Design of a steel structure for a large span roof with emphasis on the verification of bolted connections

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Abstract

The main objective of the thesis is the conceptual and detailed design of a steel structure for large span roofing by means of lattice girders.

These procedures include a conceptual analysis of a proposed roofing system (36x56 meters) as well as the detailed checking of the members and connections in accordance to *EN 1993.* For the purpose of analysis, the structure is modelled with the software SAP2000 as a series of 2D structures, effectively simulating the path of forces in the structure.

Regarding the connections, focus is given to detailed design under ultimate limit state of gusset plates as well as spliced plate connections used for chord continuity. Serviceability is evaluated in terms of overall deflection and taking into account the effects of slack recovery.

Key-words: Lattice girders, bolted joints, large span roofs

Resumo

O trabalho tem por objetivo a conceção e dimensionamento de uma estrutura metálica para uma cobertura de grande vão utilizando estruturas reticuladas de aço.

Para o efeito, o trabalho envolve a conceção duma estrutura triangulada, com perfis laminados a quente, para cobertura dum vão grande (36x56) e, posteriormente, a verificação da segurança dos principais elementos, sistema de contraventamento e ligações, assim como do estado limite de deformação de acordo com a *EN 1993*. Por forma a ter em conta o encaminhamento das cargas nos vários elementos estruturais, foi elaborada uma sequência de modelos 2D usando o programa de cálculo SAP2000.

Neste contexto, é dado destaque ao dimensionamento e pormenorização de juntas com ligações aparafusadas e recurso a chapas *gousset*; em relação ao estado limite de deformação, é avaliado o efeito das folgas no caso das ligações aparafusadas.

Palavras chave: Estruturas reticuladas, juntas aparafusadas, coberturas de grande vão

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Symbols

Chapter 3

A	gross cross-section area of bolts
d	nominal bolt diameter
d_0	hole diameter for bolts
E	modulus of elasticity
f_y	yield strength for structural steel
f_u	ultimate strength for structural steel
f_{yb}	yield strength for bolts
f_{ub}	ultimate strength for bolts
G	shear modulus

Chapter 4

C_{dir}	directional factor
C_e	exposure coefficient
C_{season}	season factor
C_t	thermal coefficient
C_z	coefficient (from *NP EN 1991-1-3*)
c_0	orography factor
c_r	roughness factor
E_d	design value of effect of actions
G_k	characteristic value of a permanent action
H	height of the roof above ground
I_v	turbulence intensity
k_I	turbulence factor
k_r	terrain factor
Q_k	characteristic value of a concentrated load
q_b	basic velocity pressure
q_k	characteristic value of a uniformly distributed load
q_p	peak velocity pressure
s	snow load on the roof
s_k	characteristic value of snow load on the ground at the relevant site
v_b	basic wind velocity
$v_{b,0}$	fundamental value of the basic wind velocity
v_m	mean wind velocity
z	height above ground
z_0	roughness length

$z_{0,II}$	terrain category II
μ_1	shape coefficient for snow loads
γ_g	partial factor for permanent action
γ_q	partial factor for variable action
ψ	factor for combination value of an action

Chapter 5

x-x	axis along a member
y-y	major axis of a cross-section
z-z	minor axis of a cross-section
v-v	minor axis of a cross-section (where this does not coincide with z-z)
A_a	area of the angle's cross-section
A_{eff}	effective cross-section area
A_{net}	net area of angle
$b_{,eff}$	effective width
C_{ay}	equivalent uniform moment factor
C_{mz}	equivalent uniform moment factor
C_{mLT}	equivalent uniform moment factor
e_0	maximum amplitude of a member imperfection
e_N	shift of the centroid of the effective area relative to the original center of gravity
I_a	moment of inertia of the angle about the relevant axis
k_{yy}	interaction factor
k_{zy}	interaction factor
k_σ	buckling factor for plates
L_{cr}	buckling length
$M_{a,Ed}$	bending moment on the angle
$M_{g,Ed}$	bending moment on the gusset
$M_{pl,Rd}$	design plastic resistance to bending about the relevant axis
m	number of braced elements
$N_{a,Ed}$	axial force on the angle
$N_{b,Rd}$	design buckling resistance of a compression member
$N_{c,Rd}$	design resistance of the net cross-section for uniform compression
N_{cr}	elastic critical force for the relevant buckling mode based on the gross cross section
$N_{g,Ed}$	axial force on the gusset
$N_{pl,Rd}$	design plastic resistance to normal forces of the gross cross-section
$N_{t,Rd}$	design value of the resistance to tension force
$N_{u,Rd}$	design ultimate resistance to normal forces of the net cross-section
Q_p	equivalent force
q_d	equivalent force per unit length
$V_{a,Ed}$	shear force on the angle

$V_{g,Ed}$	shear force on the gusset
W_{eff}	effective elastic section modulus
σ_a	normal stress on the angle
σ_g	normal stress on the gusset
α_m	reduction factor to assess the equivalent stabilizing load
γ_{M0}	partial factor for resistance of cross-sections whatever the class is
γ_{M1}	partial factor for buckling resistance of members
γ_{M2}	partial factor for resistance of cross-section in tension
Ψ	stress ratio
ϕ_i	value to determine the imperfection factor χ
$\bar{\lambda}_i$	non dimensional slenderness about the relevant axis
$\bar{\lambda}_p$	plate slenderness
ρ	reduction factor for plate buckling
χ_i	reduction factor about the relevant axis

Chapter 6

$A_{f,net}$	net area in the flange component
A_g	area of the gusset's cross section
A_{nn}	net area subjected to shear
A_{nt}	net area subjected to tension
$A_{p,net}$	net area in the plate component
$A_{w,net}$	net area in the web component
a_{min}	minimum throat thickness
b_i	number of the bolt
e_1	end distance from the center of bolt hole to the adjacent end of any part
e_2	edge distance from the center of bolt hole to the adjacent edge of any part
e_f	eccentricity from the edge of the flange to the center of the bolt group
$F_{b,Rd}$	design bearing resistance per bolt
$F_{M,bi}$	design shear force on each bolt due to the acting moment
$F_{M,bi,h'}$	component along axis h' of the design shear force on each bolt due to the acting moment
$F_{M,bi,v'}$	component along axis v' of the design shear force due on each bolt to the acting moment
$F_{N,bi}$	design shear force on each bolt due to axial force on the gusset~
$F_{p,C}$	design value of pre-loading force
$F_{s,Rd}$	design slip resistance per bolt
$F_{V,bi,Ed}$	design shear force on each bolt
$F_{V,bi,h',Ed}$	component along axis h' of the design shear force on the gusset due to axial force
$F_{V,bi,h,Ed}$	component along axis h of the design shear force on each bolt
$F_{V,bi,v',Ed}$	component along axis v' of the design shear force on the gusset due to axial force
$F_{V,hi,v,Ed}$	component along axis v of the design shear force on each bolt
$F_{V,Rd}$	design shear resistance per bolt

$F_{V,Ed,w}$	design shear force acting on the bolt group on the web
$F_{V,Ed,p}$	design shear force acting on the bolt group on the flange
F_w	design value of the weld force per unit length
$F_{w,h}$	design value of the weld force per unit length, horizontal component
$F_{w,v}$	design value of the weld force per unit length, vertical component
$f_{vw,d}$	design value of the shear strength of the weld
$h_{i'}$	distance from the center of gravity of the bolt group to bolt b_i along the axis h'
I_g	moment of inertia about the minor axis of the gusset cross section
K	Thornton factor
M_f	design bending moment acting on the flange of the chord in a spliced connection
N_f	design force acting on one flange of the chord in a spliced connection
$N_{f,net,Rd}$	design force acting on the net area of the flange component
$N_{i,g,bt,Ed}$	design acting normal force on the gusset.
$N_{p,net,Rd}$	design force acting on the net area of the plate component
N_w	design force acting on the web of the chord in a spliced connection
$N_{w,net,Rd}$	design force acting on the net area of the web component
n	number of friction surfaces
n_b	number of bolts resisting in a line of action for net cross-section
n_{bt}	total number of bolts in a connection for net cross-section resistance
p_1	spacing between the centers of bolts in a line in the direction of load transfer
p_2	spacing perpendicular to the load transfer direction between adjacent lines of bolts
$r_{i'}$	distance from the center of gravity of the bolt group to bolt b_i
t	thickness of the plate
$V_{eff,1,Rd}$	design block tearing resistance for a symmetric bolt group subjected to concentric loading
$V_{eff,2,Rd}$	design block tearing resistance for a symmetric bolt group subjected to eccentric loading
V_f	design shear force acting on the flange of the chord in a spliced connection
$v_{i'}$	distance from the center of gravity of the bolt group to bolt b_i along the axis v'
z_g	vertical distance from the centroid of gusset cross section to the outmost fiber
τ_g	elastic shear stress in the gusset cross section
μ	slip factor

1. Introduction

1.1 General historical overview

In every structural engineer's first course in statics the concepts needed to analyse statically determinate structures are defined. Apart from the simply supported beam, the truss stands as the backbone of structural engineering.

The concepts needed to analyse these structures were largely developed in the seventeenth and eighteenth century by the likes of Galileo, Stevin, Newton, Varignon, Bernoulli, Euler, Lagrange and others.

It was in France, during the nineteenth century, that advanced mathematical and scientific concepts related with civil engineering began to be taught, to the *Ingéniurs* of *Ecole des Ponts et Chaussées* and *Ecole Polytechnique*. One can find the first mathematical analyses of trusses in Navier's 1826 *"Résumé de Leçons Données à L'Ecole des Ponts et Chaussées sur l'Appication de la Mécanique"* [1]. Navier determined the forces in simple statically determined trusses as well as in statically indeterminate trusses, but it was the analogy between forces in beams and forces in chords that perhaps had the greatest influence on the design of trusses. By noticing that trusses with parallel chords could be treated as beams with stiffness proportional to the area of both chords multiplied by the distance between them, the formulas of Navier greatly expanded design practices and were later incorporated in the 1830's and 1840's designs of American wooden truss bridges.

Indeed, engineers such as Stephen Long, William Howe, James Warren, Thomas Pratt and many others greatly understood the teachings of Navier and successfully applied them both in timber and iron structures. Further dissemination of Navier's work in the United States is due to Dennis Mahan, who in 1837 published his textbook "*An Elementary Course of Civil Engineering for Use of the Cadets of the United States Military Academy"* and states in the introduction that "the best counsel that the author could give to every young engineer, is to place in his library every work of science to which Mr. Navier's name is in any way attached" [1].

In Europe and in the United States, trusses were first adopted as roofs structure rather than bridges. In France, Camille Polonceau patented a truss in 1837, displayed in Figure 1 (left), that was used in the terminals for the railroad from Paris to Versailles [1]. In Britain, a lasting example of roof truss design of this period is Robert Stephenson's locomotive roundhouse, Figure 1 (right), designed for the Birmingham railway.

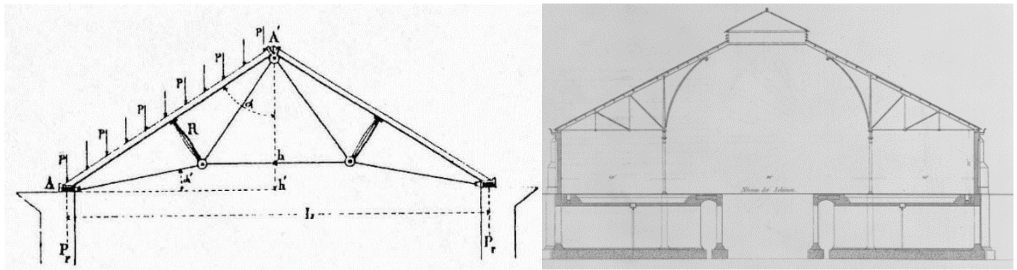


Figure 1 – Camille Polonceau truss (left); Robert Stephenson's locomotive roundhouse (right)

In its essence, a truss is framed structure in which members are connected at their ends forming a triangulated system, arranged in a pre-determined pattern depending on the span, type of loading and general function. The members are subjected to essentially axial forces due to externally applied loads at each node. Where these loads lie in the same plane one may consider a plane truss, or where loads may act in any direction, in which case one should consider space trusses so that members can be oriented in three dimensions. From a theoretical standpoint, the members are assumed to be connected to the joints so that rotation is permitted, and thereby it follows from equilibrium that the individual structural members act as bars – carrying solely axial force either in compression or tension. Often, joints are detailed such that free rotation does not occur, in which case the hinged property of the joint is an assumption. Even if so, the approximation is valid - to be discussed further on – which greatly simplifies the manual analysis of the forces in the structure and undoubtedly contributed for their popularity in bridges and roof structures, and later in cranes, offshore structures, high rise buildings and many other.

1.2 Main objectives and framework

The main objective, as stated in the Abstract is the conceptual and detailed design of a steel structure for large span roofing by means of lattice girders.

In chapter 1, a historical overview of truss structures is outlined. This serves as an introduction to how these type of structures emerged in our society and why they became so popular.

In chapter 2, general aspects relating to geometry and type of cross sections used for trusses in single story buildings are discussed.

In chapter 3, starts by presenting a design layout for the roof structure. Further discussion follows, where general considerations of the type of connections existing in the structure are made.

In chapter 4 the loads and finite element model are determined and a brief analysis of the effect of slack is considered.

In chapter 5 and 6 the safety checking of the elements and connections are carried out.

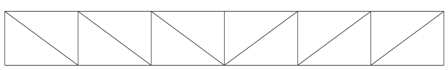
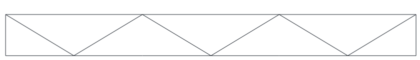
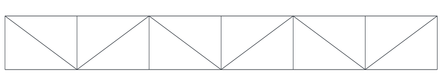
2 Trusses in single story buildings

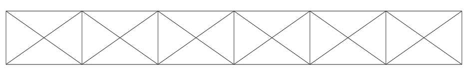
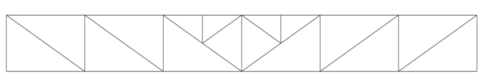
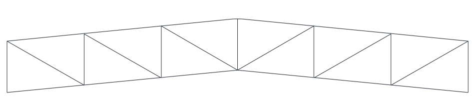
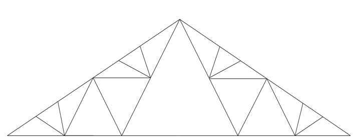
2.1 Main functions

In single story buildings, namely industrial buildings, airplane hangars, sports pavilions, stadiums etc., trusses are usually used for two main purposes. First, to provide a path by which the loads (gravity, wind etc.) can discharge on the columns. Second, to provide lateral stability to the series of portal trusses. These are known as bracing trusses, which can be longitudinal or transverse to the series of portal trusses; the bracing systems can be introduced on the roof and in the side walls such that the loads can find a viable path to discharge at the foundation.

2.2 Truss layouts

There are many possible layouts for trusses in single story buildings. A short list is presented bellow outlining the main attributes of each one.

	Pratt truss [1) 2)] : Regarded as cost effective structures, these trusses are used where gravity loads are dominant, as such all the diagonal members are in tension and the posts are in compression. For predominant uplift forces, as it may be the case in open buildings, inverted diagonals are used and the resulting force in them is tension.
(a) Original Warren truss  (b) Modified Warren truss 	Warren truss[1) 2)]: The diagonals in these trusses are alternatively in compression and in tension, providing a good solution for distributed loads. Where vertical posts are not used (a), these trusses have about half the number or joints and brace members when compared with Pratt's solution. Where vertical posts are used (b) a few observations should be noted. First these additional vertical members exist mainly to control high compression of the chords as it reduces the buckling length of these members. Secondly they provide a path of for loading by purlins that can exist at intermediate points. In

	the case were these intermediate purlins do not exist, these additional vertical posts will have zero axial force. If CHS/RHS members are used there are considerable opportunities to use gap joints as the layout is more open. Additionally, these members provide good resistance to compression.
	X truss[1) 2)]: These trusses are commonly used as wind girders. One can design such structures considering the diagonals as compression resistant, in which case the truss is a superposition of two Warren trusses, or alternatively by ignoring the members in compression in which case the behaviour is that of the Pratt truss.
	Additional members[1) 2)]: Additional members are adopted primarily to reduce the buckling length of members in compression. Another reason for these additional members is the added loading points that are established therefore avoiding additional bending to the chords.
	Slope[1) 2) 3)]: For all of the above structures, slope may be provided to fit architectural demands or to guarantee the drainage of the upper cladding. Either simple or double slope may be provided to the upper chord.
	Fink truss[3)]: The most common use of this type of trusses is in the roof structure of residential low density housing.

1) May be used either in portal trusses – transmitting bending moments to the columns – or in simply supported trusses.

2) Adequate for spans that range from 20 to 100 meters. [3]

3) Adequate as simply supported and with spans that range from 10 to 15 meters.

2.3 Roof structures

2.3.1 General geometry

Focusing on roof structures, those that span more than 20 – 25 meters are often more economical if designed as trusses instead of portal frames [2]. Savings stem from the fact that trusses are lighter, using less steel, than solid profiles. Indeed, for the same weight, better performance in both of resistance and stiffness is managed when considering trusses. Although aesthetics is a matter of taste, the general consensus is that trusses are of a superior appearance when compared to portal frames. However, relating to the installation process, hot rolled beams are less time consuming as they have much fewer connections. For a cost effective truss, the engineer has to balance several aspects such as equipment, man-hours, and cost of steel.

As with beams, the ratio depth to span of flat trusses at mid span, otherwise known as slenderness, should range from 1/7.5 to 1/12 [2] so that good structural performance, regarding deflection and forces on each element, is achieved. Moreover, efficient layouts should consider point loads applied only at nodes with diagonals connecting with chords at 35º to 55º. The reason behind these two numbers is a simple one. As the inclination of each diagonal increases (becoming more vertical) so will the number of total diagonals in the truss. Thus, for the same loading, the axial force on each element will decrease making the case for savings by means of a less robust cross-section. Evidently, the validity of this line of thought breaks down when the total number of additional diagonals and connections result in such additional cost that the savings in material are outweigh.

2.3.2 Cross-sections of members

There are two main families of cross-sections used in truss members: open sections and closed sections.

Open sections offer greater ease to establish connections as they require little to no welding, resorting primarily to bolts. For small to intermediate spans, a popular design is using single angles for diagonals and T profiles for chords. In this choice of design, it is recommended that vertical and diagonal members be placed on the same side of the T section as to avoid additional bending of the web and twisting of the chords [2]. For large spans and member forces, a popular design is using double angles or channels back-to-back spaced intermediately with battens for the diagonal members and I or H profiles (i.e. IPE, HEA, HEB) for the chords. The chords can be placed either vertically (standing up) or horizontally (flat). In both layouts there are advantages to be noted. First, in the horizontal layout the obvious advantage is that, for chords in compression, it is easier to increase in-plane buckling resistance, by shortening the buckling length by means of additional diagonals, than to increase out-of-plane buckling resistance. In the vertical orientation, the advantage stems from the fact that it is easier to establish a connection between the purlins and chord.

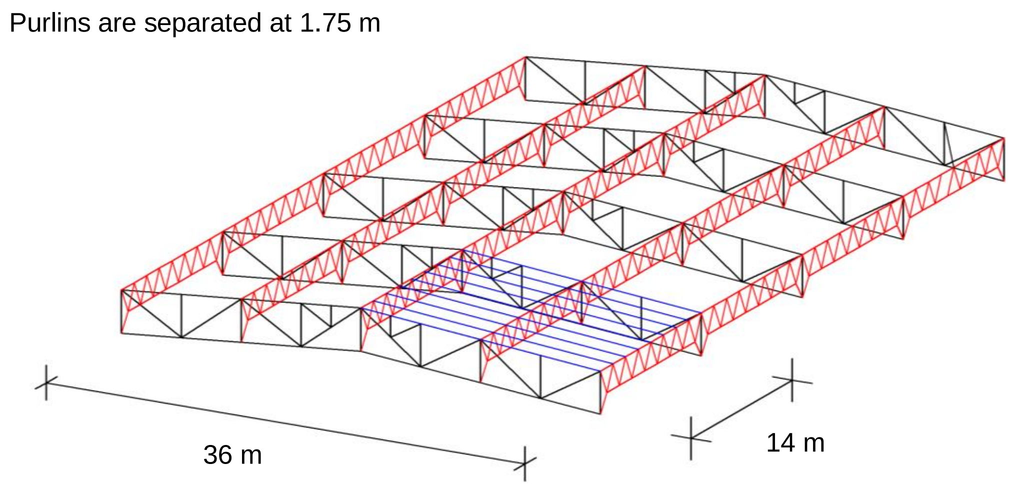
Closed sections have several advantages that need mentioning. Primarily, CHS and RHS sections are much more efficient cross-sections under compression when compared with open cross-sections. The radius of gyration is the same in all directions, hence greater efficiency. They are considered to be more aesthetic and are generally better appreciated by the public at large. As maintenance is regarded, tubular trusses require less paint per linear meter [2], reducing the cost of the corrosion protection treatment

3. Adopted solution

3.1 General overview

In deciding the appropriate layout of a roof structure the major problem is to find the right balance between economy and structural efficiency. There is little difficulty in assuming a truss structure instead of a I beam as it has already been noted that with increasing spans the latter become less efficient. Other questions arise such as what is the ideal spacing of the main trusses? What is the best slope of the chords and should both have the same slope? What is the best layout for the different truss? Is it preferable to have transverse or longitudinal purlins? What is the best type of cross-section to adopt?

A solution is presented in this thesis, Figure 2, bearing the principles outlined in the previous section and giving an answer to the issues mentioned above, although not in an exhaustive manner. The global bracing system is not indicated in Figure 2 as it is not evaluated in the document. Figures 3 and 4 display the layout of the main truss and bracing truss with the respective dimensions.



Purlins (blue); Bracing truss (red); Main truss (black)

Figure 2 – General layout of the roof structure

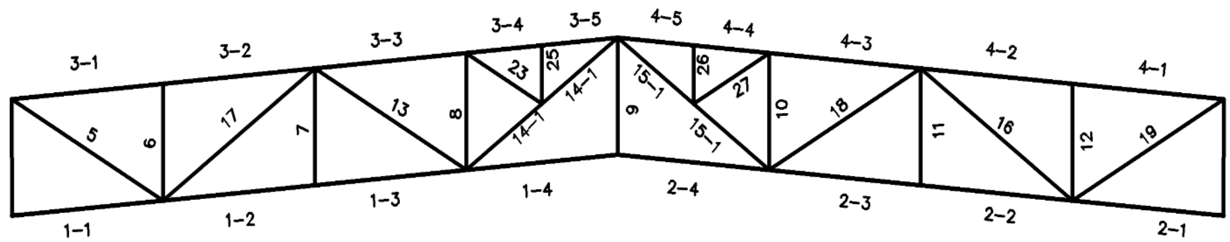


Figure 3 – Main truss and member numbering

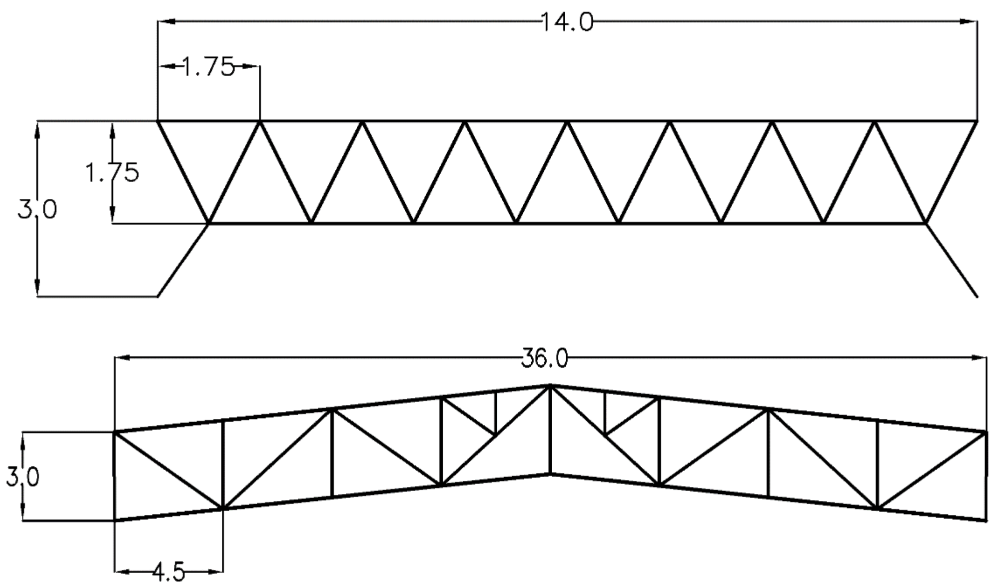


Figure 4 – Layout of the bracing truss (upper) and main truss (lower); (Dimensions in [m])

3.2 Members and materials

All structural steel members, including gusset plates, have the same grade of steel.

Table 1 – Considered steel properties

Members	Type	f_y [Mpa]	f_u [Mpa]	E [Gpa]	G [Gpa]
Structural Steel	S 355	355	510	210	81

The members that make up the structure are summarized in Table 2. As is shown in the table, all the diagonals of the bracing truss have the same profile and the same is true for the main truss. The main reason for this decision is to reduce the complexity of the installation on-site. It would be possible to adjust the robustness of the profiles according to the internal forces but so has not been done.

Table 2 – List of members of the roof structure

Structure	Members	Profile
Cladding	-	TR 45.333.1000 Negative
Purlins	-	IPE 160 (vertical)
Bracing truss	Upper & Lower chord	IPE 160 (Flat)
	Diagonals	L 100x100x10
Main truss	Upper chord	IPE 600 (Flat)
	Lower chord	IPE 400 (Flat)
	Diagonals	2L 150x150x15

The connections are established by welding and bolting. For the latter, depending on where the connection is, several types of bolts are adopted so to best fit the needed resistance.

Table 3 – List of bolts used in the connections

	Type & Class	f_{yb} [Mpa]	f_{ub} [Mpa]	d [mm]	d_0 [mm]	A [mm^2]
Bolts	M 20 cl. 10.9	900	1000	20	22	245
	M 24 cl. 10.9	900	1000	24	26	353
	M 27 cl. 10.9	900	1000	27	30	459

3.3 Connections

3.3.1 General overview

Connections are perhaps the most critical of parts in the design process. Indeed often enough, the cause of structural failure is due to poorly designed and detailed connections [3]. Modern steel structures are connected by welding or bolting – either high-strength or standard. Rivets were common in the past, but since the publication in 1951 of the first specification from the Council of Riveted and Bolted Structural Joints authorizing the substitution of rivets for high strength bolts, their use has plummeted [3].

The choice between welding and bolting depends on several factors. A possible shortlist includes customer acceptance, cost of both material and installation/execution, and safety. Welding and bolting have their advantages that should be considered in the design process.

Table 4 – Advantages of welded and bolted connections

Welded	Bolted
Less sorting of materials and reduction in installation cost	Saving in transportation outweigh additional installation costs
Less staging area required	Reduced manufacturing cost
Less hardware and reduced chance of short shipments as fewer components are involved	Easier to reconfigure and repair
Defects in manufactured frame braces are discovered in shop when welding is applied	Easier to install on-site
Seismic base plates allow for column placement at ground level	Easier to dismantle

The adopted structural design has several types of connections, these can be summarized as follows:

Table 5 – List of connections in the roof structure

Connecting members	Type
Purlin to bracing truss	Bolted and welded
Chord continuity in both the bracing and main truss	Spliced plate with bolts
Gusset to Chord	Welded
Diagonals to Gusset	Bolted
Main truss to columns	Bolted

Despite the interest in analysing all of the above, only the continuity chord connection, gusset to chord and diagonals to gusset will be fully analysed in this document. Although not fully analysed, a brief discussion on particular aspect of the connection of the main truss to the columns follows.

3.3.2 Main truss to columns

The main truss is designed as simply supported on the columns. So, one of the chord members could be omitted (namely the first lower chord member, with the arrangement of diagonals as shown in this case); however, it is advantageous to keep this member at the connection of the truss to the column in order to supply lateral stability to the lower chord of the truss. Thus, in order to enable the global in-plane rotation, the connection of one of the chords to the column must allow for relative horizontal displacement. Usually, the horizontal displacement is released at the node where the diagonal does not meet – in this case, the lower node.

In the truss being studied, the horizontal displacement in node B shown in Figure 5 (with no member 1-1 in the structural model) due to the gravity loads is +36 mm. Thus, a possible solution for the connection of member 1-1 to the column is as shown in Figure 5, comprising a plate welded to the column with a hole that has enough length (say, 50 mm) to accommodate the expected displacement.

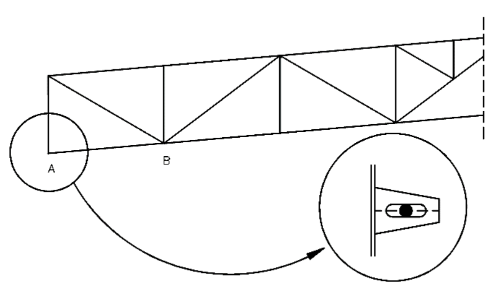


Figure 5 – Connection concept for the lower chord

3.3.3 Continuity of the chords

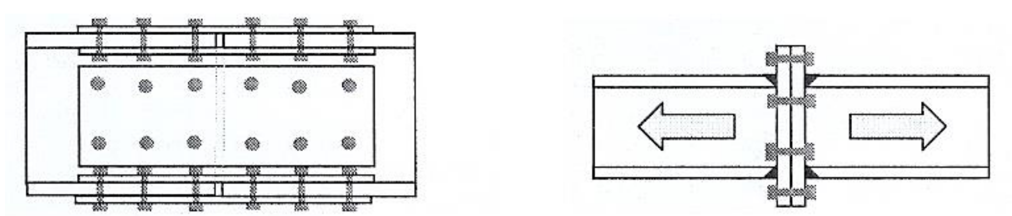
When designing large spans, one has to consider the maximum length of the members provided by the fabricator. These typically limit the length at about 12 meters due to the nature of the transportation method – trucking has limited allowable length, therefore limiting the length of profiles to be transported.

The proposed structure has a 36 meter span and therefore, in order to guaranty continuity, the connection has to be rigid and there are several options that can be considered.

Bolted connections are usually adopted instead of welded, the reason being that welded connections need a greater control in quality, and so better efficiency is achieved in shop rather than on site. Two types of bolted connections are possible with different implications: end-plate and splice-plate connections. End-plate connections are possible for I, H, and hollow profiles. Here, bolts are in tension and, with increasing force, the transverse plates will tend to bend in a complex three-dimensional manner. A simplified approach may be considered in the analysis of such connections, based on the so-called 'equivalent T-stub model'. Splice-plate connections are generally used for I, H, T, L and U profiles. The main difference from the end-plate type is that bolts are loaded with shear instead of tension.

In the adopted solution, splice-plates are considered.

Figure 6 – Spliced connection (left); end-plate connection (right)



3.3.4 Diagonals to chords

Depending on the assumptions considered in modelling the structure, as well as on the type of profiles chosen as diagonals, welding and bolting may be considered. It is common to use gusset plates as additional elements in the structure to assist in connecting diagonals to chords. These plates may be bolted or welded to the chords and the diagonals may as well be bolted or welded to the gusset.

For chords that are of T or U profiles a typical connection is shown in Figure 7 - left, with the gusset connected to the chord with bolts.

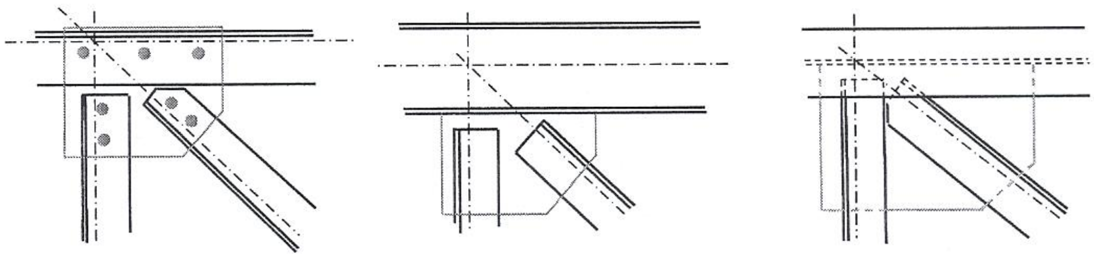


Figure 7 – Bolted gusset to T chord (left); welded gusset to H or I chord flange (centre); welded gusset to H or I chord web (right)

For chords that are I or H profiles the gusset is typically connected to these through welding and the diagonals may be bolted or welded to the chords (Figure 7 - centre and right).

The chords may be arranged vertically (standing up) or horizontally (flat), with the gusset connecting to the flange or web respectively. The discussion on the implications of a vertical or flat layout of the chords is provided further in section 4.2.2.

Relating to the gusset plate design and analysis, *EN1993* does not give any specific indication on safety checking of these members. In mid twentieth century, Whitmore and Thornton developed methods for analysing cross-sectional resistance as well as buckling of gusset plates that are adopted in this document.

4. Design Loads and Modelling

4.1 Loads

The only loads considered are the dead, live, wind and snow loads. Temperature has been opted out as the structure is modelled as a series of 2D statically determinate structures with slotted holes in the connections.

4.1.1 Dead Load (DL)

The main components of DL on roof trusses in single story industrial buildings are the self-weight of the following elements: cladding, purlins, chords, diagonals and connection elements such as bolts and gusset plates.

4.1.2 Live load (LL)

The gravity load due to maintenance is regarded as the main LL on roof trusses. In accordance to *EN 1991-1-1*, the roof is of category H and as such the characteristic value is defined in Table 6.

Table 6 – Live loads on roof category H

q_k [kN/m^2]	Q_k [kN]
0.4	1

4.1.3 Snow load (SN)

Snow loads are quantified with the assumptions indicated in Table 7, according to *NP EN 1991-1-3*.

Table 7 – Evaluation of the characteristic snow load

s	2.24	[kN/m^2]
c_e	1	
c_t	1	
μ_1	0.8	

s_k	2.8	[kN/m^2]
C_z	0.1	
H	500	[m]

4.1.4 Wind load (WL)

Given the slope of 5º, and in accordance to *EN 1991-1-4*, the predominant wind load on the roof truss is uplift force perpendicular to the roof, due to the suction effect of the wind blowing over. Hence, the wind loads act contrary to gravity loads and with greater magnitude. To illustrate this result, Table 9 and Table 10 provide the design wind pressures in the roof (with the wind velocity and the division into zones according to *NP EN 1991-1-4*) already taking into account the results of Table 8 and the internal pressures.

Table 8 – Evaluation of the characteristic wind pressure

Basic wind velocity	
V_b [m/s]	27
c_{dir}	1
c_{season}	1
$v_{b,0}$ [m/s]	27

Mean wind velocity	
V_m [m/s]	21.45
c_r	0.79
c_o	1
V_b [m/s]	27
k_r	0.22
z [m]	12
z_0	0.3
$z_{0,II}$	0.05

Peak velocity pressure	
q_p [kN/m^2]	0.83
I_v	0.2711
K_I	1

Exposure coefficient	
c_e	1.829
q_b [kN/m^2]	0.456
q_p [kN/m^2]	0.834

Table 9 – Wind pressure (in kN/m^2) for each zone of the roof, for wind direction θ=0º and 5º slope

	F		**G**		**H**		**I**		**J**	
	Cp,10	Cp,1	**Cp,10**	Cp,1	**Cp,10**	Cp,1	**Cp,10**	Cp,1	**Cp,10**	Cp,1
Internal (-)	-1.58	-2.25	-1.17	-1.83	-0.67	-1.17	-0.67	-0.67	0.00	0.00
	-0.17	-0.17	-0.17	-0.17	-0.17	-0.17	-0.67	-0.67	-0.67	-0.67

	F		**G**		**H**		**I**		**J**	
	Cp,10	Cp,1	**Cp,10**	Cp,1	**Cp,10**	Cp,1	**Cp,10**	Cp,1	**Cp,10**	Cp,1
Internal (+)	-1.17	-1.83	-0.75	-1.42	-0.25	-0.75	-0.25	-0.25	0.42	0.42
	0.25	0.25	0.25	0.25	0.25	0.25	-0.25	-0.25	-0.25	-0.25

Table 10 - Wind pressure [in kN/m^2] for each zone of the roof, for wind direction θ=90º and 5º slope

	F		**G**		**H**		**I**	
	Cp,10	Cp,1	**Cp,10**	Cp,1	**Cp,10**	Cp,1	**Cp,10**	Cp,1
Internal (-)	-1.50	-2.00	-1.25	-1.83	-0.75	-1.17	-0.67	-0.67

	F		**G**		**H**		**I**	
	Cp,10	Cp,1	**Cp,10**	Cp,1	**Cp,10**	Cp,1	**Cp,10**	Cp,1
Internal (+)	-1.08	-1.58	-0.83	-1.42	-0.33	-0.75	-0.25	-0.25

4.1.5 Load Combinations

The load combinations are summarized in Table 11 and Table 12, in accordance to *EN 1990.* As the temperature is not considered in the model the partial safety factors are not indicated.

<u>Ultimate Limit state (ULS)</u>

$$E_d = \sum_{i=1}^{m} \gamma_{gi} G_{i,k} + \gamma_q \left[Q_{i,k} + \sum_{j=2}^{n} \Psi_{0j} Q_{j,k} \right]$$

Table 11 – Summary of the partial factors for ULS

Combination	DL	LL	W	S
Live Load	1.35/1.0	1.5/0.0	0.6/0.0	0.5/0.0
Wind	1.00/0.0	-	1.5/0.0	0.5/0.0
Snow	1.35/0.0	-	0.6/0.0	1.5/0.0

<u>Serviceability limit state (SLS)</u>

The frequent combination is adopted to verify deflection.

$$E_d = \sum_{i=1}^{m} \gamma_{gi} G_{i,k} + \gamma_q \left[Q_{i,k} + \sum_{j=2}^{n} \Psi_{0j} Q_{j,k} \right]$$

Table 12 – Summary of the partial factors for SLS (frequent combinations)

Combination	DL	LL	W	S
Live Load	1	1	0.2	0.2
Wind	1	-	1	-
Snow	1	-	0.2	1

4.2 Modelling

4.2.1 General overview

As outlined in chapter 1, from the assumption of pinned joints, the members are subjected only to axial forces. However some deviations from the theoretical model must be noted as follows:

- Both diagonals and chords are frequently joined by more than one bolt – which would enable greater freedom of rotation. When several bolts are used or where welding is applied, as is the case with gusset plates, the restriction in rotation is considerably higher. Further, some members, such as chords, are generally continuous over several nodes. From this, members of the truss experience bending and shear in addition to the axial forces – these are known as secondary internal forces; the more rigid the chords the greater these forces will be.
- Loads may be applied in between nodes of the truss, resulting in bending and shear on the chords.
- Another type of secondary forces of bending and shear appear with eccentric connections of members at joints. The magnitude of these forces depends upon the eccentricity - increasing proportionally to size of the eccentricity.

As the load path is from the cladding to the purlins, from these to the bracing system and finally discharging on the main truss, several 2D models are adopted with each following model loaded with the reactions of the previous. All modelling is conducted in SAP2000.

The purlins are modelled as simply supported beams with a 5º slope. The loads applied to the purlin result from the quantification outlined in section 4.1.

The bracing truss is modelled as a Warren truss with a continuous chords and pinned diagonals as displayed in Figure 8. Four supports are considered so that the reactions on the main truss are distributed between both the upper and lower chords. The loading on this truss is the self-weight of its members as well as the reactions of the purlins.

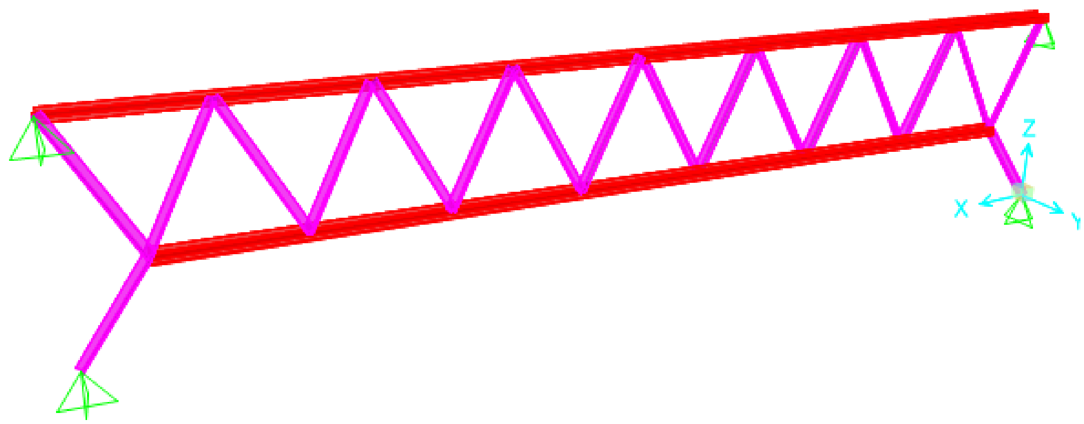


Figure 8 – Model of the bracing truss.

The main truss differs from the bracing one – a modified Warren truss is adopted with additional members. Like the purlins, the main truss has a 5º slope. The upper and lower chords are modelled as 2 continuous bars for each slope. All the diagonal and vertical posts have moment releases at their ends – pinned to the chords.

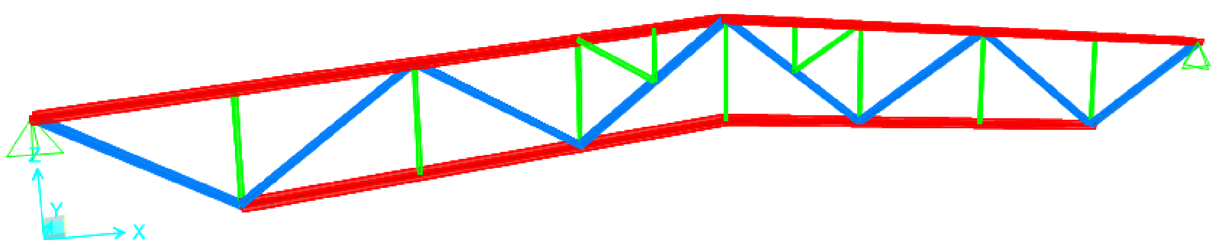


Figure 9 – Model of the main truss

4.2.2 Stiffness and secondary forces

In the previous section secondary forces were described as originating from essentially two different reasons – geometric and boundary conditions. For the first, little explanation is needed as one can easily perceive that an applied force eccentric to the centroid of a member will result in additional bending and shear. For the latter, in particular, that with increasing stiffness of the chords increasing bending follows, further explanation is required. The phenomenon is an interesting one and can be illustrated by comparing the internal forces with the upper and lower chords arranged either standing up or flat.

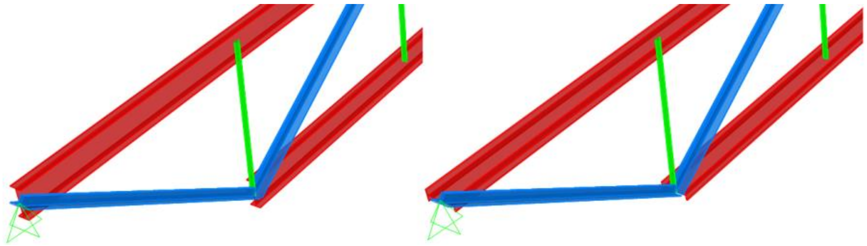


Figure 10 – Different layouts for the chords: standing up (left) and flat (right)

Under the LL combination of ULS both chords bend in the plane of the truss. In the first layout, with both chords standing up, bending in this plane mobilizes the strong inertia of the IPEs, thus increased bending moment when compared with the profiles layout as flat. The bending moment increases from 6.3 kN.m, to 136.5 kN.m as profiles change position from flat to vertical (22 times greater). In Figure 11 the bending moment diagram on the chords with standing up layout is displayed.

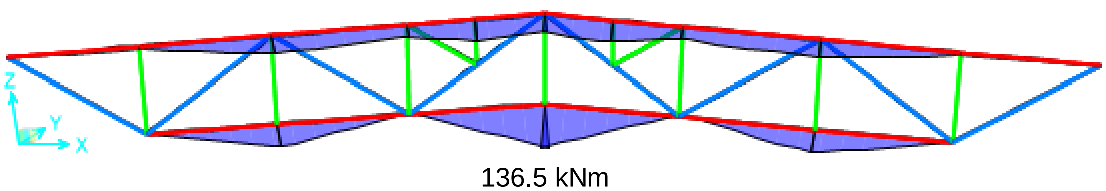


Figure 11 –Bending diagram for standing up layout.

Analysing further the effect of member stiffness in the general behaviour of the structure, another evaluation is considered. It has already been stated that the diagonals and posts are modelled as pinned to the chords but in reality these are connected to a gusset plate with several pre-loaded bolts and the gusset welded to the chord. Thus, one may assume that the connection is closer to a rigid one than to the pinned assumption. If so, during deformation the ends of all members that connect at the node will rotate with the same angle around the node whilst maintaining the angle between each one. To demonstrate the viability, from the analysis standpoint, of the pinned assumption, the comparison of the bending moments between rigid and pinned diagonals with different chord layouts is carried out (Table 13).

Table 13 – Bending moment [in kN.m] comparison between rigid and pinned diagonals with different chord layouts

	Vertical	**Flat**
End moment in diagonal 13 (rigid)	1.33	0.70
End moment in diagonal 17 (rigid)	1.55	0.67
Maximum moment due to self-weight in diagonals 13 and 17 (pinned)	3.20	3.20

It is evident that the bending moment considered at the end of the diagonals with rigid connections is of the same magnitude as the bending moments due to self-weight in the same diagonals.

Moreover, the transformation from pinned to rigid has very little influence on the axial force in the chords, as the shear at the end of the diagonals changes only slightly the value of the axial force.

Table 14 – Axial force [in kN] comparison between rigid and pinned diagonals with different chord layouts

Boundary condition	Member	Vertical	Flat
With rigid joints	3-2	381	408
	3-3	1316	1345
With pinned diagonals and posts	3-2	381	408
	3-3	1317	1346

Therefore, it is no surprise that common practice in designing trusses is to assume chord continuity with pinned posts and diagonals.

4.2.3 Clearance and deflection

Firstly, the consequences of increased deflection should be outlined:

- Discomfort is perhaps the most important as users of the building may not feel safe when noticing slopes that are evidently not desired.
- In statically indeterminate structures, additional forces arise and may place the structure at greater risk of failure.
- Increasing risk of pitch inversion follows increasing deflection of the truss. If pitch inversion does effectively occur, water accumulation could seriously impact the building.

It is therefore evident that controlling deflection is not of minor importance.

In discussing the effect of deflection of a truss structure with bolted connections it is necessary to distinguish between two rather different aspects that play an important role: general slenderness of the truss and clearance of the bolts. For both these aspects the principal of virtual work is a simple yet very convenient form of analysis. For framed structures in general and using standard notation, the principal states that:

$$\bar{1}\delta = \sum_{members} \int_0^L \left(\frac{V_1 V'_1}{GA_1} + \frac{V_2 V'_2}{GA_2} + \frac{NN'}{EA} + \frac{M_1 M'_1}{EI_1} + \frac{M_2 M'_2}{EI_2} + \frac{TT'}{GJ} \right) dx_3$$

Let us consider for the first aspect the notion understood by Navier, that a truss could be analysed as a beam. In this scenario, where depth increases so will inertia and, consequentially, slenderness decreases. Hence, by modifying the overall geometry of the truss as to increase its depth, one can control deflection.

However, even with an adequate depth, clearance of bolts, as unexpected as it might seem, can have a major contribution for deflection. When bolts are in shear, for the successful transmission of the force these have to come in contact with the adjacent members, either by their grip or by their thread. Either way, the initial slack or clearance, that is typically 2 mm, is rearranged as the adjacent members slip, establishing contact with the bolt – otherwise known as taking up slack. This can be assimilated to a reduction or increase of the length of the members in compression or tension, respectively. To further illustrate this point the main truss mentioned in chapter 3 is analysed.

The bolts in the spliced connections of the chords as well as in the connections to the gusset plates of the diagonals are inserted in holes that are drilled with 2 mm of clearance. Assuming that the bolts are initially installed at the centre of each hole, as self-weight comes in to action, the available clearance is readjusted and the members experience a 4 mm extension or reduction (the transmission of forces through the connections take place only after this readjustment). Figure 12 illustrates this phenomena for a spliced connection between plates in tension.

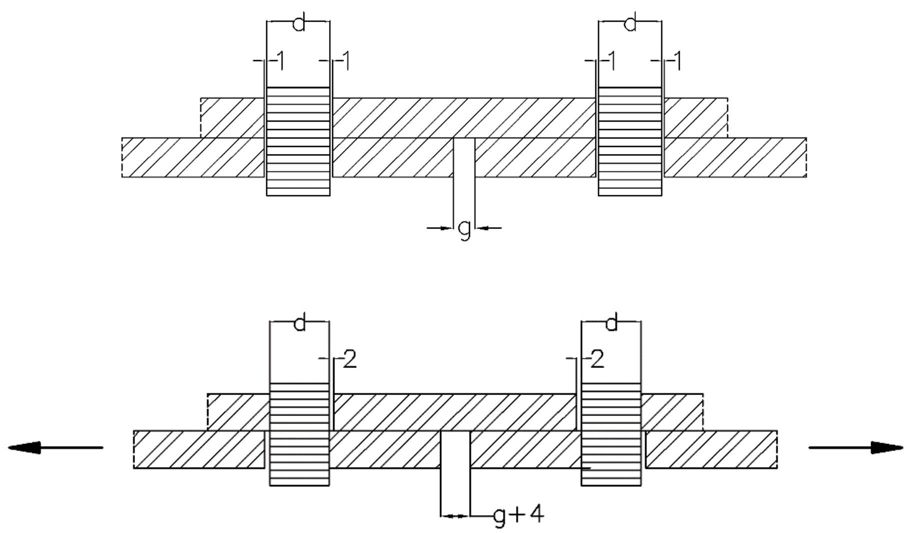


Figure 12 – Taking up slack under gravity loading (dimensions in [mm])

As previously mentioned, the principal of virtual work is applied to evaluate these effects of the clearances (i.e., the effects from the bolts taking up slack). Considering a virtual unit load applied to the truss at mid-span, the corresponding axial forces in the members are those shown in Table 15. The virtual unit load is applied such that the internal virtual forces have the same sign as when gravity loading is considered –

members that are in compression/tension under gravity loading are also in compression/tension under the virtual load.

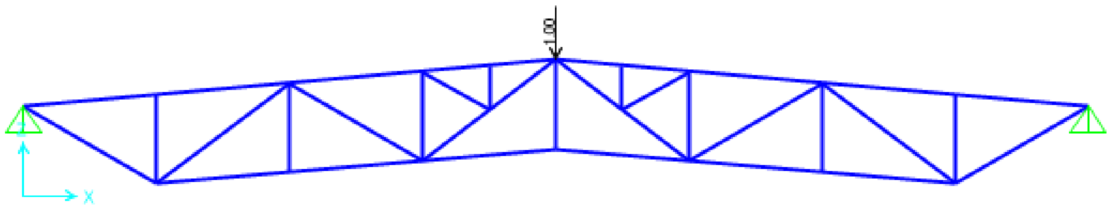


Figure 13 – Unit load applied on truss

Table 15 – Axial forces due to the point load (for each pair of members, the sum of the axial forces is presented)

Member	14 + 15	8 +10	23 + 27	25 + 26	13 + 18	7 + 11	17 + 16	6 + 12	5 + 19	1 + 2	3 + 4	9
N	1.914	0.017	0.010	0.004	1.808	0.005	1.947	0.001	1.802	3.120	2.126	0.536

The internal axial deformation of each member due to the effect of taking up slack under gravity loading is $\int \frac{N}{EA} dx_3 = \pm 4\ mm$; thus, the vertical deflection at mid-span can be calculated as follows:

$$\bar{1}\delta = \sum_{members} \int_0^L \left(\frac{N}{EA}\right) \cdot N'\, dx_3 = 4\ \times \sum_{members} |N'| =$$

$$= 4\ \times (1.914 + 0.017 + 0.010 + 0.004 + 1.808 + 0.005 + 1.947 + 0.001 + 1.802 + 3.120 + 2.126 + 0.536)$$
$$= 53.2\ mm$$

Considering that under the LL combination in SLS the total vertical deflection is 42.7 mm, this added deflection due to the recovery of slack at the bolts is considerable; it represents roughly 125% in addition.

In order to control this additional deflection several measures may be adopted, such as:

- Changing the connections by considering only welding instead of bolts;
- Drilling a smaller clearance if a category A connection is chosen (i.e. drilling +1 mm or even +0.5 mm, instead of +2 mm);
- Choosing pre-loaded bolts and category C connections.

If the maximum displacement, under SLS conditions, is taken as $\delta_{max} = L/200$, the limit state would be verified in the present case (for *L* = 36,00 m, δ_{max} = 180 mm). Anyway, preloaded bolts of category C are used in all connections the presented solution, so that no deflection from recovery of slack needs be considered.

5. Verification of Members

In this chapter, the proper subject of concern is to determine the profiles that satisfy the safety checking in accordance to *EN1993.* In this process, one should consider all load combinations and all critical sections for each member. This procedure can be quite lengthy as it implies repetition of the same checking as each load combination is considered. Therefore, and perhaps to better illustrate the principals and checking procedures that have to be considered in the design of such structures, only some sections are analysed under LL combination in ULS.

5.1 Members in Compression

Table 16 shows the checklist for the design of members in compression.

Table 16 - Design checklist for members in compression.

<table>
<tr><th>No.</th><th>Member</th><th>Check</th></tr>
<tr><td>Check 1</td><td>Diagonals of the main trusses</td><td rowspan="3">• Resistance of the cross-section
• Buckling resistance of the member</td></tr>
<tr><td>Check 2</td><td>Upper Chord of the main trusses</td></tr>
<tr><td>Check 3</td><td>Upper Chord of the bracing trusses</td></tr>
</table>

***Check 1* - Diagonals of the main trusses**

The diagonals of the main trusses under compression have precisely the same checking as the diagonals of the bracing truss. For this reason, only one diagonal member of the main truss is considered – diagonal 17. For the purpose of simplicity, the diagonal analysed in this section follows the numbering presented further in Figure 17 – diagonal 17 is from now onwards denoted as 1.

Forces

The axial force carried by the diagonal discharges on the gusset plate eccentrically to the centre of gravity of the bolts that connect these members. As such, an additional moment should be considered in the safety checking of the diagonal. The design forces acting on the gusset and angle, located at the onset of the joint shown in Figure 17, are denoted with the letter "*g*" and "*a*" respectively.

Table 17 – Design forces on the gusset and angle (diagonal member 17)

$N_{1,g,Ed}$	711.5	[kN]
e_{g1}	42.5	[mm]
$M_{1,g,Ed}$	30.2	[kN.m]
$N_{1,a,Ed}$	355.7	[kN]
$M_{1,a,Ed}$	15.1	[kN.m]

Classification of cross-section

For the purpose of classification, a single angle is considered (L150x150x15). Considering the limiting values for Class 3 cross-sections, the following results are obtained:

$$\frac{h}{t} \leq 15\varepsilon \rightarrow \frac{150}{15} = 10 \leq 15 \times 0.81 = 12.2$$

$$\frac{b+h}{2t} \leq 11.5\varepsilon \rightarrow \frac{150+150}{2 \times 15} = 10 \nleq 11.5 \times 0.81 = 9.4$$

Therefore, the cross-section of the angle is Class 4.

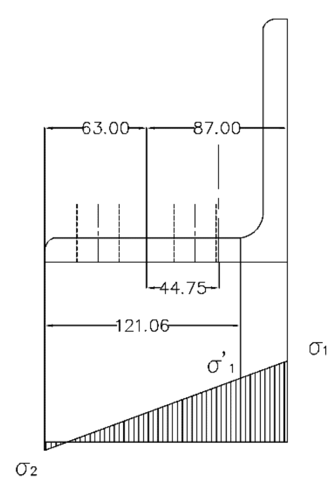
Effective cross-section

The effective area is computed for the components (legs) under compression that are parallel and at a right angle to the bending axis (i.e., the direction of the bolts that connect the angle to the gusset). The letter "*u*" in the expression below represents the distance between the centre of gravity of the effective cross-section and its outermost fibre.

$$\sigma_{a,max} = \frac{N_{1,a,Ed}}{A_{1,a}} + \frac{M_{1,a,Ed}}{I_{1,a}/u} \qquad \Psi = \frac{\sigma_2}{\sigma_1'}$$

Table 18 – Design normal stresses and stress ratio

σ_1	229	[MPa]
σ_1'	181	[MPa]
σ_2	-23.4	[MPa]
Ψ	0.129	



In analysing the parallel leg, a conservative assumption is undertaken, considering that uniform compression is present where in reality the applied stresses show a slight gradient. Thus, the stress ratio Ψ taken is equal to 1.

In determining the buckling factor k_σ, a distinction is made between the internal and outstanding components – function of boundary conditions. The parallel leg is considered outstanding and the perpendicular leg is considered internal due to the presence of the battens (that connect the 2 angles along the length of the members) and of the gusset.

Considering the plate slenderness and the buckling factor for each component – parallel and perpendicular –, in accordance to *EN1993-1-5* no reduction of resistance need be considered ($\rho = 1$, so that $W_{eff} = W_{el}$).

Table 19 – Effective area relative to the parallel (outstanding) leg and perpendicular (internal) leg

Ψ	1
k_σ	0.430
$\overline{\lambda_p}$	0.594
ρ	1

Ψ	-0.129
k_σ	8.786
$\overline{\lambda_p}$	0.103
ρ	1

<u>Resistance of the cross-section</u>

In order to evaluate cross-sectional resistance of the diagonal in compression, checking is conducted considering the compression force and the secondary moment that appears at the joint due to the eccentricity. For the resistance of the cross-section, only one angle is considered.

Table 20 – Checking of cross-sectional resistance

σ_x (= σ_1)	229.1	[MPa]
σ_{Rd} (=f_y)	355	MPa]
$\sigma_x \leq \sigma_{Rd}$	OK	

<u>Buckling resistance of the member</u>

Three flexural buckling modes are analysed in this section, namely:

- Buckling of the angles under uniform compression about the y-y and z-z axis considering the homogeneous cross-section and the full length of the diagonal
- Buckling of a single angle under uniform compression about the v-v axis considering the distance between battens
- Buckling of the homogeneous cross-section under compression and bending about the y-y and z-z axis considering the full length of the diagonal – otherwise known as column-beam.

For the first, the following is considered in accordance to *EN1993-1-1*:

$$N_{cr,i} = \frac{\pi^2 EI_i}{L_{cr,i}^2} \qquad \bar{\lambda}_i = \sqrt{\frac{Af_y}{N_{cr,i}}} \qquad \Phi_i = 0.5\left(1 + (\bar{\lambda}_i - 0.2) + \bar{\lambda}_i^{\,2}\right) \qquad \chi_i = \frac{1}{\Phi_i + \sqrt{\Phi_i^{\,2} - \bar{\lambda}_i^{\,2}}}$$

where:

$$i = y\ ;\ z \qquad L_{cr,y} = L \qquad L_{cr,z} = 0.9L \qquad I_y = 2I_{y,a} + 2A_a\left(y_{CG} + \frac{t_g}{2}\right)^2 \qquad I_z = 2I_{z,a}$$

and y_{CG} is the distance from the centre of gravity of the angle to the edge along the y axis.

Table 21 – Checking of flexural buckling under uniform compression

About y-y		
$I_{,y}$	0.440	[mm^4]
$L_{cr,y}$	5.42	[m]
$N_{cr,y}$	3098.2	[kN]
$\bar{\lambda}_y$	0.993	
α	0.34	
Φ_y	1.128	
χ_y	0.602	

About z-z		
$I_{,z}$	0.180	[mm^4]
$L_{cr,z}$	4.88	[m]
$N_{cr,z}$	1562.6	[kN]
$\bar{\lambda}_z$	1.398	
α	0.34	
Φ_z	1.681	
χ_z	0.383	

$N_{1,g,Ed}$	711.5	[kN]
$N_{b,Rd}$	1168.3	[kN]
$N_{1,g,Ed} \leq N_{b,Rd}$	OK	

In order to enforce the initial assumption of the homogeneous cross-section, the battens should be placed no more than 15 times the minimum radius of gyration of an isolated angle (according to *EN 1993-1-1, Table 6.9*). This condition is rather restrictive and, considering the previous results (that show a reasonable margin of safety) and for purposes of economy, only three battens are placed.

Therefore, the additional calculation for buckling of a single angle between battens [7] is performed as follows:

$$N_{cr,v} = \frac{\pi^2 EI_v}{L_v^2} \qquad \bar{\lambda}_v = \sqrt{\frac{Af_y}{N_{cr,v}}} \qquad \Phi_v = 0.5\left(1 + (\bar{\lambda}_v - 0.2) + \bar{\lambda}_v^{\,2}\right) \qquad \chi_v = \frac{1}{\Phi_v + \sqrt{\Phi_v^{\,2} - \bar{\lambda}_v^{\,2}}}$$

Conservatively, the reduction factor is taken as the product of the reduction factor for a single angle, with length equal to the distance in between battens, by the reduction factor of the whole member [7], i.e.;

$$\chi = \chi_v \cdot \min\{\chi_y; \chi_z\}$$

Table 22 – Buckling in between battens under uniform compression

$L_{diagonal}$	5.42	[m]
$L_{between\ battens}$	1.36	[m]
L_{cr}	0.95	[m]
$N_{cr,v}$	8489	[kN]
λ_v	0.424	
α	0.34	
Φ_v	0.628	
χ_v	0.916	
	0.351	
N_{Ed}	711.5	[kN]
$N_{b,Rd}$	1071	[kN]
$N_{Ed} \leq N_{b,Rd}$	OK	

Notwithstanding the dominance of the axial force, the self-weight of the diagonal produces bending that should be accounted for. According to *EN 1993-1-1, clause 6.3.3, Table A1 and Table A2*, the following calculations are summarized in Table 23.

$$R_y = \frac{N_{Ed}}{\chi_y \cdot \frac{A \cdot f_y}{\gamma_{M1}}} + k_{yy} \cdot \frac{M_{y,Ed}}{\frac{W_{el,y} \cdot f_y}{\gamma_{M1}}} \leq 1 \qquad R_z = \frac{N_{Ed}}{\chi_z \cdot \frac{A \cdot f_y}{\gamma_{M1}}} + k_{zy} \cdot \frac{M_{y,Ed}}{\frac{W_{el,y} \cdot f_y}{\gamma_{M1}}} \leq 1$$

$$k_{yy} = C_{my} C_{mLT} \frac{\mu_y}{1 - \frac{N_{Ed}}{N_{cr,y}}} \qquad k_{zy} = C_{my} C_{mLT} \frac{\mu_z}{1 - \frac{N_{Ed}}{N_{cr,y}}} \qquad C_{my} = C_{my,0} = 1 + 0.03 \frac{N_{Ed}}{N_{cr,y}}$$

Table 23 – Verification of flexural buckling under uniform compression and bending

C_{my}	1.01
C_{mLT}	1.00
μ_y	0.88
k_{yy}	1.15

C_{my}	1.01
C_{mLT}	1.00
μ_z	0.65
k_{yz}	1.20

$R_y \leq 1$	0.46 < 1
$R_z \leq 1$	0.70 < 1

The conclusion is more striking than one would expect, as there is a 15% increase from considering axial compression and bending (if only the axial compression is considered, $\frac{N_{Ed}}{N_{b,Rd}} = \frac{711.5}{1168.3}$ = 0.61 instead of 0.70).

***Check 2* - Upper chord of the main trusses**

Forces

Even though the effects of axial compression are largely predominant, the checking procedure will include the effects of the bending moment to completely illustrate the application of the code.

Table 24 – Design forces for the upper chord of the main trusses

N_{Ed}	1348	[kN]
V_{Ed}	0	[kN]
M_{Ed}	3.6	[kN.m]

Classification of the cross-section

Bending will provide a gradient in the distribution of stresses and should be considered in a strict application of *EN1993-1-1*. However, as it has already been noted, the predominant force is axial compression and therefore classification follows the conservative assumption that uniform compression is present.

From *EN1993-1-1 Table 5.2* and considering the limits for each component (internal and outstanding), the following table summarizes the classification of the cross-section.

Table 25 – Classification of the cross-section under uniform compression

Component	c/t	Classification
Outstanding	13.33	Class 1
Internal	43.37	Class 4

Therefore the cross-section is of Class 4 and effective properties need be considered.

Effective cross-section

The properties of the effective area are calculated firstly under compression (only N_{Ed} applied) and secondly under bending (only M_{Ed} applied). For each one, and similarly to check 1, the first component analysed is the one parallel to the axis of bending (i.e., the web), and then the components at a right angle to the same axis (i.e., the flanges).

Table 26 – Effective area of both the web and the flanges

Component	b [mm]	Ψ	k_σ	$\overline{\lambda_p}$	ρ	b_{eff} [mm]	A_{eff} [mm²]	e_{Nz} [mm]
Web	373	1.0	4.0	0.938	0.816	304.3	74767	0
Flanges	180	1.0	0.43	0.419	1.0	180		

Table 27 - Effective elastic modulus

Component	b [mm]	Ψ	k_σ	$\overline{\lambda_p}$	ρ	b_{eff} [mm]	$W_{eff,z} = W_{z,el}$ [mm³]
Web	373	-1.0	23.9	0.383	1.0	373	146425
Flanges	180	-1.0	0.85	0.298	1.0	180	

<u>*Resistance of the cross-section*</u>

As the section is Class 4, the code allows for two methods of checking the cross-sectional resistance. According to *EN1993-1-1 section 6.2.9.3 (2),* the following is considered:

$$\frac{N_{Ed}}{A_{eff} \cdot f_y / \gamma_{M0}} + \frac{M_{z,Ed}}{W_{eff,z} \cdot f_y / \gamma_{M0}} \leq 1 \ \rightarrow \ 0.31 \leq 1$$

<u>*Buckling resistance of the member*</u>

For buckling under uniform compression the same considerations mentioned in check 1 continue valid. A difference should be noted regarding the buckling length. For buckling in plane of the truss, about the z-z axis of the cross-section, the buckling length is taken as 4.5 m as it is assumed that at the nodes where the diagonals and posts connect to the chord there is sufficient rigidity. For buckling out of plane, about the y-y axis, the buckling length is taken as 9 m, i.e., the spacing in-between the bracing trusses.

Table 28 – Flexural buckling under uniform compression

about y-y		
$L_{cr,y}$	9	[m]
$N_{cr,y}$	23561	[kN]
$\overline{\lambda}_y$	0.452	
α	0.21	
Φ_y	0.629	
χ_y	0.713	

about z-z		
$L_{cr,z}$	4.5	[m]
$N_{cr,z}$	3467	[kN]
$\overline{\lambda}_z$	1.179	
α	0.34	
Φ_z	1.361	
χ_z	0.316	

N_{Ed}	1348	[kN]
$N_{b,Rd}$	1524	[kN]
$N_{Ed} \leq N_{b,Rd}$	OK	

Although the axial force is without doubt the dominant force, under *EN1993-1-1 section 6.3.3* the following check must be satisfied:

$$R_y = \frac{N_{Ed}}{\chi_y \cdot \frac{A \cdot f_y}{\gamma_{M1}}} + k_{yz} \cdot \frac{M_{z,Ed}}{\frac{W_{el,y} \cdot f_y}{\gamma_{M1}}} \leq 1 \qquad\qquad R_z = \frac{N_{Ed}}{\chi_z \cdot \frac{A \cdot f_y}{\gamma_{M1}}} + k_{zz} \cdot \frac{M_{z,Ed}}{\frac{W_{el,y} \cdot f_y}{\gamma_{M1}}} \leq 1$$

$$k_{zy} = k_{zz} = C_{mz}\left(1 + 0.6\frac{N_{Ed}}{X_z \, {N_{Rd}}/{\gamma_{M1}}}\right)$$

where C_{mz} is taken equal to 0.95.

Table 29 – Verification of flexural buckling resistance under bending and axial compression

k_{yz}	1.45
k_{zz}	1.45
$R_y \leq 1$	0.39
$R_z \leq 1$	0.88

<u>Check 3</u> - Upper chord of the bracing trusses

As many of the considerations outlined in check 2 remain valid, the only commentary that will be added is where deviations should be accounted for.

<u>Forces</u>

These trusses have already been described, namely their role in the bracing system of the main trusses. As so, the initial bow imperfections of the members to be restrained – the upper chord of the main truss - are replaced by an equivalent stabilizing force. According to *EN1993-1-1 section 5.3.3,* the quantification of this force is as follows:

$$q_d = \sum N_{Ed}\, 8\frac{e_0 + \delta_q}{L^2} \qquad e_0 = \alpha_m \, {L}/{500} \qquad \alpha_m = \sqrt{0.5\left(1 + \frac{1}{m}\right)}$$

Axial force along the main chord varies and so does the influence length that each bracing truss exerts in absorbing part of the equivalent stabilizing force. These combined aspects tend to complicate further calculations, and so a simplified and conservative approach is adopted.

This approach implies four basic assumptions, namely: (1) the chord is subjected to the maximum axial force (N_{Ed} = 1347.6 kN) along all its length, (2) there is only one braced truss (α_m =1), (3) the influence length (L*) is taken as the spacing between the bracing trusses, 9 m, (4) the in-plane deflection of the bracing system due to *q* plus any external loads calculated from first order analysis is taken as equal to 1000th of the span of the main truss.

$$q_d = \frac{8 \cdot \sum N_{Ed}}{L^2}\left(\frac{L}{500} + \frac{L}{1000}\right) = 2.4\% \frac{N_{Ed}}{L} \qquad \rightarrow \qquad Q_p = \ q_d \cdot L^* = 9.1 \text{ kN}$$

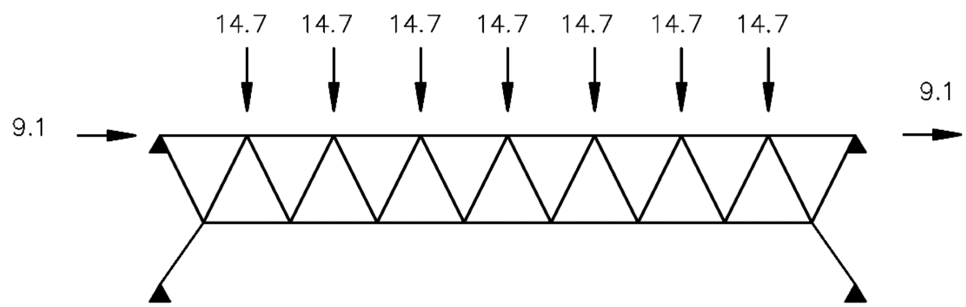


Figure 14 – Bracing truss loading under LL combination in ULS (forces in [kN])

Therefore, the internal forces on the upper chord of the bracing truss resulting from first order analysis and considering the above calculation are as summarized as follows:

Table 30 – Design forces for the upper chord of the bracing trusses

N_{Ed}	51.27	[kN]
V_{Ed}	0	[kN]
M_{Ed}	0.11	[kN.m]

Classification of the cross-section

Table 31 - Classification of the cross-section under uniform compression

Component	c/t	Classification
Outstanding	5.20	Class 1
Internal	29.04	Class 2

Resistance of the cross-section

The resistance of the cross subjected to bending and axial compression has to satisfy the following conditions, according to *EN1993-1-1 section 6.2.3* and *section 6.2.5.*

Table 32 – General resistance of the cross-section under compression and bending

N_{Ed}	51.3	[kN]
$N_{c,Rd}$	713.2	[kN]
$N_{Ed} \leq N_{c,Rd}$	OK	

$M_{z,Ed}$	0.11	[kN.m]
$M_{z,pl,Rd}$	9.27	[kN.m]
$M_{z,Ed} \leq M_{z,pl,Rd}$	OK	

The interaction M-N may be discarded according to EN1993-1-1 section 6.2.9.1 (4) when

$$N_{Ed} \leq \frac{h_w t_w f_y}{\gamma_{M0}} \quad \rightarrow \quad 51.3 \ \leq 257.7$$

Therefore the interaction M-N is discarded.

Buckling resistance of the member

Table 33 – Flexural buckling under bending and axial compression

about y-y		
$L_{cr,y}$	14	[m]
$N_{cr,y}$	91.92	[kN]
$\bar{\lambda}_y$	2.786	
α	0.21	
Φ_y	4.651	
χ_y	0.099	
N_{Ed}	51.27	[kN]
$N_{b,Rd}$	70.81	[kN]
$N_{Ed} \leq N_{b,Rd}$	OK	

about z-z		
$L_{cr,z}$	1.75	[m]
$N_{cr,z}$	462.33	[kN]
$\bar{\lambda}_z$	1.242	
α	0.34	
Φ_z	1.449	
χ_z	0.298	

Table 34 – Verification of resistance to flexural buckling under bending and axial compression

k_{yz}	1.09
k_{zz}	1.09
$R_y \leq 1$	0.74
$R_z \leq 1$	0.26

5.2 Members in Tension

Table 35 shows the checklist for the design of members in tension.

Table 35 – Design checklist for tension members

No.	Member	Check
Check 1	Diagonals of the main trusses	• Resistance of the cross-section
Check 2	Lower chord of the main trusses	
Check 3	Lower chord of the bracing trusses	

***Check 1** - Diagonals of the main trusses*

To evaluate the cross-sectional resistance, two separate sections of diagonal No.13 are analysed (from now on designated as diagonal No.3 as shown in Figure 17). First, at mid span, and second, at the connection with joint No. 10 (shown in Figure 16).

Forces

The design forces at the connection with joint No. 10 and at mid span are as follows:

Table 36 – Design forces at the joint and mid-span

	Joint No. 10	Mid-span
$N_{3,Ed}$ [kN]	423.1	421.8
$M_{3,Ed}$ [kN.m]	17.9	2.9

The diagonals are modelled as pinned and the moment at the joint is due to the eccentricity, like it has already been mentioned in section 5.1.

Cross-sectional resistance

Axial tension and bending are present, both at mid-span and at joint No 10. For checking at mid-span, according *to EN1993-1-1* and *EN1993-1-8,* the resistances of the gross and net cross-sections are to be evaluated as follows:

$$N_{pl,Rd} = \frac{2A \cdot f_y}{\gamma_{M0}} \qquad N_{u,Rd} = \frac{\beta_3 \cdot A_{net} \cdot f_y}{\gamma_{M2}} \qquad N_{t,Rd} = \min\{N_{pl,Rd}; N_{u,Rd}\} \qquad M_{el,Rd} = \frac{2W_{el} \cdot f_y}{\gamma_{M0}}$$

$$A_{net} = 2A - 2d_0 t_a$$

β_2 taken as 0.5. This approach is slightly conservative as there is more than one bolt row. $N_{u,Rd}$ could be taken as shown in check 2

Table 37 – Tension and bending resistance

$N_{pl,Rd}$	3054.8	[kN]
$N_{u,Rd}$	2222.3	[kN]
$N_{3,Ed} \leq N_{t,Rd}$	OK	

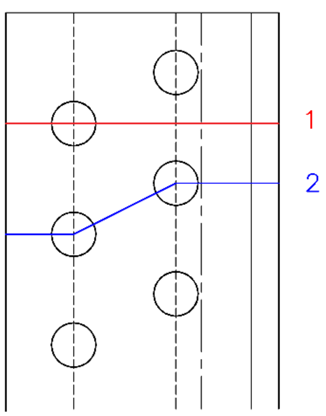
$M_{3,Ed}$	2.9	[kN]
$M_{el,Rd}$	59.3	[kN]
$M_{3,Ed} \leq M_{el,Rd}$	OK	

For checking at the joint, two separate evaluations are considered. First, an evaluation that combines the effect of axial force and bending moment, as described in section 5.1 - Check 1, is applied.

Table 38 – Checking regarding the normal stress

σ_x	156.8	[MPa]
σ_{Rd}	355	[MPa]

Second, the resistance of the net cross section, considering only one angle, is evaluated. There are two possible net areas to be considered. As Figure 15 illustrates, area 1 considers only one fastener and area 2 considers two fasteners.



$A_{1,net}$	2010	[mm]
$A_{2,net}$	1726	[mm]
$N_{a,net,Rd}$	612	[kN]
$N_{Ed,a} \leq N_{a,net,Rd}$	OK	

Table 39 – Checking regarding net area resistance

Figure 15 – Net areas

<u>Check 2</u> - Lower chord of the main trusses

In this check, the resistance of the gross cross-section as well as the net cross-section are evaluated considering the design forces displayed in Table 40. The additional moment that appears at the continuity connection of the chord, due to eccentricity between the centre of gravity of the bolt group in the flanges and the applied shear force, is considered further on in section 6.2.3.

Forces

Table 40 – Design forces on the lower chord of the main trusses

N_{Ed}	1866	[kN]
V_{Ed}	0	[kN]
M_{Ed}	6.3	[kN.m]

Cross-section resistance

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \qquad N_{u,Rd} = \frac{0.9\, A_{net} \cdot f_u}{\gamma_{M2}} \qquad N_{t,Rd} = \min\{N_{pl,,Rd};\ N_{u,Rd}\} \qquad M_{pl,Rd} = \frac{W_{z,pl} \cdot f_y}{\gamma_{M0}}$$

$$A_{net} = A - (\,4d_0t_f + 3d_0t_w)$$

Table 41 – Cross sectional resistance under tension and bending

$N_{pl,Rd}$	2999	[kN]
$N_{u,Rd}$	2298	[kN]
$N_{Ed} \leq N_{t,Rd}$	OK	

M_{Ed}	3.2	[kN.m]
$M_{pl,Rd}$	81.3	[kN.m]
$M_{Ed} \leq M_{pl,Rd}$	OK	

***Check 3* - Lower chord of the bracing trusses**

For the chord in tension of the bracing trusses, the checking that has to be satisfied is precisely the same as for the chord in tension of the main trusses. Therefore no further calculations are shown in this document.

6 Verification of Connections

In this chapter, three of the connections mentioned in section 3.3 are analysed, fully satisfying all checks according to *EN1993-1-1* and *EN1993-1-8* under the LL combination in ULS.

6.1 Detailed design of KT joint No. 10

In designing the KT joints between the bracing members and the chords of the trusses, there are two main connections to consider: (i) welded gusset to chord, and (ii) bolted angles to gusset.

6.1.1 Loads and general geometry

The location of joint No. 10 and the internal forces at the members that connect at that joint are shown in Figure 16 and Table 42, respectively.

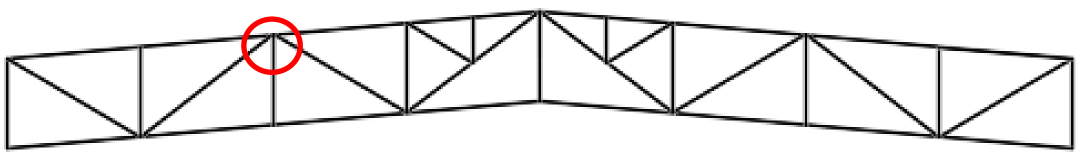
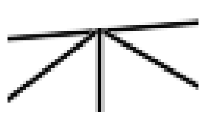


Figure 16- Location of joint No. 10

Table 42 – Internal forces at joint No 10 (LL combination in ULS)

Members at joint No 10		N [kN]	V [kN]	M [kN]
	Chord 3-2	-1346.4	2.8	-0.9
	Chord 3-3	-408.0	-2.3	-0.9
	Diagonal 17	-711.5	-2.3	0
	Diagonal 13	423.1	2.3	0
	Post 7	118.4	0	0

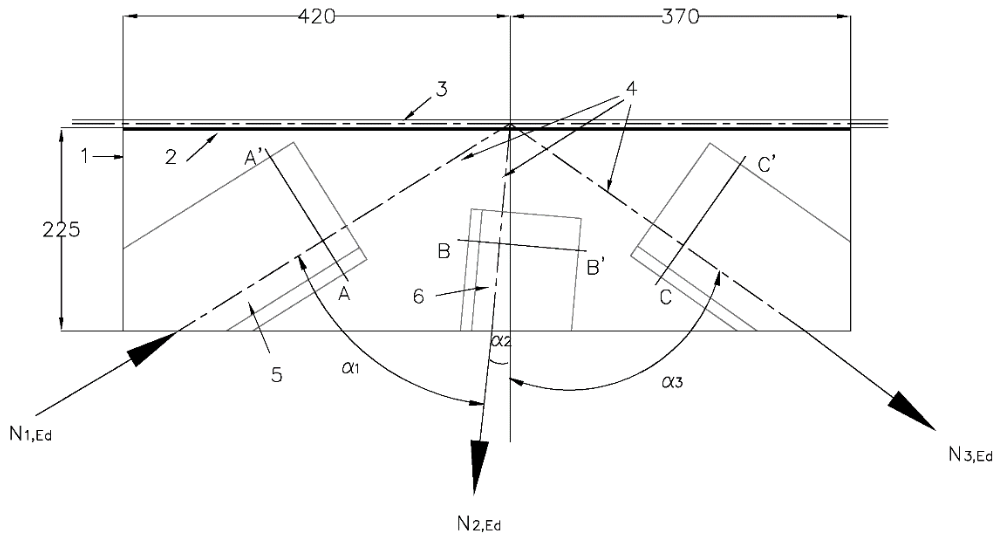


Figure 17 – General layout of joint No 10

1 – Gusset plate; 2 – Fillet weld; 3 – Chord's web (IPE); 4 – Centroid lines; 5 – Diagonal 17; 6 – Post 7

To simplify, the three bracing members that meet at the joint are labelled from 1 to 3 as shown in Figure 17; the corresponding internal forces are summarized in Table 43.

Table 43 – Design forces on the bracing members - diagonals (1 and 3) and post (2)
(Note: Positive values correspond to tension forces)

member *i*	$N_{Ed,i}$ [kN]	α_i [º]
1	-711.5	58
2	118.4	5
3	423.1	55

The diagonals and the post are positioned in a way such that their centroid lines meet on a point in the mid-plane of the chord's web (point O in Figure 18). The gusset is positioned eccentrically to that point. This eccentricity is both horizontal (e_x = 25 mm) and vertical (e_y = 4.5 mm). The moment resulting from the vertical eccentricity is not considered in the calculations that follow.

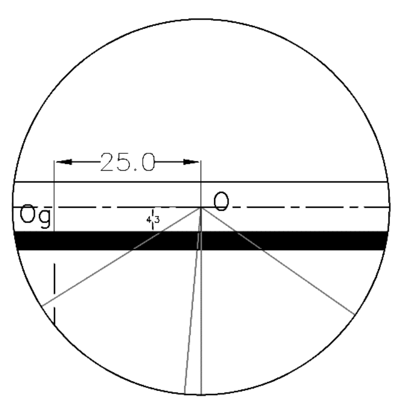


Figure 18 – Gusset to chord eccentricity detail

6.1.2 Gusset to chord

The forces exhibited in Table 43 follow a path that can be interpreted as a sequential discharge from the members to the gusset and from the gusset to the web of the chord. As forces transfer to the chord, the gusset must have the carrying ability for a successful transmission. The sequence of checking for the gusset to chord connection can be summarized as shown in Table 44.

Table 44 – Design checklist for gusset to chord connection

No.	***Member***	***Check***
Check 1	Gusset plate	• Resistance of the cross-section at the onset of welding
Check 2	Fillet welds	• Shear resistance

<u>Check 1</u> - Gusset plate

The design forces in the gusset plate at the intersection with the chord's web, displayed in Figure 19, are determined as follows:

$$N_{g,Ed} = \sum_{i=1}^{3} N_i \cdot \cos(\alpha_i) \qquad V_{g,Ed} = \sum_{i=1}^{3} N_i \cdot \sin(\alpha_i) \qquad M_{g,Ed} = e_x \cdot N_{g,Ed}$$

Table 45 – Design forces for checking of gusset to chord connection

$N_{g,Ed}$	16.4	[kN]
$V_{g,Ed}$	939.6	[kN]
$M_{g,Ed}$	0.41	[kN.m]

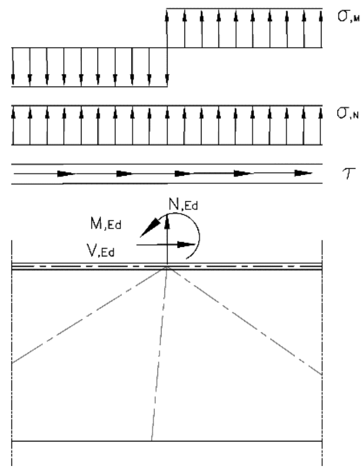


Figure 19 – Design stresses on the gusset in front of welds

The cross-section properties and resulting normal and shear stresses from the forces previously calculated are displayed in Table 46 and computed as follows:

$$A_g = t_g \cdot L_w \qquad I_g = \frac{t_g \cdot L_w^3}{12} \qquad \sigma_{g,max} = \frac{N_{g,Ed}}{A_g} + \frac{M_{g,Ed}}{I_g/z} \qquad \tau_g = \frac{V_{g,Ed}}{A_g}$$

where L_w is the length of the weld (790 mm) and t_g is the thickness of the gusset plate (25 mm).

Table 46 – Properties of the plate cross-section and design value of normal and shear stresses

A_g	15800	[mm^2]
I_g	8.22x10^8	[mm^4]
z_g	395	[mm]
$\sigma_{g,max}$	0.99	[MPa]
τ_g	47.58	[MPa]

The adopted checking criteria is in accordance to *EN 1993-1-1, 6.2.1 (5),* and shown below:

$$\left(\frac{\sigma_{g,max}}{f_y/\gamma_{M0}}\right)^2 + 3\left(\frac{\tau_g}{f_y/\gamma_{M0}}\right)^2 \leq 1 \Leftrightarrow 0.054 \leq 1$$

<u>Check 2</u> - Fillet welds

Two methods are referred to in *EN1993-1-8* for designing fillet welds: the directional method and the simplified method. The latter is adopted. The code outlines that the applied shear stress at the weld's throat is to be compared with its shear strength. Table 47 summarizes the shear stresses according to the following calculations:

$$F_{w,v} = \frac{N_{g,Ed}}{2L_w} + 2\frac{M_{g,Ed}}{L_w^2} \qquad F_{w,h} = {}^{V_{g,Ed}}\!/_{2L_w} \qquad F_{w,Ed} = \sqrt{\left(F_{w,v}\right)^2 + \left(F_{w,h}\right)^2}$$

Table 47 – Design value of the weld forces per unit length

$F_{w,v}$	11.1	[kN/m]
$F_{w,h}$	594.8	[kN/m]
$F_{w,Ed}$	594.8	[kN/m]

The shear strength of the weld as well as the minimum throat thickness needed to verify the checking are summarized in Table 48 and calculated as follows:

$$f_{vw,d} = \frac{{}^{f_u}\!/_{\sqrt{3}}}{\beta_w \gamma_{M2}} \qquad a_{min} = \frac{F_{w,Ed}}{f_{vw,d}}$$

Table 48 – Design shear stress resistance and effective throat thickness of the welds

$f_{vw,d}$	261.7	[MPa]
a_{min}	2.3	[mm]

According to *EN 1993-1-8*, *clause 4.5.2*, the minimum throat thickness is set at 3 mm. Thus, fillet welds with a = 4mm are adopted.

6.1.3 Diagonals to gusset

In connecting lattice members (diagonals and posts) to a gusset, several aspects have to be considered and so a brief discussion outlining how these members interact, and therefore the safety checking that follows, is presented. On the one hand, forces transfer from the respective members to the gusset which must have enough cross-sectional resistance as well as bucking resistance at a local level. On the other hand, the connection between the bracing members and the gusset is a category C bolted connection and, therefore, conditions of bearing and slip resistance have to be satisfied, and where the diagonal is in tension, additional block tearing and net cross-section resistances should be accounted for.

The terms global and local are used to describe two different situations: considering all the forces transmitted by the members to the gusset and considering the individual forces of each member separately.

6.1.3.1 Global elastic resistance of the gusset

The global elastic resistance of the gusset builds on the checking already carried out in 6.1.2. There are two main differences to be considered: firstly, two cross sections are analysed – shown in blue in Figure 20. Secondly, the approach is a more a conservative one as all favourable forces are discarded. Table 49 summarizes the safety checking.

Table 49 - Design checklist for verification of resistance of the gusset

No.	***Member***	***Check***
Check 1	Gusset plate	• Resistance of cross-section 1 (‘)
Check 2	Gusset plate	• Resistance of cross-section 2 (“)

The forces acting on the two cross-sections result from decomposing the acting forces into normal and shear components, shown in Figure 20, in relation to each cross-section.

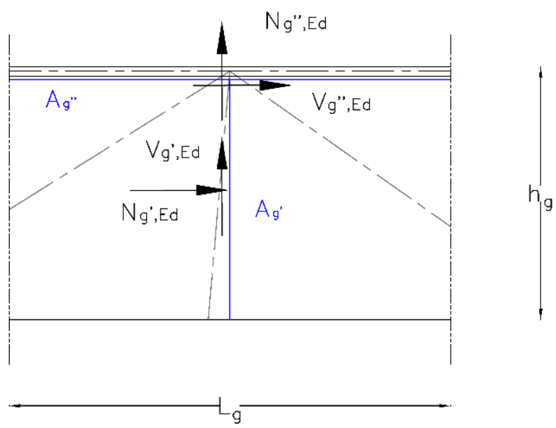


Figure 20 – Design global forces and cross-sections of gusset

The area of both cross-sections is summarized in Table 50.

$$A_{g'} = t_g \cdot h_g \qquad\qquad A_{g''} = t_g \cdot L_g$$

where t_g is the thickness of the gusset plate (25 mm), h_g the height of the gusset plate (225 mm) and L_g the length of the gusset plate (790 mm).

Table 50 – Areas of the gusset cross-sections

$A_{g'}$	5.63	[mm^2]
$A_{g''}$	19.75	[mm^2]

Check 1 ***- Gusset plate -*** Resistance of cross-section 1 (‘)

For cross-section1 (‘), the normal and shear design forces as well as the corresponding resistances are computed as follows, in accordance to *EN1993-1-1*:

$$V_{g',Ed} = \max\{N_1 \cos(\alpha_1)\,;N_2 \cos(\alpha_2) + N_3 \cos(\alpha_3)\} \qquad\qquad V_{g',Rd} = \frac{A_{g'} \cdot f_y/\sqrt{3}}{\gamma_{M0}}$$

Table 51 – Design shear force and checking on cross-section 1 (‘)

$V_{g',Ed}$	377	[kN]
$V_{g',Rd}$	1153	[kN]
$V_{g',Ed} \leq V_{g',Rd}$	OK	

$$N_{g',Ed} = \max\{N_1 \sin(\alpha_1) + N_3 \sin(\alpha_3)\,;N_2 \sin(\alpha_2)\} \qquad\qquad N_{g',Rd} = \frac{A_{g'} \cdot f_y}{\gamma_{M0}}$$

Table 52 – Design normal force and checking on cross-section 1 (‘)

$N_{g',Ed}$	950	[kN]
$N_{g',Rd}$	1997	[kN]
$N_{g',Ed} \leq N_{g',Rd}$	OK	

***Check 2* - Gusset plate -** Resistance of cross-section 2 ('')

A similar procedure is undertaken for cross-section2 (''), as follows:

$$V_{g'',Ed} = N_{g',Ed} \qquad V_{g'',Rd} = \frac{A_{g''} \cdot f_y/\sqrt{3}}{\gamma_{M0}}$$

Table 53 – Design shear force and checking on cross-section 2 ('')

$V_{g'',Ed}$	950	[kN]
$V_{g'',Rd}$	4048	[kN]
$V_{g'',Ed} \leq V_{g'',Rd}$	OK	

$$N_{g'',Ed} = V_{g',Ed} \qquad N_{g'',Rd} = \frac{A_{g''} \cdot f_y}{\gamma_{M0}}$$

Table 54 – Design normal force and checking on cross-section 2 ('')

$N_{g'',Ed}$	377	[kN]
$N_{g'',Rd}$	7011	[kN]
$N_{g'',Ed} \leq N_{g'',Rd}$	OK	

6.1.3.2 Diagonal 17 to gusset

Table 55 – Design checklist for connection of diagonal 17 to gusset

No.	***Member***	***Check***
Check 1	Gusset plate	• Resistance of the cross-section • Buckling resistance
Check 2	Bolts - regarding the gusset	• Bearing resistance • Slip resistance
Check 3	Bolts - regarding the angle	• Bearing resistance • Slip resistance

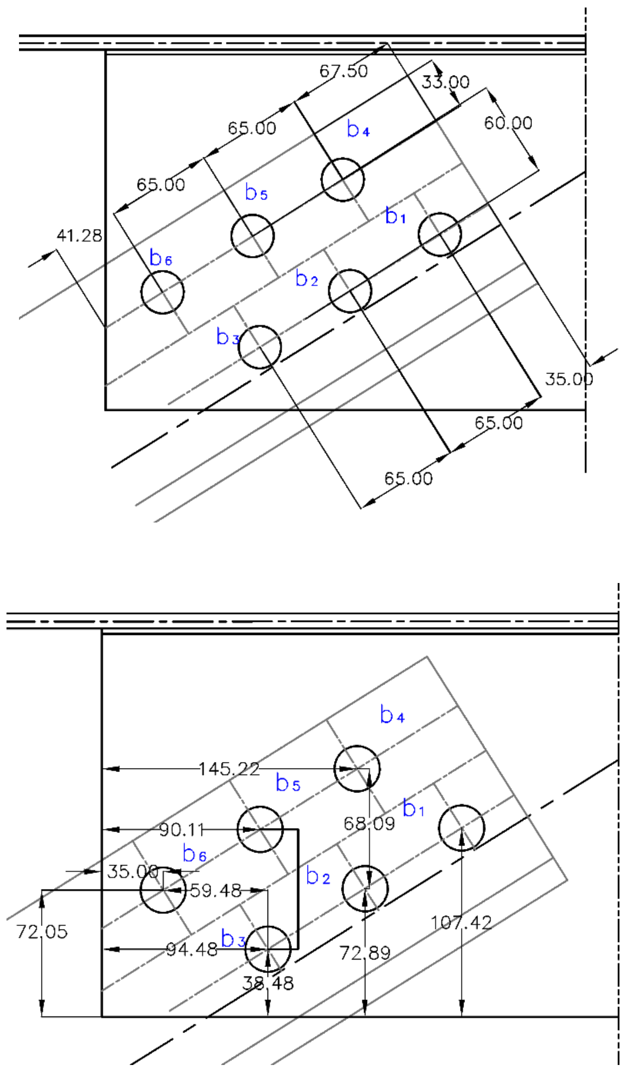


Figure 21 - Positioning of bolts in diagonal 17 connecting to the gusset (dimensions in [mm])

<u>Check 1</u> - Gusset plate

It has already been mentioned that the gusset plate should be checked for global and local cross-sectional resistance as well as local buckling. The first has already been covered in the previous section, remaining only the two latter to be analysed.

<u>Local resistance of the cross-section</u>

Whitmore suggested a simple and straight forward way to determine how forces from a bracing system distribute through a gusset plate. In order to determine the peak stress in the plate, either in compression or in tension, an effective area – called the "Whitmore section" – is determined by multiplying an effective length by the plate thickness. The effective length is established by spreading the force 30º from each side of the connection elements – bolt rows – from start to end (Figure 22). [4][6]

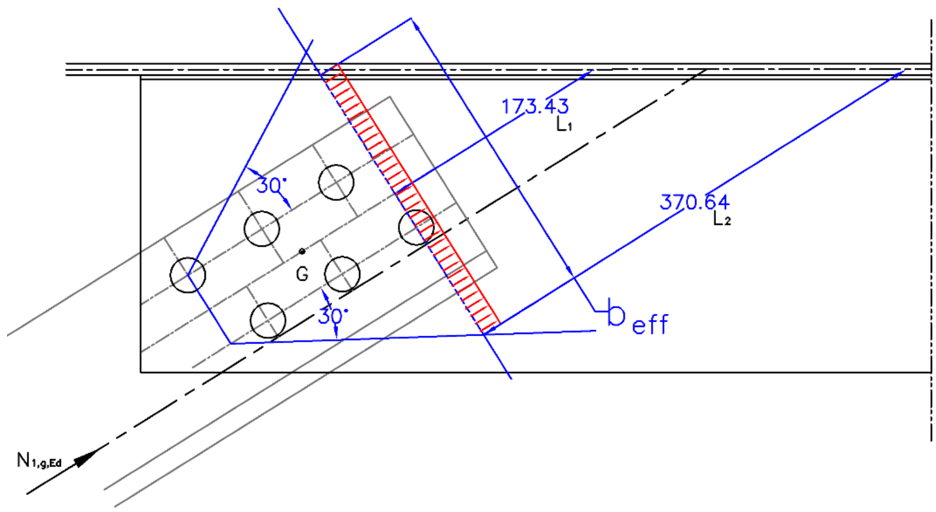


Figure 22 – Whitmore cross-section and buckling length

The cross-sectional properties and design normal stress determination are as follows and summarized in Table 56. The distance to the outmost fibre is denoted as "z" and represents half of $b_{eff.}$

$$A_{1,g} = t_g \cdot b_{1,eff} \qquad I_{1,g} = \frac{t_g \cdot b_{1,eff}^3}{12} \qquad \sigma_{Ed} = \frac{N_{1,g,Ed}}{A_{1,g}} + \frac{M_{1,g,Ed}}{I_{1,g}/z}$$

Table 56 – Cross-sectional properties, design forces and checking

$b_{1,eff}$	227.74	[mm]	$N_{1,g,Ed}$ (vd. Table 17)	711.5	[kN]
t_g	25	[mm]	$M_{1,g,Ed}$ (vd. Table 17)	30.2	[kN.m]
$A_{1,g}$	5693.5	[mm^2]	σ_{Ed}	125.1	[MPa]
z	113.87	[mm]	σ_{Rd}	355	[MPa]
$I_{1,g}$	24608022	[mm^4]	$\sigma_{Ed} \leq \sigma_{Rd}$	OK	

Buckling resistance

Thornton further suggested that the buckling resistance of the gusset could be modelled as an embedded column with cross-section equal to the Whitmore section. The length of that embedded column, L', is taken as the greatest distance of L_1, L_2 (see Figure 22) multiplied by a factor K of 0.65. As the column is embedded, the buckling length is taken as 2L' [4]. The cross-sectional properties and the buckling length are determined as follows and summarized in Table 57.

$$L' = K \cdot \min\{L_1; L_2\} \qquad I_{1,g} = \frac{t_g^3 \cdot b_{1,eff}}{12}$$

Table 57 – Buckling length and moment of inertia about the weak axis

L_1	173.43	[mm]
L_2	370.64	[mm]
L'	240.92	[mm]
$I_{1,g}$	296536	[mm^4]

The buckling resistance is evaluated according to *EN 1993-1-1, clause 6.3.1.2*. As the Whitmore section is a solid rectangle, according to table 6.2 in the referred section of *EN 1993-1-1,* α = 0.49 (curve c):

$$\bar{\lambda} = \sqrt{\frac{4L'^2 A_{1,g} f_y}{\pi^2 E I_{1,g}}} \qquad \Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right)\bar{\lambda}^2\right] \qquad \chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \qquad N_{1,g,b,Rd} = \chi \frac{A_{3,g} f_y}{\gamma_{M1}}$$

Table 58 - Design normal forces and checking

$\bar{\lambda}$	0.874
α	0.49
Φ	1.05
χ	0.616

$N_{1,Ed}$	711.5	[kN]
$N_{1,g,b,Rd}$	1245	[kN]
$N_{1,Ed} \leq N_{1,g,b,Rd}$	OK	

The approaches made by Whitmore and Thornton are not mentioned in Eurocode but are widely used and are considered as well calibrated [6].

***Check 2** - Bolts - regarding the gusset*

The bolts are loaded in shear and designed as category C. Thus, according to *EN 1993-1-8, Table 3.2*, the bolts must be of class 8.8 or greater and three criteria must be attended to:

$$1)\ F_{V,Ed} \leq F_{b,Rd} \qquad 2)\ F_{V,Ed} \leq F_{s,Rd} \qquad 3)\ F_{V,Ed} \leq N_{net,Rd}\ \text{(only in case of tensioned members)}$$

Shear Forces

When axial loads are applied on a line of action that does not pass through the centre of gravity of a bolt group, an eccentric loading effect takes place. The axial load at an eccentricity is statically equivalent to both the axial load and a moment applied at the centre of gravity. Since both the concentric load and the moment result in shear effects on the bolt group this type of loading is known as eccentric shear.

When analysing these situations a possible approach is the traditional elastic (vector) analysis in which it is assumed that no friction occurs between a rigid plate and elastic fasteners. This procedure has been shown to be a conservative one and its popularity steams from the simplified application of mechanics.

In order to pursue the above checking, and in accordance to *EN 1993-1-8, Table 3.4*[3)], the bolt shear forces are analysed in two different sets of local axis, shown in Figures 23-24 and referred to, respectively, as the {h', v'} and {h, v} reference systems, so that the resistance may be verified for the load components that are parallel and normal to the end of both the gusset plate (at the connection to the chord) and the diagonal member.

First, shear forces are computed in the reference system {h', v'}, whose origin is located at the centre of gravity of the bolt rows. The shear force applied to each bolt is determined as follows:

$$F_{N,bi,h'} = \frac{N_{1,g,Ed}}{\sum_{i=1}^{n} n_b} \qquad F_{M,bi} = \frac{M_{1,g,Ed} \cdot r_i'}{\sum_{i=1}^{n} r_i^2}$$

The shear force due to the moment $F_{M,bi}$ is decomposed in the two components in reference system {h', v'}

$$F_{M,bi,v'} = \frac{M_{1,g,Ed} \cdot h_i'}{\sum_{i=1}^{n} r_i^2} \qquad F_{M,bi,h'} = \frac{M_{1,g,Ed} \cdot v_i'}{\sum_{i=1}^{n} r_i^2}$$

The total components as well as the resulting force on each bolt are determined as follows:

$$F_{V,bi,h',Ed} = F_{N,bi,h'} + F_{M,bi,h'} \qquad F_{V,bi,v',Ed} = F_{M,bi,v'} \qquad F_{V,bi,Ed} = \sqrt{F_{V,bi,h',Ed}^2 + F_{V,bi,v',Ed}^2}$$

In Table 59 the distances needed for the calculations as well as the outcome of these are summarized.

Table 59 - Design shear forces in the reference system {h',v'} - regarding the gusset

Bolt	b_1	b_2	b_3	b_4	b_5	b_6
$h_{i'}$ [mm]	81.25	16.25	-48.75	48.75	-16.25	-81.25
$v_{i'}$ [mm]	-30.00	-30.00	-30.00	30.00	30.00	30.00
ri' [mm]	86.67	34.15	57.18	57.18	34.15	86.67
$F_{M,bi}$ [kN]	-109.7	-43.2	-72.4	-72.4	-43.2	-109.7
$F_{M,bi,h'}$ [kN]	37.9	37.9	37.9	-37.9	-37.9	-37.9
$F_{M,bi,v'}$ [kN]	102.8	20.6	-61.7	61.7	-20.6	-102.8
$F_{N,bi}$ [kN]	118.6	118.6	118.6	118.6	118.6	118.6
$F_{V,bi,h',Ed}$ [kN]	156.5	156.5	156.5	80.6	80.6	80.6
$F_{V,bi,v',Ed}$ [kN]	102.8	20.6	-61.7	61.7	-20.6	-102.8
$F_{V,bi,Ed}$ [kN]	**187.3**	**157.9**	**168.3**	**101.5**	**83.2**	**130.7**

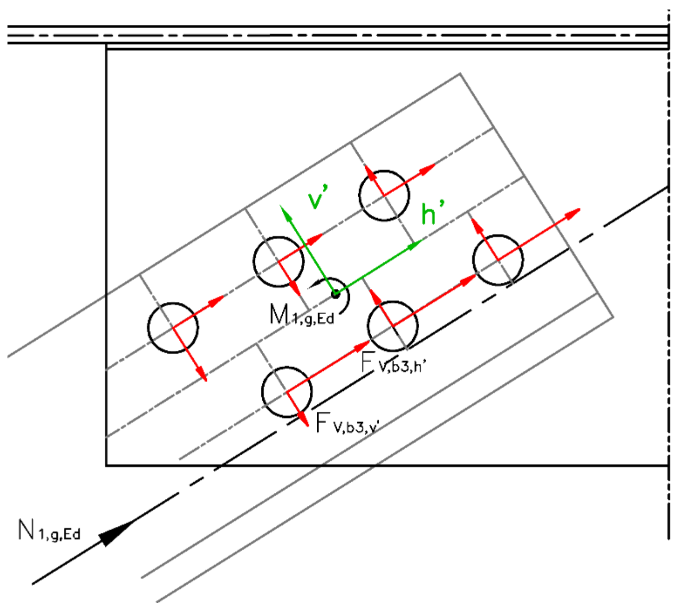


Figure 23 – Loading on bolts (forces in the {h', v'} system) - regarding the gusset

Secondly, the computed shear forces are switched from the {h', v'} to the {h, v} system:

$$F_{V,bi,h,Ed} = -F_{V,bi,h',Ed}\sin(\alpha_1) + F_{V,bi,v',Ed}\cos(\alpha_1)$$

$$F_{V,bi,v,Ed} = F_{V,bi,h',Ed}\cos(\alpha_1) + F_{V,bi,v',Ed}\sin(\alpha_1)$$

Table 60 - Design shear forces in the reference system {h, v} - regarding the gusset

Bolt	b_1	b_2	b_3	b_4	b_5	b_6
$F_{V,bi,Ed}$ [kN]	187.3	157.9	168.2	101.5	83.2	130.7
$F_{V,bi,h,Ed}$ [kN]	-78.3	-121.9	-165.4	-35.7	-79.3	-122.9
$F_{V,bi,v,Ed}$ [kN]	170.1	100.4	30.6	95.0	25.3	-44.5

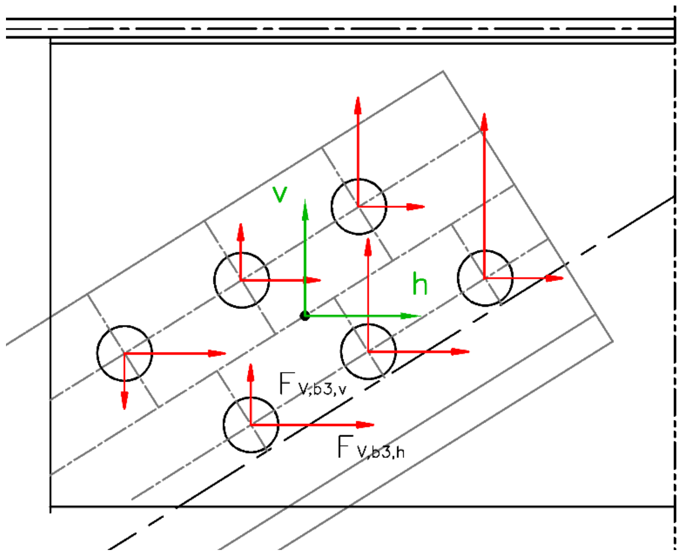


Figure 24 – Loading on bolts (forces in the {h, v} system) - regarding the gusset

<u>*Design bearing resistance*</u>

With the forces decomposed in their horizontal and vertical components, the bearing resistance is checked according to *EN1993-1-8, table 3.4.*

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}}$$

$$\alpha_{b,end} = \min\left\{\frac{e_1}{3d_0}\ ;\ \frac{f_{ub}}{f_u}\ ; 1.0\right\} \qquad k_{1,end} = \min\left\{1.4\frac{p_2}{d_0} - 1.7\ ; 2.8\frac{e_2}{d_0} - 1.7\ ; 2.5\right\}$$

$$\alpha_{b,inner} = \min\left\{\frac{p_1}{3d_0} - \frac{1}{4}\ ;\ \frac{f_{ub}}{f_u}\ ; 1.0\right\} \qquad k_{1,inner} = \min\left\{1.4\frac{p_2}{d_0} - 1.7\ ; 2.5\right\}$$

In determining the distances e_1, e_2, p_1 and p_2, the direction of loading has to be interpreted as each one accounts for different phenomena. But forces may invert, as is the case with wind loading, and a bolt previously considered inner may very well be considered end and vice-versa. Hence, where there is that possibility, the following is adopted:

$$\alpha_{b,min} = \min\{\alpha_{b,end}\,;\alpha_{b,inner}\} \qquad\qquad k_{1,min} = \min\{k_{1,end}\,;k_{1,inner}\}$$

The horizontal and vertical design bearing resistances are summarized in Table 61 and Table 62.

Table 61 – Design bearing resistance, regarding the gusset, for the horizontal component

Bolt	b_1	b_2	b_3	b_4	b_5	b_6
e_1 [mm]	-----	-----	94.48	145.22	90.11	35.00
e_2 [mm]	107.42	72.89	38.48	-----	-----	72.05
p_1 [mm]	59.48	59.48	59.48	59.48	59.48	59.48
p_2 [mm]	68.09	68.09	68.09	68.09	68.09	68.09
α_b	$\alpha_{b,inner}$	$\alpha_{b,inner}$	$\alpha_{b,min}$	$\alpha_{b,min}$	$\alpha_{b,min}$	$\alpha_{b,min}$
	0.51	0.51	0.51	0.51	0.51	0.45
k_1	$k_{1,min}$	$k_{1,min}$	$k_{1,min}$	$k1_{,inner}$	$k1_{,inner}$	$k_{1,min}$
	1.97	1.97	1.97	1.97	1.97	1.97
$F_{b,bi,h,Rd}$ [kN]	246.7	246.7	246.7	246.7	246.7	216.0

Table 62 - Design bearing resistance, regarding the gusset, for the vertical component

Bolt	b_1	b_2	b_3	b_4	b_5	b_6
e_1 [mm]	107.42	72.89	38.48	-----	-----	72.05
e_2 [mm]	-----	-----	94.48	145.22	90.11	35.00
p_1 [mm]	68.09	68.09	68.09	68.09	68.09	68.09
p_2 [mm]	59.48	59.48	59.48	59.48	59.48	59.48
α_b	$\alpha_{b,min}$	$\alpha_{b,min}$	$\alpha_{b,min}$	$\alpha_{b,inner}$	$\alpha_{b,inner}$	$\alpha_{b,min}$
	0.62	0.62	0.49	0.62	0.62	0.62
k_1	$k_{1,inner}$	$k_{1,inner}$	$k_{1,min}$	$k_{1,min}$	$k_{1,min}$	$k_{1,min}$
	1.50	1.50	1.50	1.50	1.50	1.50
$F_{b,bi,v,Rd}$ [kN]	229.5	229.5	181.5	229.2	229.2	229.2

Design slip resistance

The design slip resistance of a pre-loaded bolt of Class 10.9 is determined in accordance to *EN1993-1-8 section 3.9.1* as follows:

$$F_{s,Rd} = \frac{k_s \cdot n \cdot \mu \cdot F_{p,C}}{\gamma_{M3}} \qquad F_{p,C} = 0.7 \cdot f_{ub} \cdot A_s$$

The following table summarizes the above calculations.

Table 63 – Design slip resistance - regarding the gusset

A_s	353	[mm^2]
$F_{p,C}$	247.1	[kN]
n	2	
k_s	1	
μ	0.5	
$F_{s,Rd}$	197.7	[kN]

Individual bolt checking

All bolts are individually checked and it is clear that slip resistance is the defining resistance. Regarding the bearing resistance, an additional simplified criteria is adopted, according to ECCS [7], to account for the interaction between the two components of force, namely:

$$\left(\frac{F_{V,bi,h,Ed}}{F_{b,bi,h,Rd}}\right)^2 + \left(\frac{F_{V,bi,v,Ed}}{F_{b,bi,v,Rd}}\right)^2 \leq 1$$

Table 64 – Individual checking of the bolts (bearing and slip resistance - regarding the gusset)

Bolt	$F_{V,bi,Ed}$ [kN]	$F_{s,Rd}$ [kN]	$F_{V,bi,h,Ed}$ [kN]	$F_{b,bi,h,Rd}$ [kN]	$F_{V,bi,v,Ed}$ [kN]	$F_{b,bi,v,Rd}$ [kN]	Interaction
b_1	187.3	197.7	78.3	246.7	170.1	229.2	0.65
b_2	157.9	197.7	121.9	246.7	100.4	229.2	0.44
b_3	168.3	197.7	165.4	246.7	30.6	181.5	0.48
b_4	101.5	197.7	35.7	246.7	95.0	229.2	0.19
b_5	83.2	197.7	79.3	246.7	25.3	229.2	0.12
b_6	130.7	197.7	122.9	216.0	44.5	229.2	0.36

Check 3 *- Bolts - regarding the angle*

It has already been mentioned that the diagonals are comprised of two angles back to back separated by battens and connecting to the gusset. In the Check 2 above, the bearing and slip resistances were compared with the acting shear force that discharges on the gusset. Now, a similar sequence is considered but with half the acting force as each diagonal is assumed to carry the load equally. All of the considerations mentioned in Check 2, relating to individual bearing and checking of bolts, remain valid.

Shear Forces

The gauge lines are both parallel and perpendicular to the angle borders and so it is noted that shear forces are analysed in the {h', v'} local axis. The corresponding values are shown in Table 65 (the acting forces are half the values shown in Table 59).

Table 65 – Design shear forces in the reference system {h',v'} - regarding the angle

Bolt	b_1	b_2	b_3	b_4	b_5	b_6
h_i' [mm]	81.25	16.25	-48.75	48.75	-16.25	-81.25
v_i' [mm]	-30.00	-30.00	-30.00	30.00	30.00	30.00
r_i' [mm]	86.67	34.15	57.18	57.18	34.15	86.67
$F_{M,bi}$ [kN]	54.8	21.61	36.18	36.18	21.61	54.84
$F_{M,bi,h'}$ [kN]	-19.0	-19.0	-19.0	19.0	19.0	19.0
$F_{M,bi,v'}$ [kN]	-51.4	-10.3	30.8	-30.8	10.3	51.4
$F_{N,bi}$ [kN]	-59.3	-59.3	-59.3	-59.3	-59.3	-59.3
$F_{V,bi,h',Ed}$ [kN]	-78.3	-78.3	-78.3	-40.3	-40.3	-40.3
$F_{V,bi,v',Ed}$ [kN]	-51.4	-10.3	30.8	-30.8	10.3	51.4
$F_{V,bi,Ed}$ [kN]	**93.6**	**78.9**	**84.1**	**50.8**	**41.6**	**65.3**

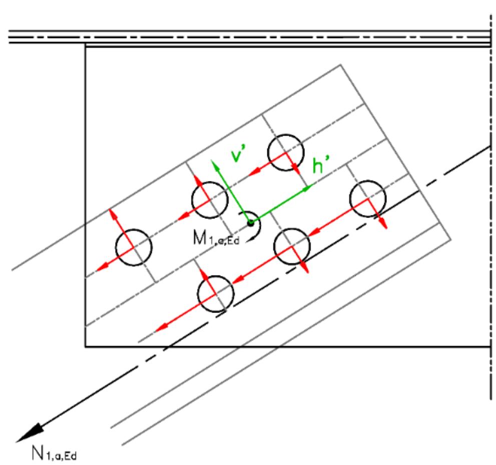


Figure 25 - Loading on bolts (forces in the {h', v'} system) - regarding the angle

Design bearing resistance

The design bearing resistances are summarized in Table 66 and Table *67*.

Table 66 – "Horizontal" component of the design bearing resistance, regarding the angle.

Bolt	**b_1**	**b_2**	**b_3**	**b_4**	**b_5**	**b_6**
e_1 [mm]	35	-----	-----	67.5	-----	-----
e_2 [mm]	-----	-----	-----	33	33	33
p_1 [mm]	65	65	65	65	65	65
p_2 [mm]	60	60	60	60	60	60
α_b	$\alpha_{b,end}$ 0.45	$\alpha_{b,inner}$ 0.58	$\alpha_{b,inner}$ 0.58	$\alpha_{b,min}$ 0.58	$\alpha_{b,inner}$ 0.58	$\alpha_{b,inner}$ 0.58
k_1	$k_{1,inner}$ 1.53	$k_{1,inner}$ 1.53	$k_{1,inner}$ 1.53	$k_{1,min}$ 1.53	$k_{1,min}$ 1.53	$k_{1,min}$ 1.53
$F_{b,bi,h',Rd}$ [kN]	100.9	131.2	131.2	131.2	131.2	131.2

Table 67 – "Vertical" component of the design bearing resistance, regarding the angle

Bolt	**b_1**	**b_2**	**b_3**	**b_4**	**b_5**	**b_6**
e_1 [mm]	-----	-----	-----	-----	33	33
e_2 [mm]	35	-----	-----	67.5	-----	-----
p_1 [mm]	60	60	60	60	60	60
p_2 [mm]	65	65	65	65	65	65
α_b	$\alpha_{b,inner}$ 0.52	$\alpha_{b,inner}$ 0.52	$\alpha_{b,inner}$ 0.52	$\alpha_{b,inner}$ 0.52	$\alpha_{b,min}$ 0.42	$\alpha_{b,min}$ 0.42
k_1	$k_{1,min}$ 1.80	$k_{1,inner}$ 1.80	$k_{1,inner}$ 1.80	$k_{1,min}$ 1.80	$k_{1,inner}$ 1.80	$k_{1,inner}$ 1.80
$F_{b,bi,v',Rd}$ [kN]	137.3	137.3	137.3	137.3	111.9	111.9

Design slip resistance

Table 68 – Design slip resistance - regarding the angle

As	353	[mm^2]
$F_{p,C}$	247.1	[kN]
n	1	
k_s	1	
μ	0.5	
$F_{s,Rd}$	98.8	[kN]

Individual bolt checking

Table 69 – Individual checking of the bolts (bearing and slip resistance - regarding the gusset)

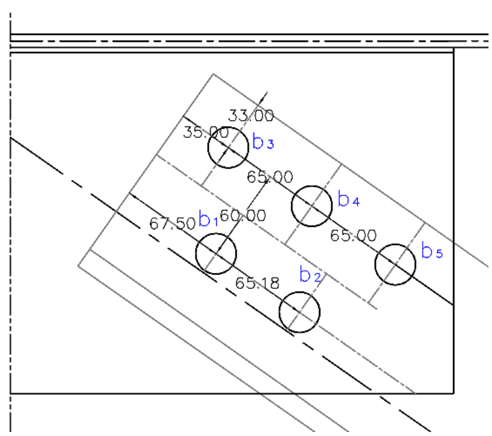
Bolt	$F_{V,bi,Ed}$ [kN]	$F_{s,Rd}$ [kN]	$F_{V,bi,h',Ed}$ [kN]	$F_{b,bi,h',Rd}$ [kN]	$F_{V,bi,v',Ed}$ [kN]	$F_{b,bi,v',Rd}$ [kN]	Interaction
b_1	93.6	98.8	78.3	100.9	51.4	137.3	0.74
b_2	78.9	98.8	78.3	131.2	10.3	137.3	0.36
b_3	84.1	98.8	78.3	131.2	30.8	137.3	0.41
b_4	50.8	98.8	40.3	131.2	30.8	137.3	0.14
b_5	41.6	98.8	40.3	131.2	10.3	111.9	0.10
b_6	65.3	98.8	40.3	131.2	51.4	111.9	0.31

6.1.3.3 Diagonal 13 to gusset

The checking carried out for the connection between the gusset and diagonal 13 is similar to that of diagonal 17. The only difference is that in this case, as the member is in tension, two additional checks are considered. For all the checks already considered in section 6.1.3.2, little to no commentary is added.

Table 70 – Design checklist for connection of diagonal 13 to gusset

No.	***Member***	***Check***
Check 1	Gusset plate	• Resistance of the cross-section
Check 2	Bolts regarding the gusset	• Bearing resistance • Slip resistance
Check 3	Bolts regarding the angle	• Bearing resistance • Slip resistance • Shear resistance
Check 4	Gusset plate & angle	• Net cross-section • Block tearing



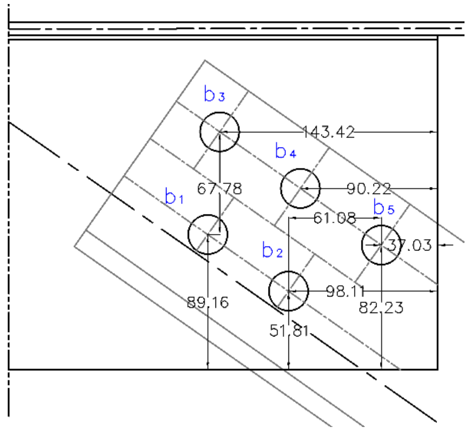


Figure 26 – Positioning of bolts in diagonal 13 connecting to the gusset (dimensions in [mm])

***Check 1* - Gusset plate**

Resistance of the cross-section

Although the diagonal is in tension, the same methodology as outlined in section 6.1.3.2, Check 1, is used for the evaluation of the cross-sectional resistance. The applied stress is determined as shown in Figure 27.

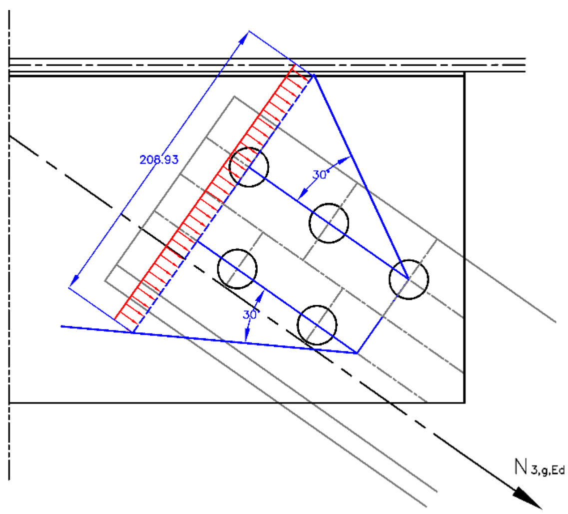


Figure 27 - Whitmore cross-section

The cross-sectional properties and design normal stress determination are as follows and summarized in Table 71.

$$A_{3,g} = t_g \cdot b_{3,eff} \qquad I_{3,g} = \frac{t_g \cdot b_{3,eff}^3}{12} \qquad \sigma_{Ed} = \frac{N_{3,g,Ed}}{A_{3,g}} + \frac{M_{3,g,Ed}}{I_{3,g}/z}$$

$b_{3,eff}$	208.93	[mm]		$N_{3,g,Ed}$ (vd. Table 17)	423	[kN]
t_g	25	[mm]		$M_{3,g,Ed}$ (vd. Table 17)	17.9	[kN.m]
$A_{3,g}$	5223	[mm^2]		σ_{Ed}	179.9	[MPa]
z	104.5	[mm]		σ_{Rd}	355	[MPa]
I_{3g}	19000331	[mm^4]		$\sigma_{Ed} \leq \sigma_{Rd}$	OK	

Table 71 – Cross-sectional properties, design forces and checking

<u>Check 2</u> – Bolts - regarding the gusset

<u>Shear Forces</u>

The same methodology adopted in 6.1.3.2 is now applied to diagonal 13. The results are shown in the following table.

Table 72 - Design shear forces in the reference system {h',v'} - regarding the gusset

Bolt	$\mathbf{b_1}$	$\mathbf{b_2}$	$\mathbf{b_3}$	$\mathbf{b_4}$	$\mathbf{b_5}$
$h_{i'}$ [mm]	-32.50	32.50	-65.00	0.00	65.00
$v_{i'}$ [mm]	-32.50	-32.50	32.50	32.50	32.50
$r_{i'}$ [mm]	43.23	43.23	71.59	30.00	71.59
$F_{M,bi}$ [kN]	52.2	52.2	86.5	36.2	86.5
$F_{M,bi,h'}$ [kN]	39.3	39.3	-39.3	-39.3	-39.3
$F_{M,bi,v'}$ [kN]	-39.3	39.3	-78.5	0.0	78.5
$F_{N,bi}$ [kN]	84.6	84.6	84.6	84.6	84.6
$F_{V,bi,h',Ed}$ [kN]	123.9	123.9	45.4	45.4	45.4
$F_{V,bi,v',Ed}$ [kN]	-39.3	39.3	-78.5	0.00	78.5
$\mathbf{F_{V,bi,Ed}}$ [kN]	**129.9**	**129.9**	**90.7**	**45.4**	**90.7**

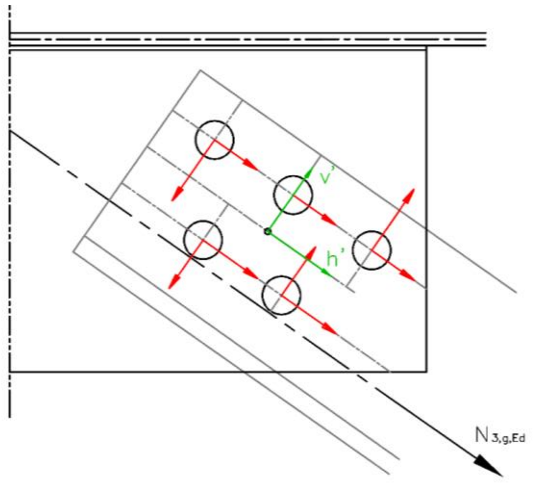


Figure 28 - Loading on bolts (forces in the {h', v'} system) - regarding the gusset

The forces are switched from the {h', v'} to the {h, v} system as follows:

$$F_{V,bi,h,Ed} = F_{V,bi,h',Ed}\cos(\alpha_3) + F_{V,bi,v',Ed}\sin(\alpha_3)$$

$$F_{V,bi,v,Ed} = -F_{V,bi,h',Ed}\sin(\alpha_3) + F_{V,bi,v',Ed}\cos(\alpha_3)$$

Table 73 - Design shear forces in the reference system {h,v} - regarding the gusset

Bolt	b_1	b_2	b_3	b_4	b_5
$F_{V,bi,Ed}$ [kN]	129.9	129.9	90.7	45.4	90.7
$F_{V,bi,h,Ed}$ [kN]	38.9	103.2	-38.3	26.0	90.3
$F_{V,bi,v,Ed}$ [kN]	-123.9	-78.9	-82.2	-37.2	7.9

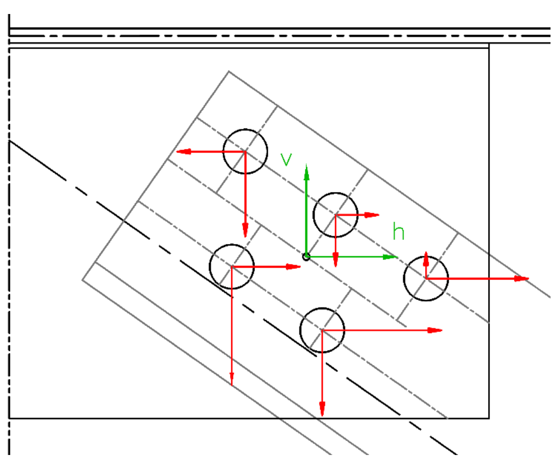


Figure 29 - Loading on bolts (forces in the {h, v} system) - regarding the gusset

Design bearing resistance

The horizontal and vertical design bearing resistances are summarized in Table 74 -and

Table *75* and were calculated with the same considerations mentioned in 6.1.3.2.

Table 74 - Design bearing resistance, regarding the gusset, for the horizontal component

Bolt	**b_1**	**b_2**	**b_3**	**b_4**	**b_5**
e_1 [mm]	-----	98.11	143.42	90.22	37.03
e_2 [mm]	89.16	51.81	-----	-----	82.23
p_1 [mm]	5.48	59.48	59.48	59.48	59.48
p_2 [mm]	68.09	68.09	68.09	68.09	68.09
α_b	$\alpha_{b,inner}$ 0.51	$\alpha_{b,min}$ 0.51	$\alpha_{b,min}$ 0.51	$\alpha_{b,min}$ 0.51	$\alpha_{b,min}$ 0.47
k_1	$k_{1,min}$ 1.97	$k_{1,min}$ 1.97	$k_{1,inner}$ 1.97	$k_{1,inner}$ 1.97	$k_{1,min}$ 1.97
$F_{b,bi,h,Rd}$ [kN]	246.7	246.7	246.7	246.7	228.5

Table 75 - Design bearing resistance, regarding the gusset, for the vertical component

Bolt	**b_1**	**b_2**	**b_3**	**b_4**	**b_5**
e_1 [mm]	89.16	51.81	-----	-----	82.23
e_2 [mm]	-----	98.11	143.42	90.22	37.03
p_1 [mm]	68.09	68.09	68.09	68.09	68.09
p_2 [mm]	59.48	59.48	59.48	59.48	59.48
α_b	$\alpha_{b,min}$ 0.62	$\alpha_{b,min}$ 0.62	$\alpha_{b,inner}$ 0.62	$\alpha_{b,inner}$ 0.62	$\alpha_{b,min}$ 0.62
k_1	$k_{1,inner}$ 1.50	$k_{1,min}$ 1.50	$k_{1,min}$ 1.50	$k_{1,min}$ 1.50	$k_{1,min}$ 1.50
$F_{b,bi,v,Rd}$ [kN]	229.2	229.2	229.2	229.2	229.2

Design slip resistance

Table 76 – Design slip resistance - regarding the gusset

A_s	353	[mm^2]
$F_{p,C}$	247.1	[kN]
n	2	
k_s	1	
μ	0.5	
$F_{s,Rd}$	197.7	[kN]

Individual bolt checking

Table 77 – Individual checking of the bolts (bearing and slip resistance - regarding the gusset)

Bolt	$F_{V,bi,Ed}$ [kN]	$F_{s,Rd}$ [kN]	$F_{V,bi,h,Ed}$ [kN]	$F_{b,bi,h,Rd}$ [kN]	$F_{V,bi,v,Ed}$ [kN]	$F_{b,bi,v,Rd}$ [kN]	Interaction
b_1	129.9	197.7	38.9	246.7	123.9	229.2	0.32
b_2	129.9	197.7	103.2	246.7	78.9	229.2	0.29
b_3	90.7	197.7	38.3	246.7	82.2	229.2	0.15
b_4	45.4	197.7	26.0	246.7	37.2	229.2	0.04
b_5	90.7	197.7	90.3	228.5	7.9	229.2	0.16

Check 3 ***- Bolts regarding the angle***

Shear Forces

Table 78 – Design shear forces in the reference system {h',v'} - regarding the angle

Bolt	b_1	b_2	b_3	b_4	b_5
$h_{i'}$ [mm]	-32.50	32.50	-65.00	0.00	65.00
$v_{i'}$ [mm]	-32.50	-32.50	32.50	32.50	32.50
$r_{i'}$ [mm]	43.23	43.23	71.59	30.00	71.59
$F_{M,bi}$ [kN]	-26.1	-26.11	-43.23	-18.12	-43.23
$F_{M,bi,h'}$ [kN]	-19.6	-19.6	19.6	19.6	19.6
$F_{M,bi,v'}$ [kN]	19.6	-19.6	39.3	0.0	-39.3
$F_{N,bi}$ [kN]	-42.3	-42.3	-42.3	-42.3	-42.3
$F_{V,bi,h',Ed}$ [kN]	-61.9	-61.9	-22.7	-22.7	-22.7
$F_{V,bi,v',Ed}$ [kN]	19.6	-19.6	39.3	0.00	-39.3
$\mathbf{F_{V,bi,Ed}}$ [kN]	**64.9**	**64.9**	**45.3**	**22.7**	**45.3**

Design bearing resistance

Table 79 – "Horizontal" component of the design bearing resistance, regarding the angle

Bolt	b_1	b_2	b_3	b_4	b_5
e_1 [mm]	67.5	-----	35	-----	-----
e_2 [mm]	-----	-----	33	33	33
p_1 [mm]	65	65	65	65	65
p_2 [mm]	60	60	60	60	60
α_b	$\alpha_{b,min}$ 0.58	$\alpha_{b,inner}$ 0.58	$\alpha_{b,min}$ 0.45	$\alpha_{b,inner}$ 0.58	$\alpha_{b,inner}$ 0.58
k_1	$k_{1,inner}$ 1.53	$k_{1,inner}$ 1.53	$k_{1,min}$ 1.53	$k_{1,min}$ 1.53	$k_{1,min}$ 1.53
$F_{b,bi,h',Rd}$ [kN]	131.2	131.2	100.9	131.2	131.2

Table 80 – "Vertical" component of the design bearing resistance, regarding the angle

Bolt	b_1	b_2	b_3	b_4	b_5
e_1 [mm]	-----	-----	33	33	33
e_2 [mm]	67.5	-----	35	-----	-----
p_1 [mm]	60	60	60	60	60
p_2 [mm]	65	65	65	65	65
α_b	$\alpha_{b,inner}$	$\alpha_{b,inner}$	$\alpha_{b,min}$	$\alpha_{b,inner}$	$\alpha_{b,inner}$
	0.52	0.52	0.42	0.52	0.52
k_1	$k_{1,min}$	$k_{1,inner}$	$k_{1,min}$	$k_{1,inner}$	$k_{1,inner}$
	1.80	1.80	1.80	1.80	1.80
$F_{b,bi,v',Rd}$ [kN]	137.3	137.3	111.9	137.3	137.3

Design slip resistance

Table 81 – Design slip resistance - regarding the angle

A_s	353	[mm^2]
$F_{p,C}$	247.1	[kN]
n	1	
k_s	1	
μ	0.5	
$F_{s,Rd}$	98.8	[kN]

Individual bolt checking

Table 82 - Individual checking of the bolts (bearing and slip resistance - regarding the angle)

Bolt	$F_{V,bi,Ed}$ [kN]	$F_{s,Rd}$ [kN]	$F_{V,bi,h',Ed}$ [kN]	$F_{b,bi,h',Rd}$ [kN]	$F_{V,bi,v',Ed}$ [kN]	$F_{b,bi,v',Rd}$ [kN]	Interaction
b_1	64.9	98.8	61.9	131.2	19.6	137.3	0.24
b_2	64.9	98.8	61.9	131.2	19.6	137.3	0.24
b_3	45.3	98.8	22.7	100.9	39.3	111.9	0.17
b_4	22.7	98.8	22.7	131.2	0	137.3	-
b_5	45.3	98.8	22.7	131.2	39.3	137.3	0.11

Shear Resistance

Table 83 - Group of bolts checking - regarding the angle

$N_{3,a,h',Ed}$	173	[kN]
$F_{gr,b,h',Rd}$	626	[kN]
$N_{3,a,h',Ed} \leq F_{gr,b,h',Rd}$	OK	

$N_{3,a,v',Ed}$	121	[kN]
$F_{gr,b,v',Rd}$	661	[kN]
$N_{3,a,v',Ed} \leq F_{gr,b,v',Rd}$	OK	

Check 4 - ***Gusset and angle***

According to *EN1993-1-8 Table 3.2,* for a connection in tension the resistance of the net cross-section of the members has to be verified.

Net cross-section - Gusset component

There is no indication in the code for determining the acting force on the net area but a possible determination [7] is presented as follows:

$$N_{3,g,bt,Ed} = n_b \frac{N_{3,g,Ed}}{n_{bt}} \qquad N_{Rd} = \frac{A_{net,3} f_y}{\gamma_{M0}} \qquad A_{net,3} = t_g \cdot (34.5 + 72.7 + 72.1)$$

where n_b is the number of bolts at the net cross-section, n_{bt} is the total number of bolts in the connection and $A_{net,3}$ is the area represented in Figure 30 with a red line (the lengths of the segments are given above).

Table 84 – Net cross-section and design force and resistance

n_b	2	
n_{bt}	5	
$A_{net,3}$	3927	[mm^2]
$N_{3,g,bt,Ed}$	169.2	[kN]
N_{Rd}	1394	[kN]
$N_{3,g,bt,Ed} \leq N_{Rd}$	OK	

Net cross-section - Angle component

The angle component has already been verified in section 5.2

Block tearing - Overview

The areas associated to the shear face and the tension face of the bolt group are different when analysing the gusset and angle components. Therefore, similarly to bearing, the checking of block tearing resistance is conducted in both the gusset and the angle. According to *EN 1993-1-8, clause 3.10.2 (3),* the design block tearing resistance for a bolt group subjected to eccentric loading is given by:

$$V_{eff,2,Rd} = \frac{0.5 \cdot f_u \cdot A_{nt}}{\gamma_{M2}} + \frac{f_y \cdot A_{nv}/\sqrt{3}}{\gamma_{M0}}$$

The following sections show the areas considered in both components as well as the respective design resistances and forces.

Block tearing - Gusset component

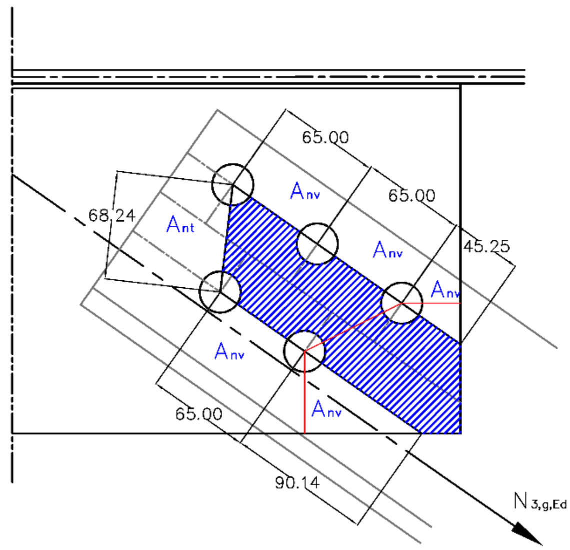


Figure 30 – Definition of block tearing areas - regarding the gusset

Table 85 - Check of block tearing resistance - regarding the gusset

A_{nt}	1706	[mm^2]
A_{nv}	4956	[mm^2]
$V_{eff,2,Rd}$	1886	[kN]
$N_{3,g,Ed}$	423.1	[kN]
$N_{3,g,Ed} \leq V_{eff,2,Rd}$	OK	

Block tearing - Angle component

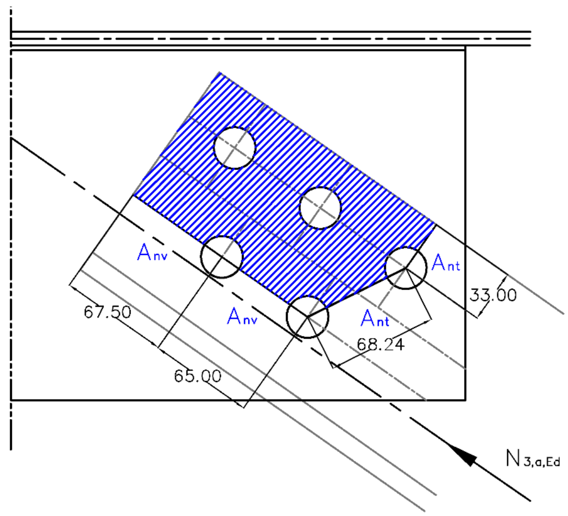


Figure 31 - Definition of block tearing areas - regarding the angle

Table 86 – Check of block tearing resistance - regarding the angle

A_{nt}	1549	[mm^2]
A_{nv}	2963	[mm^2]
$V_{eff,2,Rd}$	1397	[kN]
$N_{3,a,Ed}$	211.6	[kN]
$N_{3,a,Ed} \leq V_{eff,2,Rd}$	OK	

6.2 Detailed design of a continuous chord connection using a splice plate

The ability to assure continuity in the chords implies that equilibrium between the two connecting sides has to be established. This can be done by means of welding. A different approach, and one that is assumed in this thesis, is the use of splice plates with bolts, both at the web and flanges.

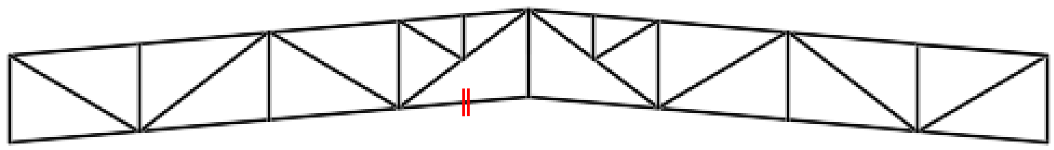


Figure 32- Location of the spliced connection in the lower chord (in red)

6.2.1 Loads and general geometry

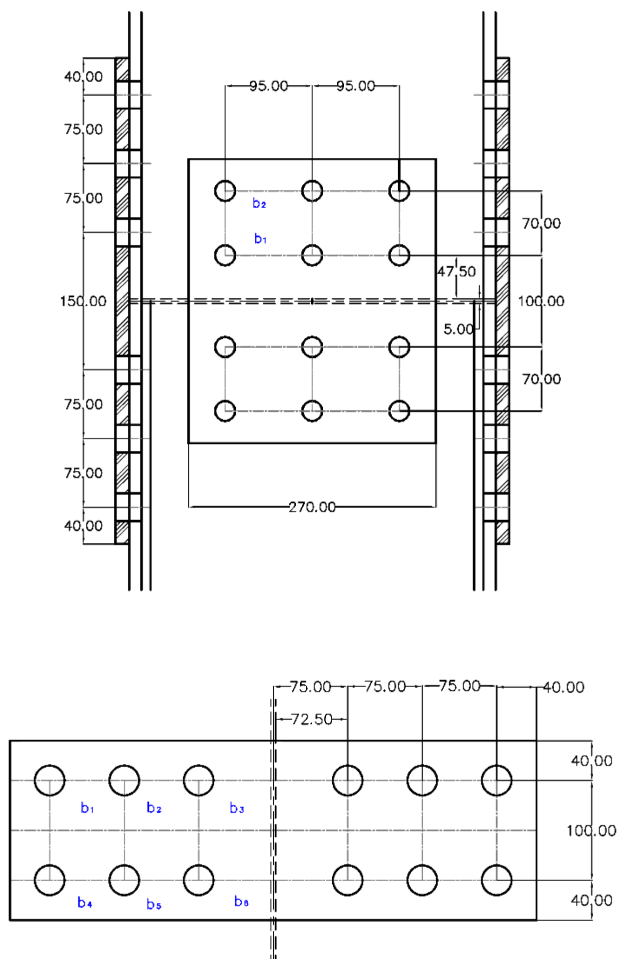


Figure 33 - Positioning of plates and holes in the spliced connection

The acting force at the connection is as follows:

Table 87 – Design forces at the spliced connection

Member	N [kN]	V [kN]	M [kN.m]
Chord 1-4	1866	1.2	-5.7

For the design of this connection a plastic distribution of internal forces is considered. The axial force is distributed between the web and flanges proportionally to the area of each component of the cross section. The shear force and bending moment are carried by the flanges.

$$A_w = (h - 2t_f) \cdot t_w \qquad A_f = \frac{(A - A_w)}{2}$$

Table 88 – Areas of the web and flange; eccentricity in flange

A_w	3208	[mm^2]
A_f	2430	[mm^2]
e_f	150	[mm]

$$N_w = \frac{N_{Ed} \cdot A_w}{A} \qquad N_f = \frac{(N_{Ed} - N_w)}{2} \qquad V_f = \frac{V_{Ed}}{2} \qquad M_f = \frac{M_{Ed}}{2} + e_f \cdot V_f$$

Table 89 – Internal forces on the web and flange

Web	N_w	708.7	[kN]
Flange	N_f	578.7	[kN]
	V_f	0.6	[kN]
	M_f	2.9	[kN.m]

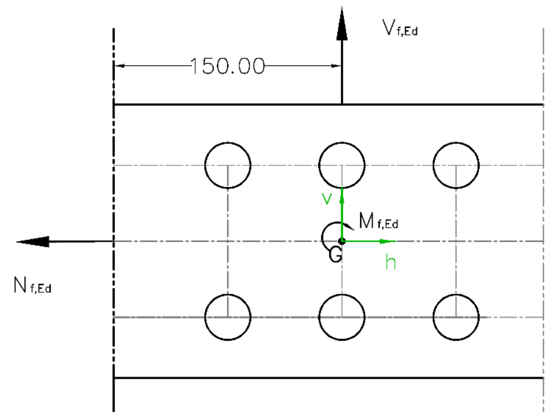


Figure 34 – Statically equivalent forces at the centre of gravity of the flange bolt group

6.2.2 Web component

Table 90 - Design checklist for the web component of the chord connection

No.	***Member***	***Check***
Check 1	Bolts - regarding the web & plate	• Bearing resistance • Slip resistance • Shear resistance
Check 2	Web & Plate	• Resistance of cross-section • Block tearing

In this section, three slight differences should be noted. Firstly, in opposition to chapter 5 (where all bolts were considered), only two bolts need be considered (labelled as bolts 1 and 2 in Figure 33). The reason is that, for the purpose of safety checking, the boundary conditions and the loading only differ between these two, and therefore there is no need for all bolts to be considered. Secondly, the results are presented simultaneously for both the web and plate component; this differs from the presentation in 6.1.3.2 where the gusset and angle components were separated for an easier interpretation. Thirdly, block tearing is not under eccentric loading and a modification to the formula previously presented is needed. Other than these aspects, bearing, slip and group of bolts resistances are evaluated under precisely the same considerations mentioned in 6.1.3.2 and 6.1.3.3 and therefore little to no commentary is added.

<u>Check 1</u> - Bolts regarding the web and the plate

<u>Shear forces</u>

All the checks mentioned in Table 90 have to be carried out, and therefore quantifying the design force on each bolt is necessary. The calculation is provided as follows:

$$F_{V,Ed,w} = \frac{N_w}{6} \qquad F_{V,Ed,p} = \frac{N_W/_2}{6}$$

Table 91 – Shear forces acting on each component (web and plate)

$F_{V,Ed,w}$	118.1	[kN]
$F_{V,Ed,p}$	59.1	[kN]

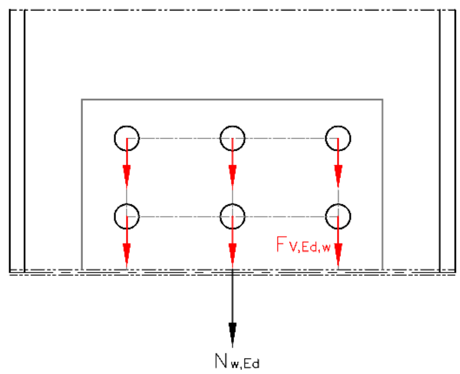


Figure 35 - Loading on bolts - regarding the web

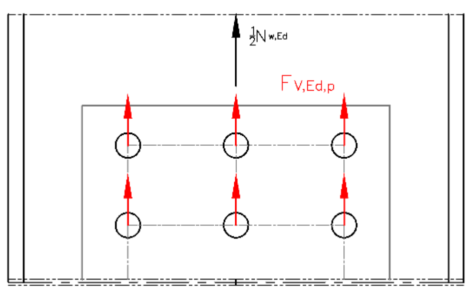


Figure 36 - Loading on bolts - regarding the plate

Bearing resistance

Table 92 – Design bearing resistances of the web and plate components

Web component

Bolt	b_1	b_2
e_1 [mm]	47.5	-----
e_2 [mm]	-----	-----
p_1 [mm]	70	70
p_2 [mm]	95	95
α_b	$\alpha_{b,end}$ 0.72	$\alpha_{b,inner}$ 0.81
k_1	$k_{1,inner}$ 2.50	$k_{1,inner}$ 2.50
$F_{b,bi,Rd}$ [kN]	126.3	142.2

Plate component

Bolt	b_1	b_2
e_1 [mm]	-----	35
e_2 [mm]	40	40
p_1 [mm]	70	70
p_2 [mm]	95	95
α_b	$\alpha_{b,inner}$ 0.81	$\alpha_{b,min}$ 0.53
k_1	$k_{1,min}$ 2.50	$k_{1,min}$ 2.50
$F_{b,bi,Rd}$ [kN]	248.1	162.3

Slip resistance

A_s	245	[mm^2]
$F_{p,C}$	171.5	[kN]

Table 93 – Design slip resistance regarding the web and the plate

	Web	Plate
n	2	1
k_s	1	1
μ	0.5	0.5
$F_{s,Rd}$ [kN]	137.2	68.6

Individual bolt checking

Table 94 – Individual checking of the bolts regarding the web component

Web component

Bolt	$F_{V,Ed,w}$ [kN]	$F_{s,Rd,w}$ [kN]	$F_{V,Ed,w}$ [kN]	$F_{b,bi,Rd,w}$ [kN]
b_1	118.1	137.2	118.1	126.3
b_2	118.1	137.2	118.1	142.2

Table 95 – Individual checking of the bolts regarding the plate component

Plate component				
Bolt	**$F_{V,Ed,p}$ [kN]**	**$F_{s,Rd,p}$ [kN]**	**$F_{V,Ed,p}$ [kN]**	**$F_{b,bi,Rd,p}$ [kN]**
b_1	59.1	68.6	59.1	248.1
b_2	59.1	68.6	59.1	162.3

Shear resistance

Table 96 - Group of bolts checking regarding the web components

Web component		
A_s	459	[mm^2]
α_v	0.6	
$F_{v,Rd}$	220.3	[kN]
n	6	
min	126.3	[kN]
$F_{gr,b,Rd,w}$	757.6	[kN]
$N_w \leq F_{gr,b,Rd,w}$	OK	

***Check 2* - Web and Plate**

Local cross-sectional resistance

The local cross-sectional resistance is evaluated in terms of net cross-section resistance. According to *EN1993-1-8, Table 3.2*, the plastic resistance of the net cross-section should be verified as follows:

$$\sum_{1}^{n_b} F_{V,Ed} \leq N_{net,Rd}$$

The web and plate components are analysed separately giving meaning to the term "local" as both components will have different implications.

$$A_{w,net} = A_w - 3d_{0,w}t_w \qquad N_{w,net,Rd} = \frac{A_{w,net}f_y}{\gamma_{M0}} \qquad A_{p,net} = A_p - 3d_{0,p}t_p \qquad N_{p,net,Rd} = \frac{A_{p,net}f_y}{\gamma_{M0}}$$

Table 97 – Checking of net area resistance

Web component		
$A_{w,net}$	2317	-----
$N_{w,net,Rd}$	822.5	[kN]
$\sum F_{V,Ed}$	354.4	[kN]
$F_{V,Ed} \leq N_{W,net,Rd}$	OK	

Plate component		
$A_{p,net}$	4560	[mm^2]
$N_{p,net,Rd}$	1619	[kN]
$\sum F_{V,Ed}$	177.2	[kN]
$F_{V,Ed} \leq N_{p,net,Rd}$	OK	

Block tearing

According to *EN1993-1-8 section 3.10.2*, for a symmetric bolt group subjected to concentric loading the design block tearing resistance is determined as follows:

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

For block tearing regarding the web component only one area is considered (shown in Figure 37). Regarding the plate component, whilst two different areas are possible (shown in Figure 38)

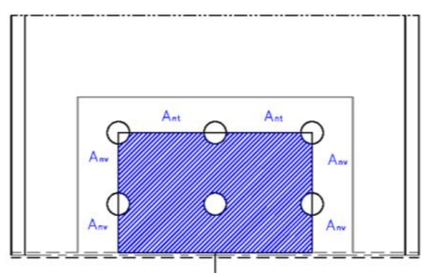


Figure 37 - Block tearing area - regarding the web component

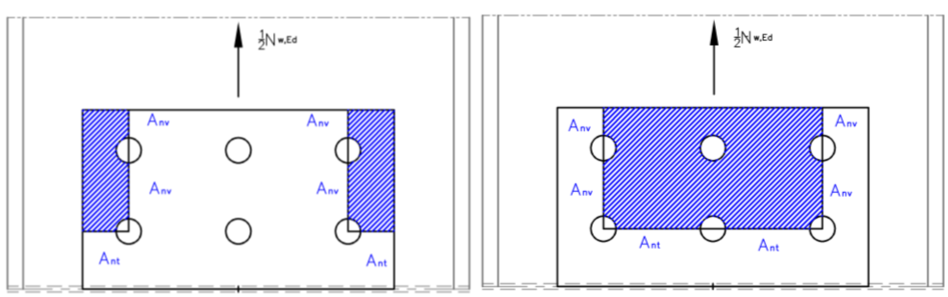


Figure 38 - Block tearing area - regarding the flange component

Table 98 - Check of block tearing resistance - regarding the web and plate

Web component		
A_{nt}	1971	[mm^2]
A_{nv}	2282	[mm^2]
$V_{eff,1,Rd}$	1272	[kN]
$N_W \leq V_{eff,1,Rd}$	OK	

Plate component		
A_{nt}	870	[mm^2]
A_{nv}	2160	[mm^2]
$V_{eff,1,Rd}$	798	[kN]
$N_W \leq V_{eff,1,Rd}$	OK	

6.2.3 Flange component

Table 99 - Design checklist for the flange component of the chord connection

No.	***Member***	***Check***
Check 1	Bolts (regarding the flange and the plate)	• Bearing resistance • Slip resistance • Group fasteners
Check 2	Flange and Plate	• Local Resistance of cross-section
Check 3	Flange and Plate	• Block tearing

There are two important differences with all the checks carried out at this stage. The first, regarding shear forces, is the fact that both the flange and the plate are loaded with the same force. As each flange is connected to only one plate, in opposition to the web component where two plates are used one on each side, the applied shear force on the flange transfers entirely to the plate. Both components, flange and plate, are evaluated as they have different thicknesses and different boundary conditions (by means of e_1 and $e_{2,}$ as shown in table 101 and 102) The second difference is regarding block tearing where previously only concentric or eccentric loading existed. Now, both types are present and additional checking is carried out. For all the other checks, such as bearing, slip group and net cross-section, little to no commentary is added as the considerations of previous sections maintain their validity.

Check 1 – Bolts regarding the flange

Shear Forces

As usual, when bending shear and axial force exist simultaneously, a decomposition of the effect of these on the bolts is perhaps the easiest and effective way of understanding the loading to which each bolt is subjected to. Similarly to section 6.1.3.2, the shear forces are summarized as follows:

Table 100 – Design shear forces on the bolts (regarding both the flange and plate)

Bolt	**b_1**	**b_2**	**b_3**	**b_4**	**b_5**	**b_6**
h_i [mm]	-75	0	75	-75	0	75
v_i [mm]	50	50	50	-50	-50	-50
r_i [mm]	90.14	50	90.14	90.14	50	90.14
$F_{M,bi}$ [kN]	7.1	3.9	7.1	7.1	3.9	7.1
$F_{M,bi,h}$ [kN]	3.9	3.9	3.9	-3.9	-3.9	-3.9
$F_{M,bi,v}$ [kN]	5.9	0	-5.9	5.9	0	-5.9
$F_{N,bi}$ [kN]	-96.5	-96.5	-96.5	-96.5	-96.5	-96.5
$F_{V,bi}$ [kN]	0.1	0.1	0.1	0.1	0.1	0.1
$F_{V,bi,h,Ed}$ [kN]	-92.5	-92.5	-92.5	-100.4	-100.4	-100.4
$F_{V,bi,v,Ed}$ [kN]	5.9	0.1	-5.8	5.9	0.1	-5.8
$F_{V,bi,Ed}$ [kN]	**92.7**	**92.5**	**92.7**	**100.5**	**100.4**	**100.5**

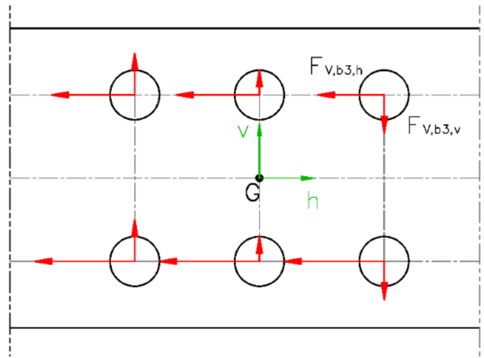


Figure 39 - Loading of bolts in reference system {h, v} - regarding the flange

Bearing Resistance

Horizontal loading

Table 101 - Design bearing resistance for horizontal component (regarding both the flange and plate)

Flange component

Bolt	**b_1**	**b_2**	**b_3**	**b_4**	**b_5**	**b_6**
e_1 [mm]	67.5	-----	-----	67.5	-----	-----
e_2 [mm]	40	40	40	40	40	40
p_1 [mm]	70	70	70	70	70	70
p_2 [mm]	100	100	100	100	100	100
α_b	$\alpha_{b,min}$ 0.53	$\alpha_{b,inner}$ 0.53	$\alpha_{b,inner}$ 0.53	$\alpha_{b,min}$ 0.53	$\alpha_{b,inner}$ 0.53	$\alpha_{b,inner}$ 0.53
k_1	$k_{1,min}$ 2.50	$k_{1,min}$ 2.50	$k_{1,min}$ 2.50	$k_{1,min}$ 2.50	$k_{1,min}$ 2.50	$k_{1,min}$ 2.50
$F_{b,bi,Rd}$ [kN]	196.2	196.2	196.2	196.2	196.2	196.2

Plate component

Bolt	**b_1**	**b_2**	**b_3**	**b_4**	**b_5**	**b_6**
e_1 [mm]	-----	-----	35	-----	-----	35
e_2 [mm]	40	40	40	40	40	40
p_1 [mm]	70	70	70	70	70	70
p_2 [mm]	100	100	100	100	100	100
α_b	$\alpha_{b,inner}$ 0.53	$\alpha_{b,inner}$ 0.53	$\alpha_{b,min}$ 0.39	$\alpha_{b,inner}$ 0.53	$\alpha_{b,inner}$ 0.53	$\alpha_{b,min}$ 0.39
k_1	$k_{1,min}$ 2.50	$k_{1,min}$ 2.50	$k_{1,min}$ 2.50	$k_{1,min}$ 2.50	$k_{1,min}$ 2.50	$k_{1,min}$ 2.50
$F_{b,bi,Rd}$ [kN]	218	218	160.7	218	218	160.7

Vertical loading

Table 102 - Design bearing resistance for vertical component (regarding both the flange and plate)

Flange component

Bolt	**b_1**	**b_2**	**b_3**	**b_4**	**b_5**	**b_6**
e_1 [mm]	40	40	40	40	40	40
e_2 [mm]	67.5	-----	-----	67.5	-----	-----
p_1 [mm]	100	100	100	100	100	100
p_2 [mm]	70	70	70	70	70	70
α_b	$\alpha_{b,min}$	$\alpha_{b,min}$	$\alpha_{b,min}$	$\alpha_{b,min}$	$\alpha_{b,min}$	$\alpha_{b,min}$
	0.44	0.44	0.44	0.44	0.44	0.44
k_1	$k_{1,min}$	$k_{1,inner}$	$k_{1,inner}$	$k_{1,min}$	$k_{1,inner}$	$k_{1,inner}$
	1.57	1.57	1.57	1.57	1.57	1.57
$F_{b,bi,Rd}$ [kN]	103.6	103.6	103.6	103.6	103.6	103.6

Plate component

Bolt	**b_1**	**b_2**	**b_3**	**b_4**	**b_5**	**b_6**
e_1 [mm]	40	40	40	40	40	40
e_2 [mm]	-----	-----	35	-----	-----	35
p_1 [mm]	100	100	100	100	100	100
p_2 [mm]	70	70	70	70	70	70
α_b	$\alpha_{b,inner}$	$\alpha_{b,min}$	$\alpha_{b,min}$	$\alpha_{b,min}$	$\alpha_{b,min}$	$\alpha_{b,min}$
	0.44	0.44	0.44	0.44	0.44	0.44
k_1	$k_{1,inner}$	$k_{1,inner}$	$k_{1,min}$	$k_{1,inner}$	$k_{1,inner}$	$k_{1,min}$
	1.57	1.57	1.57	1.57	1.57	1.57
$F_{b,bi,Rd}$ [kN]	115.1	115.1	115.1	115.1	115.1	115.1

<u>Slip resistance</u>

Table 103 - Design slip resistance (regarding both the flange and the plate)

A_s	459	[mm^2]
$F_{p,C}$	321.3	[kN]
n_w	1	
k_s	1	
μ	0.5	
$F_{s,Rd}$	128.5	[kN]

Table 104 - Individual checking of the bolts (regarding both the flange and plate)

Flange component

Bolt	$F_{V,Ed,f}$ [kN]	$F_{s,Rd,f}$ [kN]	$F_{V,bi,h,Ed}$ [kN]	$F_{b,bi,h,Rd}$ [kN]	$F_{V,bi,v,Ed}$ [kN]	$F_{b,bi,v,Rd}$ [kN]	Interaction
b_1	92.7	128.5	92.5	196.2	5.9	103.6	0.23
b_2	92.5	128.5	92.5	196.2	0.1	103.6	0.22
b_3	92.5	128.5	92.5	196.2	5.8	103.6	0.23
b_4	100.4	128.5	100.4	196.2	5.9	103.6	0.26
b_5	100.4	128.5	100.4	196.2	0.1	103.6	0.26
b_6	100.4	128.5	100.4	196.2	5.8	103.6	0.26

Plate component

Bolt	$F_{V,Ed,p}$ [kN]	$F_{s,Rd,p}$ [kN]	$F_{V,bi,h,Ed}$ [kN]	$F_{b,bi,h,Rd}$ [kN]	$F_{V,bi,v,Ed}$ [kN]	$F_{b,bi,v,Rd}$ [kN]	Interaction
b_1	92.5	128.5	92.5	218	5.9	115.1	0.18
b_2	92.5	128.5	92.5	218	0.1	115.1	0.18
b_3	92.5	128.5	92.5	160.7	5.8	115.1	0.33
b_4	100.4	128.5	100.4	218	5.9	115.1	0.21
b_5	100.4	128.5	100.4	218	0.1	115.1	0.21
b_6	100.4	128.5	100.4	160.7	5.8	115.1	0.39

Shear resistance

Table 105 - Group of bolts checking regarding the flange component

Flange component		
N_f	578.7	[kN]
$F_{gr,b,h,Rd,f}$	596.9	[kN]
$N_f \leq F_{gr,b,h,Rd,f}$	OK	

Check 2 - Flange and Plate

Table 106 – Checking of net area resistance

Web component		
$A_{f,net}$	1620	[mm^2]
$N_{f,net,Rd}$	575.1	[kN]
$F_{V,Ed}$	201.1	[kN]
$F_{V,Ed} \leq N_{f,net,Rd}$	OK	

Plate component		
$A_{p,net}$	1530	[mm^2]
$N_{p,net,Rd}$	543.2	[kN]
$F_{V,Ed}$	201.1	[kN]
$F_{V,Ed} \leq N_{p,net,Rd}$	OK	

Check 3 - Flange and Plate

Up until this point, block tearing was checked under concentric or eccentric loading. Here, a different case is analysed as both concentric and eccentric loading are in effect. Thus, two separate checks have to be conducted for each component - concentric and eccentric loading for the flange and plate. Regarding the flange, as it is attached to the web, the eventual shear face of the tearing area would be interrupted by the web and therefore only concentric loading is considered (as shown in Figure 40). Figure 41 displays the block tearing areas for the eccentric loading.

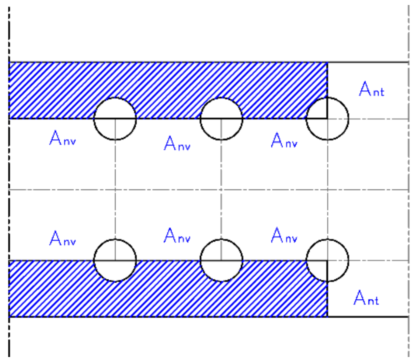


Figure 40 – Block tearing - regarding the flange

Table 107 – Block tearing resistance - regarding the flange (concentric loading)

A_{nt}	675	[mm^2]
A_{nv}	4050	[mm^2]
$V_{eff,1,Rd}$	1106	[kN]
N_f	578.7	[kN]
$N_f \leq V_{eff,1,Rd}$	OK	

Table 108 – Checking of block tearing resistance - regarding the plate (concentric and eccentric loading)

concentric loading			eccentric loading		
A_{nt}	750	[mm^2]	A_{nt}	1725	[mm^2]
A_{nv}	3450	[mm^2]	A_{nv}	1425	[mm^2]
$V_{eff,1,Rd}$	1013	[kN]	$V_{eff,2,Rd}$	644	[kN]
N_p	578.7	[kN]	V_p	0.6	[kN]
$N_p \leq V_{eff,1,Rd}$	OK		$N_p \leq V_{eff,2,Rd}$	OK	

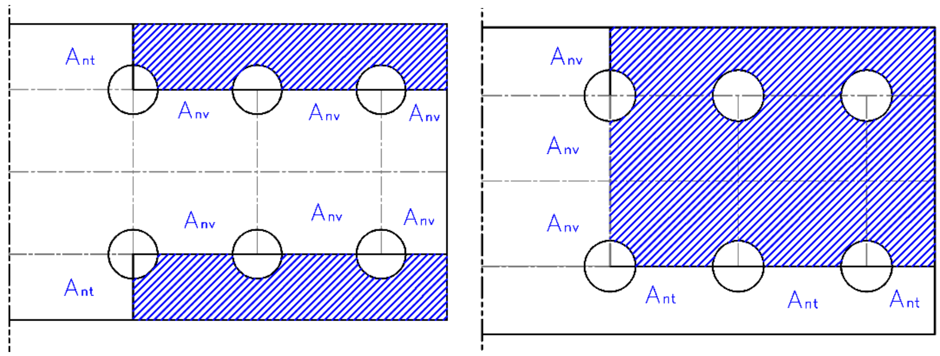


Figure 41 – Concentric and eccentric block tearing regarding the plate

7 Conclusion and Future Developments

7.1 General Conclusions

Throughout the thesis many aspects of *EN 1993* - Part 1 and Part 8 were explored and a greater understanding on the design process and behaviour of large span roof structures was attained. All of the objectives initially mentioned were achieved by verifying safety for all the main members that comprise the structure, under ULS, including the bracing system. Connections were analysed in detail and all the necessary checks, under ULS, were satisfied. Safety was also satisfied regarding SLS of the main truss as well as the evaluation of the effects of slack in the bolted connections of the structure (main trusses). Further analyses were done regarding the layout of the beams – positioned as standing up or flat – as well as confirming the validity of the usual model that assumes chord continuity with pined diagonals and posts.

7.2 Future Developments

Possible improvements and future developments to the material presented are as follows:

- A 3D model comprising the roof, columns and foundations in order to fully design the industrial building. Here the steel columns, the connections of these to the concrete foundations as well as the foundations would be designed. Dynamic analysis would be needed to evaluate the effect of earthquakes on the columns.
- Detailed design of the connections between the purlins and the chords.
- A 3D model of the gusset plate and connection bolts for comparison with the approach introduced by Whitmore for the evaluation of the peak stresses.
- Compare different layouts of the roof structure in order to compare both structural performance as well as cost.
- Develop a quantity work map and estimate budget of the proposed structure.

7 References

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Eurocodes

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EN1991-4 – European committee of standardization (CEN), 'Eurocode 1 Part 1-4: General actions: Wind actions, March 2009.

EN993-1-1 – European committee of standardization (CEN), 'Eurocode 3 Part 1-3: General rules: and rules for buildings, March 2004.

EN1993-1-5 – European committee of standardization (CEN), 'Eurocode 3 Part 1-3: Plated structural elements, March 2006.

EN1993-1-8 – European committee of standardization (CEN), 'Eurocode 3 Part 1-3: Design of structural joints elements, March 2006.