2	136	0.134		1	3807.0	567.35	0.15
Out-of-plane	350x350x12.5	6678.0	6010.2	136	0.471	0.677	0.859	3270.2	567.35	0.17

$L_{cr}=0.9L_{sys}$
$\alpha=0.49$
For $\bar{\lambda} \leq 0.2$ only cross-sectional checks apply.
For out-of-plane buckling is assumed that the axial force is constant between the lateral supports with its maximum value.
Interaction (M+N) is not relevant since there is no in-plane buckling (see 4.1.3.2. for the explanations)

Table A.32: Internal forces and design checks (bending and axial forces) - connecting beam (upper chord)

		max. N (compression), max. M				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
C1	$X_{1,ULS}$	125.68	12.57	0.03	0.03	0.06
	$X_{2,ULS}$	600.07	52.09	0.16	0.11	0.27
	$X_{3,ULS}$	557.06	51.73	0.15	0.11	0.26
	$X_{4,ULS}$	643.87	52.52	0.17	0.11	0.28
C2	$X_{1,ULS}$	114.97	14.09	0.03	0.03	0.06
	$X_{2,ULS}$	478.52	72.55	0.13	0.15	0.28
	$X_{3,ULS}$	495.22	72.66	0.13	0.15	0.28
	$X_{4,ULS}$	462.56	72.52	0.12	0.15	0.27



C1, $X_{4,ULS}$

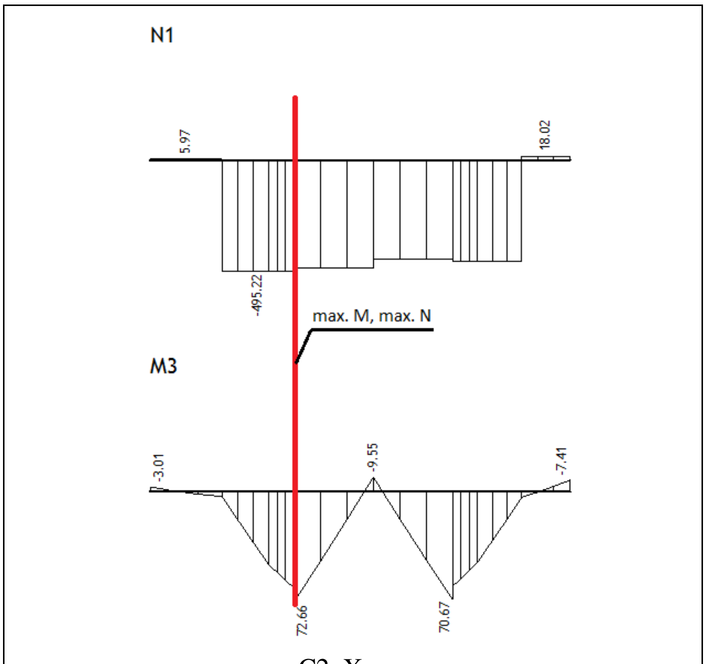


Table A.33: Internal forces and design checks (shear) - connecting beam (upper chord)

	Shear forces [kN]					
Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
C1	18.14	53.30	53.35	53.37	53.37	0.04
C2	13.51	49.46	50.07	48.92	50.07	0.04

Table A.34: Stability checks - connecting beam (upper chord)

Buckling plane	Section	L_{sys} [mm]	L_{cr} [mm]	i [mm]	$\bar{\lambda}$	Φ	χ	$N_{b,Rd}$ [kN]	$N_{Ed,max}$ [kN]	$N_{Ed}/N_{b,Rd}$
In-plane	350x350x12.5	3400.0	3060.0	136	0.24	0.539	0.979	3726.4	643.87	0.17
Out-of-plane	350x350x12.5	3400.0	3060.0	136	0.24	0.539	0.979	3726.4	643.87	0.17

L_{cr}=0.9L_{sys}
α=0.49
Interaction check (member C1, combination $X_{4,ULS}$ as the most unfavorable case)
C_{my}=1 (bending moment diagram almost rectangular)

$$k_{yy}=1\left(1+(0.24-0.2)\frac{643.87}{3807}\right)=1.007$$

k_{zy}=0
0.17+1.007·0.11=0.281<1 (EN1993-1-1, equation 6.61)
0.17<1 (EN1993-1-1, equation 6.62, with k_{zy}=0)

Bracket - diagonal

The diagonal on the right side is relevant for the design because it is longer and has higher loading.

Table A.35: Cross-section classification (bracket-diagonal)

Cross-sectional part	c [mm]	t [mm]	c/t	Class
HEB 360, flange	116.75	22.5	5.19	1
HEB 360, web	261	12.5	20.88	1

Cross-section is classified for pure compression
Limiting values (S235):
Class 1: flange c/t=9, web c/t=33
Class 2: flange c/t=10, web c/t=38
Class 3: flange c/t=14, web c/t=42

Table A.36: Compression forces [kN] - bracket (diagonal)

Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
Bracket-diagonal	118.25	709.75	710.89	709.37	710.89

Bending moments (resulting from the self-weight) are negligible and will not be considered further.

Table A.37: Bracket (diagonal) - design resistances

HEB 360	
N_{Rd} [kN]	4244.1
M_{Rd} [kNm]	631.21

Table A.38: Design checks (compression + stability) - bracket (diagonal)

Buckling plane	Section	L_{sys} [mm]	L_{cr} [mm]	i [mm]	$\bar{\lambda}$	Φ	χ	$N_{b,Rd}$ [kN]	$N_{Ed,max}$ [kN]	$N_{Ed}/N_{b,Rd}$
In-plane (y-y)	HEB 360	3518	3518	154.6	0.242	0.536	0.985	4182.2	710.89	0.17
Out-of-plane (z-z)	HEB 360	3518	3518	74.9	0.5	0.699	0.843	3576.2	710.89	0.20
$L_{cr}=L_{sys}$ α=0.34 (in-plane buckling) α=0.49 (out-of-plane buckling)										

Bracket - vertical

Table A.39: Cross-section classification (bracket-vertical)

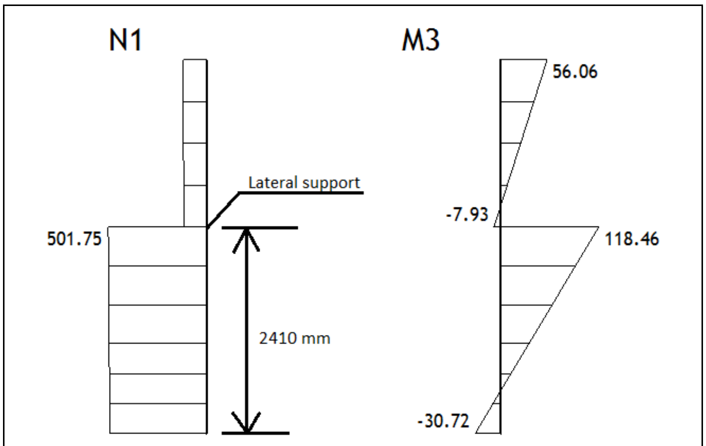
Cross-sectional part	c [mm]	t [mm]	c/t	Class
HEB 360, flange	116.75	22.5	5.19	1
HEB 360, web	261	12.5	20.88	1
Cross-section is classified for pure bending (safe-side assumption) Limiting values (S235): Class 1: flange c/t=9, web c/t=72 Class 2: flange c/t=10, web c/t=83 Class 3: flange c/t=14, web c/t=124				

Table A.40: Bracket (vertical) - design resistances

HEB 360	
N_{Rd} [kN]	4244.1
M_{Rd} [kNm]	631.21

Table A.41: Internal forces and design checks (bending and axial forces) - bracket (vertical)

		max. N (tension), max. M				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
Right side vertical	$X_{1,ULS}$	91.03	21.26	0.02	0.03	0.05
	$X_{2,ULS}$	499.81	118.02	0.12	0.19	0.31
	$X_{3,ULS}$	501.75	118.46	0.12	0.19	0.31
	$X_{4,ULS}$	498.45	117.73	0.12	0.19	0.31
Left side vertical	$X_{1,ULS}$	92.70	18.38	0.02	0.03	0.05
	$X_{2,ULS}$	517.25	104.18	0.12	0.17	0.29
	$X_{3,ULS}$	519.07	104.68	0.12	0.17	0.29
	$X_{4,ULS}$	516.02	103.80	0.12	0.16	0.28



Right side vertical, $X_{3,ULS}$

Table A.42: Stability checks - bracket (vertical)

Combination $X_{3,ULS}$ is the most unfavorable case in bending

M_{cr}=13819.31 kNm

$$\bar{\lambda}_{LT}=\sqrt{\frac{631.21}{13819.31}}=0.214$$

For $\bar{\lambda}_{LT} \leq 0.4$ lateral-torsional buckling check is not necessary

Bracket - horizontal

Table A.43: Cross-section classification (bracket-horizontal)

Cross-sectional part	c [mm]	t [mm]	c/t	Class
HEA 800, flange	112.5	28	4.02	1
HEA 800, web	337	15	22.47	1

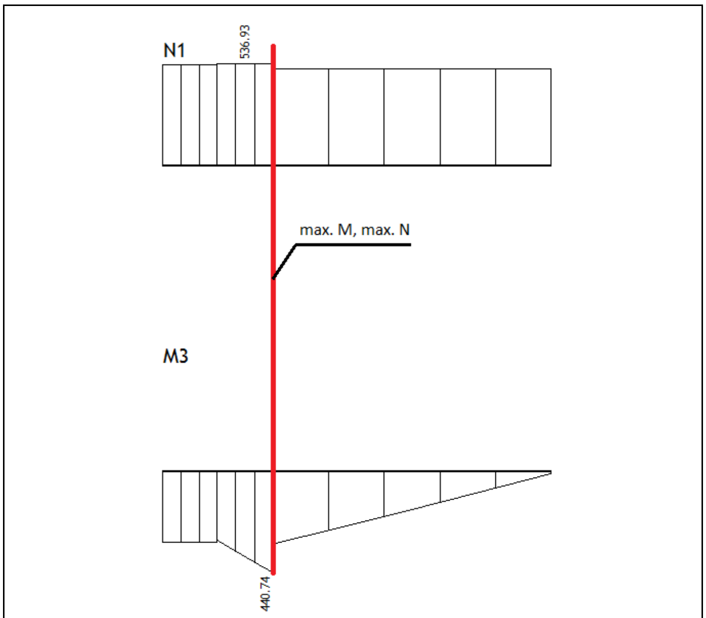
Cross-section is classified for bending (safe-side assumption)
Limiting values (S235):
Class 1: flange c/t=9, web c/t=72
Class 2: flange c/t=10, web c/t=83
Class 3: flange c/t=14, web c/t=124

Table A.44: Bracket (horizontal) - design resistances

HEA 800	
N_{Rd} [kN]	6716.3
M_{Rd} [kNm]	2044.27

Table A.45: Internal forces and design checks (bending and axial forces) - bracket (horizontal)

		max. N (tension), max. M				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
Right side horizontal	$X_{1,ULS}$	88.35	79.38	0.01	0.04	0.05
	$X_{2,ULS}$	533.36	436.37	0.08	0.21	0.29
	$X_{3,ULS}$	536.93	440.74	0.08	0.22	0.30
	$X_{4,ULS}$	530.31	432.51	0.08	0.21	0.29
Left side horizontal	$X_{1,ULS}$	71.53	69.63	0.01	0.03	0.04
	$X_{2,ULS}$	435.94	386.26	0.06	0.19	0.25
	$X_{3,ULS}$	439.78	391.16	0.07	0.19	0.26
	$X_{4,ULS}$	432.56	381.80	0.06	0.19	0.25



Right side horizontal, $X_{3,ULS}$

Table A.46: Stability checks - bracket (horizontal)

Combination $X_{3,ULS}$ is the most unfavorable case in bending
M_{cr}=27638.3 kNm
$\bar{\lambda}_{LT}=\sqrt{\frac{2044.27}{27638.3}}=0.272$
For $\bar{\lambda}_{LT} \leq 0.4$ lateral-torsional buckling check is not necessary

Design of joints

Table A.47: Designation - joints

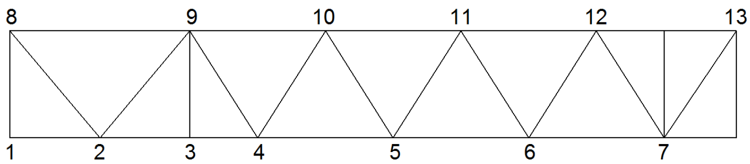
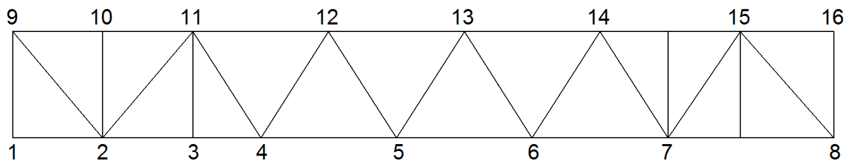
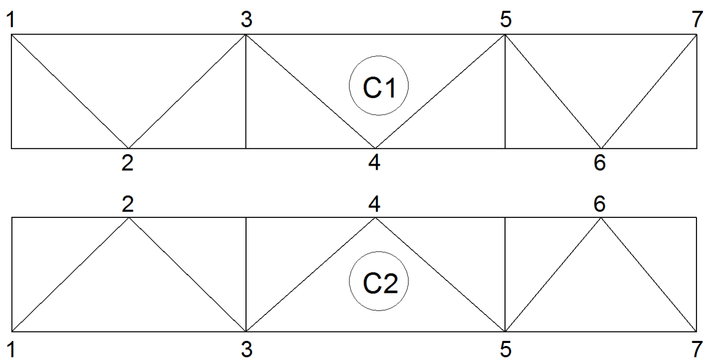
Right/left side member	
Right/left supporting beam	
Connecting beam	

Table A.48: Design checks - Joint 1 (right/left side member)

Joint 1 (right/left side member)			
Geometry:	T joint		
Brace 1		Chord	Validity limits check
SM1		SHS 350x350x12.5	b_1/b_0=200/350=0.571>0.25
RHS 200x100x6		$N_{0,Ed}$=0	b_1/t_1=200/6=33.33<35
θ=90°		$M_{0,Ed}$=0	h_1/t_1=100/6=16.66<35
$N_{1,Ed}$=-154.04 kN			Brace 1 cross-section class 1
T joint resistance			h_0/b_0=350/350=1>0.5
$N_{1,Rd}$=273.3 kN (chord face failure because β<0.85)			h_0/b_0=350/350=1<2
Joint resistance check			h_1/b_1=100/200=0.5=0.5
$N_{1,Ed}/N_{1,Rd}$=154.04/273.3=0.564<1			b_0/t_0=350/12.5=28<35

Table A.49: Design checks - Joint 2 (right/left side member)

Joint 2 (right/left side member)			
Geometry:	K gap	g=100.7 mm	e=-40 mm
Brace 1	Brace 2	Chord	Validity limits check
SM2	SM3	SHS 350x350x12.5	$b_{1(2)}/b_0$=200/350=0.571>0.35
RHS 200x100x6	RHS 200x100x8	$N_{0,Ed}$=187.3 kN	$b_{1(2)}/b_0$=200/350=0.571>0.1+0.01·350/12.5
θ=50.68°	θ=47.84°	$M_{0,Ed}$=32.54 kNm	b_1/t_1=200/6=33.33<35
$N_{1,Ed}$=83.04 kN	$N_{2,Ed}$=-213.1 kN		h_1/t_1=100/6=16.66<35
K joint resistance (brace 1: 83.04 kN, brace 2: -83.04 kN)			b_2/t_2=200/8=25<35
$N_{1,Rd}$=706.9 kN (chord face failure is the governing failure mode)			h_2/t_2=100/8=12.5<35
$N_{2,Rd}$=677.4 kN (chord face failure is the governing failure mode)			Brace 2 cross-section class 1
$N_{0,Rd}$=3924 kN (chord shear failure)			h_0/b_0=350/350=1>0.5
Y joint resistance (brace 2: -133.06 kN)			h_0/b_0=350/350=1<2
$N_{2,Rd}$=391.8 kN (chord face failure is the governing failure mode)			h_1/b_1=100/200=0.5=0.5
Joint resistance check			h_2/b_2=100/200=0.5=0.5
$N_{1,Ed}/N_{1,Rd}$=83.04/706.9=0.118<1			b_0/t_0=350/12.5=28<35
$N_{2,Ed}/N_{2,Rd}$=83.04/677.4+113.06/391.8=0.462<1			g/b_0=100.7/350=0.288>0.5(1-0.571)
$N_{0,Ed}/N_{0,Rd}$=187.3/3924=0.05<1			g/b_0=100.7/350=0.288<1.5(1-0.571)
			e=-40 >-0.55·350=-192.5

Table A.50: Design checks - Joint 3 (right/left side member)

Joint 3 (right/left side member)			
Geometry:	T joint		
Brace 1		Chord	Validity limits check
SM4		SHS 350x350x12.5	b_1/b_0=150/350=0.429>0.25
SHS 150x150x6		$N_{0,Ed}$=180.52 kN	b_1/t_1=150/6=25<35
θ=90°		$M_{0,Ed}$=-74.6 kNm	h_1/t_1=150/6=25<35
$N_{1,Ed}$=-199.83 kN			Brace 1 cross-section class 1
T joint resistance			h_0/b_0=350/350=1>0.5
$N_{1,Rd}$=249.4 kN (chord face failure because β<0.85)			h_0/b_0=350/350=1<2
Joint resistance check			h_1/b_1=150/150=1>0.5
$N_{1,Ed}/N_{1,Rd}$=199.83/249.4=0.801<1			h_1/b_1=150/150=1<2
			b_0/t_0=350/12.5=28<35

Table A.51: Design checks - Joint 4 (right/left side member)

Joint 4 (right/left side member)			
Geometry:	K gap	g=108 mm	e=0 mm
Brace 1	Brace 2	Chord	Validity limits check $b_{1(2)}/b_0$=200/350=0.571>0.35 $b_{1(2)}/b_0$=200/350=0.571>0.1+0.01·350/12.5 b_1/t_1=200/8=25<35 h_1/t_1=100/8=12.5<35 b_2/t_2=200/6=33.33<35 h_2/t_2=100/6=16.66<35 Brace 2 cross-section class 1 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 h_2/b_2=100/200=0.5=0.5 b_0/t_0=350/12.5=28<35 g/b_0=108/350=0.309>0.5(1-0.571) g/b_0=108/350=0.309<1.5(1-0.571)
SM5	SM6	SHS 350x350x12.5	
RHS 200x100x8	RHS 200x100x6	$N_{0,Ed}$=488.81 kN	
θ=56.48°	θ=57.54°	$M_{0,Ed}$=21.78 kNm	
$N_{1,Ed}$=256.15 kN	$N_{2,Ed}$=-341.55 kN		
K joint resistance (forces ratio 75%) $N_{1,Rd}$=628.6 kN (chord face failure is the governing failure mode) $N_{2,Rd}$=621.1 kN (chord face failure is the governing failure mode) $N_{0,Rd}$=3895 kN (chord shear failure) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=256.15/628.6=0.41<1 $N_{2,Ed}/N_{2,Rd}$=341.55/621.1=0.55<1 $N_{0,Ed}/N_{0,Rd}$=488.81/3895=0.13<1			

Table A.52: Design checks - Joint 5 (right/left side member)

Joint 5 (right/left side member)			
Geometry:	K gap	g=104.1 mm	e=0 mm
Brace 1	Brace 2	Chord	Validity limits check $b_{1(2)}/b_0$=200/350=0.571>0.35 $b_{1(2)}/b_0$=200/350=0.571>0.1+0.01·350/12.5 b_1/t_1=200/6=33.33<35 h_1/t_1=100/6=16.66<35 b_2/t_2=200/6=33.33<35 h_2/t_2=100/6=16.66<35 Brace 2 cross-section class 1 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 h_2/b_2=100/200=0.5=0.5 b_0/t_0=350/12.5=28<35 g/b_0=104.1/350=0.297>0.5(1-0.571) g/b_0=104.1/350=0.297<1.5(1-0.571)
SM7	SM8	SHS 350x350x12.5	
RHS 200x100x6	RHS 200x100x6	$N_{0,Ed}$=568.21 kN	
θ=57.54°	θ=57.54°	$M_{0,Ed}$=37.45 kNm	
$N_{1,Ed}$=83.81 kN	$N_{2,Ed}$=-59.15 kN		
K joint resistance (brace 1: 59.15 kN, brace 2: -59.15 kN) $N_{1,Rd}$=621.1 kN (chord face failure is the governing failure mode) $N_{2,Rd}$=621.1 kN (chord face failure is the governing failure mode) $N_{0,Rd}$=3924 kN (chord shear failure) Y joint resistance (brace 1: 24.66 kN) $N_{1,Rd}$=334.7 kN (chord face failure is the governing failure mode) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=59.15/621.1+24.66/334.7=0.169<1 $N_{2,Ed}/N_{2,Rd}$=59.15/621.18=0.095<1 $N_{0,Ed}/N_{0,Rd}$=568.21/3924=0.145<1			

Table A.53: Design checks - Joint 6 (right/left side member)

Joint 6 (right/left side member)			
Geometry:	K gap	g=101.6 mm	e=0 mm
Brace 1	Brace 2	Chord	Validity limits check $b_{1(2)}/b_0$=200/350=0.571>0.35 $b_{1(2)}/b_0$=200/350=0.571>0.1+0.01·350/12.5 b_1/t_1=200/6=33.33<35 h_1/t_1=100/6=16.66<35 Brace 1 cross-section class 1 b_2/t_2=200/6=33.33<35 h_2/t_2=100/6=16.66<35 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 h_2/b_2=100/200=0.5=0.5 b_0/t_0=350/12.5=28<35 g/b_0=101.6/350=0.290>0.5(1-0.571) g/b_0=101.6/350=0.290<1.5(1-0.571)
SM9	SM10	SHS 350x350x12.5	
RHS 200x100x6	RHS 200x100x6	$N_{0,Ed}$=546.07 kN	
θ=57.54°	θ=58.22°	$M_{0,Ed}$=20.18 kNm	
$N_{1,Ed}$=-152.64 kN	$N_{2,Ed}$=170.67 kN		
K joint resistance (forces ratio 89%) $N_{1,Rd}$=621.1 kN (chord face failure is the governing failure mode) $N_{2,Rd}$=616.5 kN (chord face failure is the governing failure mode) $N_{0,Rd}$=3912 kN (chord shear failure) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=152.64/621.1=0.246<1 $N_{2,Ed}/N_{2,Rd}$=170.67/616.5=0.277<1 $N_{0,Ed}/N_{0,Rd}$=546.07/3912=0.140<1			

Table A.54: Design checks - Joint 7 (right/left side member)

Joint 7 (right/left side member)			
Geometry:	KT overlap	λ_{ov}=(44.95+39.01)%	e=30 mm
Brace 1 (overlapped - j)	Brace 2 (overlapping - i)	Brace 3 (overlapped - j)	Validity limits check $b_2/b_0=150/350=0.429>0.25$ $b_3/t_3=200/10=20<35$ $h_3/t_3=200/10=20<35$ $b_3/t_3=150/6=25<35$ $h_3/t_3=150/6=25<35$ Brace 1 cross-section class 1 $h_0/b_0=350/350=1>0.5$ $h_0/b_0=350/350=1<2$ $h_1/b_1=200/200=1>0.5$ $h_1/b_1=200/200=1<2$ $h_2/b_2=150/150=1>0.5$ $h_2/b_2=150/150=1<2$ $h_3/b_3=200/200=1>0.5$ $h_3/b_3=200/200=1<2$ Chord cross-section class 1 $\lambda_{ov}>25$ %; $b_2/b_1=150/200=0.75$ $b_2/b_3=150/200=0.75$ $e=30<0.25\cdot 350=87.5$
SM11	SM12	SM13	
RHS 200x200x10	RHS 150x150x6	RHS 200x200x10	
θ=58.72°	θ=90°	θ=56.31°	
$N_{1,Ed}$=-369.75 kN	$N_{2,Ed}$=17.34 kN	$N_{3,Ed}$=331.76 kN	
Chord SHS 350x350x12.5 $N_{0,Ed}$=372.25 kN $M_{0,Ed}$=23.34 kNm			
KT joint resistance $b_{e,ov}$=125 mm $N_{2,Rd}$=741.66 kN (brace failure is the governing failure mode) $N_{1,Rd}$=1602.52 kN $N_{3,Rd}$=1602.52 kN Joint resistance check $N_{1,Ed}/N_{1,Rd}=369.75/1602.52=0.231<1$ $N_{2,Ed}/N_{2,Rd}=17.34/741.66=0.023<1$ $N_{3,Ed}/N_{3,Rd}=331.76/1602.52=0.207<1$ $\lambda_{ov}>80\%$=> local shear check is necessary $369.75\cos 58.72°+331.76\cos 56.31°<V_L=1972.0$ kN			

Table A.55: Design checks - Joint 8 (right/left side member)

Joint 8 (right/left side member)			
Geometry:	N gap	g=100.41 mm	e=87.5 mm
Brace 1	Brace 2	Chord	Validity limits check $b_{1(2)}/b_0=200/350=0.571>0.35$ $b_{1(2)}/b_0=200/350=0.571>0.1+0.01\cdot 350/12.5$ $b_1/t_1=200/6=33.33<35$ $h_1/t_1=100/6=16.66<35$ Brace 1 cross-section class 1 $b_2/t_2=200/6=33.33<35$ $h_2/t_2=100/6=16.66<35$ $h_0/b_0=350/350=1>0.5$ $h_0/b_0=350/350=1<2$ $h_1/b_1=100/200=0.5=0.5$ $h_2/b_2=100/200=0.5=0.5$ $b_0/t_0=350/12.5=28<35$ $g/b_0=100.41/350=0.287>0.5(1-0.571)$ $g/b_0=100.41/350=0.287<1.5(1-0.571)$ $e=87.5=0.25\cdot 350=87.5$
SM1	SM2	SHS 350x350x12.5	
RHS 200x100x6	RHS 200x100x6	$N_{0,Ed}$=-49.65 kN	
θ=90°	θ=50.68°	$M_{0,Ed}$=0 kNm	
$N_{1,Ed}$=-154.04 kN	$N_{2,Ed}$=83.04 kN		
N joint resistance (brace 1: -64.24 kN, brace 2: 83.04 kN) $N_{1,Rd}$=524.0 kN (chord face failure is the governing failure mode) $N_{2,Rd}$=677.4 kN (chord face failure is the governing failure mode) $N_{0,Rd}$=3921 kN (chord shear failure) T joint resistance (brace 1: -89.8 kN) $N_{1,Rd}$=273.3 kN (chord face failure is the governing failure mode) Joint resistance check $N_{1,Ed}/N_{1,Rd}=64.24/524.0+89.8/273.3=0.451<1$ $N_{2,Ed}/N_{2,Rd}=83.04/677.4=0.123<1$ $N_{0,Ed}/N_{0,Rd}=49.65/3921=0.013<1$			

Table A.56: Design checks - Joint 9 (right/left side member)

Joint 9 (right/left side member)			
Geometry:	KT overlap	λ_{ov}=(25.55+39.19)%	e=-60 mm
Brace 1 (overlapped - j)	Brace 2 (overlapping - i)	Brace 3 (overlapped - j)	Validity limits check $b_2/b_0=150/350=0.429>0.25$ $b_3/t_3=200/8=25<35$ $h_3/t_3=100/8=12.5<35$ Brace 1 and brace 2 cross-section class 1 $h_0/b_0=350/350=1>0.5$ $h_0/b_0=350/350=1<2$ $h_1/b_1=100/200=0.5=0.5$ $h_2/b_2=150/150=1>0.5$ $h_2/b_2=150/150=1<2$ $h_3/b_3=100/200=0.5=0.5$
SM3	SM4	SM5	
RHS 200x100x8	RHS 150x150x6	RHS 200x100x8	
θ=47.84°	θ=90°	θ=56.48°	
$N_{1,Ed}$=-213.06 kN	$N_{2,Ed}$=-192.85 kN	$N_{3,Ed}$=253.28 kN	
Chord SHS 350x350x12.5 $N_{0,Ed}$=-318.68 kN $M_{0,Ed}$=32.04 kNm			
KT joint resistance $b_{e,ov}$=80 mm			

$N_{2,Rd}$=614.76 kN (brace failure is the governing failure mode) $N_{1,Rd}$=790.41 kN $N_{3,Rd}$=790.41 kN Joint resistance check $N_{1,Ed}/N_{1,Rd}$=213.06/790.41=0.270<1 $N_{2,Ed}/N_{2,Rd}$=192.85/614.76=0.314<1 $N_{3,Ed}/N_{3,Rd}$=253.28/790.41=0.320<1 $h_j<b_j$=> local shear check is necessary 213.06cos47.84°+253.28cos56.48°<V_L=847.5 kN	Chord cross-section class 1 λ_{ov}>25 %; b_2/b_1=150/200=0.75 b_2/b_3=150/200=0.75 e=-60 >-0.55·350=-192.5

Table A.57: Design checks - Joint 10 (right/left side member)

<table>
<tr><td colspan="4">Joint 10 (right/left side member)</td></tr>
<tr><td>Geometry:</td><td>K gap</td><td>g=104.1 mm</td><td>e=0 mm</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="6">Validity limits check
$b_{1(2)}/b_0$=200/350=0.571>0.35
$b_{1(2)}/b_0$=200/350=0.571>0.1+0.01·350/12.5
b_1/t_1=200/6=33.33<35
h_1/t_1=100/6=16.66<35
Brace 1 cross-section class 1
b_2/t_2=200/6=33.33<35
h_2/t_2=100/6=16.66<35
h_0/b_0=350/350=1>0.5
h_0/b_0=350/350=1<2
h_1/b_1=100/200=0.5=0.5
h_2/b_2=100/200=0.5=0.5
b_0/t_0=350/12.5=28<35
g/b_0=104.1/350=0.297>0.5(1-0.571)
g/b_0=104.1/350=0.297<1.5(1-0.571)</td></tr>
<tr><td>SM6</td><td>SM7</td><td>SHS 350x350x12.5</td></tr>
<tr><td>RHS 200x100x6</td><td>RHS 200x100x6</td><td>$N_{0,Ed}$=-539.52 kN</td></tr>
<tr><td>θ=57.54°</td><td>θ=57.54°</td><td>$M_{0,Ed}$=12.20 kNm</td></tr>
<tr><td>$N_{1,Ed}$=-341.55 kN</td><td>$N_{2,Ed}$=83.81 kN</td><td></td></tr>
<tr><td colspan="3">K joint resistance (brace 1: -83.81 kN, brace 2: 83.81 kN)
$N_{1,Rd}$=621.1 kN (chord face failure is the governing failure mode)
$N_{2,Rd}$=621.1 kN (chord face failure is the governing failure mode)
$N_{0,Rd}$=3923 kN (chord shear failure)
Y joint resistance (brace 1: -257.74 kN)
$N_{1,Rd}$=334.7 kN (chord face failure is the governing failure mode)
Joint resistance check
$N_{1,Ed}/N_{1,Rd}$=83.81/621.1+257.74/334.7=0.905<1
$N_{2,Ed}/N_{2,Rd}$=82.81/621.1=0.133<1
$N_{0,Ed}/N_{0,Rd}$=539.52/3923=0.138<1</td></tr>
</table>

Table A.58: Design checks - Joint 11 (right/left side member)

<table>
<tr><td colspan="4">Joint 11 (right/left side member)</td></tr>
<tr><td>Geometry:</td><td>K gap</td><td>g=104.1 mm</td><td>e=0 mm</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="6">Validity limits check
$b_{1(2)}/b_0$=200/350=0.571>0.35
$b_{1(2)}/b_0$=200/350=0.571>0.1+0.01·350/12.5
b_1/t_1=200/6=33.33<35
h_1/t_1=100/6=16.66<35
b_2/t_2=200/6=33.33<35
h_2/t_2=100/6=16.66<35
Brace 1 and brace 2: cross-section class 1
h_0/b_0=350/350=1>0.5
h_0/b_0=350/350=1<2
h_1/b_1=100/200=0.5=0.5
h_2/b_2=100/200=0.5=0.5
b_0/t_0=350/12.5=28<35
g/b_0=104.1/350=0.297>0.5(1-0.571)
g/b_0=104.1/350=0.297<1.5(1-0.571)</td></tr>
<tr><td>SM8</td><td>SM9</td><td>SHS 350x350x12.5</td></tr>
<tr><td>RHS 200x100x6</td><td>RHS 200x100x6</td><td>$N_{0,Ed}$=-532.55 kN</td></tr>
<tr><td>θ=57.54°</td><td>θ=57.54°</td><td>$M_{0,Ed}$=0 kNm</td></tr>
<tr><td>$N_{1,Ed}$=-46.36 kN</td><td>$N_{2,Ed}$=-150.67 kN</td><td></td></tr>
<tr><td colspan="3">Unidirectional K joint resistance
b_{eq}=(200+200)/2=200 mm
h_{eq}=100/sin57.57°+104.1+100/sin57.57°=341.12 mm
β=200/350=0.571 (chord face failure is the governing mode)
$N_{eq,Rd}$=391.15 kN
$N_{1,Rd}$=109.07 kN
$N_{2,Rd}$=354.50 kN
Joint resistance check
$N_{1,Ed}/N_{1,Rd}$=46.36/109.07=0.425<1
$N_{2,Ed}/N_{2,Rd}$=150.67/354.50=0.425<1</td></tr>
</table>

Table A.59: Design checks - Joint 12 (right/left side member)

Joint 12 (right/left side member)			
Geometry:	K gap	g=88 mm	e=40 mm
Brace 1	Brace 2	Chord	Validity limits check $b_{1(2)}/b_0$=200/350=0.571>0.35 $b_{1(2)}/b_0$=200/350=0.571>0.1+0.01·350/12.5 b_1/t_1=200/6=33.33<35 h_1/t_1=100/6=16.66<35 b_2/t_2=200/10=20<35 h_2/t_2=200/10=20<35 Brace 1 cross-section class 1 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 h_2/b_2=200/200=1>0.5 h_2/b_2=200/200=1<2 b_0/t_0=350/12.5=28<35 g/b_0=88/350=0.251>0.5(1-0.571) g/b_0=88/350=0.251<1.5(1-0.571) e=40 <0.25·350=87.5
SM10	SM11	SHS 350x350x12.5	
RHS 200x100x6	SHS 200x200x10	$N_{0,Ed}$=-486.28 kN	
θ=58.22°	θ=58.72°	$M_{0,Ed}$=-4.85 kNm	
$N_{1,Ed}$=170.67 kN	$N_{2,Ed}$=-369.75 kN		
K joint resistance (brace 1: 170.67 kN, brace 2: -170.67 kN) $N_{1,Rd}$=719.2 kN (chord face failure is the governing failure mode) $N_{2,Rd}$=715.4 kN (chord face failure is the governing failure mode) $N_{0,Rd}$=3912 kN (chord shear failure) Y joint resistance (brace 1: -199.08 kN) $N_{2,Rd}$=396.6 kN (chord face failure is the governing failure mode) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=170.67/719.2=0.237<1 $N_{2,Ed}/N_{2,Rd}$=170.67/715.4+199.08/396.6=0.741<1 $N_{0,Ed}/N_{0,Rd}$=486.28/3912=0.124<1			

Table A.60: Design checks - Joint 13 (right/left side member)

Joint 13 (right/left side member)			
Geometry:	N overlap	λ_{ov}=52.34 %	e=0 mm
Brace 1 (overlapped - j)	Brace 2 (overlapping - i)	Chord	Validity limits check b_2/b_0=150/350=0.429>0.25 b_1/t_1=200/10=20<35 h_1/t_1=200/10=20<35 Brace 2 cross-section class 1 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=200/200=1>0.5 h_1/b_1=200/200=1<2 h_2/b_2=150/150=1>0.5 h_2/b_2=150/150=1<2 Chord cross-section class 1 λ_{ov}>25 % b_2/b_1=150/200=0.75
SM13	SM14	SHS 350x350x12.5	
SHS 200x200x10	RHS 150x150x6	$N_{0,Ed}$=-212.28 kN	
θ=56.31°	θ=90°	$M_{0,Ed}$=-9.13 kNm	
$N_{1,Ed}$=339.53 kN	$N_{2,Ed}$=-175.77 kN		
N joint resistance b_{eff}=111.61 mm $b_{e,ov}$=125 mm $N_{2,Rd}$=722.78 kN (brace failure is the governing failure mode) $N_{1,Rd}$=1561.72 kN Joint resistance check $N_{1,Ed}/N_{1,Rd}$=339.53/1561.72=0.217<1 $N_{2,Ed}/N_{2,Rd}$=175.77/722.78=0.243<1 $h_i=b_i$, $h_j=b_j$, λ_{ov}<60% => local shear check not necessary			

Table A.61: Design checks - Joint 1 (right/left supporting beam)

Joint 1 (right/left supporting beam)			
Geometry:	T joint		
Brace 1		Chord	Validity limits check b_1/b_0=200/350=0.571>0.25 b_1/t_1=200/6=33.33<35 h_1/t_1=100/6=16.66<35 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 b_0/t_0=350/12.5=28<35
SB1		SHS 350x350x12.5	
RHS 200x100x6		$N_{0,Ed}$=20.02 kN	
θ=90°		$M_{0,Ed}$=9.86 kNm	
$N_{1,Ed}$=104.29 kN			
T joint resistance $N_{1,Rd}$=273.3 kN (chord face failure because β<0.85) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=104.29/273.3=0.382<1			

Table A.62: Design checks - Joint 2 (right/left supporting beam)

Joint 2 (right/left supporting beam)			
Geometry:	KT overlap	λ_{ov}=(25.0+62.5)%	e=-65 mm
Brace 1 (overlapped - j)	Brace 2 (overlapping - i)	Brace 3 (overlapped - j)	Validity limits check $b_{1(2)}/b_0$=200/350=0.571>0.25 Brace , brace 2 and brace 3: cross-section class 1 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 h_2/b_2=150/150=1>0.5 h_2/b_2=150/150=1<2 h_3/b_3=100/200=0.5=0.5 b_0/t_0=350/12.5=28<35 λ_{ov}>25 % b_2/b_1=150/200=0.75 b_2/b_3=150/200=0.75
SB2	SB3	SB4	
RHS 200x100x6	RHS 150x150x6	RHS 200x100x6	
θ=50.22°	θ=90°	θ=47.23°	
$N_{1,Ed}$=-87.71 kN	$N_{2,Ed}$=-130.28 kN	$N_{3,Ed}$=-370.95 kN	
Chord SHS 350x350x12.5 $N_{0,Ed}$=204.27 kN $M_{0,Ed}$=-116.11 kNm			
Unidirectional KT joint resistance b_{eq}=(200+200+150)/3=183.33 mm ; h_{eq}=326.5 mm β=183.33/350=0.524 (chord face failure is the governing mode) Stiffening is necessary. Plate t_p=15 mm, b_p=350 mm is used to reinforce the chord face. β_p=0.524; η_p=0.933 $N_{eq,Rd}$=513.83 kN $N_{1,Rd}$=95.58 kN; $N_{2,Rd}$=141.97 kN; $N_{3,Rd}$=404.23 kN Joint resistance check $N_{1,Ed}/N_{1,Rd}$=87.71/95.58=0.918<1 $N_{2,Ed}/N_{2,Rd}$=130.28/141.97=0.918<1 $N_{3,Ed}/N_{3,Rd}$=370.95/404.23=0.918<1			

Table A.63: Design checks - Joint 3 (right/left supporting beam)

Joint 3 (right/left supporting beam)			
Geometry:	T joint		
Brace 1		Chord	Validity limits check b_1/b_0=150/350=0.429>0.25 b_1/t_1=150/6=25<35 h_1/t_1=150/6=25<35 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=150/150=1>0.5 h_1/b_1=150/150=1<2 b_0/t_0=350/12.5=28<35
SB5		SHS 350x350x12.5	
SHS 150x150x6		$N_{0,Ed}$=191.62 kN	
θ=90°		$M_{0,Ed}$=70.62 kNm	
$N_{1,Ed}$=72.72 kN			
T joint resistance $N_{1,Rd}$=249.4 kN (chord face failure because β<0.85) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=72.72/249.4=0.292<1			

Table A.64: Design checks - Joint 4 (right/left supporting beam)

Joint 4 (right/left supporting beam)			
Geometry:	K gap	g=120.2 mm	e=0 mm
Brace 1	Brace 2	Chord	Validity limits check b_1/b_0=200/350=0.571>0.35 b_1/b_0=200/350=0.571>0.1+0.01·350/12.5 b_2/b_0=160/350=0.457>0.35 b_2/b_0=160/350=0.457>0.1+0.01·350/12.5 b_1/t_1=200/6=33.33<35 h_1/t_1=100/6=16.66<35 b_2/t_2=160/5=32<35 h_2/t_2=80/5=16<35 Brace 2 cross-section class 1 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 h_2/b_2=80/160=0.5=0.5 b_0/t_0=350/12.5=28<35 g/b_0=120.2/350=0.343>0.5(1-0.457) g/b_0=120.2/350=0.343<1.5(1-0.457)
SB6	SB7	SHS 350x350x12.5	
RHS 200x100x6	RHS 160x80x5	$N_{0,Ed}$=388.19 kN	
θ=56.38°	θ=57.54°	$M_{0,Ed}$=3.27 kNm	
$N_{1,Ed}$=208.33 kN	$N_{2,Ed}$=-110.72 kN		
K joint resistance (brace 1: 110.72 kN, brace 2: -110.72 kN) $N_{1,Rd}$=566.4 kN (chord face failure is the governing failure mode) $N_{2,Rd}$=520.4 kN (chord face failure is the governing failure mode) $N_{0,Rd}$=3905 kN (chord shear failure) Y joint resistance (brace 1: 97.61 kN) $N_{1,Rd}$=340.0 kN (chord face failure is the governing failure mode) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=110.72/566.4+97.61/340.0=0.483<1 $N_{2,Ed}/N_{2,Rd}$=110.72/520.4=0.213<1 $N_{0,Ed}/N_{0,Rd}$=388.19/3905=0.099<1			

Table A.65: Design checks - Joint 5 (right/left supporting beam)

<table>
<tr><td colspan="4">Joint 5 (right/left supporting beam)</td></tr>
<tr><td>Geometry:</td><td>K gap</td><td>g=127.8 mm</td><td>e=0 mm</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="6">Validity limits check
$b_{1(2)}/b_0$=160/350=0.457>0.35
$b_{1(2)}/b_0$=160/350=0.457>0.1+0.01·350/12.5
b_1/t_1=160/5=32<35
h_1/t_1=80/5=16<35
b_2/t_2=160/5=32<35
h_2/t_2=80/5=16<35
Brace 2 cross-section class 1
h_0/b_0=350/350=1>0.5
h_0/b_0=350/350=1<2
h_1/b_1=80/160=0.5=0.5
h_2/b_2=80/160=0.5=0.5
b_0/t_0=350/12.5=28<35
g/b_0=127.8/350=0.365>0.5(1-0.457)
g/b_0=127.8/350=0.365<1.5(1-0.457)</td></tr>
<tr><td>SB8</td><td>SB9</td><td>SHS 350x350x12.5</td></tr>
<tr><td>RHS 160x80x5</td><td>RHS 160x80x5</td><td>$N_{0,Ed}$=493.35 kN</td></tr>
<tr><td>θ=57.54°</td><td>θ=57.54°</td><td>$M_{0,Ed}$=12.36 kNm</td></tr>
<tr><td>$N_{1,Ed}$=100.15 kN</td><td>$N_{2,Ed}$=-95.20 kN</td><td></td></tr>
<tr><td colspan="3">K joint resistance (forces ratio 95%)
$N_{1,Rd}$=496.9 kN (chord face failure is the governing failure mode)
$N_{2,Rd}$=496.9 kN (chord face failure is the governing failure mode)
$N_{0,Rd}$=3921 kN (chord shear failure)
Joint resistance check
$N_{1,Ed}/N_{1,Rd}$=100.15/496.9=0.202<1
$N_{2,Ed}/N_{2,Rd}$=95.2/496.9=0.192<1
$N_{0,Ed}/N_{0,Rd}$=493.35/3921=0.126<1</td></tr>
</table>

Table A.66: Design checks - Joint 6 (right/left supporting beam)

<table>
<tr><td colspan="4">Joint 6 (right/left supporting beam)</td></tr>
<tr><td>Geometry:</td><td>K gap</td><td>g=127.8 mm</td><td>e=0 mm</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="6">Validity limits check
$b_{1(2)}/b_0$=160/350=0.457>0.35
$b_{1(2)}/b_0$=160/350=0.457>0.1+0.01·350/12.5
b_1/t_1=160/5=32<35
h_1/t_1=80/5=16<35
b_2/t_2=160/5=32<35
h_2/t_2=80/5=16<35
Brace 2 cross-section class 1
h_0/b_0=350/350=1>0.5
h_0/b_0=350/350=1<2
h_1/b_1=80/160=0.5=0.5
h_2/b_2=80/160=0.5=0.5
b_0/t_0=350/12.5=28<35
g/b_0=127.8/350=0.365>0.5(1-0.457)
g/b_0=127.8/350=0.365<1.5(1-0.457)</td></tr>
<tr><td>SB10</td><td>SB11</td><td>SHS 350x350x12.5</td></tr>
<tr><td>RHS 160x80x5</td><td>RHS 160x80x5</td><td>$N_{0,Ed}$=588.36 kN</td></tr>
<tr><td>θ=57.54°</td><td>θ=57.54°</td><td>$M_{0,Ed}$=21.11 kNm</td></tr>
<tr><td>$N_{1,Ed}$=87.92 kN</td><td>$N_{2,Ed}$=-88.30 kN</td><td></td></tr>
<tr><td colspan="3">K joint resistance (forces ratio 99.5%)
$N_{1,Rd}$=496.9 kN (chord face failure is the governing failure mode)
$N_{2,Rd}$=496.9 kN (chord face failure is the governing failure mode)
$N_{0,Rd}$=3922 kN (chord shear failure)
Joint resistance check
$N_{1,Ed}/N_{1,Rd}$=87.92/496.9=0.177<1
$N_{2,Ed}/N_{2,Rd}$=88.3/496.9=0.178<1
$N_{0,Ed}/N_{0,Rd}$=588.36/3922=0.150<1</td></tr>
</table>

Table A.67: Design checks - Joint 7 (right/left supporting beam)

<table>
<tr><td colspan="4">Joint 7 (right/left supporting beam)</td></tr>
<tr><td>Geometry:</td><td>KT overlap</td><td>λ_{ov}=(25.0+62.5)%</td><td>e=-25 mm</td></tr>
<tr><td>Brace 1
(overlapped - j)</td><td>Brace 2
(overlapping - i)</td><td>Brace 3
(overlapped - j)</td><td rowspan="7">Validity limits check
b_2/b_0=150/350=0.429>0.25
Brace 1 and brace 2 cross-section class 1
b_3/t_3=200/10=20<35
h_3/t_3=200/10=20<35
h_0/b_0=350/350=1>0.5
h_0/b_0=350/350=1<2
h_1/b_1=100/200=0.5=0.5
h_2/b_2=150/150=1>0.5
h_2/b_2=150/150=1<2
h_3/b_3=200/200=1>0.5
h_3/b_3=200/200=1<2
Chord cross-section class 1
λ_{ov}>25 %;
b_2/b_1=150/200=0.75
b_2/b_3=150/200=0.75
e=-25>-0.55·350=-192.5</td></tr>
<tr><td>SB12</td><td>SB13</td><td>SB14</td></tr>
<tr><td>RHS 200x100x6</td><td>RHS 150x150x6</td><td>RHS 200x200x10</td></tr>
<tr><td>θ=57.11°</td><td>θ=90°</td><td>θ=55.70°</td></tr>
<tr><td>$N_{1,Ed}$=-5.89 kN</td><td>$N_{2,Ed}$=-132.63 kN</td><td>$N_{3,Ed}$=278.92 kN</td></tr>
<tr><td colspan="3">Chord SHS 350x350x12.5
$N_{0,Ed}$=541.18 kN
$M_{0,Ed}$=19.98 kNm</td></tr>
<tr><td colspan="3">KT joint resistance
$b_{e,ov,1}$=45 mm
$b_{e,ov,2}$=125 mm
$N_{2,Rd}$=628.86 kN (brace failure is the governing failure mode)
$N_{1,Rd}$= 628.86 kN
$N_{3,Rd}$=1358.79 kN
Joint resistance check
$N_{1,Ed}/N_{1,Rd}$=5.89/628.86=0.009<1</td></tr>
</table>

$N_{2,Ed}/N_{2,Rd}$=132.63/628.86=0.211<1 $N_{3,Ed}/N_{3,Rd}$=278.92/1358.79=0.205<1 $h_1<b_j$, λ_{ov}>80%=> local shear check is necessary 5.89cos57.11°+278.92cos55.70°<V_L=1303.5 kN	

Table A.68: Design checks - Joint 8 (right/left supporting beam)

Joint 8 (right/left supporting beam)			
Geometry:	N overlap	λ_{ov}=37.10 %	e=0 mm
Brace 1 (overlapped - j)	Brace 2 (overlapping - i)	Chord	Validity limits check b_2/b_0=150/350=0.429>0.25 Brace 1 and brace 2 cross-section class 1 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=200/200=1>0.5 h_1/b_1=200/200=1<2 h_2/b_2=150/150=1>0.5 h_2/b_2=150/150=1<2 Chord cross-section class 1 λ_{ov}>25 % b_2/b_1=150/200=0.75
SM15	SM16	SHS 350x350x12.5	
SHS 200x200x10	RHS 150x150x6	$N_{0,Ed}$=372.01 kN	
θ=49.09°	θ=90°	$M_{0,Ed}$=0 kNm	
$N_{1,Ed}$=-654.62 kN	$N_{2,Ed}$=-18.09 kN		
N joint resistance b_{eff}=111.61 mm $b_{e,ov}$=125 mm $N_{2,Rd}$=613.65 kN (brace failure is the governing failure mode) $N_{1,Rd}$=1325.91 kN Joint resistance check $N_{1,Ed}/N_{1,Rd}$=654.62/1325.91=0.494<1 $N_{2,Ed}/N_{2,Rd}$=18.09/613.65=0.029<1 $h_i=b_i$, $h_j=b_j$, λ_{ov}<60% => local shear check not necessary			

Table A.69: Design checks - Joint 9 (right/left supporting beam)

Joint 9 (right/left supporting beam)			
Geometry:	N gap	g=103.5 mm	e=87.5 mm
Brace 1	Brace 2	Chord	Validity limits check $b_{1(2)}/b_0$=200/350=0.571>0.35 $b_{1(2)}/b_0$=200/350=0.571>0.1+0.01·350/12.5 b_1/t_1=200/6=33.33<35 h_1/t_1=100/6=16.66<35 b_2/t_2=200/6=33.33<35 h_2/t_2=100/6=16.66<35 Brace 1 cross-section class 1 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 h_2/b_2=100/200=0.5=0.5 b_0/t_0=350/12.5=28<35 g/b_0=103.5/350=0.296>0.5(1-0.571) g/b_0=103.5/350=0.296<1.5(1-0.571) e=87.5 =0.25·350=87.5
SB1	SB2	SHS 350x350x12.5	
RHS 200x100x6	RHS 200x100x6	$N_{0,Ed}$=58.31 kN	
θ=90°	θ=50.22°	$M_{0,Ed}$=0 kNm	
$N_{1,Ed}$=104.29 kN	$N_{2,Ed}$=-99.93 kN		
N joint resistance (brace 1: 76.34 kN, brace 2: -99.33 kN) $N_{1,Rd}$=681.9 kN (chord face failure is the governing failure mode) $N_{2,Rd}$=524.0kN (chord face failure is the governing failure mode) $N_{0,Rd}$=3924 kN (chord shear failure) T joint resistance (brace 1: 27.95 kN) $N_{1,Rd}$=273.3 kN (chord face failure is the governing failure mode) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=76.34/681.9+27.95/273.3=0.214<1 $N_{2,Ed}/N_{2,Rd}$=99.33/524.0=0.190<1 $N_{0,Ed}/N_{0,Rd}$=58.31/3924=0.015<1			

Table A.70: Design checks - Joint 10 (right/left supporting beam)

Joint 10 (right/left supporting beam)			
Geometry:	T joint		
Brace 1		Chord	Validity limits check b_1/b_0=150/350=0.429>0.25 b_1/t_1=150/6=25<35 h_1/t_1=150/6=25<35 Brace 1 cross-section class 1 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=150/150=1>0.5 h_1/b_1=150/150=1<2 b_0/t_0=350/12.5=28<35
SB3		SHS 350x350x12.5	
SHS 150x150x6		$N_{0,Ed}$=-51.43 kN	
θ=90°		$M_{0,Ed}$=63.67 kNm	
$N_{1,Ed}$=-130.28 kN			
T joint resistance $N_{1,Rd}$=249.4 kN (chord face failure because β<0.85) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=130.28/249.4=0.522<1			

Table A.71: Design checks - Joint 11 (right/left supporting beam)

Joint 11 (right/left supporting beam)			
Geometry:	KT overlap	λ_{ov}=(27.57+41.27)%	e=-65 mm
Brace 1 (overlapped - j)	Brace 2 (overlapping - i)	Brace 3 (overlapped - j)	Validity limits check b_2/b_0=150/350=0.429>0.25 Brace 1 cross-section class 1 b_2/t_2=150/6=25<35 h_2/t_2=150/6=25<35 b_3/t_3=200/6=33.33<35 h_3/t_3=100/6=16.66<35 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 h_2/b_2=150/150=1>0.5 h_2/b_2=150/150=1<2 h_3/b_3=100/200=0.5=0.5 Chord cross-section class 1 λ_{ov}>25 %; b_2/b_1=150/200=0.75 b_2/b_3=150/200=0.75 e=-65 >-0.55·350=-192.5
SB4	SB5	SB6	
RHS 200x100x6	RHS 150x150x6	RHS 200x100x6	
θ=47.23°	θ=90°	θ=56.38°	
$N_{1,Ed}$=-370.95 kN	$N_{2,Ed}$=67.6 kN	$N_{3,Ed}$=208.83 kN	
Chord SHS 350x350x12.5 $N_{0,Ed}$=-308.73 kN $M_{0,Ed}$=-35.21 kNm			
KT joint resistance $b_{e,ov}$=45 mm $N_{2,Rd}$=516.06 kN (brace failure is the governing failure mode) $N_{1,Rd}$=516.06 kN $N_{3,Rd}$=516.06 kN Joint resistance check $N_{1,Ed}/N_{1,Rd}$=370.95/516.06=0.719<1 $N_{2,Ed}/N_{2,Rd}$=67.6/516.06=0.131<1 $N_{3,Ed}/N_{3,Rd}$=208.83/516.06=0.405<1 h_j<b_j=> local shear check is necessary 370.95cos47.23°+208.83cos56.38°<V_L=852.37 kN			

Table A.72: Design checks - Joint 12 (right/left supporting beam)

Joint 12 (right/left supporting beam)			
Geometry:	K gap	g=127.8 mm	e=0 mm
Brace 1	Brace 2	Chord	Validity limits check $b_{1(2)}/b_0$=160/350=0.457>0.35 $b_{1(2)}/b_0$=160/350=0.457>0.1+0.01·350/12.5 b_1/t_1=160/5=32<35 h_1/t_1=80/5=16<35 Brace 1 cross-section class 1 b_2/t_2=160/5=32<35 h_2/t_2=80/5=16<35 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=80/160=0.5=0.5 h_2/b_2=80/160=0.5=0.5 b_0/t_0=350/12.5=28<35 g/b_0=127.8/350=0.365>0.5(1-0.457) g/b_0=127.8/350=0.365<1.5(1-0.457)
SB7	SB8	SHS 350x350x12.5	
RHS 160x80x5	RHS 160x80x5	$N_{0,Ed}$=-421.56 kN	
θ=57.54°	θ=57.54°	$M_{0,Ed}$=-11.75 kNm	
$N_{1,Ed}$=-110.72 kN	$N_{2,Ed}$=100.15 kN		
K joint resistance (forces ratio 90%) $N_{1,Rd}$=496.9 kN (chord face failure is the governing failure mode) $N_{2,Rd}$=496.9 kN (chord face failure is the governing failure mode) $N_{0,Rd}$=3921 kN (chord shear failure) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=110.72/496.9=0.223<1 $N_{2,Ed}/N_{2,Rd}$=100.15/496.9=0.202<1 $N_{0,Ed}/N_{0,Rd}$=421.56/3921=0.108<1			

Table A.73: Design checks - Joint 13 (right/left supporting beam)

Joint 13 (right/left supporting beam)			
Geometry:	K gap	g=127.8 mm	e=0 mm
Brace 1	Brace 2	Chord	Validity limits check $b_{1(2)}/b_0$=160/350=0.457>0.35 $b_{1(2)}/b_0$=160/350=0.457>0.1+0.01·350/12.5 b_1/t_1=160/5=32<35 h_1/t_1=80/5=16<35 Brace 1 cross-section class 1 b_2/t_2=160/5=32<35 h_2/t_2=80/5=16<35 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=80/160=0.5=0.5 h_2/b_2=80/160=0.5=0.5
SB9	SB10	SHS 350x350x12.5	
RHS 160x80x5	RHS 160x80x5	$N_{0,Ed}$=-519.51 kN	
θ=57.54°	θ=57.54°	$M_{0,Ed}$=-12.38 kNm	
$N_{1,Ed}$=-95.20 kN	$N_{2,Ed}$=87.92 kN		
K joint resistance (forces ratio 92%) $N_{1,Rd}$=496.9 kN (chord face failure is the governing failure mode) $N_{2,Rd}$=496.9 kN (chord face failure is the governing failure mode) $N_{0,Rd}$=3922 kN (chord shear failure) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=95.20/496.9=0.192<1 $N_{2,Ed}/N_{2,Rd}$=87.92/496.9=0.177<1			

$N_{0,Ed}/N_{0,Rd}=519.51/3922=0.132<1$	$b_0/t_0=350/12.5=28<35$ $g/b_0=127.8/350=0.365>0.5(1-0.457)$ $g/b_0=127.8/350=0.365<1.5(1-0.457)$

Table A.74: Design checks - Joint 14 (right/left supporting beam)

Joint 14 (right/left supporting beam)			
Geometry:	K gap	g=117.5 mm	e=0 mm
Brace 1	Brace 2	Chord	Validity limits check $b_1/b_0=160/350=0.457>0.35$ $b_1/b_0=160/350=0.457>0.1+0.01\cdot 350/12.5$ $b_2/b_0=200/350=0.571>0.35$ $b_2/b_0=200/350=0.571>0.1+0.01\cdot 350/12.5$ $b_1/t_1=160/5=32<35$ $h_1/t_1=80/5=16<35$ Brace 1 cross-section class 1 $b_2/t_2=200/6=33.33<35$ $h_2/t_2=100/6=16.66<35$ $h_0/b_0=350/350=1>0.5$ $h_0/b_0=350/350=1<2$ $h_1/b_1=80/160=0.5=0.5$ $h_2/b_2=8100/200=0.5=0.5$ $b_0/t_0=350/12.5=28<35$ $g/b_0=117.5/350=0.336>0.5(1-0.457)$ $g/b_0=127.8/350=0.336<1.5(1-0.457)$
SB11	SB12	SHS 350x350x12.5	
RHS 160x80x5	RHS 200x100x6	$N_{0,Ed}$=-567.33 kN	
θ=57.54°	θ=57.11°	$M_{0,Ed}$=-17.96 kNm	
$N_{1,Ed}$=-88.33 kN	$N_{2,Ed}$=1.46 kN		
Y joint resistance (Force in Brace 2≈0) $N_{1,Rd}$=279.7 kN (chord face failure is the governing failure mode) Joint resistance check $N_{1,Ed}/N_{1,Rd}=88.33/279.7=0.316<1$			

Table A.75: Design checks - Joint 15 (right/left supporting beam)

Joint 15 (right/left supporting beam)			
Geometry:	KT overlap	λ_{ov}=(42.01+25.57)%	e=20 mm
Brace 1 (overlapped - j)	Brace 2 (overlapping - i)	Brace 3 (overlapped - j)	Validity limits check $b_2/b_0=150/350=0.429>0.25$ $b_1/t_1=200/10=20<35$ $h_1/t_1=200/10=20<35$ $b_2/t_2=150/6=25<35$ $h_2/t_2=150/6=25<35$ Brace 3 cross-section class 1 $h_0/b_0=350/350=1>0.5$ $h_0/b_0=350/350=1<2$ $h_1/b_1=200/200=1<0.5$ $h_1/b_1=200/200=1<2$ $h_2/b_2=150/150=1>0.5$ $h_2/b_2=150/150=1<2$ $h_3/b_3=200/200=1<0.5$ $h_3/b_3=200/200=1<2$ Chord cross-section class 1 $\lambda_{ov}>25$ %; $b_2/b_1=150/200=0.75$ $b_2/b_3=150/200=0.75$ $e=20<0.25\cdot 350=87.5$
SB14	SB15	SB16	
RHS 200x200x10	RHS 150x150x6	RHS 200x200x10	
θ=55.7°	θ=90°	θ=49.09°	
$N_{1,Ed}$=278.92 kN	$N_{2,Ed}$=184.23 kN	$N_{3,Ed}$=-654.62 kN	
Chord SHS 350x350x12.5 $N_{0,Ed}$=-565.83 kN $M_{0,Ed}$=-21.57 kNm			
KT joint resistance $b_{e,ov}$=125 mm $N_{2,Rd}$=741.66 kN (brace failure is the governing failure mode) $N_{1,Rd}$=1602.52 kN $N_{3,Rd}$=1602.52 kN Joint resistance check $N_{1,Ed}/N_{1,Rd}=278.92/1602.52=0.174<1$ $N_{2,Ed}/N_{2,Rd}=184.23/741.66=0.248<1$ $N_{3,Ed}/N_{3,Rd}=654.62/1602.52=0.408<1$ $h_i=b_i$, $h_j=b_j$, $\lambda_{ov}<80\%$ => local shear check not necessary			

Table A.76: Design checks - Joint 16 (right/left supporting beam)

Joint 16 (right/left supporting beam)			
Geometry:	T joint		
Brace 1		Chord	Validity limits check $b_1/b_0=150/350=0.429>0.25$ $b_1/t_1=150/6=25<35$ $h_1/t_1=150/6=25<35$ Brace 1 cross-section class 1
SB16		SHS 350x350x12.5	
SHS 150x150x6		$N_{0,Ed}$=0kN	
θ=90°		$M_{0,Ed}$=0 kNm	
$N_{1,Ed}$=-19.62 kN			

T joint resistance $N_{1,Rd}$=249.4 kN (chord face failure because $\beta<0.85$) Joint resistance check $N_{1,Ed}/N_{1,Rd}=19.62/249.4=0.079<1$	$h_0/b_0=350/350=1>0.5$ $h_0/b_0=350/350=1<2$ $h_1/b_1=150/150=1>0.5$ $h_1/b_1=150/150=1<2$ $b_0/t_0=350/12.5=28<35$

Table A.77: Design checks - Joint 1 (connecting beam C1)

Joint 1 (connecting beam C1)			
Geometry:	N gap	g=105.1 mm	e=80 mm
Brace 1	Brace 2	Chord	Validity limits check
SB13	C1B1	SHS 350x350x12.5	$b_1/b_0=150/350=0.429>0.35$
SHS 150x150x6	RHS 200x100x8	$N_{0,Ed}$=-315.58 kN	$b_1/b_0=150/350=0.429>0.1+0.01\cdot 350/12.5$
θ=90°	θ=45.55°	$M_{0,Ed}$=-6 kNm	$b_1/t_1=150/6=25<35$
$N_{1,Ed}$=-125.30 kN	$N_{2,Ed}$=365.44 kN		$h_1/t_1=150/6=25<35$
N joint resistance (brace 1: -125.30 kN, brace 2: 175.52 kN) $N_{1,Rd}$=524.0 kN (chord face failure is the governing failure mode) $N_{2,Rd}$=734.1 kN (chord face failure is the governing failure mode) $N_{0,Rd}$=3915 kN (chord shear failure) Y joint resistance (brace 1: 189.92 kN) $N_{2,Rd}$=410.4 kN (chord face failure is the governing failure mode) Joint resistance check $N_{1,Ed}/N_{1,Rd}=125.3/524.0=0.239<1$ $N_{2,Ed}/N_{2,Rd}=175.52/734.1+189.92/410.4=0.702<1$ $N_{0,Ed}/N_{0,Rd}=315.58/3915=0.081<1$			Brace 1 cross-section class 1 $b_2/t_2=200/8=25<35$ $h_2/t_2=100/8=12.5<35$ $h_0/b_0=350/350=1>0.5$ $h_0/b_0=350/350=1<2$ $h_1/b_1=150/150=1>0.5$ $h_1/b_1=150/150=1<2$ $h_2/b_2=100/200=0.5=0.5$ $b_0/t_0=350/12.5=28<35$ $g/b_0=105.1/350=0.300>0.5(1-0.429)$ $g/b_0=103.5/350=0.300<1.5(1-0.429)$ $e=80 =0.25\cdot 350=87.5$

Table A.78: Design checks - Joint 2 (connecting beam C1)

Joint 2 (connecting beam C1)			
Geometry:	K gap	g=183.6 mm	e=-10 mm
Brace 1	Brace 2	Chord	Validity limits check
C1B1	C1B2	SHS 350x350x12.5	$b_{1(2)}/b_0=200/350=0.571>0.35$
RHS 200x100x8	RHS 200x100x8	$N_{0,Ed}$=480.72 kN	$b_{1(2)}/b_0=200/350=0.571>0.1+0.01\cdot 350/12.5$
θ=45.55°	θ=45.55°	$M_{0,Ed}$=13.80 kNm	$b_1/t_1=200/8=25<35$
$N_{1,Ed}$=365.44 kN	$N_{2,Ed}$=-346.45 kN		$h_1/t_1=100/8=12.5<35$
K joint resistance (forces ratio 95%) $N_{1,Rd}$=734.1 kN (chord face failure is the governing failure mode) $N_{2,Rd}$=734.1 kN (chord face failure is the governing failure mode) $N_{0,Rd}$=3877 kN (chord shear failure) Joint resistance check $N_{1,Ed}/N_{1,Rd}=365.44/734.1=0.498<1$ $N_{2,Ed}/N_{2,Rd}=346.45/734.1=0.472<1$ $N_{0,Ed}/N_{0,Rd}=480.72/3877=0.124<1$			$b_2/t_2=200/8=25<35$ $h_2/t_2=100/8=12.5<35$ Brace 2 cross-section class 1 $h_0/b_0=350/350=1>0.5$ $h_0/b_0=350/350=1<2$ $h_1/b_1=100/200=0.5=0.5$ $h_2/b_2=100/200=0.5=0.5$ $b_0/t_0=350/12.5=28<35$ $g/b_0=183.6/350=0.525>0.5(1-0.571)$ $g/b_0=183.6/350=0.525<1.5(1-0.571)$ $e=-10 >-0.55\cdot 350=-192.5$

Table A.79: Design checks - Joint 3 (connecting beam C1)

Joint 3 (connecting beam C1)			
Geometry:	KT gap	g=133 mm	e=80 mm
Brace 1	Brace 2	Brace 3	Validity limits check
C1B2	SM12	C1B3	$b_2/b_0=150/350=0.429>0.25$
RHS 200x100x8	RHS 150x150x6	RHS 200x100x6	$b_2/b_0=150/350=0.429>0.1+0.01\cdot 350/12.5$
θ=45.55°	θ=90°	θ=41.87°	$b_1/t_1=200/8=25<35$
$N_{1,Ed}$=-346.45 kN	$N_{2,Ed}$=17.34 kN	$N_{3,Ed}$=38.88 kN	$h_1/t_1=100/8=12.5<35$
Chord SHS 350x350x12.5 $N_{0,Ed}$=-643.87 $M_{0,Ed}$=-15.64 kNm			Brace 1 cross-section class 1 $b_2/t_2=150/6=25<35$ $h_2/t_2=150/6=25<35$

Y joint resistance (because: $N_{1,Ed}$>>$N_{2,Ed}$ and $N_{1,Ed}$>>$N_{3,Ed}$) $N_{1,Rd}$=410.4 kN (chord face failure is the governing failure mode) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=346.45/410.4=0.844<1	b_3/t_3=200/6=33.33<35 h_3/t_3=100/6=16.66<35 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 h_2/b_2=150/150=1>0.5 h_2/b_2=150/150=1<2 h_3/b_3=100/200=0.5=0.5 b_0/t_0=350/12.5=28<35 g/b_0=133/350=0.380>0.5(1-0.429) g/b_0=133/350=0.380<1.5(1-0.429) e=80 <0.25·350=87.5

Table A.80: Design checks - Joint 4 (connecting beam C1)

Joint 4 (connecting beam C1)			
Geometry:	K gap	g=118 mm	e=-55 mm
Brace 1	Brace 2	Chord	Validity limits check $b_{1(2)}/b_0$=200/350=0.571>0.35 $b_{1(2)}/b_0$=200/350=0.571>0.1+0.01·350/12.5 b_1/t_1=200/6=33.33<35 h_1/t_1=100/6=16.66<35 b_2/t_2=200/6=33.33<35 h_2/t_2=100/6=16.66<35 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 h_2/b_2=100/200=0.5=0.5 b_0/t_0=350/12.5=28<35 g/b_0=118/350=0.337>0.5(1-0.571) g/b_0=118/350=0.337<1.5(1-0.571) e=-55 >-0.55·350=-192.5
C1B3	C1B4	SHS 350x350x12.5	
RHS 200x100x6	RHS 200x100x6	$N_{0,Ed}$=-532.55 kN	
θ=41.87°	θ=41.89°	$M_{0,Ed}$=0 kNm	
$N_{1,Ed}$=41.41 kN	$N_{2,Ed}$=91.53 kN		
Unidirectional K joint resistance b_{eq}=(200+200)/2=200 mm h_{eq}=100/sin41.87°+118+100/sin41.89°=417.62 mm β=200/350=0.571 (chord face failure is the governing mode) $N_{eq,Rd}$=428.46 kN $N_{1,Rd}$=199.91 kN $N_{2,Rd}$=441.86 kN Joint resistance check $N_{1,Ed}/N_{1,Rd}$=41.41/199.91=0.207<1 $N_{2,Ed}/N_{2,Rd}$=91.53/441.86=0.207<1			

Table A.81: Design checks - Joint 5 (connecting beam C1)

Joint 5 (connecting beam C1)			
Geometry:	KT gap	g=100.9 mm	e=80 mm
Brace 1	Brace 2	Brace 3	Validity limits check b_2/b_0=150/350=0.429>0.25 b_2/b_0=150/350=0.429>0.1+0.01·350/12.5 b_1/t_1=200/6=33.33<35 h_1/t_1=100/6=16.66<35 b_2/t_2=150/6=25<35 h_2/t_2=150/6=25<35 b_3/t_3=200/8=25<35 h_3/t_3=100/8=12.5<35 Brace 1 cross-section class 1 h_0/b_0=350/350=1>0.5 h_0/b_0=350/350=1<2 h_1/b_1=100/200=0.5=0.5 h_2/b_2=150/150=1>0.5 h_2/b_2=150/150=1<2 h_3/b_3=100/200=0.5=0.5 b_0/t_0=350/12.5=28<35 g/b_0=100.9/350=0.288>0.5(1-0.429) g/b_0=100.9/350=0.288<1.5(1-0.429) e=80 <0.25·350=87.5
C1B4	SM12	C1B5	
RHS 200x100x6	RHS 150x150x6	RHS 200x100x8	
θ=41.89°	θ=90°	θ=46.11°	
$N_{1,Ed}$=91.53 kN	$N_{2,Ed}$=8.91 kN	$N_{3,Ed}$=-373.97 kN	
Chord SHS 350x350x12.5 $N_{0,Ed}$=-557.06 $M_{0,Ed}$=-9.15 kNm			
Y joint resistance (because: $N_{3,Ed}$>>$N_{2,Ed}$ and $N_{3,Ed}$>>$N_{1,Ed}$) $N_{1,Rd}$=405.6 kN (chord face failure is the governing failure mode) Joint resistance check $N_{1,Ed}/N_{1,Rd}$=373.97/405.6=0.922<1			

Table A.82: Design checks - Joint 6 (connecting beam C1)

<table>
<tr><td colspan="4">Joint 6 (connecting beam C1)</td></tr>
<tr><td>Geometry:</td><td>K gap</td><td>g=102.1 mm</td><td>e=-40 mm</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="6">Validity limits check
$b_{1(2)}/b_0$=200/350=0.571>0.35
$b_{1(2)}/b_0$=200/350=0.571>0.1+0.01·350/12.5
b_1/t_1=200/8=25<35
h_1/t_1=100/8=12.5<35
Brace 1 cross-section class 1
b_2/t_2=200/8=25<35
h_2/t_2=100/8=12.5<35
h_0/b_0=350/350=1>0.5
h_0/b_0=350/350=1<2
h_1/b_1=100/200=0.5=0.5
h_2/b_2=100/200=0.5=0.5
b_0/t_0=350/12.5=28<35
g/b_0=183.6/350=0.525>0.5(1-0.571)
g/b_0=183.6/350=0.525<1.5(1-0.571)
e=-40 >-0.55·350=-192.5</td></tr>
<tr><td>C1B5</td><td>C1B6</td><td>SHS 350x350x12.5</td></tr>
<tr><td>RHS 200x100x8</td><td>RHS 200x100x8</td><td>$N_{0,Ed}$=438.38 kN</td></tr>
<tr><td>θ=46.11°</td><td>θ=52.57°</td><td>$M_{0,Ed}$=13.31 kNm</td></tr>
<tr><td>$N_{1,Ed}$=-385.38 kN</td><td>$N_{2,Ed}$=337.47 kN</td><td></td></tr>
<tr><td colspan="3">K joint resistance (forces ratio 88%)
$N_{1,Rd}$=727.2 kN (chord face failure is the governing failure mode)
$N_{2,Rd}$=659.9 kN (chord face failure is the governing failure mode)
$N_{0,Rd}$=3876 kN (chord shear failure)
Joint resistance check
$N_{1,Ed}/N_{1,Rd}$=385.38/727.2=0.530<1
$N_{2,Ed}/N_{2,Rd}$=337.47/659.9=0.511<1
$N_{0,Ed}/N_{0,Rd}$=438.38/3876=0.113<1</td></tr>
</table>

Table A.83: Design checks - Joint 7 (connecting beam C1)

<table>
<tr><td colspan="4">Joint 7 (connecting beam C1)</td></tr>
<tr><td>Geometry:</td><td>N overlap</td><td>λ_{ov}=56.25 %</td><td>e=-105 mm</td></tr>
<tr><td>Brace 1
(overlapped - j)</td><td>Brace 2
(overlapping - i)</td><td>Chord</td><td rowspan="6">Validity limits check
b_2/b_0=150/350=0.429>0.25
b_1/t_1=200/8=25<35
h_1/t_1=100/8=12.5<35
Brace 2 cross-section class 1
h_0/b_0=350/350=1>0.5
h_0/b_0=350/350=1<2
h_1/b_1=100/200=0.5=0.5
h_2/b_2=150/150=1>0.5
h_2/b_2=150/150=1<2
Chord cross-section class 1
λ_{ov}>25 %
b_2/b_1=150/200=0.75</td></tr>
<tr><td>C1B6</td><td>SB15</td><td>SHS 350x350x12.5</td></tr>
<tr><td>RHS 200x100x8</td><td>RHS 150x150x6</td><td>$N_{0,Ed}$=-253.45 kN</td></tr>
<tr><td>θ=52.57°</td><td>θ=90°</td><td>$M_{0,Ed}$=-9.51 kNm</td></tr>
<tr><td>$N_{1,Ed}$=337.47 kN</td><td>$N_{2,Ed}$=-132.63 kN</td><td></td></tr>
<tr><td colspan="3">N joint resistance
b_{eff}=111.61 mm; $b_{e,ov}$=125 mm
$N_{2,Rd}$=722.78 kN (brace failure is the governing failure mode)
$N_{1,Rd}$=1561.72 kN
Joint resistance check
$N_{1,Ed}/N_{1,Rd}$=337.47/1561.72=0.216<1
$N_{2,Ed}/N_{2,Rd}$=132.63/722.78=0.183<1
$h_i=b_i$, $h_j=b_j$, λ_{ov}<60% => local shear check not necessary</td></tr>
</table>

Table A.84: Design checks - Joint 1 (connecting beam C2)

<table>
<tr><td colspan="4">Joint 1 (connecting beam C2)</td></tr>
<tr><td>Geometry:</td><td>N gap</td><td>g=105.1 mm</td><td>e=80 mm</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="6">Validity limits check
b_1/b_0=150/350=0.429>0.35
b_1/b_0=150/350=0.429>0.1+0.01·350/12.5
b_1/t_1=150/6=25<35
h_1/t_1=150/6=25<35
b_2/t_2=200/8=25<35
h_2/t_2=100/8=12.5<35
Brace 2 cross-section class 1
h_0/b_0=350/350=1>0.5
h_0/b_0=350/350=1<2
h_1/b_1=150/150=1>0.5
h_1/b_1=150/150=1<2
h_2/b_2=100/200=0.5=0.5
b_0/t_0=350/12.5=28<35
g/b_0=105.1/350=0.300>0.5(1-0.429)
g/b_0=103.5/350=0.300<1.5(1-0.429)
e=80 =0.25·350=87.5</td></tr>
<tr><td>SB15</td><td>C2B1</td><td>SHS 350x350x12.5</td></tr>
<tr><td>SHS 150x150x6</td><td>RHS 200x100x8</td><td>$N_{0,Ed}$=303.88 kN</td></tr>
<tr><td>θ=90°</td><td>θ=45.55°</td><td>$M_{0,Ed}$=-6 kNm</td></tr>
<tr><td>$N_{1,Ed}$=163.69 kN</td><td>$N_{2,Ed}$=-387.51 kN</td><td></td></tr>
<tr><td colspan="3">N joint resistance (brace 1: 163.69 kN, brace 2: -229.30 kN)
$N_{1,Rd}$=524.0 kN (chord face failure is the governing failure mode)
$N_{2,Rd}$=734.1 kN (chord face failure is the governing failure mode)
$N_{0,Rd}$=3915 kN (chord shear failure)
Y joint resistance (brace 2: -158.21 kN)
$N_{2,Rd}$=410.4 kN (chord face failure is the governing failure mode)
Joint resistance check
$N_{1,Ed}/N_{1,Rd}$=163.69/524=0.312<1
$N_{2,Ed}/N_{2,Rd}$=229.3/734.1+158.21/410.4=0.698<1
$N_{0,Ed}/N_{0,Rd}$=303.88/3915=0.077<1</td></tr>
</table>

Table A.85: Design checks - Joint 2 (connecting beam C2)

Joint 2 (connecting beam C2)			
Geometry:	K gap	g=183.6 mm	e=-10 mm
Brace 1	Brace 2	Chord	Validity limits check
C2B1	C2B2	SHS 350x350x12.5	$b_{1(2)}/b_0=200/350=0.571>0.35$
RHS 200x100x8	RHS 200x100x8	$N_{0,Ed}=-462.56$ kN	$b_{1(2)}/b_0=200/350=0.571>0.1+0.01\cdot 350/12.5$
$\theta=45.55°$	$\theta=45.55°$	$M_{0,Ed}=-6.60$ kNm	$b_1/t_1=200/8=25<35$
$N_{1,Ed}=-387.51$ kN	$N_{2,Ed}=326.69$ kN		$h_1/t_1=100/8=12.5<35$
K joint resistance (forces ratio 84%)			$b_2/t_2=200/8=25<35$
$N_{1,Rd}=734.1$ kN (chord face failure is the governing failure mode)			$h_2/t_2=100/8=12.5<35$
$N_{2,Rd}=734.1$ kN (chord face failure is the governing failure mode)			Brace 2 cross-section class 1
$N_{0,Rd}=3887$ kN (chord shear failure)			$h_0/b_0=350/350=1>0.5$
Joint resistance check			$h_0/b_0=350/350=1<2$
$N_{1,Ed}/N_{1,Rd}=387.51/734.1=0.528<1$			$h_1/b_1=100/200=0.5=0.5$
$N_{2,Ed}/N_{2,Rd}=326.69/734.1=0.445<1$			$h_2/b_2=100/200=0.5=0.5$
$N_{0,Ed}/N_{0,Rd}=462.56/3887=0.119<1$			$b_0/t_0=350/12.5=28<35$
			$g/b_0=183.6/350=0.525>0.5(1-0.571)$
			$g/b_0=183.6/350=0.525<1.5(1-0.571)$
			$e=-10>-0.55\cdot 350=-192.5$

Table A.86: Design checks - Joint 3 (connecting beam C2)

Joint 3 (connecting beam C2)			
Geometry:	KT gap	g=133 mm	e=80 mm
Brace 1	Brace 2	Brace 3	Validity limits check
C2B2	SM14	C2B3	$b_2/b_0=150/350=0.429>0.35$
RHS 200x100x8	RHS 150x150x6	RHS 200x100x6	$b_2/b_0=150/350=0.429>0.1+0.01\cdot 350/12.5$
$\theta=45.55°$	$\theta=90°$	$\theta=41.87°$	$b_1/t_1=200/8=25<35$
$N_{1,Ed}=326.69$ kN	$N_{2,Ed}=-170.53$ kN	$N_{3,Ed}=-46.01$ kN	$h_1/t_1=100/8=12.5<35$
Chord SHS 350x350x12.5			$b_2/t_2=150/6=25<35$
$N_{0,Ed}=593.36$ kN			$h_2/t_2=150/6=25<35$
$M_{0,Ed}=30.48$ kNm			Brace 2 and brace 3 cross-section class 1
KT joint resistance			$b_3/t_3=200/6=33.33<35$
$N_{1,Rd}=734.8$ kN (chord face failure is the governing failure mode)			$h_3/t_3=100/6=16.66<35$
$N_{0,Rd}=3887$ kN (chord shear failure)			$h_0/b_0=350/350=1>0.5$
Joint resistance check			$h_0/b_0=350/350=1<2$
$N_{1,Ed}/N_{1,Rd}=326.69/734.830.445<1$			$h_1/b_1=100/200=0.5=0.5$
$N_{2,Ed}\sin\theta_2+N_{3,Ed}\sin\theta_3=201.24$ kN			$h_2/b_2=150/150=1>0.5$
$N_{1,Rd}\sin\theta_1=524.57$ kN			$h_2/b_2=150/150=1<2$
$201.24/524.57=0.384<1$			$h_3/b_3=100/200=0.5=0.5$
$N_{0,Ed}/N_{0,Rd}=593.36/3887=0.153<1$			$b_0/t_0=350/12.5=28<35$
			$g/b_0=133/350=0.380>0.5(1-0.429)$
			$g/b_0=133/350=0.380<1.5(1-0.429)$
			$e=80<0.25\cdot 350=87.5$

Table A.87: Design checks - Joint 4 (connecting beam C2)

Joint 4 (connecting beam C2)			
Geometry:	K gap	g=118 mm	e=-55 mm
Brace 1	Brace 2	Chord	Validity limits check
C2B3	C2B4	SHS 350x350x12.5	$b_{1(2)}/b_0=200/350=0.571>0.35$
RHS 200x100x6	RHS 200x100x6	$N_{0,Ed}=-486.04$ kN	$b_{1(2)}/b_0=200/350=0.571>0.1+0.01\cdot 350/12.5$
$\theta=41.87°$	$\theta=41.89°$	$M_{0,Ed}=9.55$ kNm	$b_1/t_1=200/6=33.33<35$
$N_{1,Ed}=-48.08$ kN	$N_{2,Ed}=-102.93$ kN		$h_1/t_1=100/6=16.66<35$
Unidirectional K joint resistance			$b_2/t_2=200/6=33.33<35$
$b_{eq}=(200+200)/2=200$ mm			$h_2/t_2=100/6=16.66<35$
$h_{eq}=100/\sin 41.87°+118+100/\sin 41.89°=417.62$ mm			$h_0/b_0=350/350=1>0.5$
$\beta=200/350=0.571$ (chord face failure is the governing mode)			$h_0/b_0=350/350=1<2$
$N_{eq,Rd}=428.46$ kN			$h_1/b_1=100/200=0.5=0.5$
			$h_2/b_2=100/200=0.5=0.5$

<table>
<tr><td>$N_{1,Rd}$=199.91 kN
$N_{2,Rd}$=441.86 kN
Joint resistance check
$N_{1,Ed}/N_{1,Rd}$=48.08/199.91=0.241<1
$N_{2,Ed}/N_{2,Rd}$=102.93/441.86=0.233<1</td><td>b_0/t_0=350/12.5=28<35
g/b_0=118/350=0.337>0.5(1-0.571)
g/b_0=118/350=0.337<1.5(1-0.571)
e=-55 >-0.55·350=-192.5</td></tr>
</table>

Table A.88: Design checks - Joint 5 (connecting beam C2)

<table>
<tr><td colspan="4">Joint 5 (connecting beam C2)</td></tr>
<tr><td>Geometry:</td><td>KT gap</td><td>g=100.9 mm</td><td>e=80 mm</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Brace 3</td><td rowspan="7">Validity limits check
b_2/b_0=150/350=0.429>0.35
b_2/b_0=150/350=0.429>0.1+0.01·350/12.5
b_1/t_1=200/6=33.33<35
h_1/t_1=100/6=16.66<35
b_2/t_2=150/6=25<35
h_2/t_2=150/6=25<35
Brace 1 and brace 2 cross-section class 1
b_3/t_3=200/8=25<35
h_3/t_3=100/8=12.5<35
h_0/b_0=350/350=1>0.5
h_0/b_0=350/350=1<2
h_1/b_1=100/200=0.5=0.5
h_2/b_2=150/150=1>0.5
h_2/b_2=150/150=1<2
h_3/b_3=100/200=0.5=0.5
b_0/t_0=350/12.5=28<35
g/b_0=100.9/350=0.288>0.5(1-0.429)
g/b_0=100.9/350=0.288<1.5(1-0.429)
e=80 <0.25·350=87.5</td></tr>
<tr><td>C2B4</td><td>SM14</td><td>C2B5</td></tr>
<tr><td>RHS 200x100x6</td><td>RHS 150x150x6</td><td>RHS 200x100x8</td></tr>
<tr><td>θ=41.89°</td><td>θ=90°</td><td>θ=46.11°</td></tr>
<tr><td>$N_{1,Ed}$=-101.55 kN</td><td>$N_{2,Ed}$=-175.77 kN</td><td>$N_{3,Ed}$=379.88 kN</td></tr>
<tr><td colspan="3">Chord SHS 350x350x12.5
$N_{0,Ed}$=593.36 kN
$M_{0,Ed}$=27.65 kNm</td></tr>
<tr><td colspan="3">KT joint resistance
$N_{3,Rd}$=727.9 kN (chord face failure is the governing failure mode)
$N_{0,Rd}$=3887 kN (chord shear failure)
Joint resistance check
$N_{3,Ed}/N_{3,Rd}$=379.88/727.9=0.520<1
$N_{1,Ed}\sin\theta_1+N_{1,Ed}\sin\theta_1$=243.58 kN
$N_{3,Rd}\sin\theta_3$=524.58 kN
243.58/524.58=0.464<1
$N_{0,Ed}/N_{0,Rd}$=593.36/3887=0.153<1</td></tr>
</table>

Table A.89: Design checks - Joint 6 (connecting beam C2)

<table>
<tr><td colspan="4">Joint 6 (connecting beam C2)</td></tr>
<tr><td>Geometry:</td><td>K gap</td><td>g=102.1 mm</td><td>e=-40 mm</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="6">Validity limits check
$b_{1(2)}/b_0$=200/350=0.571>0.35
$b_{1(2)}/b_0$=200/350=0.571>0.1+0.01·350/12.5
b_1/t_1=200/8=25<35
h_1/t_1=100/8=12.5<35
b_2/t_2=200/8=25<35
h_2/t_2=100/8=12.5<35
Brace 2 cross-section class 1
h_0/b_0=350/350=1>0.5
h_0/b_0=350/350=1<2
h_1/b_1=100/200=0.5=0.5
h_2/b_2=100/200=0.5=0.5
b_0/t_0=350/12.5=28<35
g/b_0=183.6/350=0.525>0.5(1-0.571)
g/b_0=183.6/350=0.525<1.5(1-0.571)
e=-40 >-0.55·350=-192.5</td></tr>
<tr><td>C2B5</td><td>C2B6</td><td>SHS 350x350x12.5</td></tr>
<tr><td>RHS 200x100x8</td><td>RHS 200x100x8</td><td>$N_{0,Ed}$=-422.71 kN</td></tr>
<tr><td>θ=46.11°</td><td>θ=52.57°</td><td>$M_{0,Ed}$=-4.33 kNm</td></tr>
<tr><td>$N_{1,Ed}$=379.88 kN</td><td>$N_{2,Ed}$=-372.76 kN</td><td></td></tr>
<tr><td colspan="3">K joint resistance (forces ratio 98%)
$N_{1,Rd}$=727.2 kN (chord face failure is the governing failure mode)
$N_{2,Rd}$=659.9 kN (chord face failure is the governing failure mode)
$N_{0,Rd}$=3876 kN (chord shear failure)
Joint resistance check
$N_{1,Ed}/N_{1,Rd}$=379.88/727.2=0.522<1
$N_{2,Ed}/N_{2,Rd}$=372.76/659.9=0.565<1
$N_{0,Ed}/N_{0,Rd}$=422.71/3876=0.109<1</td></tr>
</table>

Table A.90: Design checks - Joint 7 (connecting beam C2)

<table>
<tr><td colspan="4">Joint 7 (connecting beam C7)</td></tr>
<tr><td>Geometry:</td><td>N overlap</td><td>λ_{ov}=56.25 %</td><td>e=-105 mm</td></tr>
<tr><td>Brace 1
(overlapped - j)</td><td>Brace 2
(overlapping - i)</td><td>Chord</td><td rowspan="6">Validity limits check
b_2/b_0=150/350=0.429>0.25
Brace 1 cross-section class 1
b_2/t_2=150/6=25<35
h_2/t_2=150/6=25<35
h_0/b_0=350/350=1>0.5
h_0/b_0=350/350=1<2
h_1/b_1=100/200=0.5=0.5
h_2/b_2=150/150=1>0.5
h_2/b_2=150/150=1<2
Chord cross-section class 1
λ_{ov}>25 %
b_2/b_1=150/200=0.75</td></tr>
<tr><td>C2B6</td><td>SB17</td><td>SHS 350x350x12.5</td></tr>
<tr><td>RHS 200x100x8</td><td>RHS 150x150x6</td><td>$N_{0,Ed}$=233.42 kN</td></tr>
<tr><td>θ=52.57°</td><td>θ=90°</td><td>$M_{0,Ed}$=2.68 kNm</td></tr>
<tr><td>$N_{1,Ed}$=-372.76 kN</td><td>$N_{2,Ed}$=184.23 kN</td><td></td></tr>
<tr><td colspan="3">N joint resistance
b_{eff}=111.61 mm
$b_{e,ov}$=125 mm
$N_{2,Rd}$=722.78 kN (brace failure is the governing failure mode)
$N_{1,Rd}$=1561.72 kN
Joint resistance check
$N_{1,Ed}/N_{1,Rd}$=372.76/1561.72=0.239<1
$N_{2,Ed}/N_{2,Rd}$=184.23/722.78=0.255<1
$h_i=b_i$, $h_j=b_j$, λ_{ov}<60% => local shear check not necessary</td></tr>
</table>

Site joints

Table A.91: Site joints design checks (splices)

	Right/left side member (bottom chord)	Right/left supporting beam (bottom chord)	Brace SB12	Brace SM11
Bolts	(4+4)M20	(4+4)M20	(2+2)M12	(2+2)M12
$F_{t,Rd}$ [kN]	141.1	141.1	48.56	48.56
N_{Ed} [kN]	407.70	588.50	≈0	-369.75
M_{Ed} [kNm]	24.70	21.50	0	0
$N_{Ed,eff}$ [kN]	640.75	791.36	157.92 (20% of N_{Rd})	92.44 (25% of N_{Ed})
δ	0.78	0.78	0.88	0.88
a [mm]	45	45	30	30
b [mm]	40	40	25	25
a' [mm]	55	55	36	36
b' [mm]	42.5	42.5	25	25
K [1/MPa]	7.234	7.234	3.868	3.868
P_f [kN]	80.09	98.92	39.48	23.11
t_{min} [mm]	18.04	20.05	9.01	6.90
t_{max} [mm]	24.07	26.75	12.35	9.45
t_p [mm]	**22**	**22**	**12**	**12**
α	0.802	0.802	0.204	0.204
F_{Rd} [kN]	870.08	870.08	175.71	175.71
$N_{Ed,eff}/F_{Rd}$	**0.74**	**0.91**	**0.90**	**0.53**
α_{mod}	0.253	0.613	0.069	0
T_f [kN]	90.27	123.65	41.05	23.11
$T_f/N_{t,Rd}$	**0.64**	**0.88**	**0.85**	**0.48**
T_f/P_f	1.127	1.25	1.04	1
Prying [%]	12.7	25	4	0

For the upper chord splice joints, use the same layout and dimensions as for the bottom chord splices.

The upper chords are loaded in shear as well and the maximum acting force is $V_{Ed,max}$=169.12 kN.
One bolt is loaded with $F_{v,Ed}$=169.12/8=21.14 kN.
The design shear resistance of a M20 bolt is $F_{v,Rd}$=120.6 kN.
$F_{v,Ed}/F_{v,Rd}$=0.175<0.286 => No reduction of the tension resistance.

Weight of the structure

Table A.92: Weight (right/left side member-braces)

Member	Cross-section	Length [mm]	Unit weight [kg/m]	Total weight [kg]
SM1	RHS 200x100x6	1150	26.4	30.36
SM2	RHS 200x100x6	1568	26.4	41.40
SM3	RHS 200x100x8	1642	33.9	55.66
SM4	SHS 150x150x6	1150	26.4	30.36
SM4	RHS 200x100x8	1446	33.9	49.02
SM6	RHS 200x100x6	1426	26.4	37.65
SM7	RHS 200x100x6	1426	26.4	37.65
SM8	RHS 200x100x6	1426	26.4	37.65
SM9	RHS 200x100x6	1426	26.4	37.65
SM10	RHS 200x100x6	1415	26.4	37.36
SM11	SHS 200x200x10	1467	57	83.62
SM12	SHS 150x150x6	1150	26.4	30.36
SM13	SHS 200x200x10	1515	57	86.36
SM14	SHS 150x150x6	1150	26.4	30.36
			Σ	625.43
			2·Σ	1250.87

Table A.93: Weight (right/left supporting beam-braces)

Member	Cross-section	Length [mm]	Unit weight [kg/m]	Total weight [kg]
SB1	RHS 200x100x6	1150	26.4	30.36
SB2	RHS 200x100x6	1580	26.4	41.71
SB3	SHS 150x150x6	1150	26.4	30.36
SB4	RHS 200x100x6	1659	26.4	43.80
SB5	SHS 150x150x6	1150	26.4	30.36
SB6	RHS 200x100x6	1447	26.4	38.20
SB7	RHS 160x80x5	1414	17.5	24.75
SB8	RHS 160x80x5	1414	17.5	24.75
SB9	RHS 160x80x5	1414	17.5	24.75
SB10	RHS 160x80x5	1414	17.5	24.75
SB11	RHS 160x80x5	1434	17.5	25.10
SB12	RHS 200x100x6	1434	26.4	37.86
SB13	SHS 150x150x6	1150	26.4	30.36
SB14	SHS 200x200x10	1529	57	87.15
SB15	SHS 150x150x6	1150	26.4	30.36
SB16	SHS 200x200x10	1695	57	96.62
SB17	SHS 150x150x6	1150	26.4	30.36
			Σ	651.57
			2·Σ	1303.14

Table A.94: Weight (connecting beam C1-braces)

Member	Cross-section	Length [mm]	Unit weight [kg/m]	Total weight [kg]
C1B1	RHS 200x100x8	1709	33.9	57.94
C1B2	RHS 200x100x8	1709	33.9	57.94
C1B3	RHS 200x100x6	1835	26.4	48.44
C1B4	RHS 200x100x6	1835	26.4	48.44
C1B5	RHS 200x100x8	1692	33.9	57.36
C1B6	RHS 200x100x8	1525	33.9	51.70
			Σ	321.81

Table A.95: Weight (connecting beam C2-braces)

Member	Cross-section	Length [mm]	Unit weight [kg/m]	Total weight [kg]
C2B1	RHS 200x100x8	1709	33.9	57.94
C2B2	RHS 200x100x8	1709	33.9	57.94
C2B3	RHS 200x100x6	1835	26.4	48.44
C2B4	RHS 200x100x6	1835	26.4	48.44
C2B5	RHS 200x100x8	1692	33.9	57.36
C2B6	RHS 200x100x8	1525	33.9	51.70
			Σ	321.81

Table A.96: Weight (chords)

Member	Cross-section	Length [mm]	Unit weight [kg/m]	Quantity	Total weight [kg]
Right side member	SHS 350x350x12.5	9933	127	2	2522.98
Left side member	SHS 350x350x12.5	9933	127	2	2522.98
Right supporting beam	SHS 350x350x12.5	12000	127	2	3048.00
Left supporting beam	SHS 350x350x12.5	12000	127	2	3048.00
Connecting beam	SHS 350x350x12.5	8650	127	4	4394.20
				Σ	15536.16

Table A.97: Weight (brackets)

Member	Cross-section	Length [mm]	Unit weight [kg/m]	Quantity	Total weight [kg]
Vertical	HEB 360	4360	142	4	2476.48
Diagonal-left	HEB 360	3134	142	2	890.06
Diagonal-right	HEB 360	3518	142	2	999.11
Horizontal1-left	HEA 800	3020	224	2	1352.96
Horizontal1-right	HEA 800	3580	224	2	1603.84
Horizontal2-left	RHS 200x100x6	1645	26.4	2	86.86
Horizontal2-right	RHS 200x100x6	2208	26.4	2	116.58
Longitudinal	IPE 550	2535	106	2	537.42
				Σ	8063.31

B. Annex B

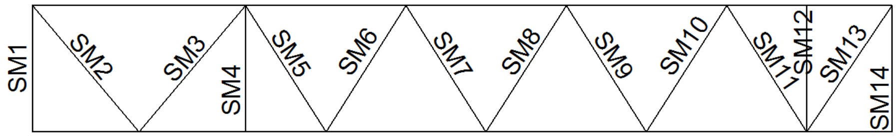
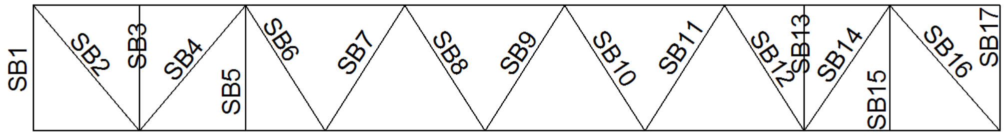
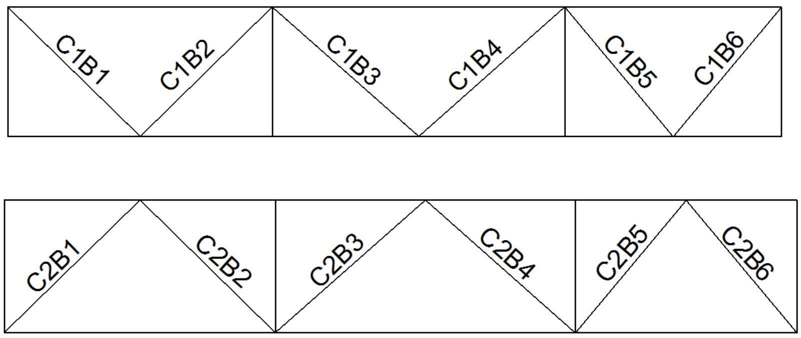
List of tables

Note: Drawings are provided at the end of the report.

Table B.1: Material properties and partial safety factors

Coeff.	Value
f_y [N/mm^2]	235
f_u [N/mm^2]	360
ε	1
γ_{M0}	1
γ_{M1}	1
γ_{M2}	1.25

Table B.2: Designation - brace members

Right/left side member	
Right/left supporting beam	
Connecting beam	

Braces loaded in tension

Table B.3: Tension forces [kN] - right/left side member (braces)

	Right side member				Left side member				
Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
SM2	14.81	64.91	69.13	60.78	14.35	59.52	64.33	54.79	69.13
SM5	45.91	252.85	256.54	249.45	47.05	255.62	259.57	251.97	259.57
SM7	14.22	75.76	80.64	70.97	16.18	83.65	88.84	78.56	88.84
SM10	31.62	168.31	167.02	171.8	29.90	162.32	159.71	165.11	171.80
SM12	6.52	12.39	10.68	14.15	4.85	8.27	6.50	10.08	14.15
SM13	42.41	325.39	322.21	328.84	45.80	335.17	332.00	338.62	338.62

Table B.4: Design checks (tension) - right/left side member (braces)

Mem.	Section	Bolts Φ	No. bolts	A [mm^2]	A_{net} [mm^2]	$N_{pl,Rd}$ [kN]	P_1 [mm]	d_0 [mm]	β	$N_{u,Rd}$ [kN]	$N_{Rd,1}$ [kN]	N_{Rd} [kN]	N_{Ed}/N_{Rd}
MB2	L100x12	12	3	2270	2156.9	533.45	70	13	0.7	559.07	533.45	533.5	0.130
MB5	2L100x12	16	3	2270	2068.9	533.45	70	18	0.611	536.27	533.45	1066.9	0.243
MB7	2L90x10	12	2	1710	1596.9	401.85	70	13	0.7	413.92	401.85	803.7	0.110
MB10	2L90x10	12	2	1710	1596.9	401.85	70	13	0.7	413.92	401.85	803.7	0.214
MB12	2L90x10	12	2	1710	1596.9	401.85	70	13	0.7	413.92	401.85	803.7	0.018
MB13	2L130x12	16	3	3000	2798.9	705.00	70	18	0.611	725.48	705.00	1410.0	0.240

Table B.5: Tension forces [kN] - right/left supporting beam (braces)

	Right supporting beam				Left supporting beam				
Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
SB1	24.21	98.52	100.96	96.22	22.49	93.87	96.47	91.41	100.96
SB5	13.99	66.52	64.29	68.85	14.29	71.04	68.52	73.66	73.66
SB6	55.50	190.61	194.37	187.19	56.31	202.14	206.23	198.40	206.23
SB8	26.41	87.62	90.07	85.32	27.07	94.95	97.54	92.52	97.54
SB10	16.96	75.57	77.99	73.25	17.58	82.71	85.23	80.30	85.23
SB14	64.23	251.80	246.02	258.00	66.54	274.47	267.90	281.45	281.45
SB15	43.86	159.38	157.64	161.40	48.54	179.51	177.58	181.75	181.75

Table B.6: Design checks (tension) - right/left supporting beam (braces)

Mem.	Section	Bolts Φ	No. bolts	A [mm^2]	A_{net} [mm^2]	$N_{pl,Rd}$ [kN]	P_1 [mm]	d_0 [mm]	β	$N_{u,Rd}$ [kN]	$N_{Rd,1}$ [kN]	N_{Rd} [kN]	N_{Ed}/N_{Rd}
SB1	2L90x10	12	2	1710	1596.9	401.85	70	13	0.7	413.92	401.85	803.7	0.126
SB5	2L90x10	12	2	1710	1596.9	401.85	70	13	0.7	413.92	401.85	803.7	0.092
SB6	2L90x10	12	3	1710	1596.9	401.85	70	13	0.7	413.92	401.85	803.7	0.257
SB8	L100x12	16	2	2270	2068.9	533.45	70	18	0.566	536.27	533.45	533.45	0.183
SB10	L100x12	16	2	2270	2068.9	533.45	70	18	0.566	536.27	533.45	533.45	0.160
SB14	2L130x12	12	3	3000	2886.9	705.00	70	13	0.7	748.29	705.00	1410.0	0.200
SB15	2L90x10	12	2	1710	1596.9	401.85	70	13	0.7	413.92	401.85	803.7	0.226

Table B.7: Tension forces [kN] - connecting beam (braces)

Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
C1B1	70.51	360.56	357.09	364.47	364.47
C1B3	11.54	37.94	39.30	36.66	39.30
C1B4	22.25	88.25	89.03	87.62	89.03
C1B6	67.10	355.51	351.50	359.93	359.93
C2B2	78.30	323.05	318.96	327.63	327.63
C2B5	83.92	342.68	338.80	347.10	347.10

Table B.8: Design checks (tension) - connecting beam (braces)

Mem.	Section	Bolts Φ	No. bolts	A [mm^2]	A_{net} [mm^2]	$N_{pl,Rd}$ [kN]	P_1 [mm]	d_0 [mm]	β	$N_{u,Rd}$ [kN]	$N_{Rd,1}$ [kN]	N_{Rd} [kN]	N_{Ed}/N_{Rd}
C1B1	2L100x12	16	4	2270	2068.9	533.45	70	18	0.611	536.27	533.45	1066.9	0.342
C1B3	2L90x10	12	2	1710	1596.9	401.85	70	13	0.7	413.92	401.85	803.7	0.049
C1B4	2L90x10	12	2	1710	1596.9	401.85	70	13	0.7	413.92	401.85	803.7	0.111
C1B6	2L100x12	16	4	2270	2068.9	533.45	70	18	0.611	536.27	533.45	1066.9	0.337
C2B2	2L100x12	16	4	2270	2068.9	533.45	70	18	0.611	536.27	533.45	1066.9	0.307
C2B5	2L100x12	16	4	2270	2068.9	533.45	70	18	0.611	536.27	533.45	1066.9	0.325

Braces loaded in compression

Table B.9: Cross-section classification for the braces loaded in compression

Cross-section	h [mm]	t [mm]	h/t	$\frac{b+h}{2t}$	Class
L 90x10	90	10	9	9	3
L 100x12	100	12	8.33	8.33	3
L 130x12	130	12	10.83	10.83	3
Limiting values (S235): Class 3: h/t<15, (b+h)/(2t)<11.5					

Table B.10: Compression forces [kN] - right/left side member (braces)

	Right side member				Left side member				
Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
SM1	14.06	142.18	145.82	138.65	13.52	137.46	141.56	133.45	145.82
SM3	31.34	201.45	208.03	195.08	31.37	197.88	205.28	190.68	208.03
SM4	24.86	190.58	190.04	191.28	25.83	196.47	195.45	197.65	197.65
SM6	44.73	346.11	351.34	341.16	46.70	353.82	359.47	348.53	359.47
SM8	tension	39.20	44.23	34.09	tension	46.69	52.06	41.24	52.06
SM9	22.78	156.56	154.62	158.63	20.82	148.89	146.60	151.31	158.63
SM11	40.03	356.44	351.41	361.72	39.66	353.12	347.89	358.61	361.72
SM14	29.48	167.21	164.2	170.41	30.50	168.02	165.17	171.07	171.07

Table B.11: Design checks (compression + stability) - right/left side member (braces)

Mem.	Section	A [mm^2]	$N_{pl,Rd}$ [kN]	L_{cr} [mm]	$i_y=i_z$ [mm]	i_v [mm]	$\bar{\lambda}_{eff,y}$	$\bar{\lambda}_{eff,v}$	Φ	χ	$N_{b,Rd,1}$ [kN]	$N_{b,Rd}$ [kN]	$N_{Ed}/N_{b,Rd}$
SM1	2L90x10	1710	401.9	1500	27.2	17.5	0.911	0.989	1.123	0.604	242.7	485.5	0.30
SM3	2L100x12	2270	533.5	1963.8	30.2	19.4	0.985	1.105	1.264	0.532	284.1	568.1	0.37
SM4	2L90x10	1710	401.9	1500	27.2	17.5	0.911	0.989	1.123	0.604	242.7	485.5	0.41
SM6	2L100x12	2270	533.5	1777.7	30.2	19.4	0.939	1.033	1.175	0.576	307.4	614.7	0.59
SM8	2L90x10	1710	401.9	1777.7	27.2	17.5	0.987	1.107	1.267	0.531	213.3	426.7	0.12
SM9	2L90x10	1710	401.9	1777.7	27.2	17.5	0.987	1.107	1.267	0.531	213.3	426.7	0.37
SM11	2L130x12	3000	705.0	1777.7	39.7	25.4	0.834	0.872	0.994	0.679	478.9	957.8	0.38
SM14	2L90x10*	3420	803.7	1500	41.9		0.382		0.604	0.933	750.1	750.1	0.23
$L_{cr}=L_{sys}$ α=0.34 Brace member SM14 is star-battened													

Table B.12: Compression forces [kN] - right/left supporting beam (braces)

	Right supporting beam				Left supporting beam				
Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
SB2	24.78	83.32	85.64	82.17	21.93	74.20	76.40	72.14	85.64
SB3	35.50	132.63	134.63	130.86	34.67	133.30	135.38	131.44	135.38
SB4	89.16	352.50	356.32	349.25	89.95	368.14	371.66	365.20	371.66
SB7	31.49	98.46	101.19	95.94	32.07	105.63	108.44	103.02	108.44
SB9	22.12	82.67	85.11	80.37	22.80	90.14	92.70	87.73	92.70
SB11	14.34	73.59	75.82	71.45	15.16	80.95	83.28	78.71	83.28
SB12	4.99	10.54	7.60	13.52	4.32	8.64	5.38	11.94	13.52
SB13	25.42	124.1	122.99	125.37	25.11	130.20	128.80	131.76	131.76
SB16	152.25	568.15	556.90	580.38	163.03	630.90	618.13	644.70	644.70
SB17	7.97	18.63	19.28	18.03	9.09	22.68	23.44	21.98	23.44

Table B.13: Design checks (compression + stability) - right/left supporting beam (braces)

Mem.	Section	A [mm^2]	$N_{pl,Rd}$ [kN]	L_{cr} [mm]	$i_y=i_z$ [mm]	i_v [mm]	$\bar{\lambda}_{eff,y}$	$\bar{\lambda}_{eff,v}$	Φ	χ	$N_{b,Rd,1}$ [kN]	$N_{b,Rd}$ [kN]	$N_{Ed}/N_{b,Rd}$
SB2	L100x12	2270	533.5	1963.8	30.2	19.4	0.985	1.105	1.264	0.532	284.0	284.0	0.30
SB3	2L90x10	1710	401.9	1500	27.2	17.5	0.911	0.989	1.123	0.604	242.7	485.5	0.28
SB4	2L90x10	1710	401.9	1963.8	27.2	17.5	1.038	1.187	1.372	0.485	195.1	390.2	0.95
SB7	L100x12	2270	533.5	1777.7	30.2	19.4	0.939	1.033	1.175	0.576	307.4	307.4	0.35
SB9	L100x12	2270	533.5	1777.7	30.2	19.4	0.939	1.033	1.175	0.576	307.4	307.4	0.30
SB11	L100x12	2270	533.5	1777.7	30.2	19.4	0.939	1.033	1.175	0.576	307.4	307.4	0.27
SB12	L100x12	2270	533.5	1777.7	30.2	19.4	0.939	1.033	1.175	0.576	307.4	307.4	0.04
SB13	2L90x10*	3420	803.7	1500	41.9		0.382		0.604	0.933	750.1	750.1	0.18
SB16	2L130x12	3000	705.0	1996.1	39.7	25.4	0.875	0.936	1.063	0.638	449.9	899.7	0.72
SB17	2L90x10	1710	401.9	1500	27.2	17.5	0.911	0.989	1.123	0.604	242.7	485.5	0.05
$L_{cr}=L_{sys}$; α=0.34 Brace member SB13 is star-battened													

Table B.14: Compression forces [kN] - connecting beam (braces)

Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
C1B2	61.64	341.81	336.87	347.15	347.15
C1B5	62.92	354.09	348.84	359.74	359.74
C2B1	97.11	384.33	381.57	387.71	387.71
C2B3	8.61	46.04	47.13	45.01	47.13
C2B4	23.07	99.87	100.59	99.29	100.59
C2B6	99.30	397.39	394.62	400.79	400.79

Table B.15: Design checks (compression + stability) - connecting beam (braces)

Mem.	Section	A [mm^2]	$N_{pl,Rd}$ [kN]	L_{cr} [mm]	$i_y=i_z$ [mm]	i_v [mm]	$\bar{\lambda}_{eff,y}$	$\bar{\lambda}_{eff,v}$	Φ	χ	$N_{b,Rd,1}$ [kN]	$N_{b,Rd}$ [kN]	$N_{Ed}/N_{b,Rd}$
C1B2	2L100x12	2270	533.5	2149.8	30.2	19.4	1.031	1.176	1.358	0.491	262.1	524.1	0.66
C1B5	2L100x12	2270	533.5	1959	30.2	19.4	0.984	1.103	1.262	0.534	284.6	569.3	0.63
C2B1	2L100x12	2270	533.5	2149.8	30.2	19.4	1.031	1.176	1.358	0.491	262.1	524.1	0.74
C2B3	2L90x10	1710	401.9	2267.2	27.2	17.5	1.121	1.316	1.555	0.419	168.5	337.0	0.14
C2B4	2L90x10	1710	401.9	2267.2	27.2	17.5	1.121	1.316	1.555	0.419	168.5	337.0	0.30
C2B6	2L100x12	2270	533.5	1959	30.2	19.4	0.984	1.103	1.262	0.534	284.6	569.3	0.70
$L_{cr}=L_{sys}$; α=0.34													

Bottom chords

Table B.16: Cross-section classification (bottom chords)

Cross-section	c [mm]	t [mm]	c/t	Class
SHS 350x350x12.5	312.5	12.5	25	1

c≈b-3t
Bottom chord is classified for pure bending (safe-side assumption)
Limiting values for the compression flange(S235):
Class 1: c/t=33
Class 2: c/t=38
Class 3: c/t=42

Table B.17: Chords - design resistances

SHS 350x350x12.5	
N_{Rd} [kN]	3807
M_{Rd} [kNm]	474.7
V_{Rd} [kN]	1187.18

Table B.18: Internal forces and design checks (bending and axial forces) - right/left side member (bottom ch.)

		max. N (tension), corr. M					max. M, corr. N (tension)				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
Right side member	$X_{1,ULS}$	78.92	2.75	0.02	0.01	0.03	29.58	10.36	0.01	0.02	0.03
	$X_{2,ULS}$	533.37	31.54	0.14	0.07	0.21	172.94	68.42	0.05	0.14	0.19
	$X_{3,ULS}$	562.81	31.72	0.15	0.07	0.21	175.32	70.22	0.05	0.15	0.19
	$X_{4,ULS}$	544.76	31.45	0.14	0.07	0.21	170.73	66.70	0.04	0.14	0.19
Left side member	$X_{1,ULS}$	82.10	2.76	0.02	0.01	0.03	29.45	11.01	0.01	0.02	0.03
	$X_{2,ULS}$	559.97	31.48	0.15	0.07	0.21	166.69	72.32	0.04	0.15	0.20
	$X_{3,ULS}$	567.36	31.65	0.15	0.07	0.22	168.49	74.32	0.04	0.16	0.20
	$X_{4,ULS}$	553.12	31.33	0.15	0.07	0.21	165.14	70.40	0.04	0.15	0.19

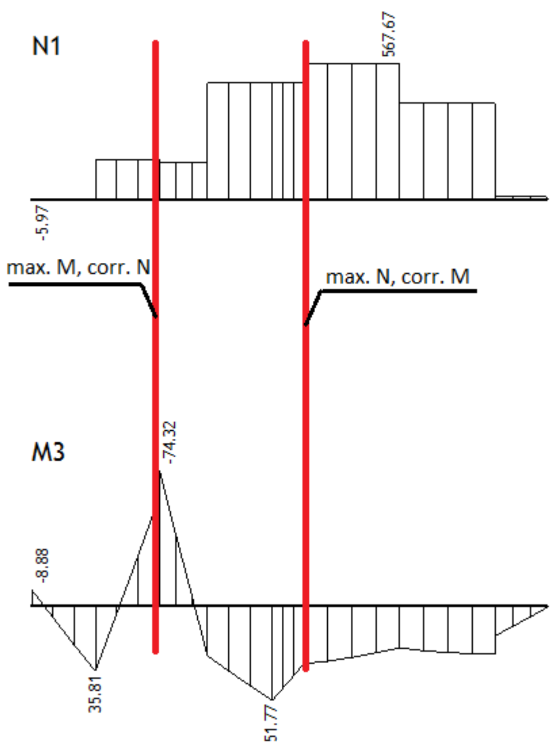


Diagram is given for $X_{3,ULS}$, left side member

Table B.19: Internal forces and design checks (shear) - right/left side member (bottom chord)

Member	Shear forces [kN]					
	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
Right side member	20.99	101.64	103.21	100.15	103.21	0.09
Left side member	20.99	105.77	107.49	104.15	107.49	0.09

Table B.20: Internal forces and design checks (bending and axial forces) - right/left suppor. beam (bottom ch.)

		max. N (tension), corr. M					max. M, corr. N (tension)				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
Right supporting beam	$X_{1,ULS}$	134.53	3.41	0.04	0.01	0.04	45.20	30.16	0.01	0.06	0.08
	$X_{2,ULS}$	516.23	22.08	0.14	0.05	0.18	188.49	116.89	0.05	0.25	0.30
	$X_{3,ULS}$	537.15	22.58	0.14	0.05	0.19	194.18	118.25	0.05	0.25	0.30
	$X_{4,ULS}$	496.09	21.62	0.13	0.05	0.18	183.07	115.73	0.05	0.24	0.29
Left supporting beam	$X_{1,ULS}$	139.77	3.09	0.04	0.01	0.04	47.61	29.30	0.01	0.06	0.07
	$X_{2,ULS}$	561.77	23.82	0.15	0.05	0.20	205.62	117.18	0.05	0.25	0.30
	$X_{3,ULS}$	586.06	24.33	0.15	0.05	0.21	212.32	118.61	0.06	0.25	0.31
	$X_{4,ULS}$	538.51	23.33	0.14	0.05	0.19	199.23	115.94	0.05	0.24	0.30

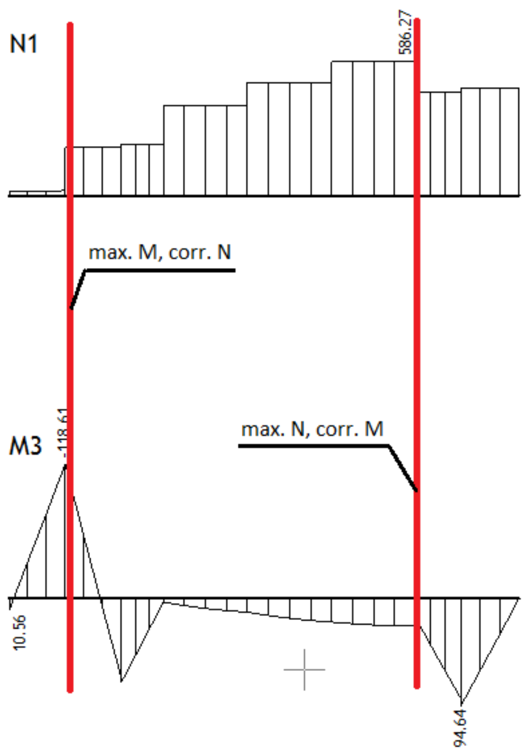


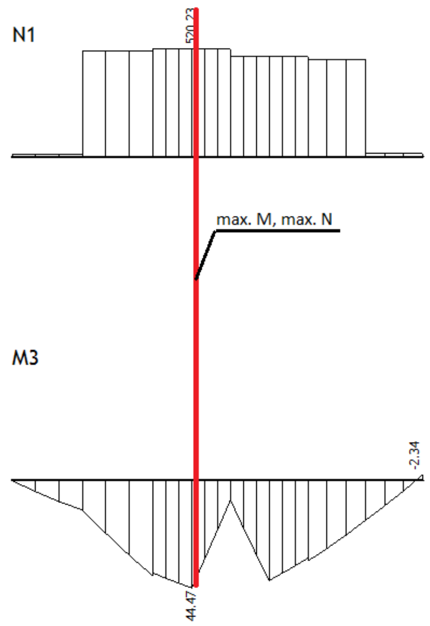
Diagram is given for $X_{3,ULS}$, left supporting beam

Table B.21: Internal forces and design checks (shear) - right/left supporting beam (bottom chord)

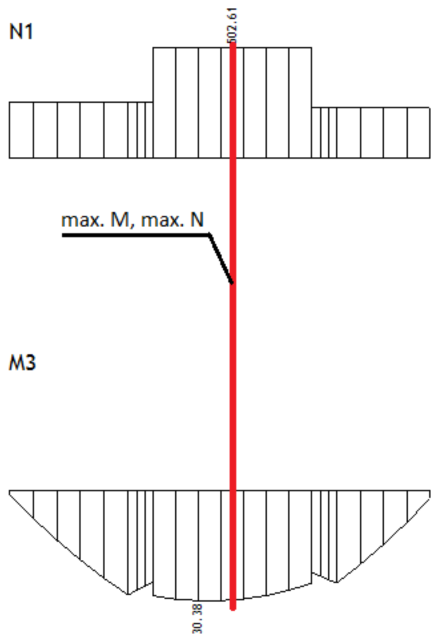
Member	Shear forces [kN]					
	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
Right supporting beam	37.14	148.66	148.66	148.90	148.90	0.13
Left supporting beam	36.37	150.36	150.02	150.92	150.92	0.13

Table B.22: Internal forces and design checks (bending and axial forces) - connecting beam (bottom chord)

		max. N (tension), max. M				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
C1	$X_{1,ULS}$	102.37	10.39	0.03	0.02	0.05
	$X_{2,ULS}$	495.26	44.86	0.13	0.09	0.22
	$X_{3,ULS}$	520.23	44.47	0.14	0.09	0.23
	$X_{4,ULS}$	458.23	45.32	0.12	0.10	0.22
C2	$X_{1,ULS}$	122.71	6.92	0.03	0.01	0.05
	$X_{2,ULS}$	546.63	30.69	0.14	0.06	0.21
	$X_{3,ULS}$	502.61	30.38	0.13	0.06	0.20
	$X_{4,ULS}$	591.43	31.04	0.16	0.07	0.22



C1, $X_{3,ULS}$



C2, $X_{3,ULS}$

Table B.23: Internal forces and design checks (shear) - connecting beam (bottom chord)

Member	Shear forces [kN]					
	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
C1	11.11	43.07	43.80	42.42	43.80	0.04
C2	13.34	13.60	13.62	13.62	13.62	0.01

Upper chords

Table B.24: Cross-section classification (upper chords)

Cross-section	c [mm]	t [mm]	c/t	Class
SHS 350x350x12.5	312.5	12.5	25	1
c≈b-3t Bottom chord is classified for pure compression (safe-side assumption) Limiting values in compression(S235): Class 1: c/t=33 Class 2: c/t=38 Class 3: c/t=42				

Table B.25: Chords - design resistances

SHS 350x350x12.5	
N_{Rd} [kN]	3807
M_{Rd} [kNm]	474.7
V_{Rd} [kN]	1187.18

Table B.26: Internal forces and design checks (bending and axial forces) - right/left side member (upper ch.)

		max. N (compression), max. M				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
Right side member	$X_{1,ULS}$	86.06	11.47	0.02	0.02	0.04
	$X_{2,ULS}$	534.01	26.93	0.14	0.06	0.20
	$X_{3,ULS}$	535.76	28.66	0.14	0.06	0.20
	$X_{4,ULS}$	547.34	25.26	0.14	0.05	0.19
Left side member	$X_{1,ULS}$	88.50	11.45	0.02	0.02	0.04
	$X_{2,ULS}$	538.10	26.93	0.14	0.06	0.20
	$X_{3,ULS}$	541.29	28.64	0.14	0.06	0.20
	$X_{4,ULS}$	550.02	25.27	0.14	0.05	0.19

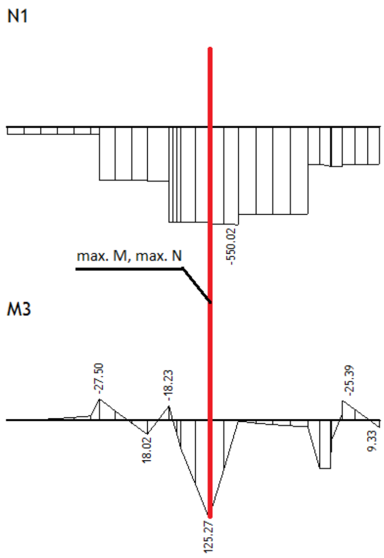


Diagram is given for $X_{4,ULS}$, left side member

Table B.27: Internal forces and design checks (shear) - right/left side member (upper chord)

Member	Shear forces [kN]					
	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
Right side member	25.74	166.76	168.76	164.90	168.76	0.14
Left side member	25.82	167.09	169.11	165.21	169.11	0.14

Table B.28: Stability checks - right/left side member (upper chord)

Buckling plane	Section	L_{sys} [mm]	L_{cr} [mm]	i [mm]	$\bar{\lambda}$	Φ	χ	$N_{b,Rd}$ [kN]	$N_{Ed,max}$ [kN]	$N_{Ed}/N_{b,Rd}$
In-plane	350x350x12.5	1908.0	1717.2	136	0.134		1	3807.0	550.02	0.14
Out-of-plane	350x350x12.5	6678.0	6010.2	136	0.471	0.677	0.859	3270.2	550.02	0.17

$L_{cr}=0.9L_{sys}$
$\alpha=0.49$
For $\bar{\lambda} \leq 0.2$ only cross-sectional checks apply.
For out-of-plane buckling is assumed that the axial force is constant between the lateral supports with its maximum value.
Interaction (M+N) is not relevant since there is no in-plane buckling (see 4.1.3.2. for the explanations)

Table B.29: Internal forces and design checks (bending and axial forces) - right/left suppor. beam (upper ch.)

		max. N (compression), max. M				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
Right supporting beam	$X_{1,ULS}$	133.43	18.63	0.04	0.04	0.08
	$X_{2,ULS}$	509.15	81.45	0.13	0.17	0.30
	$X_{3,ULS}$	517.39	81.34	0.14	0.17	0.31
	$X_{4,ULS}$	511.54	81.66	0.13	0.17	0.30
Left supporting beam	$X_{1,ULS}$	140.08	19.39	0.04	0.04	0.08
	$X_{2,ULS}$	557.59	87.65	0.15	0.18	0.33
	$X_{3,ULS}$	562.55	87.42	0.15	0.18	0.33
	$X_{4,ULS}$	560.88	87.98	0.15	0.19	0.34

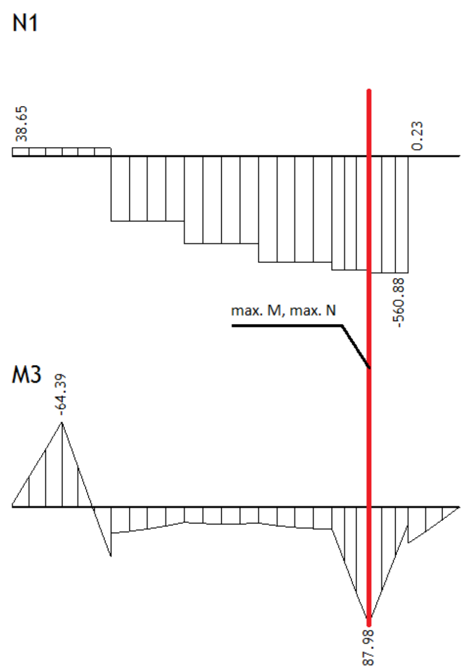


Diagram is given for $X_{4,ULS}$, left supporting beam

Table B.30: Internal forces and design checks (shear) - right/left supporting beam (upper chord)

Member	Shear forces [kN]					
	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
Right supporting beam	20.14	80.05	80.32	79.90	80.32	0.07
Left supporting beam	19.88	81.42	81.63	81.33	81.63	0.07

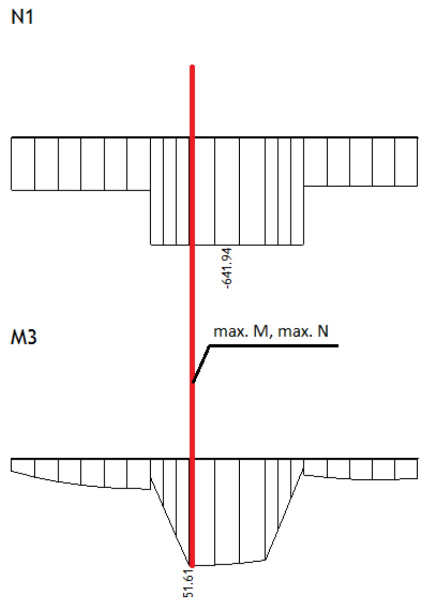
Table B.31: Stability checks - right/left supporting beam (upper chord)

Buckling plane	Section	L_{sys} [mm]	L_{cr} [mm]	i [mm]	$\bar{\lambda}$	Φ	χ	$N_{b,Rd}$ [kN]	$N_{Ed,max}$ [kN]	$N_{Ed}/N_{b,Rd}$
In-plane	350x350x12.5	1908.0	1717.2	136	0.134		1	3807.0	562.55	0.15
Out-of-plane	350x350x12.5	6678.0	6010.2	136	0.471	0.677	0.859	3270.2	562.55	0.17

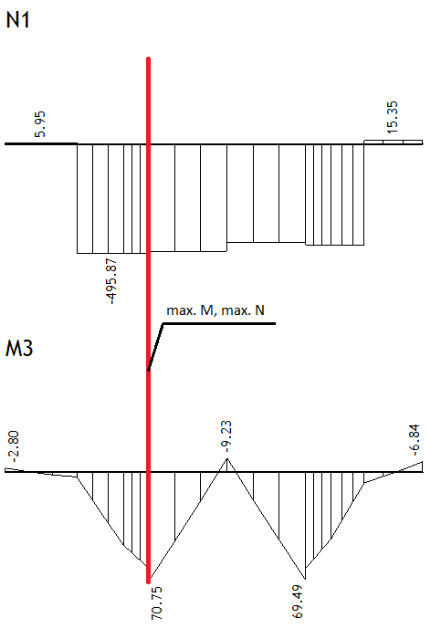
$L_{cr}=0.9L_{sys}$
$\alpha=0.49$
For $\bar{\lambda} \leq 0.2$ only cross-sectional checks apply.
For out-of-plane buckling is assumed that the axial force is constant between the lateral supports with its maximum value.
Interaction (M+N) is not relevant since there is no in-plane buckling (see 4.1.3.2. for the explanations)

Table B.32: Internal forces and design checks (bending and axial forces) - connecting beam (upper chord)

		max. N (compression), max. M				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
C1	$X_{1,ULS}$	124.28	12.35	0.03	0.03	0.06
	$X_{2,ULS}$	598.17	51.18	0.16	0.11	0.27
	$X_{3,ULS}$	555.20	50.83	0.15	0.11	0.26
	$X_{4,ULS}$	641.94	51.61	0.17	0.11	0.28
C2	$X_{1,ULS}$	115.69	13.83	0.03	0.03	0.06
	$X_{2,ULS}$	479.29	70.63	0.13	0.15	0.28
	$X_{3,ULS}$	495.87	70.75	0.13	0.15	0.28
	$X_{4,ULS}$	463.44	70.59	0.12	0.15	0.27



C1, $X_{4,ULS}$



C2, $X_{3,ULS}$

Table B.33: Internal forces and design checks (shear) - connecting beam (upper chord)

Member	Shear forces [kN]					
	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$V_{Ed,max}$	V_{Ed}/V_{Rd}
C1	18.16	53.43	53.49	53.49	53.49	0.05
C2	14.60	50.78	51.19	50.46	51.19	0.04

Table B.34: Stability checks - connecting beam (upper chord)

Buckling plane	Section	L_{sys} [mm]	L_{cr} [mm]	i [mm]	$\bar{\lambda}$	Φ	χ	$N_{b,Rd}$ [kN]	$N_{Ed,max}$ [kN]	$N_{Ed}/N_{b,Rd}$
In-plane	350x350x12.5	3400.0	3060.0	136	0.24	0.539	0.979	3726.4	641.94	0.17
Out-of-plane	350x350x12.5	3400.0	3060.0	136	0.24	0.539	0.979	3726.4	641.94	0.17

$L_{cr}=0.9L_{sys}$
$\alpha=0.49$
Interaction check (member C1, combination $X_{4,ULS}$ as the most unfavorable case)
$C_{my}=1$ (bending moment diagram almost rectangular)
$$k_{yy}=1\left(1+(0.24-0.2)\frac{641.94}{3807}\right)=1.007$$
$k_{zy}=0$
$0.17+1.007\cdot0.11=0.281<1$ (EN1993-1-1, equation 6.61)
$0.17<1$ (EN1993-1-1, equation 6.62, with $k_{zy}=0$)

Bracket - diagonal

The diagonal on the right side is relevant for the design because it is longer and has higher loading.

Table B.35: Cross-section classification (bracket-diagonal)

Cross-sectional part	c [mm]	t [mm]	c/t	Class
HEB 360, flange	116.75	22.5	5.19	1
HEB 360, web	261	12.5	20.88	1

Cross-section is classified for pure compression
Limiting values (S235):
Class 1: flange c/t=9, web c/t=33
Class 2: flange c/t=10, web c/t=38
Class 3: flange c/t=14, web c/t=42

Table B.36: Compression forces [kN] - bracket (diagonal)

Member	$X_{1,ULS}$	$X_{2,ULS}$	$X_{3,ULS}$	$X_{4,ULS}$	$N_{Ed,max}$
Bracket-diagonal	120.51	720.04	721.57	719.28	721.57

Bending moments (resulting from the self-weight) are negligible and will not be considered further.

Table B.37: Bracket (diagonal) - design resistances

HEB 360	
N_{Rd} [kN]	4244.1
M_{Rd} [kNm]	631.21

Table B.38: Design checks (compression + stability) - bracket (diagonal)

Buckling plane	Section	L_{sys} [mm]	L_{cr} [mm]	i [mm]	$\bar{\lambda}$	Φ	χ	$N_{b,Rd}$ [kN]	$N_{Ed,max}$ [kN]	N_{Ed}/ $N_{b,Rd}$
In-plane (y-y)	HEB 360	3518	3518	154.6	0.242	0.536	0.985	4182.2	721.57	0.17
Out-of-plane (z-z)	HEB 360	3518	3518	74.9	0.5	0.699	0.843	3576.2	721.57	0.20
L_{cr}=L_{sys} α=0.34 (in-plane buckling) α=0.49 (out-of-plane buckling)										

Bracket - vertical

Table B.39: Cross-section classification (bracket-vertical)

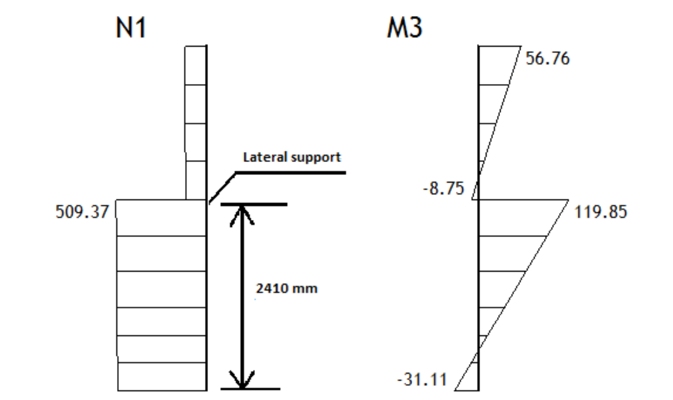
Cross-sectional part	c [mm]	t [mm]	c/t	Class
HEB 360, flange	116.75	22.5	5.19	1
HEB 360, web	261	12.5	20.88	1
Cross-section is classified for pure bending (safe-side assumption) Limiting values (S235): Class 1: flange c/t=9, web c/t=72 Class 2: flange c/t=10, web c/t=83 Class 3: flange c/t=14, web c/t=124				

Table B.40: Bracket (vertical) - design resistances

HEB 360	
N_{Rd} [kN]	4244.1
M_{Rd} [kNm]	631.21

Table B.41: Internal forces and design checks (bending and axial forces) - bracket (vertical)

		max. N (tension), max. M				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/ N_{Rd}	M_{Ed}/ M_{Rd}	N_{Ed}/N_{Rd}+ M_{Ed}/M_{Rd}
Right side vertical	$X_{1,ULS}$	92.67	21.55	0.02	0.03	0.05
	$X_{2,ULS}$	507.16	119.37	0.12	0.19	0.31
	$X_{3,ULS}$	509.37	119.85	0.12	0.19	0.31
	$X_{4,ULS}$	505.54	119.02	0.12	0.19	0.31
Left side vertical	$X_{1,ULS}$	94.33	18.61	0.02	0.03	0.05
	$X_{2,ULS}$	524.55	105.28	0.12	0.17	0.29
	$X_{3,ULS}$	526.70	105.84	0.12	0.17	0.29
	$X_{4,ULS}$	523.00	104.85	0.12	0.17	0.29



Right side vertical, $X_{3,ULS}$

Table B.42: Stability checks - bracket (vertical)

Combination $X_{3,ULS}$ is the most unfavorable case in bending M_{cr}=13819.31 kNm $\bar{\lambda}_{LT}=\sqrt{\dfrac{631.21}{13819.31}}=0.214$ For $\bar{\lambda}_{LT} \leq 0.4$ lateral-torsional buckling check is not necessary

Bracket - horizontal

Table B.43: Cross-section classification (bracket-horizontal)

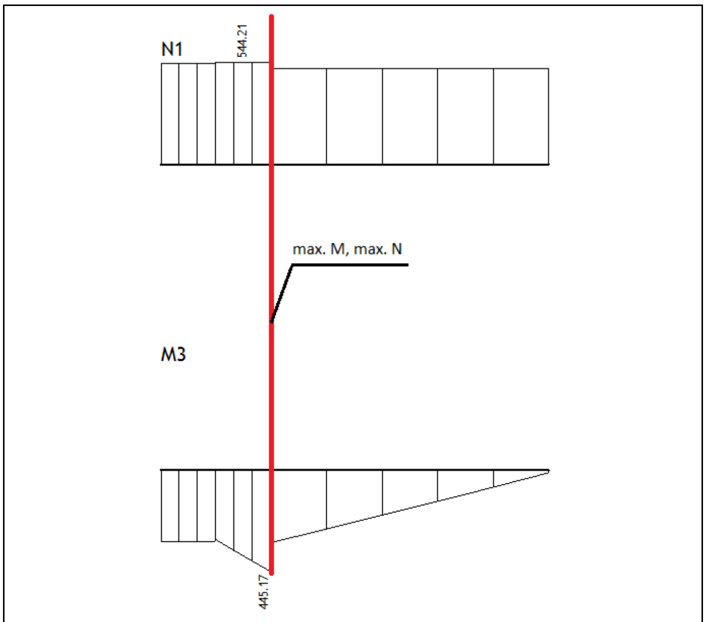
Cross-sectional part	c [mm]	t [mm]	c/t	Class
HEA 800, flange	112.5	28	4.02	1
HEA 800, web	337	15	22.47	1
Cross-section is classified for bending (safe-side assumption) Limiting values (S235): Class 1: flange c/t=9, web c/t=72 Class 2: flange c/t=10, web c/t=83 Class 3: flange c/t=14, web c/t=124				

Table B.44: Bracket (horizontal) - design resistances

HEA 800	
N_{Rd} [kN]	6716.3
M_{Rd} [kNm]	2044.27

Table B.45: Internal forces and design checks (bending and axial forces) - bracket (horizontal)

		max. N (tension), max. M				
	Comb.	N_{Ed} [kN]	M_{Ed} [kNm]	N_{Ed}/N_{Rd}	M_{Ed}/M_{Rd}	$N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$
Right side horizontal	$X_{1,ULS}$	89.87	80.32	0.01	0.04	0.05
	$X_{2,ULS}$	540.38	440.65	0.08	0.22	0.30
	$X_{3,ULS}$	544.21	445.17	0.08	0.22	0.30
	$X_{4,ULS}$	537.13	436.64	0.08	0.21	0.29
Left side horizontal	$X_{1,ULS}$	72.58	70.34	0.01	0.03	0.04
	$X_{2,ULS}$	440.80	389.52	0.07	0.19	0.26
	$X_{3,ULS}$	444.88	394.58	0.07	0.19	0.26
	$X_{4,ULS}$	437.18	384.94	0.07	0.19	0.26



Right side horizontal, $X_{3,ULS}$

Table B.46: Stability checks - bracket (horizontal)

Combination $X_{3,ULS}$ is the most unfavorable case in bending M_{cr}=27638.3 kNm $\bar{\lambda}_{LT}=\sqrt{\frac{2044.27}{27638.3}}=0.272$ For $\bar{\lambda}_{LT} \leq 0.4$ lateral-torsional buckling check is not necessary

Design of joints

Table B.47: Designation - joints

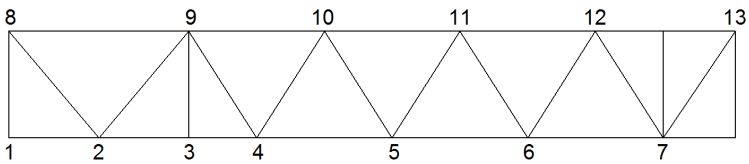
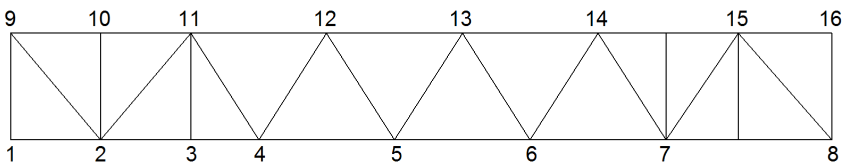
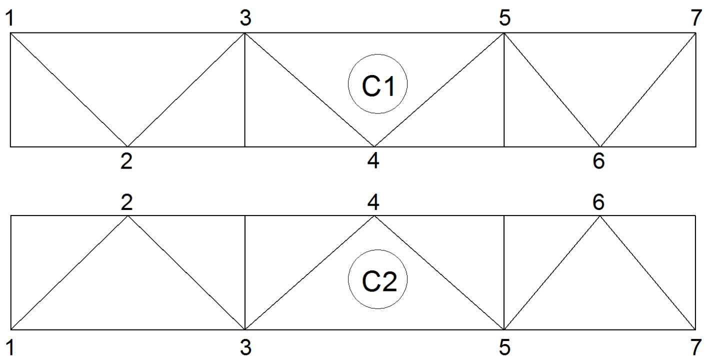
Right/left side member	
Right/left supporting beam	
Connecting beam	

Table B.48: Design checks - Joint 1 (right/left side member)

Joint 1 (right/left side member)		Geometry: T
Brace 1	Chord	Gusset plate design checks Length: h=200 mm Supported compression brace $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{b,Rd}$= $N_{pl,Rd}$ >$N_{1,Ed}$ Stresses (critical section) σ_1=-60.75 MPa<235 MPa σ_2=-60.75 MPa<235 MPa τ=0 MPa
SM1	SHS 350x350x12.5	
2L90x10	$N_{0,Ed}$=0 kN	
θ=90°	$M_{0,Ed}$=0 kNm	
2M12...8.8		
$N_{1,Ed}$=-145.81 kN		
$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.81 kN $V_{Rd,min}$=181.81 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.80<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-145.81 kN $M_{1-1,Ed}$=0 kNm N_{Rd}=186.30 kN $\frac{N_{1-1,Ed}}{N_{Rd}}$=0.78<1	

Table B.49: Design checks - Joint 2 (right/left side member)

Joint 2 (right/left side member)			Geometry: K
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=740 mm Supported tension brace $N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$ $N_{u,Rd}$=462.39 kN>$N_{1,Ed}$ Supported compression brace $N_{pl,Rd}$=455.88 kN>$N_{2,Ed}$ $N_{b,Rd}$=447.72 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=-40.53 MPa<235 MPa σ_2=16.64 MPa<235 MPa τ=30.22 MPa<135.68 MPa
SM2	SM3	SHS 350x350x12.5	
L100x12	2L100x12	$N_{0,Ed}$=175.14 kN	
θ=49.80°	θ=49.80°	$M_{0,Ed}$=36.30 kNm	
3M12...8.8	3M12...8.8		
$N_{1,Ed}$=69.13 kN	$N_{2,Ed}$=-208.03 kN		
$V_{v,Rd}$=118.81 kN $V_{b,Rd}$=283.64 kN $V_{gus,b,Rd}$ =283.64 kN $V_{eff,Rd}$=331.60 kN $V_{Rd,min}$=118.81 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.58<1	$V_{v,Rd}$=237.63 kN $V_{b,Rd}$=567.29 kN $V_{gus,b,Rd}$ =283.64 kN $V_{Rd,min}$=283.64 kN $\frac{N_{2,Ed}}{V_{Rd,min}}$=0.73<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-106.09 kN $M_{1-1,Ed}$=31.31 kNm N_{Rd}=299.60 kN $M_{ip,Rd}$=110.85 kNm Interaction=0.64<1	

Table B.50: Design checks - Joint 3 (right/left side member)

Joint 3 (right/left side member)		Geometry: T
Brace 1	Chord	Gusset plate design checks Length: h=270 mm Supported compression brace $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{b,Rd}$= $N_{pl,Rd}$ >$N_{1,Ed}$ Stresses (critical section) σ_1=-61.00MPa<235 MPa σ_2=-61.00 MPa<235 MPa τ=0 MPa
SM4	SHS 350x350x12.5	
2L90x10	$N_{0,Ed}$=169.4 kN	
θ=90°	$M_{0,Ed}$=-70.4 kNm	
2M16...8.8		
$N_{1,Ed}$=-197.65 kN		
$V_{v,Rd}$=270.77 kN $V_{b,Rd}$=364.32 kN $V_{gus,b,Rd}$ =228.24 kN $V_{Rd,min}$=228.24 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.87<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-197.65 kN $M_{1-1,Ed}$=0 kNm N_{Rd}=200.99 kN $\frac{N_{1-1,Ed}}{N_{Rd}}$=0.98<1	

Table B.51: Design checks - Joint 4 (right/left side member)

<table>
<tr><td colspan="3">Joint 4 (right/left side member)</td><td>Geometry: K</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="7">Gusset plate design checks
Length: h=620 mm
Supported tension brace
$N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$
$N_{u,Rd}$=446.84 kN>$N_{1,Ed}$
Supported compression brace
$N_{pl,Rd}$=455.88 kN>$N_{2,Ed}$
$N_{b,Rd}$=453.05 kN>$N_{2,Ed}$
Stresses (critical section)
σ_1=-86.94 MPa<235 MPa
σ_2=64.29 MPa<235 MPa
τ=66.99MPa<135.68 MPa</td></tr>
<tr><td>SM5</td><td>SM6</td><td>SHS 350x350x12.5</td></tr>
<tr><td>2L100x12</td><td>2L100x12</td><td>$N_{0,Ed}$=489.05 kN</td></tr>
<tr><td>θ=57.54°</td><td>θ=57.54°</td><td>$M_{0,Ed}$=27.02 kNm</td></tr>
<tr><td>3M16...8.8</td><td>3M16...8.8</td><td></td></tr>
<tr><td>$N_{1,Ed}$=259.57 kN</td><td>$N_{2,Ed}$=-359.47 kN</td><td></td></tr>
<tr><td>$V_{v,Rd}$=422.44 kN
$V_{b,Rd}$=700.36 kN
$V_{gus,b,Rd}$ =378.19 kN
$V_{eff,Rd}$=613.85 kN
$V_{Rd,min}$=378.19 kN
$\frac{N_{1,Ed}}{V_{Rd,min}}$=0.69<1</td><td>$V_{v,Rd}$=422.44 kN
$V_{b,Rd}$=700.36 kN
$V_{gus,b,Rd}$ =378.19 kN
$V_{Rd,min}$=378.19 kN
$\frac{N_{2,Ed}}{V_{Rd,min}}$=0.95<1</td><td>Gusset to chord
(chord face failure)
$N_{1-1,Ed}$=-84.29 kN
$M_{1-1,Ed}$=58.13 kNm
N_{Rd}=274.42 kN
$M_{ip,Rd}$=85.07 kNm
Interaction=0.99<1</td></tr>
</table>

Table B.52: Design checks - Joint 5 (right/left side member)

<table>
<tr><td colspan="3">Joint 5 (right/left side member)</td><td>Geometry: K</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="7">Gusset plate design checks
Length: h=520 mm
Supported tension brace
$N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$
$N_{u,Rd}$=210.98 kN>$N_{1,Ed}$
Supported compression brace
$N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$
$N_{b,Rd}$=227.24 kN>$N_{2,Ed}$
Stresses (critical section)
σ_1=-19.49MPa<235 MPa
σ_2=29.44 MPa<235 MPa
τ=18.18MPa<135.68 MPa</td></tr>
<tr><td>SM7</td><td>SM8</td><td>SHS 350x350x12.5</td></tr>
<tr><td>2L90x10</td><td>2L90x10</td><td>$N_{0,Ed}$=567.34 kN</td></tr>
<tr><td>θ=57.54°</td><td>θ=57.54°</td><td>$M_{0,Ed}$=31.68 kNm</td></tr>
<tr><td>2M12...8.8</td><td>2M12...8.8</td><td></td></tr>
<tr><td>$N_{1,Ed}$=88.84 kN</td><td>$N_{2,Ed}$=-52.06 kN</td><td></td></tr>
<tr><td>$V_{v,Rd}$=152.31 kN
$V_{b,Rd}$=303.00 kN
$V_{gus,b,Rd}$ =181.80 kN
$V_{eff,Rd}$=383.59 kN
$V_{Rd,min}$=152.31 kN
$\frac{N_{1,Ed}}{V_{Rd,min}}$=0.58<1</td><td>$V_{v,Rd}$=152.31 kN
$V_{b,Rd}$=303.00 kN
$V_{gus,b,Rd}$ =181.80 kN
$V_{Rd,min}$=152.31 kN
$\frac{N_{2,Ed}}{V_{Rd,min}}$=0.34<1</td><td>Gusset to chord
(chord face failure)
$N_{1-1,Ed}$=31.03 kN
$M_{1-1,Ed}$=13.23 kNm
N_{Rd}=253.44 kN
$M_{ip,Rd}$=65.90 kNm
Interaction=0.32<1</td></tr>
</table>

Table B.53: Design checks - Joint 6 (right/left side member)

<table>
<tr><td colspan="3">Joint 6 (right/left side member)</td><td>Geometry: K</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="7">Gusset plate design checks
Length: h=520 mm
Supported tension brace
$N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$
$N_{u,Rd}$=195.43 kN>$N_{2,Ed}$
Supported compression brace
$N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$
$N_{b,Rd}$=227.24 kN>$N_{1,Ed}$
Stresses (critical section)
σ_1=59.16 MPa<235 MPa
σ_2=-55.60 MPa<235 MPa
τ=42.63MPa<135.68 MPa</td></tr>
<tr><td>SM9</td><td>SM10</td><td>SHS 350x350x12.5</td></tr>
<tr><td>2L90x10</td><td>2L90x10</td><td>$N_{0,Ed}$=544.45 kN</td></tr>
<tr><td>θ=57.54°</td><td>θ=57.54°</td><td>$M_{0,Ed}$=22.22 kNm</td></tr>
<tr><td>2M16...8.8</td><td>2M16...8.8</td><td></td></tr>
<tr><td>$N_{1,Ed}$=-158.63 kN</td><td>$N_{2,Ed}$=171.80 kN</td><td></td></tr>
<tr><td>$V_{v,Rd}$=270.77 kN
$V_{b,Rd}$=364.32 kN
$V_{gus,b,Rd}$ =228.24 kN
$V_{Rd,min}$=228.24 kN
$\frac{N_{1,Ed}}{V_{Rd,min}}$=0.70<1</td><td>$V_{v,Rd}$=270.77 kN
$V_{b,Rd}$=364.32 kN
$V_{gus,b,Rd}$ =228.24 kN
$V_{eff,Rd}$=356.04 kN
$V_{Rd,min}$=228.24 kN
$\frac{N_{2,Ed}}{V_{Rd,min}}$=0.75<1</td><td>Gusset to chord
(chord face failure)
$N_{1-1,Ed}$=11.11 kN
$M_{1-1,Ed}$=31.03 kNm
N_{Rd}=253.44 kN
$M_{ip,Rd}$=65.90 kNm
Interaction=0.51<1</td></tr>
</table>

Table B.54: Design checks - Joint 7 (right/left side member)

Joint 7 (right/left side member)		Geometry: KT	
Brace 1	Brace 2	Brace 3	Gusset plate design checks Length: h=770 mm Supported tension brace $N_{pl,Rd}$=455.88 kN>$N_{3,Ed}$ $N_{u,Rd}$=446.84 kN>$N_{3,Ed}$ Supported compression brace $N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$ $N_{b,Rd}$=446.88 kN>$N_{1,Ed}$ Stresses (critical section) σ_1=55.08 MPa<235 MPa σ_2=-58.02 MPa<235 MPa τ=60.17MPa<135.68 MPa
SM11	SM12	SM13	
2L130x12	2L90x10	2L130x12	
θ=57.54°	θ=90°	θ=55.78°	
3M16...8.8	2x2M12...8.8	3M16...8.8	
$N_{1,Ed}$=-361.72 kN	$N_{2,Ed}$=14.15 kN	$N_{3,Ed}$=328.84 kN	
$V_{v,Rd}$=413.39 kN $V_{b,Rd}$=695.52 kN $V_{gus,b,Rd}$ =370.09 kN $V_{Rd,min}$=370.09 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.98<1$	$V_{v,Rd}$=304.62 kN $V_{b,Rd}$=606.00 kN $V_{gus,b,Rd}$ =363.6 kN $V_{eff,Rd}$=767.18 kN $V_{Rd,min}$=304.62 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.05<1$	$V_{v,Rd}$=413.39 kN $V_{b,Rd}$=695.52 kN $V_{gus,b,Rd}$ =370.09 kN $V_{eff,Rd}$=682.97 kN $V_{Rd,min}$=370.09 kN $\frac{N_{3,Ed}}{V_{Rd,min}}=0.89<1$	
Chord SHS 350x350x12.5 $N_{0,Ed}$=366.79 kN $M_{0,Ed}$=25.06 kNm	Gusset to chord (chord face failure) $N_{1\text{-}1,Ed}$=-13.59 kN $M_{1\text{-}1,Ed}$=67.06 kNm N_{Rd}=305.90 kN $M_{ip,Rd}$=117.77 kNm Interaction=0.61<1		

Table B.55: Design checks - Joint 8 (right/left side member)

Joint 8 (right/left side member)		Geometry: N	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=610 mm Supported tension brace $N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$ $N_{u,Rd}$=210.98 kN>$N_{2,Ed}$ Supported compression brace $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{b,Rd}$= $N_{pl,Rd}$>$N_{1,Ed}$ Stresses (critical section) σ_1=9.64 MPa<235 MPa σ_2=-34.84 MPa<235 MPa τ=8.04 MPa<135.68 MPa
SM1	SM2	SHS 350x350x12.5	
2L90x10	L100x12	$N_{0,Ed}$=-40.67 kN	
θ=90°	θ=49.80°	$M_{0,Ed}$=0 kNm	
2M12...8.8	3M12...8.8		
$N_{1,Ed}$=-145.81 kN	$N_{2,Ed}$=69.13 kN		
$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{Rd,min}$=152.31 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.96<1$	$V_{v,Rd}$=118.81 kN $V_{b,Rd}$=283.64 kN $V_{gus,b,Rd}$ =283.64 kN $V_{eff,Rd}$=322.96 kN $V_{Rd,min}$=118.81 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.58<1$	Gusset to chord (chord face failure) $N_{1\text{-}1,Ed}$=-92.23 kN $M_{1\text{-}1,Ed}$=16.55 kNm N_{Rd}=272.32 kN $M_{ip,Rd}$=83.06 kNm Interaction=0.54<1	

Table B.56: Design checks - Joint 9 (right/left side member)

Joint 9 (right/left side member)		Geometry: KT	
Brace 1	Brace 2	Brace 3	Gusset plate design checks Length: h=780 mm Supported tension brace $N_{pl,Rd}$=455.88 kN>$N_{3,Ed}$ $N_{u,Rd}$=446.84 kN>$N_{3,Ed}$ Supported compression brace 1 $N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$ $N_{b,Rd}$=447.94 kN>$N_{1,Ed}$ Supported compression brace 2 $N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$ $N_{b,Rd}$= $N_{pl,Rd}$ >$N_{2,Ed}$ Stresses (critical section) σ_1=20.48 MPa<235 MPa σ_2=-48.94 MPa<235 MPa τ=43.56 MPa<135.68 MPa
SM3	SM4	SM5	
2L100x12	2L90x10	2L100x12	
θ=49.80°	θ=90°	θ=57.54°	
3M12...8.8	2M16...8.8	3M16...8.8	
$N_{1,Ed}$=-205.28 kN	$N_{2,Ed}$=-195.45 kN	$N_{3,Ed}$=259.57 kN	
$V_{v,Rd}$=237.63 kN $V_{b,Rd}$=567.29 kN $V_{gus,b,Rd}$ =283.64 kN $V_{Rd,min}$=237.63 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.86<1$	$V_{v,Rd}$=270.77 kN $V_{b,Rd}$=364.32 kN $V_{gus,b,Rd}$ =228.24 kN $V_{Rd,min}$=228.24 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.86<1$	$V_{v,Rd}$=422.44 kN $V_{b,Rd}$=700.36 kN $V_{gus,b,Rd}$ =354.46 kN $V_{eff,Rd}$=613.85 kN $V_{Rd,min}$=354.46 kN $\frac{N_{3,Ed}}{V_{Rd,min}}=0.73<1$	
Chord SHS 350x350x12.5 $N_{0,Ed}$=-304.96 kN	Gusset to chord (chord face failure) $N_{1\text{-}1,Ed}$=-133.23 kN		

$M_{0,Ed}$=26.85 kNm	$M_{1-1,Ed}$=42.23 kNm N_{Rd}=308.00 kN $M_{ip,Rd}$=120.11 kNm Interaction=0.78<1	

Table B.57: Design checks - Joint 10 (right/left side member)

Joint 10 (right/left side member)		Geometry: K	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=800 mm Supported tension brace $N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$ $N_{u,Rd}$=210.98 kN>$N_{2,Ed}$ Supported compression brace $N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$ $N_{b,Rd}$=452.96 kN>$N_{1,Ed}$ Stresses (critical section) σ_1=0.19 MPa<235 MPa σ_2=-47.76 MPa<235 MPa τ=37.60 MPa<135.68 MPa
SM6	SM7	SHS 350x350x12.5	
2L100x12	2L90x10	$N_{0,Ed}$=-541.27 kN	
θ=57.54°	θ=57.54°	$M_{0,Ed}$=19.21 kNm	
3M16...8.8	2M12...8.8		
$N_{1,Ed}$=-359.47 kN	$N_{2,Ed}$=88.84 kN		
$V_{v,Rd}$=422.44 kN $V_{b,Rd}$=700.36 kN $V_{gus,b,Rd}$ =378.19 kN $V_{Rd,min}$=378.19 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.95<1$	$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{eff,Rd}$=383.59 kN $V_{Rd,min}$=152.31 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.58<1$	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-228.35 kN $M_{1-1,Ed}$=30-68 kNm N_{Rd}=312-19 kN $M_{ip,Rd}$=124-88 kNm Interaction=0.98<1	

Table B.58: Design checks - Joint 11 (right/left side member)

Joint 11 (right/left side member)		Geometry: K	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=600 mm Supported compression brace 1 $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{u,Rd}$=227.29 kN>$N_{1,Ed}$ Supported compression brace 2 $N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$ $N_{b,Rd}$=227.29 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=-38.83 MPa<235 MPa σ_2=-6.34 MPa<235 MPa τ=13.93MPa<135.68 MPa
SM8	SM9	SHS 350x350x12.5	
2L90x10	2L90x10	$N_{0,Ed}$=-480.53 kN	
θ=57.54°	θ=57.54°	$M_{0,Ed}$=-1.01 kNm	
2M12...8.8	2M16...8.8		
$N_{1,Ed}$=-34.09 kN	$N_{2,Ed}$=-158.63 kN		
$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=300.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{Rd,min}$=152.31 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.22<1$	$V_{v,Rd}$=270.77 kN $V_{b,Rd}$=364.32 kN $V_{gus,b,Rd}$ =227.90 kN $V_{Rd,min}$=227.90 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.70<1$	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-162.61 kN $M_{1-1,Ed}$=11.70 kNm N_{Rd}=270.23 kN $M_{ip,Rd}$=81.07 kNm Interaction=0.75<1	

Table B.59: Design checks - Joint 12 (right/left side member)

Joint 12 (right/left side member)		Geometry: K	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=700 mm Supported tension brace $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{u,Rd}$=195.43 kN>$N_{1,Ed}$ Supported compression brace $N_{pl,Rd}$=455.88 kN>$N_{2,Ed}$ $N_{b,Rd}$=446.88 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=-63.66 MPa<235 MPa σ_2=25.51 MPa<235 MPa τ=51.13 MPa<135.68 MPa
SM10	SM11	SHS 350x350x12.5	
2L90x10	2L130x12	$N_{0,Ed}$=-480.82 kN	
θ=57.54°	θ=57.54°	$M_{0,Ed}$=-7.82 kNm	
2M16...8.8	3M16...8.8		
$N_{1,Ed}$=171.80 kN	$N_{2,Ed}$=-361.72 kN		
$V_{v,Rd}$=270.77 kN $V_{b,Rd}$=364.32 kN $V_{gus,b,Rd}$ =228.24 kN $V_{eff,Rd}$=356.04 kN $V_{Rd,min}$=228.24 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.75<1$	$V_{v,Rd}$=413.39 kN $V_{b,Rd}$=695.52 kN $V_{gus,b,Rd}$ =370.09 kN $V_{Rd,min}$=370.09 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.98<1$	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-160.24 kN $M_{1-1,Ed}$=43.69 kNm N_{Rd}=291.21 kN $M_{ip,Rd}$=101.92 kNm Interaction=0.98<1	

Table B.60: Design checks - Joint 13 (right/left side member)

<table>
<tr><td colspan="3">Joint 13 (right/left side member)</td><td>Geometry: N</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="7">Gusset plate design checks
Length: h=450 mm
Supported tension brace
$N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$
$N_{u,Rd}$=446.84 kN>$N_{1,Ed}$
Supported compression brace
$N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$
$N_{b,Rd}$= 202.16 kN>$N_{2,Ed}$
Stresses (critical section)
σ_1=-36.55 MPa<235 MPa
σ_2=76.89 MPa<235 MPa
τ=52.90 MPa<135.68 MPa</td></tr>
<tr><td>SM13</td><td>SM14</td><td>SHS 350x350x12.5</td></tr>
<tr><td>2L130x12</td><td>2L90x10</td><td>$N_{0,Ed}$=-211.54 kN</td></tr>
<tr><td>θ=55.78°</td><td>θ=90°</td><td>$M_{0,Ed}$=-9.33kNm</td></tr>
<tr><td>3M16...8.8</td><td>2x2M12...8.8</td><td></td></tr>
<tr><td>$N_{1,Ed}$=338.68 kN</td><td>$N_{2,Ed}$=-171.07 kN</td><td></td></tr>
<tr><td>$V_{v,Rd}$=413.39 kN
$V_{b,Rd}$=695.52 kN
$V_{gus,b,Rd}$ =347.76 kN
$V_{eff,Rd}$=665.69 kN
$V_{Rd,min}$=347.76 kN
$\frac{N_{1,Ed}}{V_{Rd,min}}$=0.97<1</td><td>$V_{v,Rd}$=304.60 kN
$V_{b,Rd}$=606.00 kN
$V_{gus,b,Rd}$ =363.60 kN
$V_{Rd,min}$=304.60 kN
$\frac{N_{2,Ed}}{V_{Rd,min}}$=0.56<1</td><td>Gusset to chord
(chord face failure)
$N_{1-1,Ed}$=108.93 kN
$M_{1-1,Ed}$=22.97 kNm
N_{Rd}=238.75 kN
$M_{ip,Rd}$=53.72 kNm
Interaction=0.88<1</td></tr>
</table>

Table B.61: Design checks - Joint 1 (right/left supporting beam)

<table>
<tr><td colspan="2">Joint 1 (right/left supporting beam)</td><td>Geometry: T</td></tr>
<tr><td>Brace 1</td><td>Chord</td><td rowspan="7">Gusset plate design checks
Length: h=200 mm
Supported tension brace
$N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$
$N_{u,Rd}$= 210.98 kN >$N_{1,Ed}$
Stresses (critical section)
σ_1=42.07 MPa<235 MPa
σ_2=42.07 MPa<235 MPa
τ=0 MPa</td></tr>
<tr><td>SB1</td><td>SHS 350x350x12.5</td></tr>
<tr><td>2L90x10</td><td>$N_{0,Ed}$=19.25 kN</td></tr>
<tr><td>θ=90°</td><td>$M_{0,Ed}$=9.57 kNm</td></tr>
<tr><td>2M12...8.8</td><td></td></tr>
<tr><td>$N_{1,Ed}$=100.96 kN</td><td></td></tr>
<tr><td>$V_{v,Rd}$=152.31 kN
$V_{b,Rd}$=303.00 kN
$V_{gus,b,Rd}$ =181.81 kN
$V_{eff,Rd}$=383.59 kN
$V_{Rd,min}$=152.32 kN
$\frac{N_{1,Ed}}{V_{Rd,min}}$=0.66<1</td><td>Gusset to chord
(chord face failure)
$N_{1-1,Ed}$=100.96 kN
$M_{1-1,Ed}$=0 kNm
N_{Rd}=186.30 kN
$\frac{N_{1-1,Ed}}{N_{Rd}}$=0.54<1</td></tr>
</table>

Table B.62: Design checks - Joint 2 (right/left supporting beam)

<table>
<tr><td colspan="3">Joint 2 (right/left supporting beam)</td><td>Geometry: KT</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Brace 3</td><td rowspan="8">Gusset plate design checks
Length: h=790 mm
Supported compression brace 1
$N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$
$N_{u,Rd}$=446.68 kN>$N_{1,Ed}$
Supported compression brace 2
$N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$
$N_{b,Rd}$= $N_{pl,Rd}$>$N_{2,Ed}$
Supported compression brace 3
$N_{pl,Rd}$=455.88 kN>$N_{3,Ed}$
$N_{b,Rd}$= 446.68 kN >$N_{3,Ed}$
Stresses (critical section)
σ_1=-77.10 MPa<235 MPa
σ_2=-23.66 MPa<235 MPa
τ=30.15 MPa<135.68 MPa</td></tr>
<tr><td>SB2</td><td>SB3</td><td>SB4</td></tr>
<tr><td>L100x12</td><td>2L90x10</td><td>2L90x10</td></tr>
<tr><td>θ=49.80°</td><td>θ=90°</td><td>θ=49.80°</td></tr>
<tr><td>3M12...8.8</td><td>2M12...8.8</td><td>3M16...8.8</td></tr>
<tr><td>$N_{1,Ed}$=-76.40 kN</td><td>$N_{2,Ed}$=-135.38 kN</td><td>$N_{3,Ed}$=-371.66 kN</td></tr>
<tr><td>$V_{v,Rd}$=118.81 kN
$V_{b,Rd}$=283.64 kN
$V_{gus,b,Rd}$ =283.64 kN
$V_{Rd,min}$=118.81 kN
$\frac{N_{1,Ed}}{V_{Rd,min}}$=0.64<1</td><td>$V_{v,Rd}$=152.32 kN
$V_{b,Rd}$=303.00 kN
$V_{gus,b,Rd}$ =181.80 kN
$V_{Rd,min}$=152.32 kN
$\frac{N_{2,Ed}}{V_{Rd,min}}$=0.89<1</td><td>$V_{v,Rd}$=428.40 kN
$V_{b,Rd}$=582.06 kN
$V_{gus,b,Rd}$ =383.53 kN
$V_{Rd,min}$=383.53 kN
$\frac{N_{3,Ed}}{V_{Rd,min}}$=0.97<1</td></tr>
<tr><td>Chord
SHS 350x350x12.5
$N_{0,Ed}$=212.33 kN
$M_{0,Ed}$=-118.61 kNm</td><td colspan="2">Gusset to chord
(chord face failure)
$N_{1-1,Ed}$=-477.61 kN
$M_{1-1,Ed}$=33.35 kNm
Stiffened by a plate
t_p=32 mm; b_p=190 mm
N_{Rd}=579.83 kN
$M_{ip,Rd}$=235.46 kNm
Interaction=0.97<1</td></tr>
</table>

Table B.63: Design checks - Joint 3 (right/left supporting beam)

Joint 3 (right/left supporting beam)		Geometry: T
Brace 1	Chord	Gusset plate design checks Length: h=200 mm Supported tension brace $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{u,Rd}$= 210.98 kN >$N_{1,Ed}$ Stresses (critical section) σ_1=30.69 MPa<235 MPa σ_2=30.69 MPa<235 MPa τ=0 MPa
SB5	SHS 350x350x12.5	
2L90x10	$N_{0,Ed}$=199.03 kN	
θ=90°	$M_{0,Ed}$=72.91 kNm	
2M12...8.8		
$N_{1,Ed}$=73.66 kN		
$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.81 kN $V_{eff,Rd}$=383.59 kN $V_{Rd,min}$=152.32 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.48<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=73.66 kN $M_{1-1,Ed}$=0 kNm N_{Rd}=186.30 kN $\frac{N_{1-1,Ed}}{N_{Rd}}$=0.40<1	

Table B.64: Design checks - Joint 4 (right/left supporting beam)

Joint 4 (right/left supporting beam)			Geometry: K
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=600 mm Supported tension brace $N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$ $N_{u,Rd}$=462.39 kN>$N_{1,Ed}$ Supported compression brace $N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$ $N_{b,Rd}$=227.10 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=-29.58 MPa<235 MPa σ_2=52.50 MPa<235 MPa τ=35.18 MPa<135.68 MPa
SB6	SB7	SHS 350x350x12.5	
2L90x10	L100x12	$N_{0,Ed}$=392.79 kN	
θ=57.54°	θ=57.54°	$M_{0,Ed}$=3.46 kNm	
3M12...8.8	2M16...8.8		
$N_{1,Ed}$=206.23 kN	$N_{2,Ed}$=-108.44 kN		
$V_{v,Rd}$=240.98 kN $V_{b,Rd}$=479.41 kN $V_{gus,b,Rd}$ =287.65 kN $V_{eff,Rd}$=538.26 kN $V_{Rd,min}$=240.98 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.86<1	$V_{v,Rd}$=132.41 kN $V_{b,Rd}$=219.52 kN $V_{gus,b,Rd}$ =223.78 kN $V_{Rd,min}$=132.41 kN $\frac{N_{2,Ed}}{V_{Rd,min}}$=0.82<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=82.51 kN $M_{1-1,Ed}$=29.55 kNm N_{Rd}=270.22 kN $M_{ip,Rd}$=81.07 kNm Interaction=0.67<1	

Table B.65: Design checks - Joint 5 (right/left supporting beam)

Joint 5 (right/left supporting beam)			Geometry: K
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=520 mm Supported tension brace $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{u,Rd}$=195.43 kN>$N_{1,Ed}$ Supported compression brace $N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$ $N_{b,Rd}$=227.10 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=-32.38 MPa<235 MPa σ_2=33.69 MPa<235 MPa τ=24.54 MPa<135.68 MPa
SB8	SB9	SHS 350x350x12.5	
L100x12	L100x12	$N_{0,Ed}$=495.20 kN	
θ=57.54°	θ=57.54°	$M_{0,Ed}$=12.45 kNm	
2M16...8.8	2M16...8.8		
$N_{1,Ed}$=97.54 kN	$N_{2,Ed}$=-92.70 kN		
$V_{v,Rd}$=132.41 kN $V_{b,Rd}$=219.52 kN $V_{gus,b,Rd}$ =223.13 kN $V_{eff,Rd}$=222.26 kN $V_{Rd,min}$=132.41 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.74<1	$V_{v,Rd}$=132.41 kN $V_{b,Rd}$=219.52 kN $V_{gus,b,Rd}$ =223.78 kN $V_{Rd,min}$=132.41 kN $\frac{N_{2,Ed}}{V_{Rd,min}}$=0.70<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=4.09 kN $M_{1-1,Ed}$=17.87 kNm N_{Rd}=253.44 kN $M_{ip,Rd}$=65.90 kNm Interaction=0.29<1	

Table B.66: Design checks - Joint 6 (right/left supporting beam)

Joint 6 (right/left supporting beam)		Geometry: K	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=520 mm Supported tension brace $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{u,Rd}$=195.43 kN>$N_{1,Ed}$ Supported compression brace $N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$ $N_{b,Rd}$=227.10 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=-29.00 MPa<235 MPa σ_2=29.53 MPa<235 MPa τ=21.74 MPa<135.68 MPa
SB10	SB11	SHS 350x350x12.5	
L100x12	L100x12	$N_{0,Ed}$=585.98 kN	
θ=57.54°	θ=57.54°	$M_{0,Ed}$=21.08 kN	
2M16...8.8	2M16...8.8		
$N_{1,Ed}$=85.23 kN	$N_{2,Ed}$=-83.28 kN		
$V_{v,Rd}$=132.41 kN $V_{b,Rd}$=219.52 kN $V_{gus,b,Rd}$ =223.13 kN $V_{eff,Rd}$=222.26 kN $V_{Rd,min}$=132.41 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.64<1	$V_{v,Rd}$=132.41 kN $V_{b,Rd}$=219.52 kN $V_{gus,b,Rd}$ =223.78 kN $V_{Rd,min}$=132.41 kN $\frac{N_{2,Ed}}{V_{Rd,min}}$=0.63<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=1.65 kN $M_{1-1,Ed}$=15.83 kNm N_{Rd}=253.44 kN $M_{ip,Rd}$=65.90 kNm Interaction=0.25<1	

Table B.67: Design checks - Joint 7 (right/left supporting beam)

Joint 7 (right/left supporting beam)		Geometry: KT	
Brace 1	Brace 2	Brace 3	Gusset plate design checks Length: h=660 mm Supported tension brace $N_{pl,Rd}$=455.88 kN>$N_{3,Ed}$ $N_{u,Rd}$=446.84 kN>$N_{3,Ed}$ Supported compression brace 1 $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{b,Rd}$=227.10 kN>$N_{1,Ed}$ Supported compression brace 2 $N_{pl,Rd}$=455.88 kN>$N_{2,Ed}$ $N_{b,Rd}$=418.72 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=38.29 MPa<235 MPa σ_2=-15.34 MPa<235 MPa τ=31.19 MPa<135.68 MPa
SB12	SB13	SB14	
L100x12	2L90x10	2L130x12	
θ=57.54°	θ=90°	θ=55.78°	
2M12...8.8	2x2M12...8.8	3M16...8.8	
$N_{1,Ed}$=-11.94 kN	$N_{2,Ed}$=-131.76 kN	$N_{3,Ed}$=281.45 kN	
$V_{v,Rd}$=74.48 kN $V_{b,Rd}$=177.81 kN $V_{gus,b,Rd}$ =177.81 kN $V_{Rd,min}$=74.48 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.15<1	$V_{v,Rd}$=304.62 kN $V_{b,Rd}$=606.00 kN $V_{gus,b,Rd}$ =363.6 kN $V_{Rd,min}$=304.62 kN $\frac{N_{2,Ed}}{V_{Rd,min}}$=0.43<1	$V_{v,Rd}$=413.39 kN $V_{b,Rd}$=695.52 kN $V_{gus,b,Rd}$ =370.76 kN $V_{eff,Rd}$=665.69 kN $V_{Rd,min}$=370.76 kN $\frac{N_{3,Ed}}{V_{Rd,min}}$=0.76<1	
Chord SHS 350x350x12.5 $N_{0,Ed}$=538.39 kN $M_{0,Ed}$=23.31 KNm		Gusset to chord (chord face failure) $N_{1-1,Ed}$=90.89 kN $M_{1-1,Ed}$=23.36 kNm N_{Rd}=282.82 kN $M_{ip,Rd}$=93.33 kNm Interaction=0.57<1	

Table B.68: Design checks - Joint 8 (right/left supporting beam)

Joint 8 (right/left supporting beam)		Geometry: N	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=750 mm Supported compression brace 1 $N_{pl,Rd}$=830.35 kN>$N_{1,Ed}$ $N_{b,Rd}$ =824.43 kN>$N_{1,Ed}$ Supported compression brace 2 $N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$ $N_{b,Rd}$= $N_{pl,Rd}$>$N_{2,Ed}$ Stresses (critical section) σ_1=33.15 MPa<235 MPa σ_2=72.15 MPa<235 MPa τ=71.98 MPa<135.68 MPa
SB16	SB17	SHS 350x350x12.5	
2L130x12	2L90x10	$N_{0,Ed}$=372.01 kN	
θ=48.72°	θ=90°	$M_{0,Ed}$=0 kNm	
4M20...8.8	2M12...8.8		
$N_{1,Ed}$=-644.70 kN	$N_{2,Ed}$=-18.09 kN		
$V_{v,Rd}$=915.61 kN $V_{b,Rd}$=1015.43 kN $V_{gus,b,Rd}$ =655.76 kN $V_{Rd,min}$=655.76kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.98<1	$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =178.04 kN $V_{Rd,min}$=178.04 kN $\frac{N_{2,Ed}}{V_{Rd,min}}$=0.10<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-510.03 kN $M_{1-1,Ed}$=13.67 kNm Stiffened by a plate t_p=32 mm; b_p=190 mm N_{Rd}=561.5 kN $M_{ip,Rd}$=210.56 kNm Interaction=0.97<1	

Table B.69: Design checks - Joint 9 (right/left supporting beam)

Joint 9 (right/left supporting beam)		Geometry: N	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=620 mm Supported tension brace $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{u,Rd}$=210.98 kN>$N_{1,Ed}$ Supported compression brace $N_{pl,Rd}$=455.88 kN>$N_{2,Ed}$ $N_{b,Rd}$= 446.68 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=-14.92 MPa<235 MPa σ_2=22.44 MPa<235 MPa τ=13.00 MPa<135.68 MPa
SB1	SB2	SHS 350x350x12.5	
2L90x10	L100x12	$N_{0,Ed}$=58.33 kN	
θ=90°	θ=49.80°	$M_{0,Ed}$=0 kNm	
2M12...8.8	3M12...8.8		
$N_{1,Ed}$=104.29 kN	$N_{2,Ed}$=-99.93 kN		
$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{eff,Rd}$=383.59 kN $V_{Rd,min}$=152.31 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.68<1	$V_{v,Rd}$=118.81 kN $V_{b,Rd}$=283.64 kN $V_{gus,b,Rd}$ =283.64 kN $V_{eff,Rd}$=322.96 kN $V_{Rd,min}$=118.81 kN $\frac{N_{2,Ed}}{V_{Rd,min}}$=0.84<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=27.96 kN $M_{1-1,Ed}$=14.36 kNm N_{Rd}=274.42 kN $M_{ip,Rd}$=85.07 kNm Interaction=0.27<1	

Table B.70: Design checks - Joint 10 (right/left supporting beam)

Joint 10 (right/left supporting beam)	Geometry: T	
Brace 1	Chord	Gusset plate design checks Length: h=200 mm Supported compression brace $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{b,Rd}$= $N_{pl,Rd}$ >$N_{1,Ed}$ Stresses (critical section) σ_1=-56.41 MPa<235 MPa σ_2=-56.41 MPa<235 MPa τ=0 MPa
SB3	SHS 350x350x12.5	
2L90x10	$N_{0,Ed}$=-43.52 kN	
θ=90°	$M_{0,Ed}$=66.23 kNm	
2M12...8.8		
$N_{1,Ed}$=-135.38 kN		
$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.81 kN $V_{Rd,min}$=152.32 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.89<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-135.38 kN $M_{1-1,Ed}$=0 kNm N_{Rd}=186.30 kN $\frac{N_{1-1,Ed}}{N_{Rd}}$=0.73<1	

Table B.71: Design checks - Joint 11 (right/left supporting beam)

Joint 11 (right/left supporting beam)		Geometry: KT	
Brace 1	Brace 2	Brace 3	Gusset plate design checks Length: h=750 mm Supported compression brace $N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$ $N_{b,Rd}$= 448.63 kN >$N_{1,Ed}$ Supported tension brace 1 $N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$ $N_{u,Rd}$=210.98 kN>$N_{2,Ed}$ Supported tension brace 2 $N_{pl,Rd}$=227.94 kN>$N_{3,Ed}$ $N_{u,Rd}$=210.98 kN>$N_{3,Ed}$ Stresses (critical section) σ_1=48.65 MPa<235 MPa σ_2=-57.84 MPa<235 MPa τ=58.43 MPa<135.68 MPa
SB4	SB5	SB6	
2L90x10	2L90x10	2L90x10	
θ=49.80°	θ=90°	θ=57.54°	
3M16...8.8	2M12...8.8	3M12...8.8	
$N_{1,Ed}$=-371.66 kN	$N_{2,Ed}$=68.52 kN	$N_{3,Ed}$=206.23 kN	
$V_{v,Rd}$=428.40 kN $V_{b,Rd}$=582.06 kN $V_{gus,b,Rd}$ =377.07 kN $V_{Rd,min}$=377.07 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.99<1	$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=300.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{eff,Rd}$=383.59 kN $V_{Rd,min}$=152.31 kN $\frac{N_{2,Ed}}{V_{Rd,min}}$=0.45<1	$V_{v,Rd}$=240.98 kN $V_{b,Rd}$=479.41 kN $V_{gus,b,Rd}$ =287.65 kN $V_{eff,Rd}$=538.26 kN $V_{Rd,min}$=240.98 kN $\frac{N_{3,Ed}}{V_{Rd,min}}$=0.86<1	
Chord SHS 350x350x12.5 $N_{0,Ed}$=-315.69 kN $M_{0,Ed}$=-34.69 kNm	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-41.34 kN $M_{1-1,Ed}$=59.90 kNm N_{Rd}=301.70 kN $M_{ip,Rd}$=113.14 kNm Interaction=0.66<1		

Table B.72: Design checks - Joint 12 (right/left supporting beam)

<table>
<tr><td colspan="2">Joint 12 (right/left supporting beam)</td><td colspan="2">Geometry: K</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="7">Gusset plate design checks
Length: h=610 mm
Supported compression brace
$N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$
$N_{b,Rd}$=226.39 kN>$N_{1,Ed}$
Supported tension brace
$N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$
$N_{u,Rd}$=195.43 kN>$N_{2,Ed}$
Stresses (critical section)
σ_1=24.74 MPa<235 MPa
σ_2=-27.25 MPa<235 MPa
τ=22.65 MPa<135.68 MPa</td></tr>
<tr><td>SB7</td><td>SB8</td><td>SHS 350x350x12.5</td></tr>
<tr><td>L100x12</td><td>L100x12</td><td>$N_{0,Ed}$=-425.90 kN</td></tr>
<tr><td>θ=57.54°</td><td>θ=57.54°</td><td>$M_{0,Ed}$=-11.80 kNm</td></tr>
<tr><td>2M16...8.8</td><td>2M16...8.8</td><td></td></tr>
<tr><td>$N_{1,Ed}$=-108.44 kN</td><td>$N_{2,Ed}$=97.54 kN</td><td></td></tr>
<tr><td>$V_{v,Rd}$=132.41 kN
$V_{b,Rd}$=219.52 kN
$V_{gus,b,Rd}$ =223.78 kN
$V_{Rd,min}$=132.41 kN
$\frac{N_{1,Ed}}{V_{Rd,min}}=0.82<1$</td><td>$V_{v,Rd}$=132.41 kN
$V_{b,Rd}$=219.52 kN
$V_{gus,b,Rd}$ =223.78 kN
$V_{eff,Rd}$=222.26 kN
$V_{Rd,min}$=132.41 kN
$\frac{N_{2,Ed}}{V_{Rd,min}}=0.74<1$</td><td>Gusset to chord
(chord face failure)
$N_{1-1,Ed}$=-9.20 kN
$M_{1-1,Ed}$=19.34 kNm
N_{Rd}=272.33 kN
$M_{ip,Rd}$=83.06 kNm
Interaction=0.27<1</td></tr>
</table>

Table B.73: Design checks - Joint 13 (right/left supporting beam)

<table>
<tr><td colspan="2">Joint 13 (right/left supporting beam)</td><td colspan="2">Geometry: K</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="7">Gusset plate design checks
Length: h=610 mm
Supported compression brace
$N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$
$N_{b,Rd}$=226.39 kN>$N_{1,Ed}$
Supported tension brace
$N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$
$N_{u,Rd}$=195.43 kN>$N_{2,Ed}$
Stresses (critical section)
σ_1=21.59 MPa<235 MPa
σ_2=-23.32 MPa<235 MPa
τ=19.57 MPa<135.68 MPa</td></tr>
<tr><td>SB9</td><td>SB10</td><td>SHS 350x350x12.5</td></tr>
<tr><td>L100x12</td><td>L100x12</td><td>$N_{0,Ed}$=-521.07 kN</td></tr>
<tr><td>θ=57.54°</td><td>θ=57.54°</td><td>$M_{0,Ed}$=-12.71 kNm</td></tr>
<tr><td>2M16...8.8</td><td>2M16...8.8</td><td></td></tr>
<tr><td>$N_{1,Ed}$=-92.72 kN</td><td>$N_{2,Ed}$=85.23 kN</td><td></td></tr>
<tr><td>$V_{v,Rd}$=132.41 kN
$V_{b,Rd}$=219.52 kN
$V_{gus,b,Rd}$ =223.78 kN
$V_{Rd,min}$=132.41 kN
$\frac{N_{1,Ed}}{V_{Rd,min}}=0.82<1$</td><td>$V_{v,Rd}$=132.41 kN
$V_{b,Rd}$=219.52 kN
$V_{gus,b,Rd}$ =223.78 kN
$V_{eff,Rd}$=222.26 kN
$V_{Rd,min}$=132.41 kN
$\frac{N_{2,Ed}}{V_{Rd,min}}=0.74<1$</td><td>Gusset to chord
(chord face failure)
$N_{1-1,Ed}$=-6.32 kN
$M_{1-1,Ed}$=16.71 kNm
N_{Rd}=272.33 kN
$M_{ip,Rd}$=83.06 kNm
Interaction=0.22<1</td></tr>
</table>

Table B.74: Design checks - Joint 14 (right/left supporting beam)

<table>
<tr><td colspan="2">Joint 14 (right/left supporting beam)</td><td colspan="2">Geometry: K</td></tr>
<tr><td>Brace 1</td><td>Brace 2</td><td>Chord</td><td rowspan="7">Gusset plate design checks
Length: h=610 mm
Supported compression brace 1
$N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$
$N_{b,Rd}$=226.39 kN>$N_{1,Ed}$
Supported compression brace 2
$N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$
$N_{b,Rd}$=226.39 kN>$N_{2,Ed}$
Stresses (critical section)
σ_1=0.39 MPa<235 MPa
σ_2=-20.05MPa<235 MPa
τ=8.57 MPa<135.68 MPa</td></tr>
<tr><td>SB11</td><td>SB12</td><td>SHS 350x350x12.5</td></tr>
<tr><td>L100x12</td><td>L100x12</td><td>$N_{0,Ed}$=-562.61 kN</td></tr>
<tr><td>θ=57.54°</td><td>θ=57.54°</td><td>$M_{0,Ed}$=-18.40 kNm</td></tr>
<tr><td>2M16...8.8</td><td>2M12...8.8</td><td></td></tr>
<tr><td>$N_{1,Ed}$=-83.28 kN</td><td>$N_{2,Ed}$=-5.38 kN</td><td></td></tr>
<tr><td>$V_{v,Rd}$=132.41 kN
$V_{b,Rd}$=219.52 kN
$V_{gus,b,Rd}$ =223.78 kN
$V_{Rd,min}$=132.41 kN
$\frac{N_{1,Ed}}{V_{Rd,min}}=0.63<1$</td><td>$V_{v,Rd}$=74.48 kN
$V_{b,Rd}$=177.81 kN
$V_{gus,b,Rd}$ =177.81 kN
$V_{Rd,min}$=74.48 kN
$\frac{N_{2,Ed}}{V_{Rd,min}}=0.07<1$</td><td>Gusset to chord
(chord face failure)
$N_{1-1,Ed}$=-74.81 kN
$M_{1-1,Ed}$=7.32 kNm
N_{Rd}=272.33 kN
$M_{ip,Rd}$=83.06 kNm
Interaction=0.36<1</td></tr>
</table>

Table B.75: Design checks - Joint 15 (right/left supporting beam)

Joint 15 (right/left supporting beam)		Geometry: KT	
Brace 1	Brace 2	Brace 3	Gusset plate design checks Length: h=810 mm Supported tension brace 1 $N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$ $N_{u,Rd}$=446.84 kN>$N_{1,Ed}$ Supported tension brace 2 $N_{pl,Rd}$=455.88 kN>$N_{2,Ed}$ $N_{u,Rd}$=421.96 kN>$N_{2,Ed}$ Supported compression brace $N_{pl,Rd}$=830.35 kN>$N_{3,Ed}$ $N_{b,Rd}$= 806.80 kN >$N_{3,Ed}$ Stresses (critical section) σ_1=-81.57 MPa<235 MPa σ_2=67.16 MPa<235 MPa τ=90.06 MPa<135.68 MPa
SB14	SB15	SB16	
2L130x12	2L90x10	2L130x12	
θ=55.78°	θ=90°	θ=48.72°	
3M16...8.8	2x2M12...8.8	4M20...8.8	
$N_{1,Ed}$=281.45 kN	$N_{2,Ed}$=181.75 kN	$N_{3,Ed}$=-644.70 kN	
$V_{v,Rd}$=413.39 kN $V_{b,Rd}$=695.52 kN $V_{gus,b,Rd}$ =347.76 kN $V_{eff,Rd}$=648.41 kN $V_{Rd,min}$=347.76 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.81<1$	$V_{v,Rd}$=304.60 kN $V_{b,Rd}$=606.00 kN $V_{gus,b,Rd}$ =363.60 kN $V_{eff,Rd}$=767.18 kN $V_{Rd,min}$=304.60 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.60<1$	$V_{v,Rd}$=915.61 kN $V_{b,Rd}$=1009.10 kN $V_{gus,b,Rd}$ =654.75 kN $V_{Rd,min}$=654.75 kN $\frac{N_{3,Ed}}{V_{Rd,min}}=0.98<1$	
Chord SHS 350x350x12.5 $N_{0,Ed}$=-560.87 kN $M_{0,Ed}$=-26.65 kNm	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-70.01 kN $M_{1-1,Ed}$=97.59 kNm N_{Rd}=314.29 kN $M_{ip,Rd}$=127.294 kNm Interaction=0.69<1		

Table B.76: Design checks - Joint 16 (right/left supporting beam)

Joint 16 (right/left supporting beam)	Geometry: T	
Brace 1	Chord	Gusset plate design checks Length: h=200 mm Supported compression brace $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{b,Rd}$= $N_{pl,Rd}$ >$N_{1,Ed}$ Stresses (critical section) σ_1=-9.77 MPa<235 MPa σ_2=-9.77 MPa<235 MPa τ=0 MPa
SB17	SHS 350x350x12.5	
2L90x10	$N_{0,Ed}$=0 kN	
θ=90°	$M_{0,Ed}$=0 kNm	
2M12...8.8		
$N_{1,Ed}$=-23.44 kN		
$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.81 kN $V_{Rd,min}$=152.32 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.15<1$	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-23.44 kN $M_{1-1,Ed}$=0 kNm N_{Rd}=186.30 kN $\frac{N_{1-1,Ed}}{N_{Rd}}=0.13<1$	

Table B.77: Design checks - Joint 1 (connecting beam C1)

Joint 1 (connecting beam C1)		Geometry: N	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=645 mm Supported compression brace $N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$ $N_{b,Rd}$= $N_{pl,Rd}$>$N_{1,Ed}$ Supported tension brace $N_{pl,Rd}$=683.82 kN>$N_{2,Ed}$ $N_{u,Rd}$=698.25 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=39.42MPa<235 MPa σ_2=-6.10 MPa<235 MPa τ=50.60 MPa<135.68 MPa
SB13	C1B1	SHS 350x350x12.5	
2L90x10	2L100x12	$N_{0,Ed}$=-314.98 kN	
θ=90°	θ=44.25°	$M_{0,Ed}$=-6.17 kNm	
2x2M12...8.8	4M16...8.8		
$N_{1,Ed}$=-125.37 kN	$N_{2,Ed}$=364.47 kN		
$V_{v,Rd}$=304.60 kN $V_{b,Rd}$=606.00 kN $V_{gus,b,Rd}$ =363.60 kN $V_{eff,Rd}$=767.18 kN $V_{Rd,min}$=304.60 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.41<1$	$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{Rd,min}$=485.73 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.75<1$	Gusset to chord (chord face failure) $N_{1-1,Ed}$=128.95 kN $M_{1-1,Ed}$=18.94 kNm N_{Rd}=279.67 kN $M_{ip,Rd}$=90.19 kNm Interaction=0.67<1	

Table B.78: Design checks - Joint 2 (connecting beam C1)

Joint 2 (connecting beam C1)		Geometry: K	
Brace 1	Brace 2	Chord	Gusset plate design checks
C1B1	C1B2	SHS 350x350x12.5	Length: h=940 mm
2L100x12	2L100x12	$N_{0,Ed}$=481.47 kN	Supported tension brace
θ=44.25°	θ=44.25°	$M_{0,Ed}$=13.76 kNm	$N_{pl,Rd}$=683.82 kN>$N_{1,Ed}$
4M16...8.8	4M16...8.8		$N_{u,Rd}$=698.25 kN>$N_{1,Ed}$
$N_{1,Ed}$=364.47 kN	$N_{2,Ed}$=-347.15 kN		Supported compression brace
$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{eff,Rd}$=783.18 kN $V_{Rd,min}$=485.73 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.75<1$	$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{Rd,min}$=485.73 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.71<1$	Gusset to chord (chord face failure) $N_{1-1,Ed}$=12.09 kN $M_{1-1,Ed}$=89.22 kNm N_{Rd}=341.57 kN $M_{ip,Rd}$=160.54 kNm Interaction=0.59<1	$N_{pl,Rd}$=683.82 kN>$N_{2,Ed}$ $N_{b,Rd}$= 665.60 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=-49.41 MPa<235 MPa σ_2=51.56 MPa<235 MPa τ=67.78 MPa<135.68 MPa

Table B.79: Design checks - Joint 3 (connecting beam C1)

Joint 3 (connecting beam C1)		Geometry: KT	
Brace 1	Brace 2	Brace 3	Gusset plate design checks
C1B2	SM12	C1B3	Length: h=980 mm
2L100x12	2L90x10	2L90x10	Supported compression brace
θ=44.25°	θ=90°	θ=41.42°	$N_{pl,Rd}$=683.82 kN>$N_{1,Ed}$
4M16...8.8	2x2M12...8.8	2M12...8.8	$N_{b,Rd}$= 665.60 kN>$N_{1,Ed}$
$N_{1,Ed}$=-347.15 kN	$N_{2,Ed}$=14.15 kN	$N_{3,Ed}$=36.66 kN	Supported tension brace 1
$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{Rd,min}$=485.73 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.71<1$	$V_{v,Rd}$=304.60 kN $V_{b,Rd}$=606.00 kN $V_{gus,b,Rd}$ =363.60 kN $V_{eff,Rd}$=767.18 kN $V_{Rd,min}$=304.60 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.05<1$	$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{eff,Rd}$=383.59 kN $V_{Rd,min}$=152.31 kN $\frac{N_{3,Ed}}{V_{Rd,min}}=0.24<1$	$N_{pl,Rd}$=455.88 kN>$N_{2,Ed}$ $N_{u,Rd}$=421.96 kN>$N_{2,Ed}$ Supported tension brace 2 $N_{pl,Rd}$=227.94 kN>$N_{3,Ed}$ $N_{u,Rd}$=210.98 kN>$N_{3,Ed}$ Stresses (critical section) σ_1=3.59 MPa<235 MPa
Chord SHS 350x350x12.5 $N_{0,Ed}$=-641.94 kN $M_{0,Ed}$=-26.65 kNm	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-203.84 kN $M_{1-1,Ed}$=40.18 kNm N_{Rd}=349.96 kN $M_{ip,Rd}$=171.48 kNm Interaction=0.82<1		σ_2=-38.25 MPa<235 MPa τ=35.22 MPa<135.68 MPa

Table B.80: Design checks - Joint 4 (connecting beam C1)

Joint 4 (connecting beam C1)		Geometry: K	
Brace 1	Brace 2	Chord	Gusset plate design checks
C1B3	C1B4	SHS 350x350x12.5	Length: h=780 mm
2L90x10	2L90x10	$N_{0,Ed}$=520.23 kN	Supported tension brace 1
θ=41.42°	θ=41.42°	$M_{0,Ed}$=9.49 kNm	$N_{pl,Rd}$=227.94 kN >$N_{1,Ed}$
2M12...8.8	2M12...8.8		$N_{u,Rd}$=210.98 kN >$N_{1,Ed}$
$N_{1,Ed}$=39.30 kN	$N_{2,Ed}$=89.03 kN		Supported tension brace 2
$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{eff,Rd}$=383.59 kN $V_{Rd,min}$=152.31 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.26<1$	$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{eff,Rd}$=383.59 kN $V_{Rd,min}$=152.31 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.58<1$	Gusset to chord (chord face failure) $N_{1-1,Ed}$=84.90 kN $M_{1-1,Ed}$=6.53 kNm N_{Rd}=308.00 kN $M_{ip,Rd}$=120.12 kNm Interaction=0.33<1	$N_{pl,Rd}$=227.94 kN >$N_{2,Ed}$ $N_{u,Rd}$=210.98 kN >$N_{2,Ed}$ Stresses (critical section) σ_1=-14.43 MPa<235 MPa σ_2=3.71 MPa<235 MPa τ=5.98 MPa<135.68 MPa

Table B.81: Design checks - Joint 5 (connecting beam C1)

Joint 5 (connecting beam C1)		Geometry: KT	
Brace 1	Brace 2	Brace 3	Gusset plate design checks Length: h=910 mm Supported tension brace 1 $N_{pl,Rd}$=227.94 kN>$N_{1,Ed}$ $N_{u,Rd}$=210.98 kN>$N_{1,Ed}$ Supported tension brace 2 $N_{pl,Rd}$=227.94 kN>$N_{2,Ed}$ $N_{u,Rd}$=210.98 kN>$N_{2,Ed}$ Supported compression brace $N_{pl,Rd}$=683.82 kN>$N_{3,Ed}$ $N_{b,Rd}$= 672.07 kN>$N_{3,Ed}$ Stresses (critical section) σ_1=-49.76 MPa<235 MPa σ_2=11.77 MPa<235 MPa τ=40.81 MPa<135.68 MPa
C1B4	SM12	C1B5	
2L90x10	2L90x10	2L100x12	
θ=41.42°	θ=90°	θ=49.97°	
2M16...8.8	2x2M12...8.8	4M12...8.8	
$N_{1,Ed}$=87.62 kN	$N_{2,Ed}$=10.08 kN	$N_{3,Ed}$=-359.74 kN	
$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{eff,Rd}$=383.59 kN $V_{Rd,min}$=152.31 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.58<1$	$V_{v,Rd}$=304.60 kN $V_{b,Rd}$=606.00 kN $V_{gus,b,Rd}$ =363.60 kN $V_{eff,Rd}$=767.18 kN $V_{Rd,min}$=304.60 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.03<1$	$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{Rd,min}$=485.73 kN $\frac{N_{3,Ed}}{V_{Rd,min}}=0.74<1$	
Chord	Gusset to chord (chord face failure) $N_{1\text{-}1,Ed}$=-207.41 kN $M_{1\text{-}1,Ed}$=50.95 kNm N_{Rd}=335.27 kN $M_{ip,Rd}$=152.55 kNm Interaction=0.95<1		
SHS 350x350x12.5			
$N_{0,Ed}$=-641.94 kN			
$M_{0,Ed}$=-7.87 kNm			

Table B.82: Design checks - Joint 6 (connecting beam C1)

Joint 6 (connecting beam C1)		Geometry: K	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=820 mm Supported tension brace $N_{pl,Rd}$=683.82 kN>$N_{2,Ed}$ $N_{u,Rd}$=698.25 kN>$N_{2,Ed}$ Supported compression brace $N_{pl,Rd}$=683.82 kN>$N_{1,Ed}$ $N_{b,Rd}$= 672.07 kN>$N_{1,Ed}$ Stresses (critical section) σ_1=60.25 MPa<235 MPa σ_2=-60.22 MPa<235 MPa τ=70.56 MPa<135.68 MPa
C1B5	C1B6	SHS 350x350x12.5	
2L100x12	2L100x12	$N_{0,Ed}$=442.32 kN	
θ=49.97°	θ=49.97°	$M_{0,Ed}$=17.94 kNm	
4M16...8.8	4M16...8.8		
$N_{1,Ed}$=-359.74 kN	$N_{2,Ed}$=359.93 kN		
$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{Rd,min}$=485.73 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.74<1$	$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{eff,Rd}$=783.18 kN $V_{Rd,min}$=485.73 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.74<1$	Gusset to chord (chord face failure) $N_{1\text{-}1,Ed}$=0.15 kN $M_{1\text{-}1,Ed}$=81.01 kNm N_{Rd}=316.39 kN $M_{ip,Rd}$=129.72 kNm Interaction=0.62<1	

Table B.83: Design checks - Joint 7 (connecting beam C1)

Joint 7 (connecting beam C1)		Geometry: N	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=575 mm Supported compression brace $N_{pl,Rd}$=455.88 kN>$N_{2,Ed}$ $N_{b,Rd}$= $N_{pl,Rd}$>$N_{2,Ed}$ Supported tension brace $N_{pl,Rd}$=683.82 kN>$N_{1,Ed}$ $N_{u,Rd}$=698.25 kN>$N_{1,Ed}$ Stresses (critical section) σ_1=-2.90 MPa<235 MPa σ_2=44.59 MPa<235 MPa τ=50.33 MPa<135.68 MPa
C1B6	SB13	SHS 350x350x12.5	
2L100x12	2L90x10	$N_{0,Ed}$=-289.87 kN	
θ=49.97°	θ=90°	$M_{0,Ed}$=-9.42 kNm	
4M16...8.8	2x2M12...8.8		
$N_{1,Ed}$=359.93 kN	$N_{2,Ed}$=-131.76 kN		
$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{eff,Rd}$=783.18 kN $V_{Rd,min}$=485.73 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.74<1$	$V_{v,Rd}$=304.60 kN $V_{b,Rd}$=606.00 kN $V_{gus,b,Rd}$ =363.60 kN $V_{Rd,min}$=304.60 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.43<1$	Gusset to chord (chord face failure) $N_{1\text{-}1,Ed}$=143.84 kN $M_{1\text{-}1,Ed}$=15.70 kNm N_{Rd}=264.98 kN $M_{ip,Rd}$=76.18 kNm Interaction=0.75<1	

Table B.84: Design checks - Joint 1 (connecting beam C2)

Joint 1 (connecting beam C2)		Geometry: N	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=645 mm Supported compression brace $N_{pl,Rd}$=683.82 kN>$N_{2,Ed}$ $N_{b,Rd}$= 665.60 kN>$N_{2,Ed}$ Supported tension brace $N_{pl,Rd}$=455.88 kN>$N_{1,Ed}$ $N_{u,Rd}$=421.94 kN>$N_{1,Ed}$ Stresses (critical section) σ_1=-45.30MPa<235 MPa σ_2=17.10 MPa<235 MPa τ=53.82 MPa<135.68 MPa
SB13	C2B1	SHS 350x350x12.5	
2L90x10	2L100x12	$N_{0,Ed}$=303.10 kN	
θ=90°	θ=44.25°	$M_{0,Ed}$=1.44 kNm	
2x2M12...8.8	4M16...8.8		
$N_{1,Ed}$=161.40 kN	$N_{2,Ed}$=-387.71 kN		
$V_{v,Rd}$=304.60 kN $V_{b,Rd}$=606.00 kN $V_{gus,b,Rd}$ =363.60 kN $V_{eff,Rd}$=767.18 kN $V_{Rd,min}$=304.60 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.53<1	$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{Rd,min}$=485.73 kN $\frac{N_{2,Ed}}{V_{Rd,min}}$=0.80<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-109.14 kN $M_{1-1,Ed}$=25.96 kNm N_{Rd}=279.67 kN $M_{ip,Rd}$=90.19 kNm Interaction=0.68<1	

Table B.85: Design checks - Joint 2 (connecting beam C2)

Joint 2 (connecting beam C2)		Geometry: K	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=940 mm Supported tension brace $N_{pl,Rd}$=683.82 kN>$N_{1,Ed}$ $N_{u,Rd}$=698.25 kN>$N_{1,Ed}$ Supported compression brace $N_{pl,Rd}$=683.82 kN>$N_{2,Ed}$ $N_{b,Rd}$= 665.60 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=47.05 MPa<235 MPa σ_2=-54.47 MPa<235 MPa τ=68.15 MPa<135.68 MPa
C2B1	C2B2	SHS 350x350x12.5	
2L100x12	2L100x12	$N_{0,Ed}$=-463.44 kN	
θ=44.25°	θ=44.25°	$M_{0,Ed}$=-4.69 kNm	
4M16...8.8	4M16...8.8		
$N_{1,Ed}$=-387.71 kN	$N_{2,Ed}$=327.63 kN		
$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{Rd,min}$=485.73 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.80<1	$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{eff,Rd}$=783.18 kN $V_{Rd,min}$=485.73 kN $\frac{N_{2,Ed}}{V_{Rd,min}}$=0.67<1	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-41.85 kN $M_{1-1,Ed}$=89.70 kNm N_{Rd}=341.57 kN $M_{ip,Rd}$=160.54 kNm Interaction=0.68<1	

Table B.86: Design checks - Joint 3 (connecting beam C2)

Joint 3 (connecting beam C2)		Geometry: KT	
Brace 1	Brace 2	Brace 3	Gusset plate design checks Length: h=980 mm Supported tension brace $N_{pl,Rd}$=683.82 kN>$N_{1,Ed}$ $N_{u,Rd}$=698.25 kN>$N_{1,Ed}$ Supported compression brace 1 $N_{pl,Rd}$=455.90 kN>$N_{2,Ed}$ $N_{b,Rd}$= $N_{pl,Rd}$ >$N_{2,Ed}$ Supported compression brace 2 $N_{pl,Rd}$=455.88 kN>$N_{3,Ed}$ $N_{u,Rd}$=443.86 kN>$N_{3,Ed}$ Stresses (critical section) σ_1=-21.45MPa<235 MPa σ_2=26.28 MPa<235 MPa τ=34.24 MPa<135.68 MPa
C2B2	SM14	C2B3	
2L100x12	2L90x10	2L90x10	
θ=44.25°	θ=90°	θ=41.42°	
4M16...8.8	2x2M12...8.8	2M12...8.8	
$N_{1,Ed}$=327.63 kN	$N_{2,Ed}$=-170.41 kN	$N_{3,Ed}$=-45.01 kN	
$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{eff,Rd}$=783.18 kN $V_{Rd,min}$=485.73 kN $\frac{N_{1,Ed}}{V_{Rd,min}}$=0.67<1	$V_{v,Rd}$=304.60 kN $V_{b,Rd}$=606.00 kN $V_{gus,b,Rd}$ =363.60 kN $V_{Rd,min}$=304.60 kN $\frac{N_{2,Ed}}{V_{Rd,min}}$=0.56<1	$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{Rd,min}$=152.31 kN $\frac{N_{3,Ed}}{V_{Rd,min}}$=0.30<1	
Chord SHS 350x350x12.5 $N_{0,Ed}$=591.43 kN $M_{0,Ed}$=39.69 kNm	Gusset to chord (chord face failure) $N_{1-1,Ed}$=28.43 kN $M_{1-1,Ed}$=45.84 kNm N_{Rd}=349.96 kN $M_{ip,Rd}$=171.48 kNm Interaction=0.35<1		

Table B.87: Design checks - Joint 4 (connecting beam C2)

Joint 4 (connecting beam C2)		Geometry: K	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=780 mm Supported compression brace 1 $N_{pl,Rd}$=227.94 kN >$N_{1,Ed}$ $N_{b,Rd}$=221.9 kN >$N_{1,Ed}$ Supported compression brace 2 $N_{pl,Rd}$=227.94 kN >$N_{1,Ed}$ $N_{b,Rd}$=221.9 kN >$N_{1,Ed}$ Stresses (critical section) σ_1=-16.21 MPa<235 MPa σ_2=-4.68 MPa<235 MPa τ=6.42 MPa<135.68 MPa
C2B3	C2B4	SHS 350x350x12.5	
2L90x10	2L90x10	$N_{0,Ed}$=-486.97 kN	
θ=41.42°	θ=41.42°	$M_{0,Ed}$=9.23 kNm	
2M12...8.8	2M12...8.8		
$N_{1,Ed}$=-47.13 kN	$N_{2,Ed}$=-100.59 kN		
$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{Rd,min}$=152.31 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.31<1$	$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{Rd,min}$=152.31 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.66<1$	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-97.73 kN $M_{1-1,Ed}$=7.14 kNm N_{Rd}=308.00 kN $M_{ip,Rd}$=120.12 kNm Interaction=0.38<1	

Table B.88: Design checks - Joint 5 (connecting beam C2)

Joint 5 (connecting beam C2)		Geometry: KT	
Brace 1	Brace 2	Brace 3	Gusset plate design checks Length: h=910 mm Supported compression brace 1 $N_{pl,Rd}$=227.94 kN >$N_{1,Ed}$ $N_{b,Rd}$=221.9 kN >$N_{1,Ed}$ Supported compression brace 1 $N_{pl,Rd}$=455.90 kN>$N_{2,Ed}$ $N_{b,Rd}$= $N_{pl,Rd}$ >$N_{2,Ed}$ Supported tension brace $N_{pl,Rd}$=683.82 kN>$N_{3,Ed}$ $N_{u,Rd}$=698.25 kN>$N_{3,Ed}$ Stresses (critical section) σ_1=34.02 MPa<235 MPa σ_2=-28.71 MPa<235 MPa τ=40.89 MPa<135.68 MPa
C2B4	SM14	C2B5	
2L90x10	2L90x10	2L100x12	
θ=41.42°	θ=90°	θ=49.97°	
2M16...8.8	2x2M12...8.8	4M12...8.8	
$N_{1,Ed}$=-99.29 kN	$N_{2,Ed}$=-171.07 kN	$N_{3,Ed}$=347.10 kN	
$V_{v,Rd}$=152.31 kN $V_{b,Rd}$=303.00 kN $V_{gus,b,Rd}$ =181.80 kN $V_{Rd,min}$=152.31 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.65<1$	$V_{v,Rd}$=304.60 kN $V_{b,Rd}$=606.00 kN $V_{gus,b,Rd}$ =363.60 kN $V_{Rd,min}$=304.60 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.56<1$	$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{eff,Rd}$=783.18 kN $V_{Rd,min}$=485.73 kN $\frac{N_{3,Ed}}{V_{Rd,min}}=0.71<1$	
Chord SHS 350x350x12.5 $N_{0,Ed}$=591.43 kN $M_{0,Ed}$=26.22 kNm	Gusset to chord (chord face failure) $N_{1-1,Ed}$=29.02 kN $M_{1-1,Ed}$=51.95 kNm N_{Rd}=335.27 kN $M_{ip,Rd}$=152.55 kNm Interaction=0.43<1		

Table B.89: Design checks - Joint 6 (connecting beam C2)

Joint 6 (connecting beam C2)		Geometry: K	
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=820 mm Supported tension brace $N_{pl,Rd}$=683.82 kN>$N_{1,Ed}$ $N_{u,Rd}$=698.25 kN>$N_{1,Ed}$ Supported compression brace $N_{pl,Rd}$=683.82 kN>$N_{2,Ed}$ $N_{b,Rd}$= 672.07 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=-66.78 MPa<235 MPa σ_2=58.42 MPa<235 MPa τ=73.32 MPa<135.68 MPa
C2B5	C2B6	SHS 350x350x12.5	
2L100x12	2L100x12	$N_{0,Ed}$=-427.33 kN	
θ=49.97°	θ=49.97°	$M_{0,Ed}$=-7.56 kNm	
4M16...8.8	4M16...8.8		
$N_{1,Ed}$=347.10 kN	$N_{2,Ed}$=-400.79 kN		
$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{eff,Rd}$=783.18 kN $V_{Rd,min}$=485.73 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.71<1$	$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{Rd,min}$=485.73 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.83<1$	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-41.11 kN $M_{1-1,Ed}$=84.18 kNm N_{Rd}=316.39 kN $M_{ip,Rd}$=129.72 kNm Interaction=0.78<1	

Table B.90: Design checks - Joint 7 (connecting beam C2)

Joint 7 (connecting beam C2)			Geometry: N
Brace 1	Brace 2	Chord	Gusset plate design checks Length: h=575 mm Supported compression brace $N_{pl,Rd}$=683.82 kN>$N_{1,Ed}$ $N_{b,Rd}$= 672.07 kN>$N_{1,Ed}$ Supported tension brace $N_{pl,Rd}$=455.88 kN>$N_{2,Ed}$ $N_{u,Rd}$=421.94 kN>$N_{2,Ed}$ Stresses (critical section) σ_1=17.44 MPa<235 MPa σ_2=-53.71 MPa<235 MPa τ=56.04 MPa<135.68 MPa
C2B6	SB15	SHS 350x350x12.5	
2L100x12	2L90x10	$N_{0,Ed}$=274.97 kN	
θ=49.97°	θ=90°	$M_{0,Ed}$=2.76 kNm	
4M16...8.8	2x2M12...8.8		
$N_{1,Ed}$=-400.79 kN	$N_{2,Ed}$=181.75 kN		
$V_{v,Rd}$=581.15 kN $V_{b,Rd}$=963.47 kN $V_{gus,b,Rd}$ =485.73 kN $V_{Rd,min}$=485.73 kN $\frac{N_{1,Ed}}{V_{Rd,min}}=0.83<1$	$V_{v,Rd}$=304.60 kN $V_{b,Rd}$=606.00 kN $V_{gus,b,Rd}$ =363.60 kN $V_{eff,Rd}$=767.18 kN $V_{Rd,min}$=304.60 kN $\frac{N_{2,Ed}}{V_{Rd,min}}=0.60<1$	Gusset to chord (chord face failure) $N_{1-1,Ed}$=-125.14 kN $M_{1-1,Ed}$=23.53 kNm N_{Rd}=264.98 kN $M_{ip,Rd}$=76.18 kNm Interaction=0.78<1	

Site joints

Table B.91: Site joints design checks (splices)

	Right/left side member (bottom chord)	Connecting beam (bottom chord)
Bolts	(4+4)M20	(4+4)M16
$F_{t,Rd}$ [kN]	141.1	90.43
N_{Ed} [kN]	403.55	303.10
M_{Ed} [kNm]	26.47	15.17
$N_{Ed,eff}$ [kN]	606.40	446.23
δ	0.78	0.82
a [mm]	45	45
b [mm]	40	40
a' [mm]	55	53
b' [mm]	42.5	44.5
K [1/MPa]	7.234	7.574
P_f [kN]	75.80	55.78
t_{min} [mm]	17.55	15.24
t_{max} [mm]	23.42	20.55
t_p [mm]	**22**	**18**
α	0.802	0.738
F_{Rd} [kN]	870.08	549.32
$N_{Ed,eff}/F_{Rd}$	**0.70**	**0.81**
α_{mod}	0.170	0.371
T_f [kN]	82.65	66.70
$T_f/N_{t,Rd}$	**0.59**	**0.74**
T_f/P_f	1.090	1.195
Prying [%]	9	19.5

For the upper chord splice joints, use the same layout and dimensions as for the bottom chord splices.

The upper chords are loaded in shear as well and the maximum acting force is $V_{Ed,max}$=164.20 kN.
One bolt is loaded with $F_{v,Ed}$=164.20/8=20.53 kN.
The design shear resistance of a M20 bolt is $F_{v,Rd}$=120.6 kN.
$F_{v,Ed}/F_{v,Rd}$=0.170<0.286 => No reduction of the tension resistance.

Table B.92: Weight (right/left side member-braces)

Member	Single/ double config.	Cross-section	Length [mm]	Unit weight [kg/m]	Total weight [kg]
SM1	2	L 90x90x10	1090	13.4	29.21
SM2	1	L 100x100x12	1374	17.8	24.46
SM3	2	L 100x100x12	1371	17.8	48.81
SM4	2	L 90x90x10	1090	13.4	29.21
SM5	2	L 100x100x12	1249	17.8	44.46
SM6	2	L 100x100x12	1249	17.8	44.46
SM7	2	L 90x90x10	1256	13.4	33.66
SM8	2	L 90x90x10	1256	13.4	33.66
SM9	2	L 90x90x10	1256	13.4	33.66
SM10	2	L 90x90x10	1256	13.4	33.66
SM11	2	L 130x130x12	1228	23.6	57.96
SM12	2	L 90x90x10	1090	13.4	29.21
SM13	2	L 130x130x12	1253	23.6	59.14
SM14	2	L 90x90x10	960	13.4	25.73
				Σ	527.30
				2·Σ	1054.61

Table B.93: Weight (right/left supporting beam-braces)

Member	Single/ double config.	Cross-section	Length [mm]	Unit weight [kg/m]	Total weight [kg]
SB1	2	L 90x90x10	1090	13.4	29.21
SB2	1	L 100x100x12	1335	17.8	23.76
SB3	2	L 90x90x10	1060	13.4	28.41
SB4	2	L 90x90x10	1335	13.4	35.78
SB5	2	L 90x90x10	1090	13.4	29.21
SB6	2	L 90x90x10	1255	13.4	33.63
SB7	1	L 100x100x12	1256	17.8	22.36
SB8	1	L 100x100x12	1256	17.8	22.36
SB9	1	L 100x100x12	1256	17.8	22.36
SB10	1	L 100x100x12	1256	17.8	22.36
SB11	1	L 100x100x12	1256	17.8	22.36
SB12	1	L 100x100x12	1256	17.8	22.36
SB13	2	L 90x90x10	990	13.4	26.53
SB14	2	L 130x130x12	1253	23.6	59.14
SB15	2	L 90x90x10	960	13.4	25.73
SB16	2	L 130x130x12	1366	23.6	64.48
SB17	2	L 90x90x10	1060	13.4	28.41
				Σ	518.43
				2·Σ	1036.87

Table B.94: Weight (connecting beam C1-braces)

Member	Single/ double config.	Cross-section	Length [mm]	Unit weight [kg/m]	Total weight [kg]
C1B1	2	L 100x100x12	1496	17.8	53.26
C1B2	2	L 100x100x12	1496	17.8	53.26
C1B3	2	L 90x90x10	1586	13.4	42.50
C1B4	2	L 90x90x10	1586	13.4	42.50
C1B5	2	L 100x100x12	1368	17.8	48.70
C1B6	2	L 100x100x12	1368	17.8	48.70
				Σ	288.93

Table B.95: Weight (connecting beam C2-braces)

Member	Single/ double config.	Cross-section	Length [mm]	Unit weight [kg/m]	Total weight [kg]
C2B1	2	L 100x100x12	1496	17.8	53.26
C2B2	2	L 100x100x12	1496	17.8	53.26
C2B3	2	L 90x90x10	1586	13.4	42.50
C2B4	2	L 90x90x10	1586	13.4	42.50
C2B5	2	L 100x100x12	1368	17.8	48.70
C2B6	2	L 100x100x12	1368	17.8	48.70
				Σ	288.93

Table B.96: Weight (chords)

Member	Cross-section	Length [mm]	Unit weight [kg/m]	Quantity	Total weight [kg]
Right side member	SHS 350x350x12.5	9933	127	2	2522.98
Left side member	SHS 350x350x12.5	9933	127	2	2522.98
Right supporting beam	SHS 350x350x12.5	12000	127	2	3048.00
Left supporting beam	SHS 350x350x12.5	12000	127	2	3048.00
Connecting beam	SHS 350x350x12.5	8650	127	4	4394.20
				Σ	15536.16

Table B.97: Weight (brackets)

Member	Cross-section	Length [mm]	Unit weight [kg/m]	Quantity	Total weight [kg]
Vertical	HEB 360	4360	142	4	2476.48
Diagonal-left	HEB 360	3134	142	2	890.06
Diagonal-right	HEB 360	3518	142	2	999.11
Horizontal1-left	HEA 800	3020	224	2	1352.96
Horizontal1-right	HEA 800	3580	224	2	1603.84
Horizontal2-left	2L 90x90x10	1645	26.8	2	88.17
Horizontal2-right	2L 90x90x10	2208	26.8	2	118.35
Longitudinal	IPE 550	2535	106	2	537.42
				Σ	8066.39

NOTE:
This Thesis does not analyze in detail joints between the brackets and trusses, sub-assemblies intended to fix roller bogies to the structure, sub-assemblies for fixing the platform columns and caissons, hence they are not given in detail here.

MATERIAL:
STEEL GRADE: S235
BOLT GRADE: 8.8

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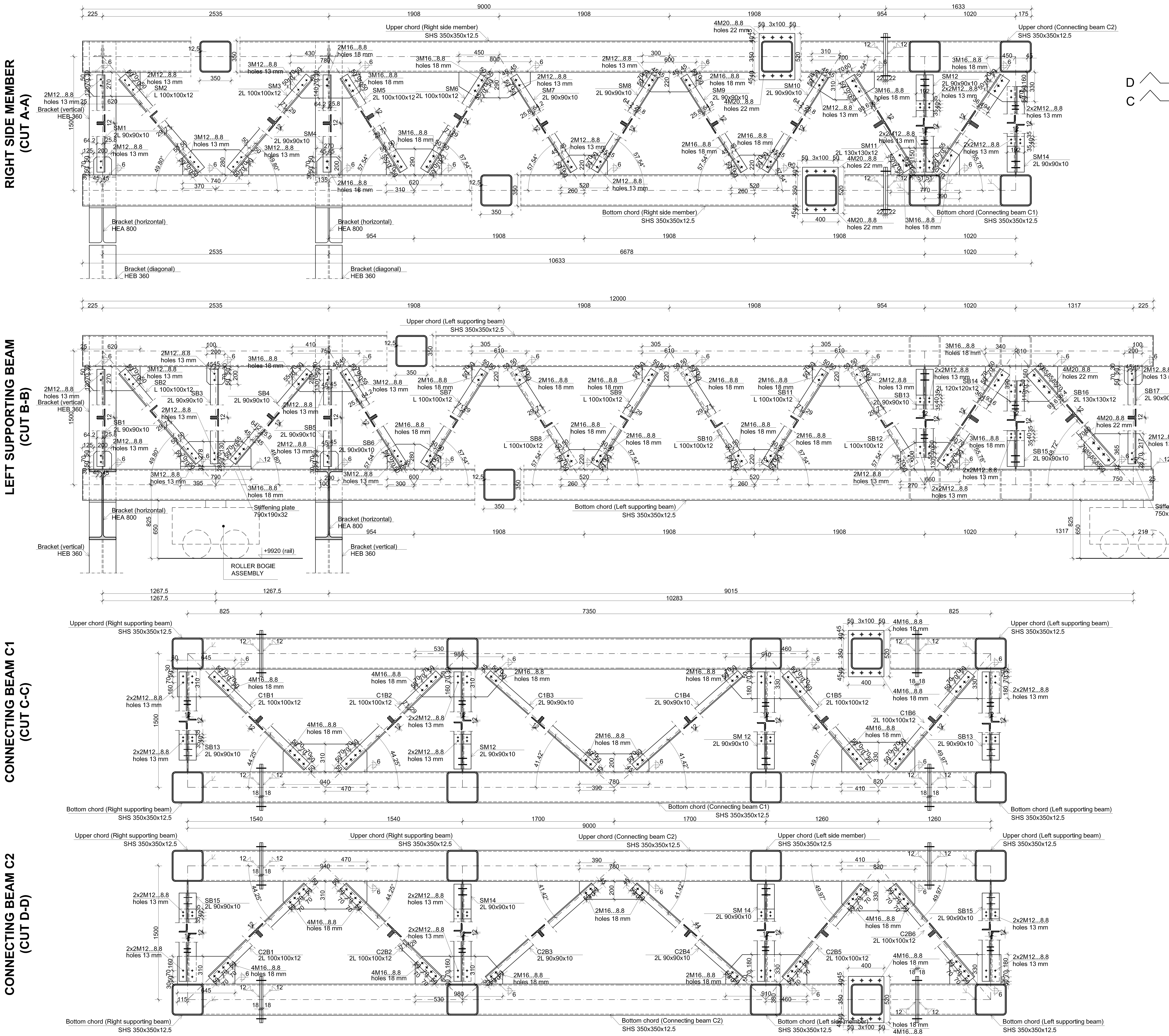
MASTER THESIS: OPTIMISATION OF THE TRANSFER CARRIAGE STRUCTURE		
DRAWING: **SOLUTION 6-1**	SCALE: 1:20	FORMAT: A1
STUDENT: VLADIMIR ŠKORIĆ	DATE: 06.01.2017	

RIGHT SIDE MEMBER (CUT A-A)

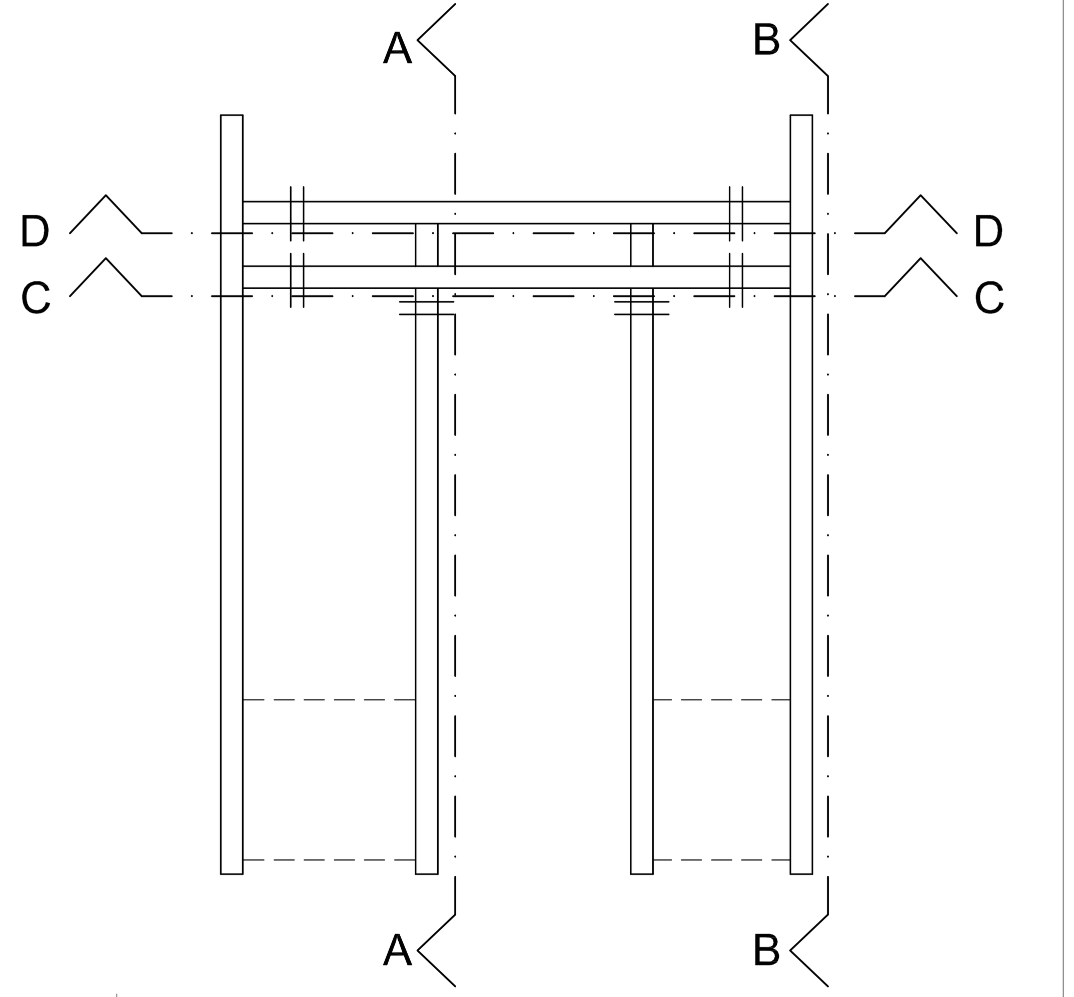
LEFT SUPPORTING BEAM (CUT B-B)

CONNECTING BEAM C1 (CUT C-C)

CONNECTING BEAM C2 (CUT D-D)



PLAN VIEW - SCHEME



NOTE:
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MATERIAL:
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DRAWING: **SOLUTION 6-2**	SCALE: 1:20	FORMAT: A1
STUDENT: VLADIMIR ŠKORIĆ	DATE: 06.01.2017	