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

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1 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

Introduction

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.

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 **Error! Reference source not found.** (top), 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, **Error! Reference source not found.** (bottom), designed for the Birmingham railway.

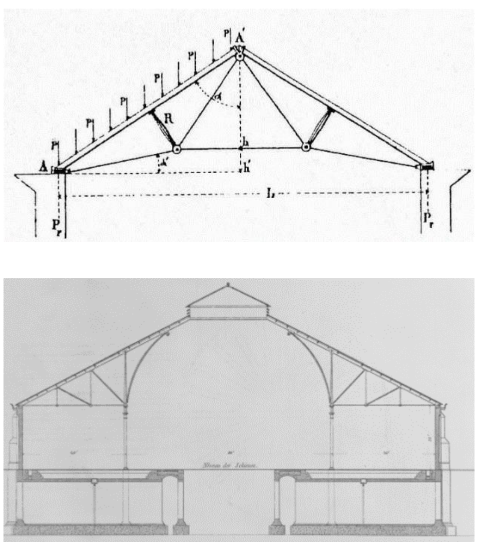


Figure 1- Camille Polonceau truss (top); Robert Stephenson's locomotive roundhouse (bottom)

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.

2 Adopted solution

The adopted solution (shown in Figure 2) is elaborated bearing the principals that are common practice in designing roof steel structures, such as the adoption of a lattice structure (in opposition to a portal frame where the span is greater than 20-25 meters) with slenderness of 1/12 and diagonals connecting with chords at angles of 35º.

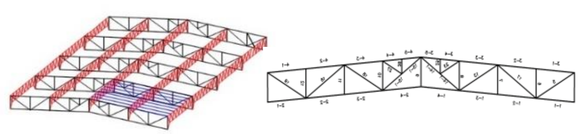


Figure 2 – General layout of the roof structure and main truss. Purlins (blue); Bracing truss (red); Main truss (black)

All structural steel members, including gusset plates, have the same grade of steel (S 355). This simplifies the computation of the multiple safety checks. The members that make up the structure are summarized in Table 1**Error! Reference source not found.**. 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.

Table 1 - 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 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.

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

Table 2 - 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.

3 Modeling

3.1 Overview

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 bracing truss is modelled as a Warren truss with a continuous chords and pinned diagonals as displayed in Figure 3. 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.

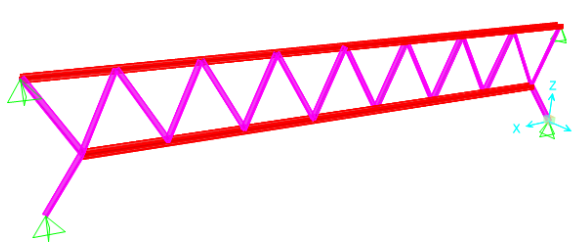


Figure 3 - Model of the bracing truss.

The main truss (shown in Figure 4) 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.

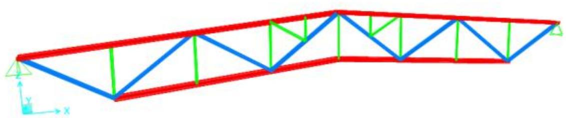


Figure 4 - Model of the main truss

3.2 Stiffness and secondary forces

With increasing stiffness of the chords increasing bending moments follow. 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 as shown in Figure 5.

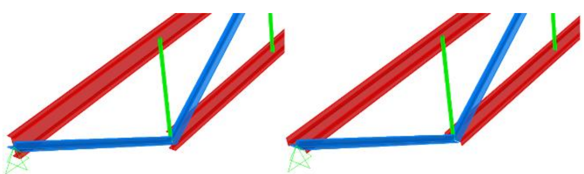


Figure 5 - Different layouts for the chords: standing up (left) and flat (right)

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 moments when compared with the profiles layout as flat. The maximum bending moment on the chord increases from 6.3 kN.m, to 136.5 kN.m as profiles change position from flat to vertical (22 times greater).

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 is 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 3).

Table 3 - 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 (shown in Table 4)

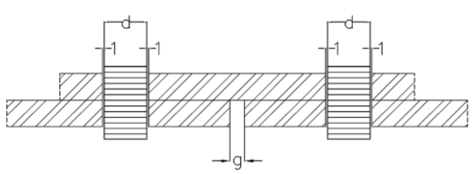
Table 4 - Axial force [in kN] comparison between rigid and pinned diagonals with different chord layouts

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

However, even with an adequate depth, the clearances of the 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 an increase of the length of the members in compression or in tension, respectively.

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 6 illustrates this phenomena for a spliced connection between plates in tension.



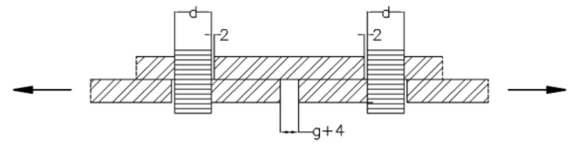


Figure 6 - Taking up slack on bolts subjected to shear (dimensions in [mm])

To analyse the effects of slack recovery in the connections, the principal of virtual work is a simple yet very convenient form of analysis. The virtual unit load is applied to the truss at mid-span 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. 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:

$$\bar{1}\delta = \sum_{members} \int_0^L \left(\frac{N}{EA}\right) \cdot N' \, dx_3 = 4 \times \sum_{members} |N'| = 53.2\, mm$$

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.

4 Verification of Members

In this chapter, the proper subject of concern is to determine the profiles that satisfy the safety checking of members in accordance to *EN1993.* To illustrate the principals and checking procedures that have to be considered in the design of such structures, only some situations are analysed under LL combination in ULS.

4.1 Members in Compression

The members in compression are checked as follows:

Table 5 – Design checklist for members in compression.

No.	Member	Check
Check 1	Diagonals of the main trusses	Resistance of the cross-section Buckling resistance of the member
Check 2	Upper Chord of the main trusses	
Check 3	Upper Chord of the bracing trusses	

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

Resistance of the cross-section

In order to evaluate the 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.

$$\sigma_{a,max} = \frac{N_{1,a,Ed}}{A_{1,a}} + \frac{M_{1,a,Ed}}{I_{1,a}/u}$$

Buckling resistance

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 – column-beam verification.

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

Forces acting on the truss

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.

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} = 1348 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 1000th of the span of the main truss. Thus, the following result is obtained:

$$q_d = \frac{8 \cdot \sum N_{Ed}}{L^2}\left(\frac{L}{500} + \frac{L}{1000}\right) = 2.4\% \frac{N_{Ed}}{L}$$

$$\rightarrow Q_p = \; q_d \cdot L^* = 9.1 \text{ kN}$$

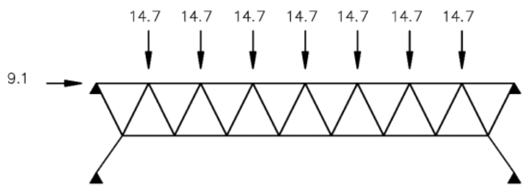


Figure 7 - Bracing truss loading under LL combination in ULS (forces in [kN])

5 Verification of Connections

5.1 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. The location and general layout of the joint analysed is shown below.

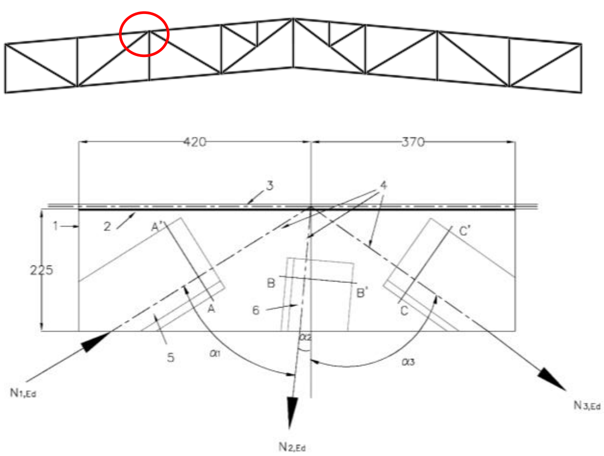


Figure 8 – Location and general layout of joint No 10

5.1.1 Gusset to chord

The sequence of checking of the connection can be summarized as shown in Table 6.

Table 6 - 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

***Check 1* - Gusset plate**

The design forces in the gusset plate at the intersection with the chord's web, displayed in Figure 9, are determined as follows:

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

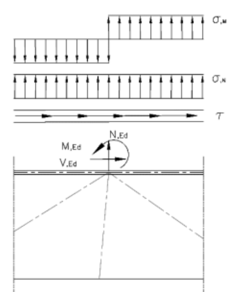


Figure 9 – Design stresses on the gusset in front of welds

The cross-sectional resistance is evaluated as follows:

$$\sigma_{g,max} = \frac{N_{g,Ed}}{A_g} + \frac{M_{g,Ed}}{I_g/_z} \qquad \tau_g = \frac{V_{g,Ed}}{A_g}$$

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

5.1.2 Diagonals to gusset

In analysing the connection between diagonals and the gusset two different aspects should be noted. 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 connections between the bracing members and the gusset are category C bolted connections and, therefore, conditions of bearing and slip resistance have to be satisfied; moreover, 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.

5.1.2.1 Global elastic resistance of the gusset

The global elastic resistance of the gusset builds on the checking already carried out in 5.1.1. There are two main differences to be considered: firstly, two cross sections are analysed – shown in blue in Figure 10; secondly, the approach is a more conservative one as all favourable forces are discarded.

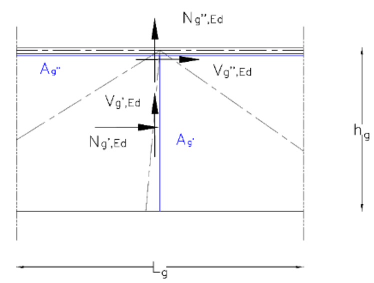


Figure 10 - Design global forces and cross-sections of the gusset

The forces acting on the two cross-sections result from decomposing the acting forces into normal and shear components, shown in Figure 10, in relation to each cross-section.

5.1.2.2 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

Check 1 - Gusset plate

Local resistance of the cross-section

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 – the bolt rows – from start to end (**Error! Reference source not found.**) [4][6].

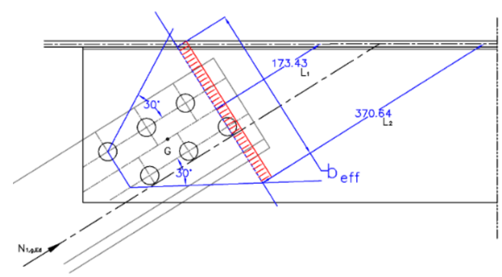


Figure 11 - Whitmore cross-section and buckling length

Buckling resistance

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 11) multiplied by the so-called Thornton factor K, equal to 0.65. As the column is embedded, the buckling length is taken as 2L' [4].

$$L' = K \cdot \min\{L_1; L_2\}$$

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) $F_{V,Ed} \leq F_{b,Rd}$ 2) $F_{V,Ed} \leq F_{s,Rd}$

3) $F_{V,Ed} \leq N_{net,Rd}$ (only in case of tensioned members)

Shear Forces

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 Figure 12 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 the 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:

$$F_{V,bi,h',Ed} = F_{N,bi,h'} + F_{M,bi,h'} \quad F_{V,bi,v',Ed} = F_{M,bi,v'}$$

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

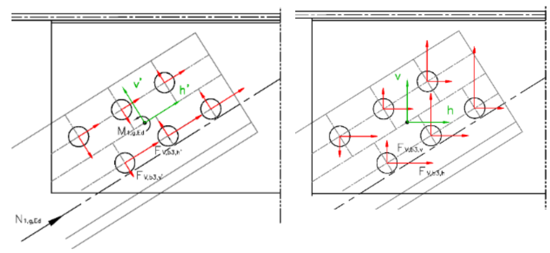


Figure 12 - Loading on bolts (forces in the {h', v'} and {h, v} system) - regarding the gusset

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

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

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

5.1.2.3 Diagonal 13 to gusset

In diagonal 13 all the safety evaluations carried out in 5.1.2.2 are valid. Additionally, the net cross-section and block tearing are checked as the diagonal is in tension.

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

Net cross-section - Gusset component

There is no indication in EN 1993-1-8 for determining the acting force on the net area, but a possible procedure [7] is presented as follows:

$$N_{3,g,bt,Ed} = n_b \frac{N_{3,g,Ed}}{n_{bt}} \quad N_{Rd} = \frac{A_{net,3} f_y}{\gamma_{M0}}$$

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 (shown in Figure 13). 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:

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

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

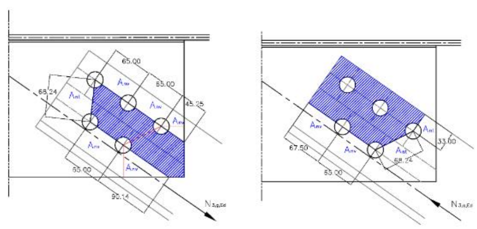


Figure 13 - Definition of block tearing areas - regarding the gusset (left) and angle (right)

5.2 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 work, is the use of splice plates with bolts, both at the web and flanges. The general layout of the connection is as follows:

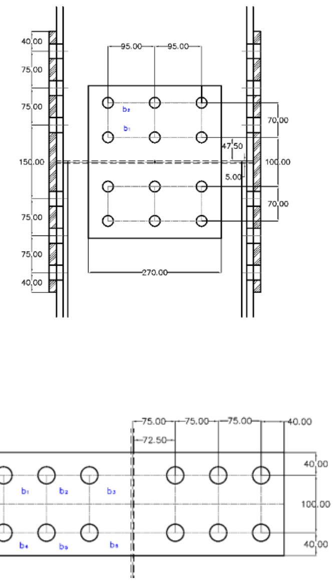


Figure 14 - Positioning of plates and holes in the spliced connection

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.

Safety is evaluated in the web and flange component separately. All the checks that are satisfied in this chapter are the same as the ones carried out in 5.1.

6 References

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