

# **STEEL BUILDINGS IN EUROPE**

## **Single-Storey Steel Buildings**

### **Part 5: Detailed Design of Trusses**



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## FOREWORD

This publication is part five of the design guide, *Single-Storey Steel Buildings*.

The 10 parts in the *Single-Storey Steel Buildings* guide are:

- Part 1: Architect's guide
- Part 2: Concept design
- Part 3: Actions
- Part 4: Detailed design of portal frames
- Part 5: Detailed design of trusses
- Part 6: Detailed design of built up columns
- Part 7: Fire engineering
- Part 8: Building envelope
- Part 9: Introduction to computer software
- Part 10: Model construction specification
- Part 11: Moment connections

*Single-Storey Steel Buildings* is one of two design guides. The second design guide is *Multi-Storey Steel Buildings*.

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The design guides have been prepared under the direction of Arcelor Mittal, Peiner Träger and Corus. The technical content has been prepared by CTICM and SCI, collaborating as the Steel Alliance.



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## **SUMMARY**

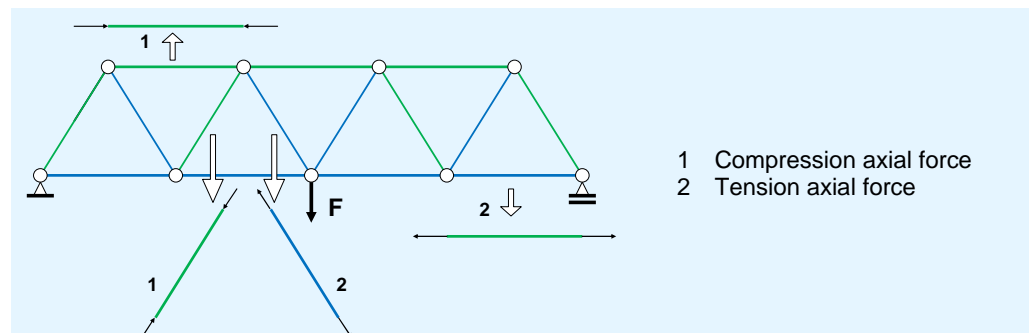
This publication provides guidance on the design of trusses for single-storey buildings. The use of the truss form of construction allows buildings of all sizes and shapes to be constructed. The document explains that both 2D and 3D truss forms can be used. The 2D form of truss is essentially a beam and is used to supporting a building roof, spanning up to 120 metres for large industrial buildings. The 3D form of truss can be used to cover large areas without intermediate supports; this form is often used for large exhibition halls. The detailed guidance in this document relates mainly to 2D truss structures composed of rolled profiles but the principles are generally applicable to all forms of truss structure.



# 1 INTRODUCTION

## 1.1 Definition

A truss is essentially a triangulated system of (usually) straight interconnected structural elements; it is sometimes referred to as an open web girder. The individual elements are connected at nodes; the connections are often assumed to be nominally pinned. The external forces applied to the system and the reactions at the supports are generally applied at the nodes. When all the members and applied forces are in a same plane, the system is a plane or 2D truss.



**Figure 1.1 Members under axial forces in a simple truss**

The principal force in each element is axial tension or compression. When the connections at the nodes are stiff, secondary bending is introduced; this effect is discussed below.

## 1.2 Use of trusses in single-storey buildings

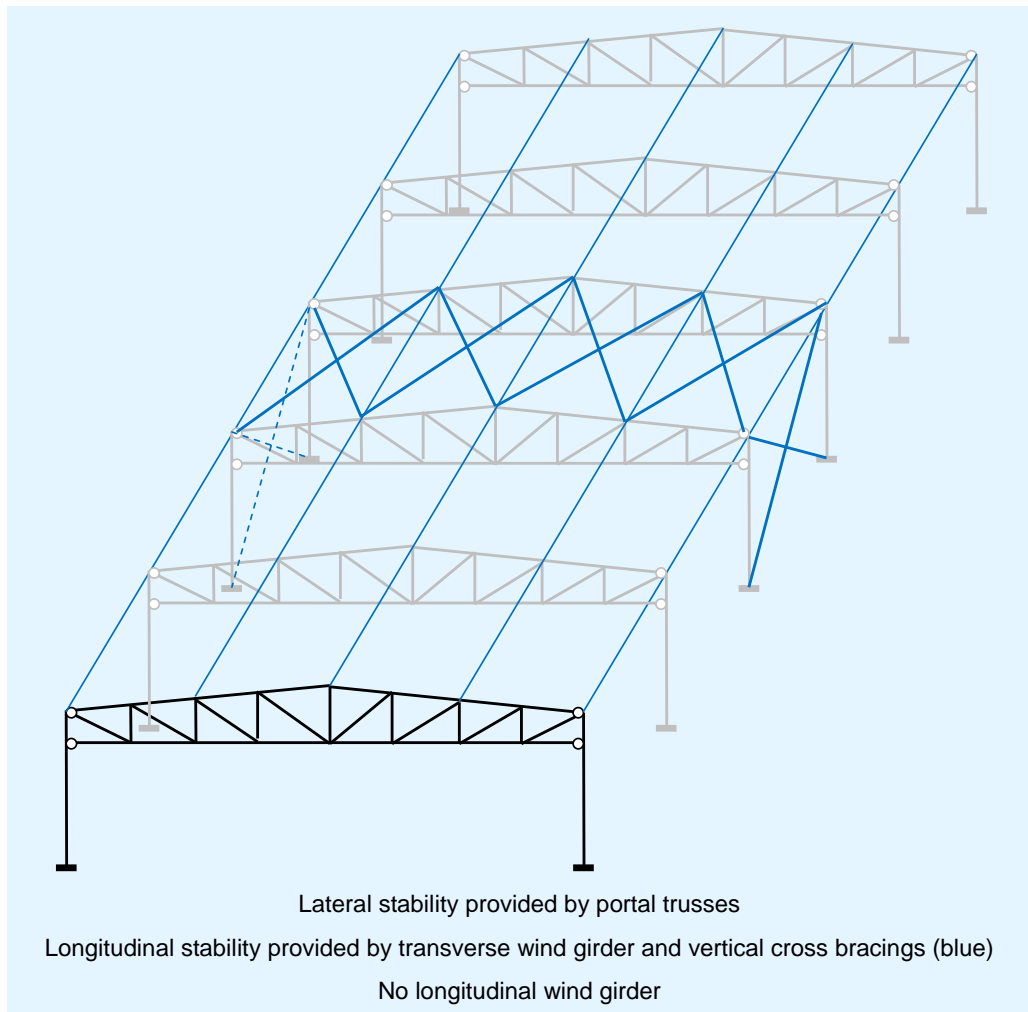
In a typical single-storey industrial building, trusses are very widely used to serve two main functions:

- To carry the roof load:
  - Gravity loads (self-weight, roofing and equipment, either on the roof or hung to the structure, snow loads)
  - Actions due to the wind (including uplift due to negative pressure).
- To provide horizontal stability:
  - Wind girders at roof level, or at intermediate levels if required
  - Vertical bracing in the side walls and/or in the gables.

Two types of general arrangement of the structure of a typical single-storey building are shown in Figure 1.2 and in Figure 1.3.

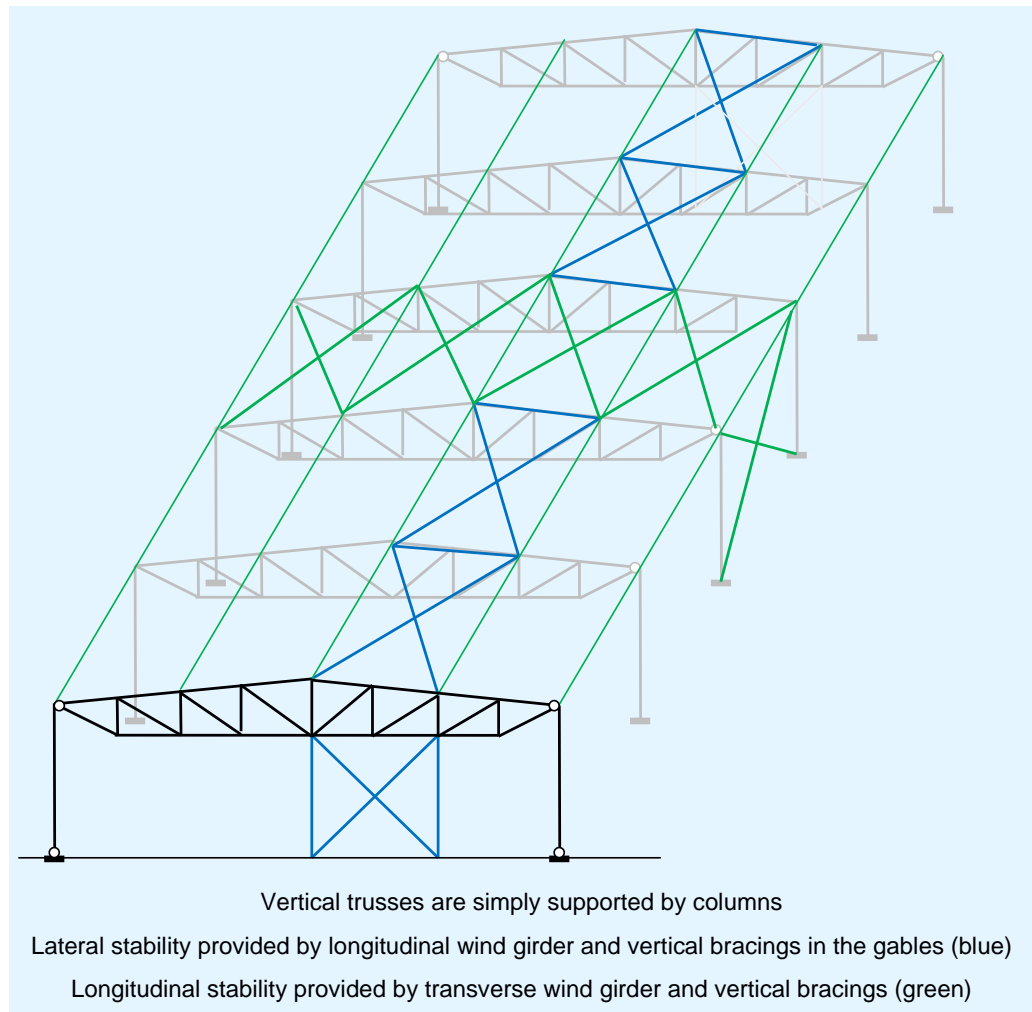
In the first case (Figure 1.2), the lateral stability of the structure is provided by a series of portal trusses: the connections between the truss and the columns provide resistance to a global bending moment. Loads are applied to the portal structure by purlins and side rails.

For the longitudinal stability of the structure, a transverse roof wind girder, together with bracing in the side walls, is used. In this arrangement the forces due to longitudinal wind loads are transferred from the gables to the side walls and then to the foundations.



**Figure 1.2 Portal frame a arrangement**

In the second case, as shown in Figure 1.3, each vertical truss and the two columns on which it spans constitute a simple beam structure: the connection between the truss and a column does not resist the global bending moment, and the two column bases are pinned. Transverse restraint is necessary at the top level of the simple structure; it is achieved by means of a longitudinal wind girder carries the transverse forces due to wind on the side walls to the braced gable walls.



**Figure 1.3 Beam and column arrangement**

A further arrangement is shown in Figure 1.4. The roof structure is arranged with main trusses spanning from column to column, and secondary trusses spanning from main truss to main truss.

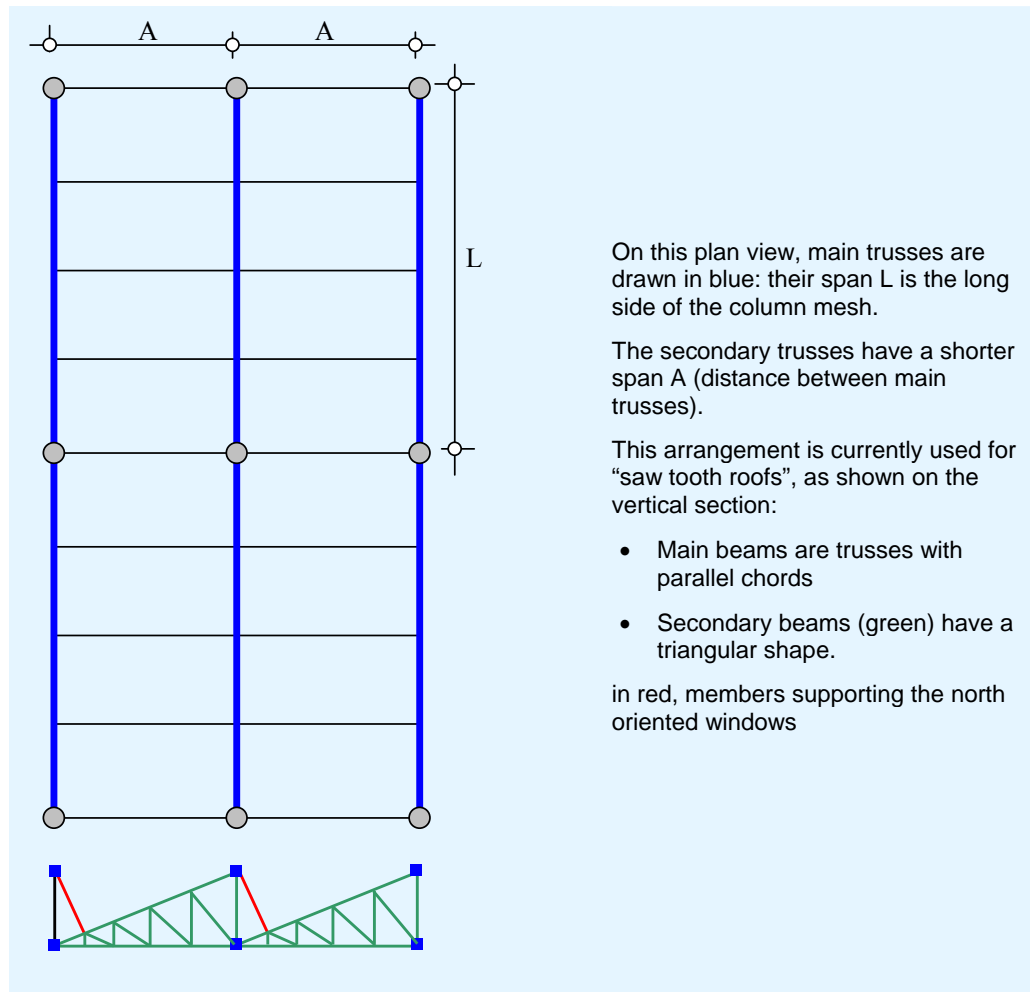


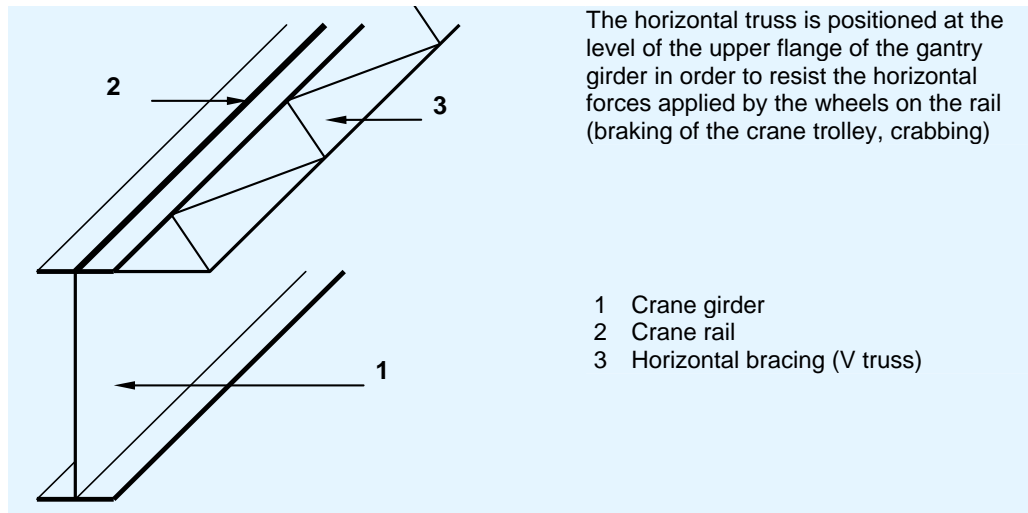
Figure 1.4 General arrangement 3

### 1.3 Different shapes of trusses

A large range is available for the general shapes of the trusses. Some of the commonly used shapes are shown in Table 1.1.

**Table 1.1 Main types of trusses**

	All these types of trusses can be used either in portal truss structures (see figure 1.2) or in simple truss structures (see figure 1.3).	Long spans: range from 20 to 100 m	<b>Pratt truss:</b> In a Pratt truss, diagonal members are in tension for gravity loads. This type of truss is used where gravity loads are predominant
			In a truss as shown, diagonal members are in tension for uplift loads. This type of truss is used where uplift loads are predominant, such as open buildings.
			<b>Warren truss:</b> In this type of truss, diagonal members are alternatively in tension and in compression This type of truss is also used for the horizontal truss of gantry/crane girders (see Figure 1.5)
	All these types of trusses can be used either in portal truss structures (see figure 1.2) or in simple truss structures (see figure 1.3).	There are two different types of X truss : <ul style="list-style-type: none"> <li>• if the diagonal members are designed to resist compression, the X truss is the superposition of two Warren trusses.</li> <li>• if the resistance of the diagonal members in compression is ignored, the behaviour is the same as a Pratt truss.</li> </ul> This shape of truss is more commonly used for wind girders, where the diagonal members are very long.	
			It is possible to add secondary members in order to : <ul style="list-style-type: none"> <li>• create intermediate loading points</li> <li>• limit the buckling length of members in compression (without influencing the global structural behaviour).</li> </ul>
			For any of the forms shown above, it is possible to provide either a single or a double slope to the upper chord of a roof supporting truss This example shows a duo-pitch truss
			Single slope upper chord for these triangular trusses, part of a “saw tooth roof” North oriented windows
	Simply supported, smaller spans Range from 10 to 15 m	<b>Fink truss:</b> This type of truss is more commonly used for the roof of houses.	



**Figure 1.5 Horizontal bracing for a crane girder**

Figure 1.6 and Figure 1.7 illustrate some of the trusses described in Table 1.1.



**Figure 1.6 N-truss – 100 m span**



Figure 1.7 N-truss (also with N-truss purlins)

## 1.4 Aspects of truss design for roof structure

### 1.4.1 Truss or I-beam

For the same steel weight, it is possible to get better performance in terms of resistance and stiffness with a truss than an I-beam. This difference is more sensitive for long spans and/or heavy loads.

The full use of this advantage is achievable if the height of the truss is not limited by criteria other than the structural efficiency (a limit on total height of the building, for example).

However, fabrication of a truss is generally more time consuming than for an I-beam, even considering that the modernisation of fabrication equipment allows the optimisation of fabrication times.

The balance between minimum weight and minimum cost depends on many conditions: the equipment of the workshop, the local cost of manufacturing; the steel unit cost, etc. Trusses generally give an economic solution for spans over 20 or 25 m.

An advantage of the truss design for roofs is that ducts and pipes that are required for operation of the buildings services can be installed through the truss web.

### 1.4.2 General geometry

In order to get a good structural performance, the ratio of span to truss depth should be chosen in the range 10 to 15.

The architectural design of the building determines its external geometry and governs the slope(s) given to the top chord of the truss.

The intended use of the internal space can lead either to the choice of a horizontal bottom chord (e.g. where conveyors must be hung under the chord), or to an inclined internal chord, to allow maximum space to be freed up (see the final example in Table 1.1).

To get an efficient layout of the truss members between the chords, the following is advisable:

- The inclination of the diagonal members in relation to the chords should be between  $35^\circ$  and  $55^\circ$
- Point loads should only be applied at nodes
- The orientation of the diagonal members should be such that the longest members are subject to tension (the shorter ones being subject to compression).

### **1.4.3 Section of the members**

Many solutions are available. The main criteria are:

- Sections should be symmetrical for bending out of the vertical plane of the truss
- For members in compression, the buckling resistance in the vertical plane of the truss should be similar to that out of the plane.

A very popular solution, especially for industrial buildings, is to use sections composed of two angles bolted on vertical gusset plates and intermediately battened, for both chords and internal members. It is a very simple and efficient solution.

For large member forces, it is a good solution to use:

- Chords having IPE, HEA or HEB sections, or a section made up of two channels (UPE)
- Diagonals formed from two battened angles.

The web of the IPE / HEA / HEB chord section is oriented either vertically or horizontally. As it is easier to increase the resistance to in-plane buckling of the chords (by adding secondary diagonal members) than to increase their to out-of-plane resistance, it is more efficient to have the web horizontal, for chords in compression. On the other hand, it is easier to connect purlins to the top chord if it has a vertical web.

It could be a good solution to have the top chord with a vertical web, and the bottom chord with a horizontal web.

Another range of solutions is given by the use of hollow sections, for chords and/or for internals.

### **1.4.4 Types of connections**

For all the types of member sections, it is possible to design either bolted connections or welded connections. Generally, bolted connections are preferred on site. Where bolted connections are used with bolts loaded perpendicular to their shank, it is necessary to evaluate the consequences of slack in



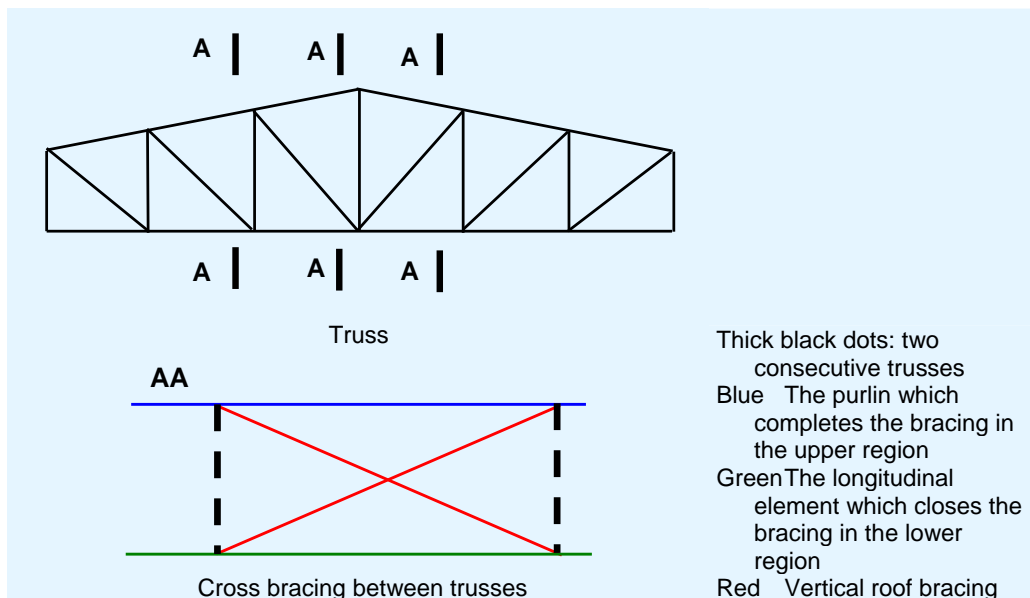
connections. In order to reduce these consequences (typically, the increase of the deflections), solutions are available such as use of pre-stressed bolts, or limiting the hole size.

### 1.4.5 Lateral stability

It is necessary to design the chords in compression against the out-of-plane buckling. For simply supported trusses, the upper chord is in compression for gravity loading, and the bottom chord is in compression for uplift loading. For portal trusses, each chord is partly in compression and partly in tension.

Lateral restraint of the upper chord is generally given by the purlins and the transverse roof wind girder.

For the restraint of the bottom chord, additional bracing may be necessary, as shown in Figure 1.8. Such bracing allows the buckling length of the bottom chord to be limited out of the plane of the truss to the distance between points laterally restrained: they serve to transfer the restraint forces to the level of the top chord, the level at which the general roof bracing is provided. This type of bracing is also used when a horizontal load is applied to the bottom chord (for example, forces due to braking from a suspended conveyor).



**Figure 1.8 Lateral bracing**

The roof purlins often serve as part of the bracing at the top chord. Introduction of longitudinal members at the lower chord allows the trusses to be stabilised by the same vertical bracing.

It is possible to create a horizontal wind girder at the level of the bottom chords, with longitudinal elements to stabilize all the trusses.

## 1.5 Design of wind girders

### 1.5.1 Transverse wind girder

In general, the form of a transverse wind girder is as follows (see Figure 1.2):

- The wind girder is arranged as an X truss, parallel to the roof plane.

- The chords of the wind girder are the upper chords of two adjacent vertical trusses. This means that the axial forces in these members due to loading on the vertical truss and those due to loads on the wind girder loading must be added together (for an appropriate combination of actions).
- The posts of the wind girder are generally the roof purlins. This means that the purlins are subject to a compression, in addition to the bending due to the roof loading.
- It is also possible, for large spans of the wind girder, to have separate posts (generally tubular section) that do not act as purlins.
- The diagonal members are connected in the plane of the posts. If the posts are the purlins, the diagonal members are connected at the bottom level of the purlins. In a large X truss, diagonals are only considered in tension and it is possible to use single angles or cables.

It is convenient to arrange a transverse wind girder at each end of the building, but it is then important to be careful about the effects of thermal expansion which can cause significant forces if longitudinal elements are attached between the two bracing systems, especially for buildings which are longer than about 60 m.

In order to release the expansion of the longitudinal elements, the transverse wind girder can be placed in the centre of the building, but then it is necessary to ensure that wind loads are transmitted from the gables to the central wind-bracing.

Transverse wind girders are sometimes placed in the second and penultimate spans of the roof because, if the roof purlins are used as the wind girder posts, these spans are less subject to bending by roof loads.

The purlins which serve as wind girder posts and are subject to compression must sometimes be reinforced:

- To reinforce IPE purlins: use welded angles or channels (UPE)
- To reinforce cold formed purlins: increase of the thickness in the relevant span, or, if that is not sufficient, double the purlin sections (with fitting for the Zed, back to back for the Sigma).

### **1.5.2 Longitudinal wind girder**

It is necessary to provide a longitudinal wind girder (between braced gable ends) in buildings where the roof trusses are not “portalized”.

The general arrangement is similar to that described for a transverse wind girder:

- X truss
- The chords are two lines of purlins in small buildings, or additional elements (usually tubular sections)
- The posts are the upper chords of the consecutive stabilized roof trusses.

## 2 INTRODUCTION TO DETAILED DESIGN

The detailed design of trusses is illustrated in the following Sections by reference to a ‘worked example’. This Section summarizes the general requirements and introduces the example. The topics covered in subsequent Sections are:

Section 3: Global analysis

Section 4: Verification of members

Section 5: Verification of connections

Fully detailed calculations for verification of a gusset plate connection and a chord splice are given in Appendices A and B.

### 2.1 General requirements

The parameters to be taken into account in design are:

- Aesthetics
- Geometry (span length, height, rise, etc)
- Actions.

The following requirements have to be considered:

- Regulatory requirements
- Contractual requirements with regard to standards
- Specific contractual requirements.

The resulting outcome of a design is the set of execution documents for the structure.

The nature of regulatory requirements varies from one country to another. Their purpose is usually to protect people. They exist in particular in the area of seismic behaviour, and for the behaviour of buildings during a fire (see *Single-Storey Steel Buildings. Fire engineering Guide*<sup>1</sup>).

The requirements in standards concern the determination of actions to be considered, the methods of analysis to be used, and the criteria for verification with respect to resistance and stiffness.

There is no limit to the number of specific requirements which may be imposed for any particular building but these mainly concern construction geometry; they influence determination of actions, in particular climatic actions.

Obligations and interface arrangements for detailed design might include:

- Banning the use of tubes for the bottom chord of trusses to which the industry client wishes to hang equipment
- Obligation to use tubes for truss chords for reasons of appearance

- Use of the roof to stabilise certain structural elements.

The flowchart below illustrates the main steps in the design of a structural element.

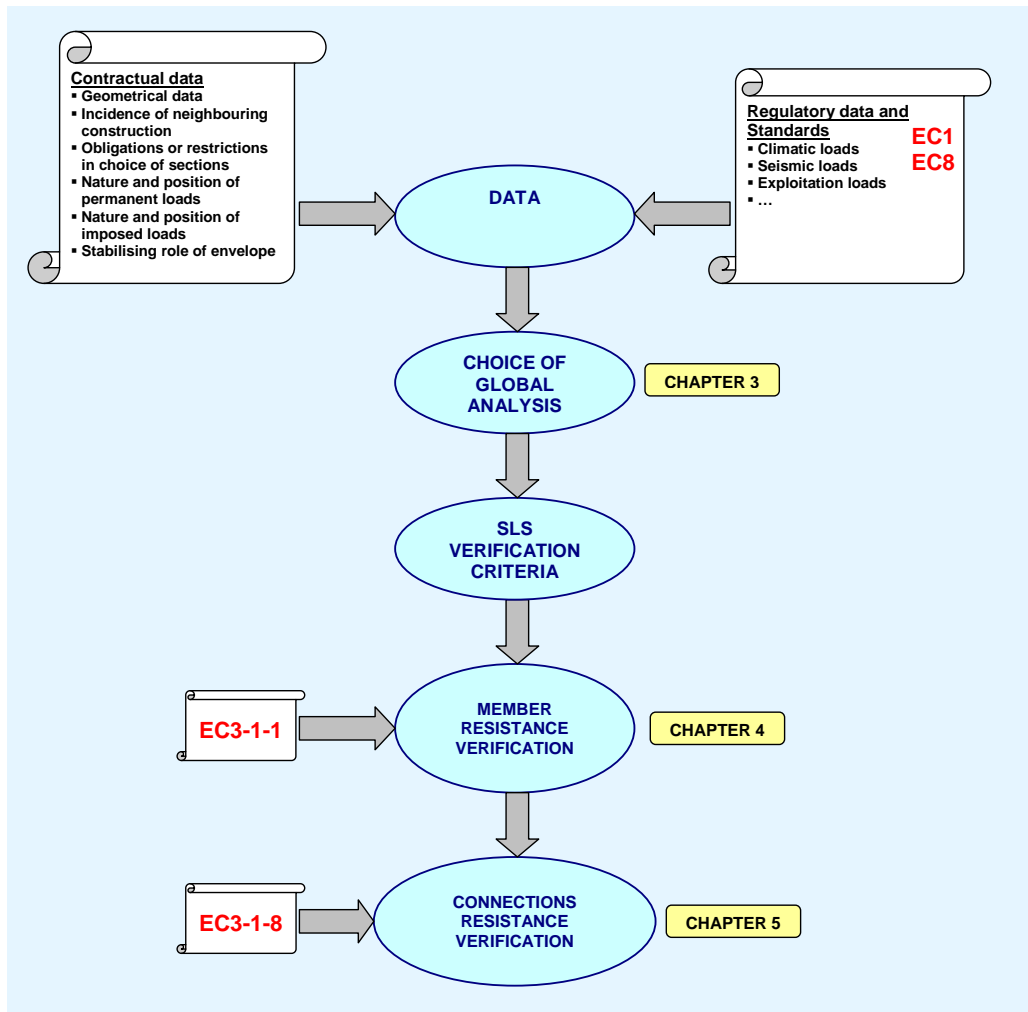
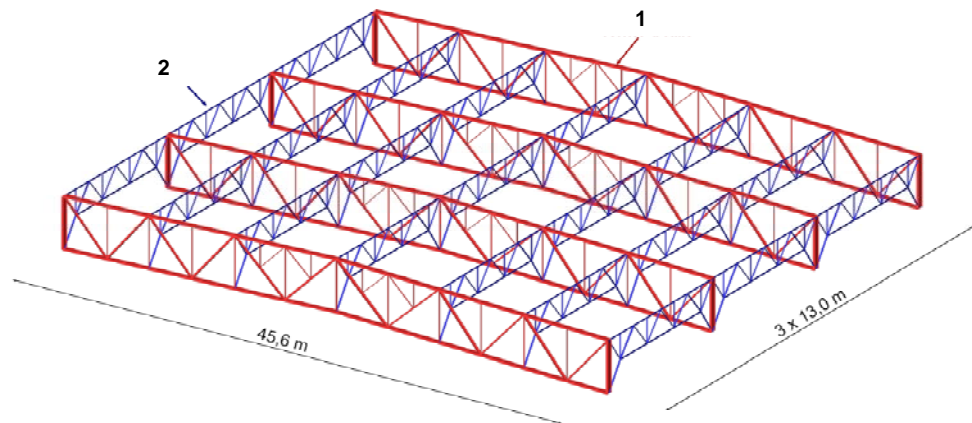


Figure 2.1 Flowchart for the design of a structural element

## 2.2 Description of the worked example

The worked example that is the subject of subsequent Sections is a large span truss supporting the roof of an industrial building, by means of purlins in the form of trusses. This example is directly transposed from a real construction and has been simplified in order to clarify the overview.



- 1 Main truss
- 2 Purlin truss

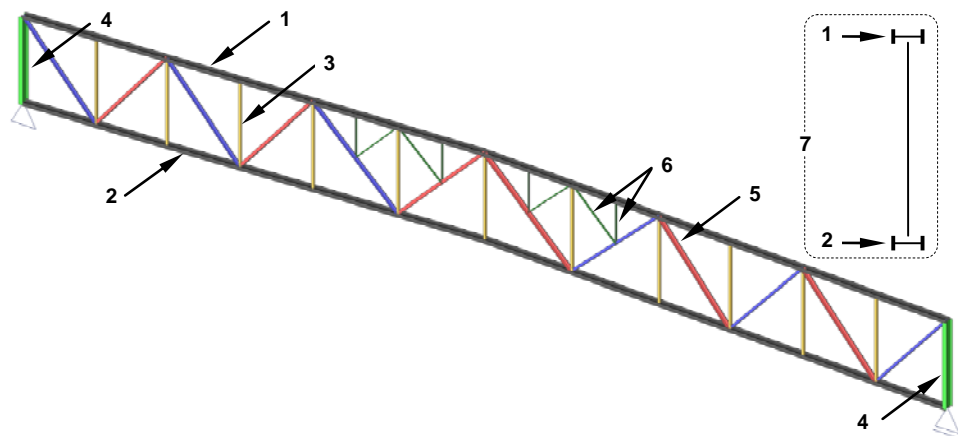
Note: the horizontal bracing is not displayed in this diagram but it is designed in such a way that the purlins provide efficient lateral restraints to the main trusses.

**Figure 2.2 Worked example - General layout of the roof**

The roof is a symmetrical pitched roof; the slope on each side is 3%.

Each main truss has a span of 45,60 m and is simply supported at the tops of the columns (there is no moment transmission between the truss and the column).

General transverse stability of the building is provided by fixity of the columns at ground level; longitudinal stability is provided by a system of roof bracings and braced bays in the walls.



- 1 Upper chord IPE 330 with horizontal web
- 2 Lower chord IPE 330 with horizontal web
- 3 Post - Single angle L100x100x10
- 4 Top of the column (IPE 450)
- 5 Diagonals - Double angle
- 6 Secondary truss members
- 7 Sketch of the cross-section

**Figure 2.3 Worked example – View of truss**

The truss is illustrated in Figure 2.3. The truss chords are parallel and are made up of IPE 330 profiles with the webs horizontal. The diagonals are made of twinned angles: two  $120 \times 120 \times 12$  angles for diagonals in tension under gravity loads (in blue in the diagram above), two  $150 \times 150 \times 15$  angles for

diagonals in compression under gravity loads (in red in the diagram above); the posts are single angles  $100 \times 100 \times 10$ .

Note that, in the central panels, secondary diagonals and posts are present. They would generally be installed with one or other of the following objectives:

- To permit application of a point load between main nodes, without causing further bending in the upper chord
- To reduce buckling, in the plane of the truss of central members of the upper chord.

In this example, the secondary trusses reduce the buckling length.

The pairs of angles which make up the section of a diagonal are joined by battens, to ensure combined action with respect to buckling between the truss nodes. To be efficient, battens must therefore prevent local slip of one angle in relation to the other. See Section 4.1.3 for more information.

Each chord is fabricated in two pieces (see Figure 3.6). The diagonals and posts are bolted at their two ends to vertical gusset plates, which are themselves welded to the horizontal webs of the IPE 330 chords. Detailed diagrams of this type of connection are given in Appendix A and in Sections 5.2 and 5.3.

The columns on which the truss is supported are IPE 450, for which the web is perpendicular to the plane of the truss beam.

In order to illustrate all of the topics here, the truss beam in the worked example is designed for two situations: a gravity load case and an uplift load case. The loads correspond to the combination of actions, determined according to EN 1990 for verification with respect to the ultimate limit state (ULS).

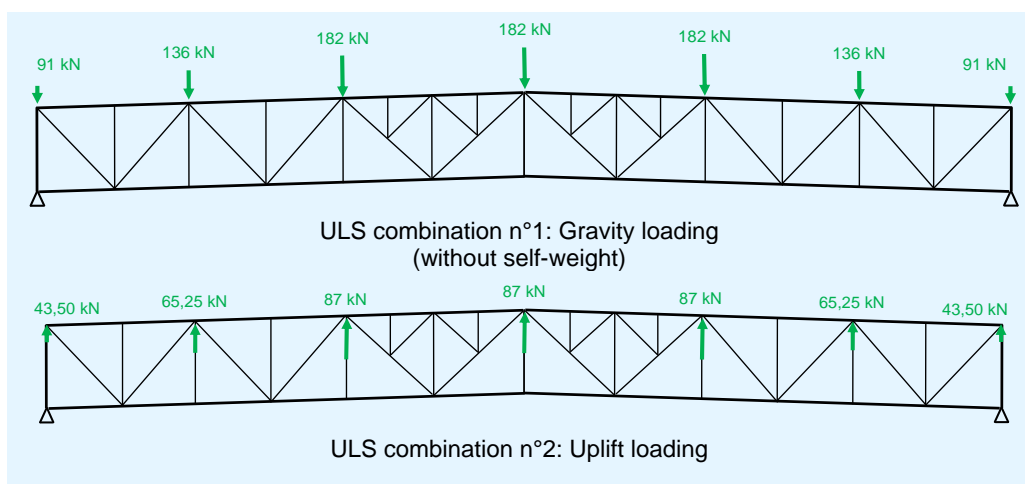


Figure 2.4 Worked example – Load Combinations

## 3 GLOBAL ANALYSIS

### 3.1 General

Section 1.1 describes the general behaviour of a truss. In reality, structures deviate from this theoretical behaviour and their global analysis involves consideration of the deviations. In particular, the deviations include the occurrence of bending in the members, in addition to the axial forces. These bending moments, known as “secondary moments”, can cause significant additional stresses in the members which make up the truss.

The deviations in design take various forms:

- All the members which make up the structure are not usually articulated at their original node and their end node. Truss chords, in particular, are usually fabricated in one length only, over several truss purlins: the continuous chord members are then connected rigidly to their original and end nodes. Rotation of the nodes, resulting from general deformation of the truss beam then causes bending moments in the rigidly connected members; the more rigid the chord members, the bigger the moments (see Section 3.4).
- The members are not always strictly aligned on their original and end nodes. Bending moments which result from a misalignment of axes increase in proportion to the size of the eccentricity and the stiffness of the members. This phenomenon is illustrated in Section 3.6.
- Loads are not always strictly applied to the nodes and, if care is not taken to introduce secondary members to triangulate the point of application of the loads between nodes, this results in bending moments.

### 3.2 Modelling

Several questions arise in respect of the modelling of a truss.

It is always convenient to work on restricted models. For example, for a standard building, it is common and usually justified to work with 2D models (portal, wind girder, vertical bracing) rather than a unique and global 3D model. A truss can even be modelled without its supporting columns when it is articulated to the columns.

Nonetheless, it is important to note that:

- If separate models are used, it may be necessary, in order to verify the resistance of certain elements, to combine the results of several analyses; example: the upper chord of a truss also serves as chord of the wind girder.
- If a global 3D model is used, “parasitic” bending can be observed, which often only creates an illusory precision of the structural behaviour process. That is why 2D models are generally preferable.

In the worked example, where the truss is simply supported on the columns, the design model chosen is that of the truss only.

Once the scope of the model has been decided and adapted according to use to be made of the results, it is important to consider the nature of the internal connections. In current modelling of member structures, the selection is made between “a pin-jointed member at a node” and a “member rigidly connected to a node”; the possibility offered by EN 1993 to model connections as semi-rigid is rarely used for truss structures.

For trusses, the model is commonly represented as either:

- **Continuous chords** (and therefore chord members rigidly connected at both ends)
- **Truss members** (diagonals and verticals) pin jointed to the chords.

### 3.3 Modelling the worked example

In the worked example, the truss diagonals are pin jointed to the chords, although the connections are carried out using high strength bolts suitable for preloading with controlled tightening. This provides a rigid connection without slack between the diagonal and the connection gusset plates. The connection can be considered as pinned due to the fact that the vertical gusset plates are welded in the middle of the horizontal, not very stiff, IPE 330 web.

The modelling is shown in Figure 3.1, with the numbering of the members.

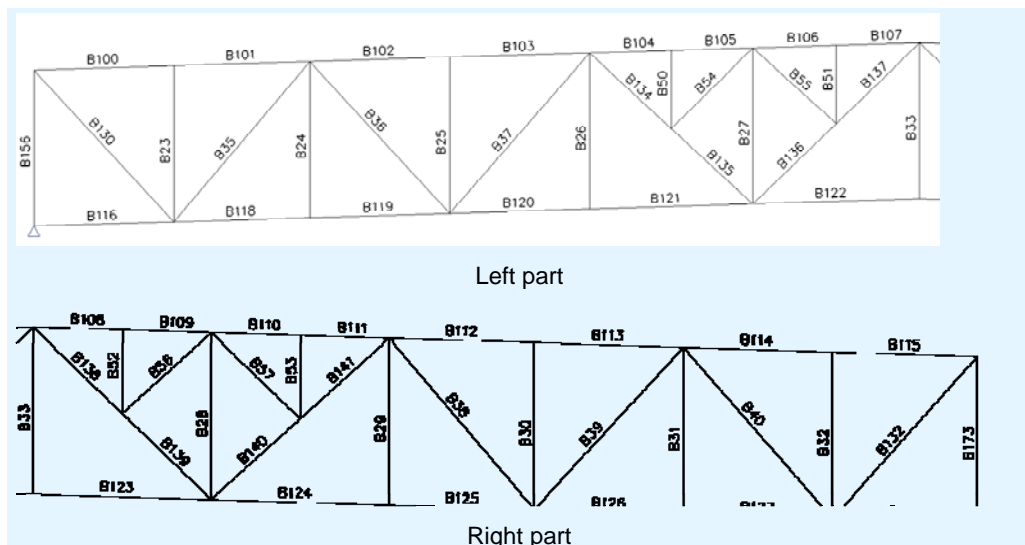


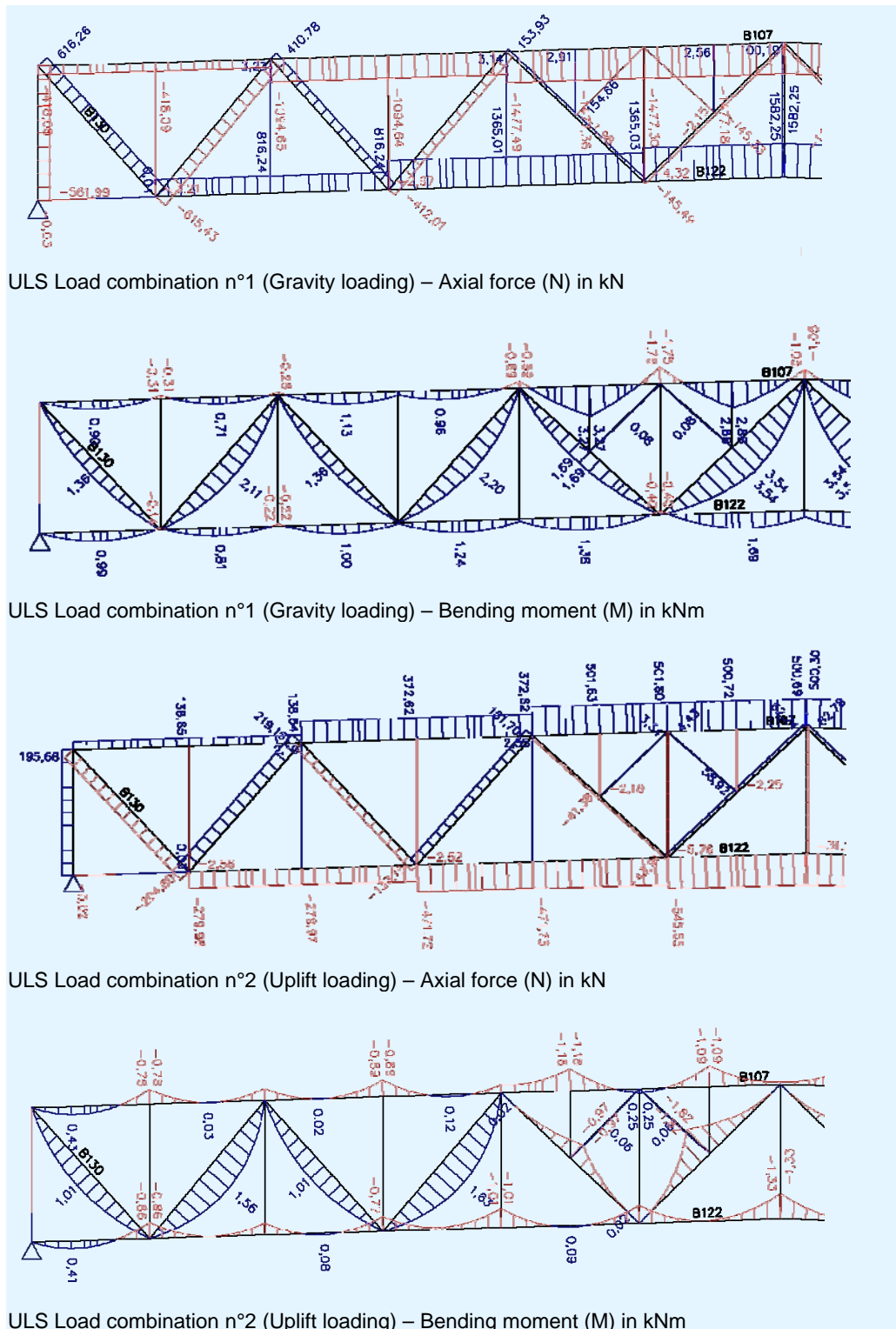
Figure 3.1 Computer model

It is important for the model to be representative of the eccentricities which exist in the real structure. They can have a significant effect, as illustrated in Section 3.6.1.

It is also important that modelling of the loads is representative of the real situation. In particular, the fact of applying to the truss nodes loads which, in reality, are applied between nodes, risks leading to neglect of the bending with quite significant outcomes.

The main results of the analysis are given in Figure 3.2 for the left part of the truss.





**Figure 3.2 Worked example – Axial forces and bending moments**

It is interesting to note the form of the moment diagrams in the member:

- In the chords and the diagonals, the self weight generates a bending moment with a parabolic shape
- In the chords, continuous modelling (members rigidly connected at both ends) leads to moments at the nodes.

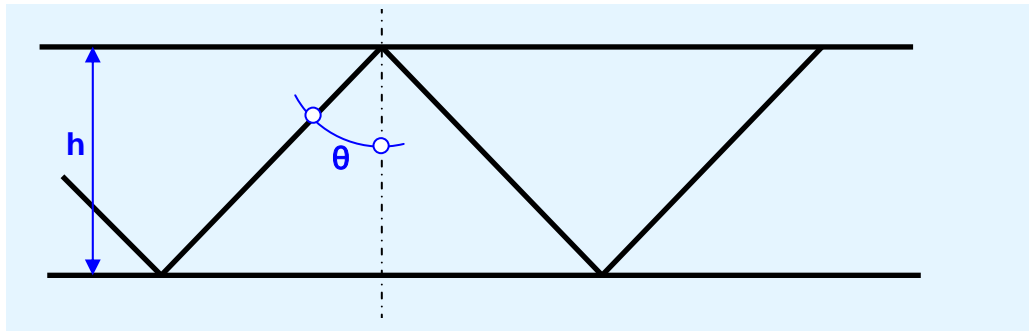
### 3.4 Simplified global analysis of the worked example

A triangulated beam, with a constant depth, can be equated to an I-beam. This equivalence is possible and provides a good approximation, for example, for a truss with parallel chords.

The global shear force  $V_{\text{global}}$  and the global bending moment  $M_{\text{global}}$  in the equivalent beam vary very little along a panel and can be equated with the mean values in the panel. Therefore the axial load can be assessed using the following expressions (see Figure 3.3 for the notations):

$$N_{\text{ch}} = \pm M_{\text{global}}/h \quad \text{in the chords}$$

$$N_{\text{d}} = \pm V_{\text{global}}/\cos \theta \quad \text{in a diagonal}$$



**Figure 3.3** Truss with parallel chords - Notation

An estimate can also be made for the deflection of the truss beam by calculating that for an equivalent beam, for the same loading. In order to do this, the classic approach is to use elementary beam theory, giving the equivalent beam a second moment of area equal to:

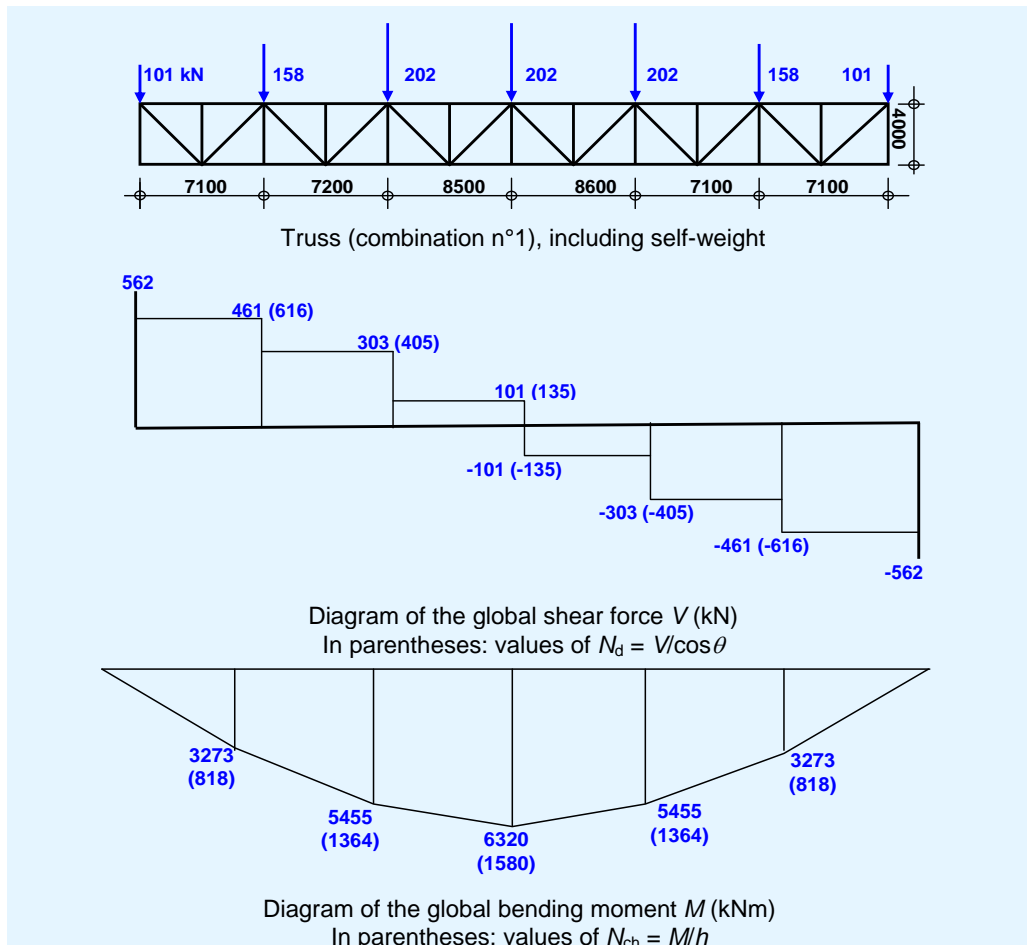
$$I = \sum_{i=1}^2 A_{\text{ch},i} d_i^2$$

where:

$A_{\text{ch},i}$  is the section area of the chord  $i$

$d_i$  is the distance from the centroid of both chords to the centroid of the chord  $i$ .

In order to take into account global shear deformations, not dealt with in elementary formulae, a reduced modulus of elasticity is used. Global shear deformations are not, in fact, negligible in the case of trusses, since they result from a variation in length of the diagonals and posts. The value of the reduced modulus of elasticity clearly varies depending on the geometry of the truss, the section of the members, etc. For a truss beam with “well proportioned” parallel chords, the reduced modulus of elasticity is about  $160000 \text{ N/mm}^2$  (instead of  $210000 \text{ N/mm}^2$ ).



**Figure 3.4 Worked example – Approximate calculation**

The values of the axial forces in the chords obtained by the simplified approach,  $M_{\text{global}}/h$ , are shown in Figure 3.4. The values are very close to the values obtained using structural analysis software (see Figure 3.2), for the sections close to the applied loads. The small difference comes from the slope (3%) of the chords of the truss in the worked example, not taken into account in the hand calculation.

The values of the axial forces in the diagonals obtained by the simplified approach,  $V_{\text{global}}/\cos\theta$ , are also very close to the values obtained using software.

## 3.5 Secondary forces

### 3.5.1 Influence of chord rigidity

Chord members in trusses which are used in construction are rarely pinned at the nodes and are more often rigidly connected; this means that members connected to the same node have to keep their respective angles. During deformation of the structure under load, the ends of the members all rotate at the same angle around the node. In these conditions, bending loads (bending moments and shear forces) called secondary forces are added to the axial loads in the members calculated assuming the nodes are pinned (primary forces).

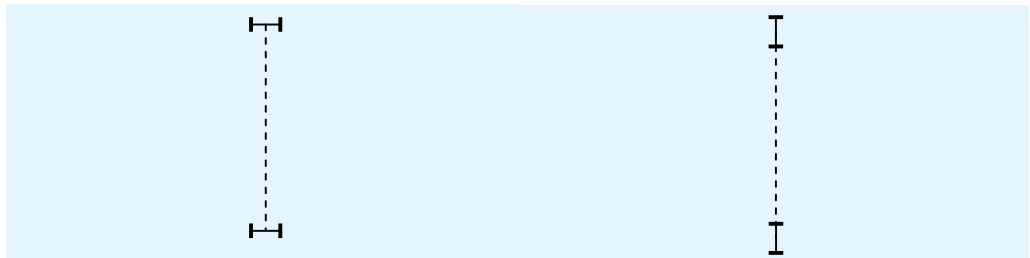
It is routine in design to use continuous chord members and to pin the truss members.

In fact, transforming pinned connections into rigid nodes hardly leads to any modification to the axial forces in the members, because the shear transmitted by the members has little influence on the equilibrium equation of nodal forces and, on the other hand, bending of the member due to secondary bending moments only causes a slight variation in the distance between the ends of this member compared to the difference in length due to axial force.

Nevertheless, it is essential that the triangulated structures be designed properly so that the members are adequately arranged to withstand bending stresses, but not too slender so as to avoid buckling. Note that the greater the stiffness of the chords (which are usually continuous), compared to the global stiffness of the truss beam, the bigger the moments developed in the chords. For instance, for a wind girder in a roof, the stiffness of the chords is relatively small and the secondary moments remain small as well.

For a stocky truss, i.e. when the flexural stiffness of the individual chords is not significantly lower than the global stiffness of the truss, it can be necessary to take into account the secondary moments. Then the members and the connections must be designed accordingly.

This phenomenon can be illustrated in the worked example by arranging the IPE 330 sections as ‘standing up’ chord members, instead of being flat in the initial design (Figure 3.5). The chords therefore bend in the vertical plane of the truss member, mobilising their strong inertia. The calculation results demonstrate well a significant increase in the secondary moments.



**Figure 3.5 Options for the orientation of the chords**

In the upper chord in a standing up IPE 300 section near the half-span, the bending moment under gravity loads (ULS) is 28,5 kNm, compared to 2,7 kNm for the flat IPE 330 section.

Similarly, in the lower chord, the bending moment is 23,4 kNm, compared to 1,7 kNm.

The multiplier of the bending moments is 11 for the upper chord, and 14 for the lower chord. This is comparable with the ratio of the inertia in an IPE 330 section (about 15).

### 3.5.2 Assumption of rigid connections

In another evaluation of the effect of member stiffness on the value of the secondary moments, the truss in the example was recalculated by making all

the internal connections rigid (diagonal and verticals fixed on their original end nodes). The comparison is summarized in Table 3.1, where it can be seen that the end moments are in the same range as the moments resulting from the self-weight of the diagonals.

**Table 3.1 Effect of rigid connection instead of pinned**

	Horizontal web	Vertical web
End moment in a diagonal in tension (Double angles 120 x12)	1,03	1,17
End moment in a diagonal in compression (Double angles 150 × 15)	1,30	2,35
<i>Moment resulting from the self-weight (for comparison)</i>	1,36	1,36
Assumption of bi-hinged diagonals	Acceptable	Acceptable

Note: the bending moments are given in kNm.

### 3.6 Effect of clearance of deflection

When the connections between elements which make up a truss beam are bolted connections, with bolts in shear (category A in EN 1993-1-8<sup>[2]</sup>), the clearance introduced into these connections can have a significant effect on displacement of the nodes.

In order to facilitate erection, the bolts are in fact inserted in holes which are larger than the bolts themselves. For standard bolt sizes, holes more than 2 mm bigger than the bolt are usually made (usually referred to as a 2mm clearance).

In order for a connection with clearance to transmit to the node the load required by the attached member, the bolt must come into contact with one or other of the connected parts: this is called often referred to as ‘taking up slack’. For a connected tension member, this slack can be assimilated as an additional extension that is added to the elastic elongation of the member in tension. Likewise, for a connected compression member, the slack is assimilated as a reduction in length that is added to the elastic shortening of the compressed member.

The total slack in the many different connections of a truss structure can lead to a significant increase in displacements, which can have various and more or less serious consequences. Amongst these, note:

- In most of the cases, the visual effect is the worst consequence.
- Increased deflection can lead to a reduction of free height under the bottom chord, which might prevent or upset the anticipated usage. For example, the additional deflection of a truss holding doors suspended in a gable of an aeroplane hangar could prevent the passage of the aeroplane.
- Increase in the deflection can result in reduction in the slope of the supported roof and even, if the nominal slope were small, to a slope inversion; a risk of water accumulation is therefore associated with an inversion in pitch.
- If the truss structure is not a statically determinate system, this may lead to unexpected internal forces.

It is therefore essential, where truss structures are concerned, to control the effect of connection slack on the displacements. In order to do this, it is often necessary:

- either to limit slack in category A connections: drilling at +1 mm, even +0,5 mm and using shear bolts on a smooth bolt shank (to limit the increase in slack by deformation) of the threads and pieces; or
- to use ‘fit bolts’; or
- to use preloaded bolts (category C connections); or
- to use welded connections instead of bolted connections.

In cases where loading in the members does not result in reversal of axial force, it is possible to calculate a value for the effect of slack in all the connections. The following calculation illustrates this phenomenon for the worked example.

Each of the chords, upper and lower, has a continuous connection with bolted splice plates around the mid-span. In addition, the diagonals are connected by bolting on gusset plates welded to the chords. Holes are 2 mm larger than the bolt diameter.

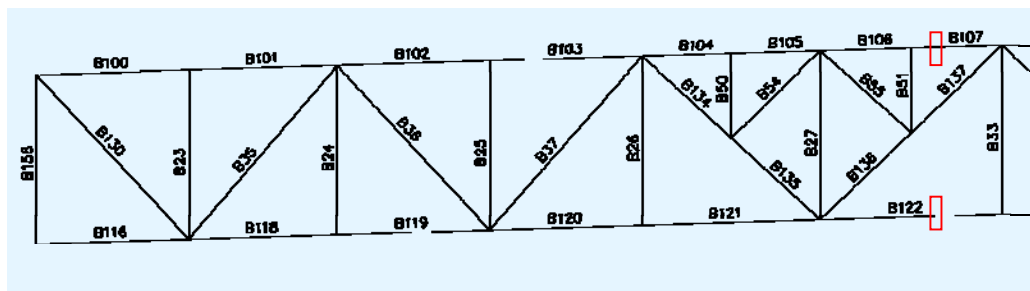


Figure 3.6 Worked example – Position of the chord connections using splice plates

In a spliced connection of a chord, the effect of slack on the deflection can be evaluated by assuming that the bolts are initially centred on their holes. If the diameter of the holes is  $d + 2$  mm (where  $d$  is the bolt diameter), a chord in tension is extended by 4 mm, as shown in Figure 3.7.

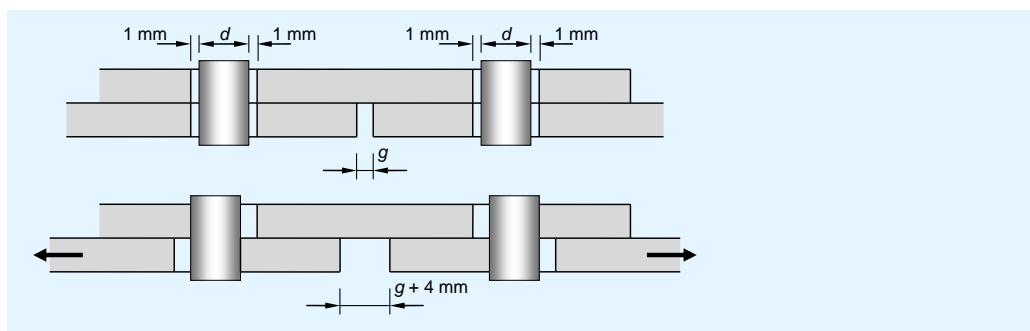


Figure 3.7 The effect of slack under load

In order for a diagonal to be loaded, 2 mm has to be recovered at each end: the length of a diagonal in tension is increased by 4 mm; a diagonal under compression is reduced by a further 4 mm.

The deflection of a truss due to the slack can be evaluated by considering the effect of a unit load applied at mid span, using the Bertrand Fontviolant equation.

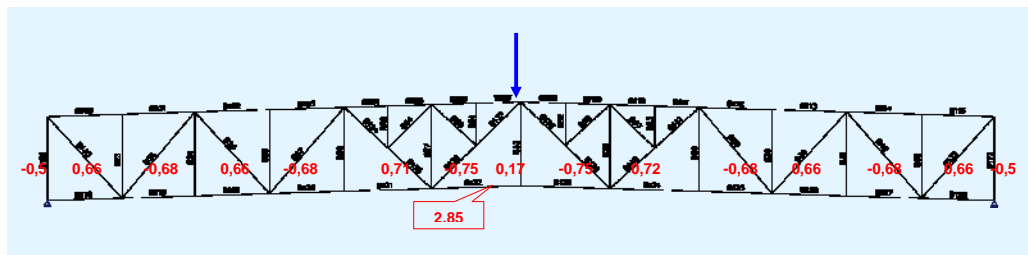


Figure 3.8 Worked example – Axial forces ( $N_{1,i}$ ) under unit load

The deflection is given by:

$$v = \sum_{i=1}^{i=b} N_{1,i} \frac{F_i l_i}{ES_i}$$

Where:

$N_{1,i}$  is the axial force produced in the member  $i$  by a unit force applied at the point where the deflection is required

$l_i$  is the length of member  $i$

$S_i$  is the section area of the member  $i$

$b$  is the number of members with bolted connection(s).

$\frac{F_i l_i}{ES_i}$  is the variation in length of member  $i$  due to the slack recovery

=  $\pm 4$  mm according to whether the chord is in compression or tension.

Then:

$$v = 4 \times (2,31 + 2,85 + 0,5 + 0,66 + 0,68 + 0,66 + 0,68 + 0,71 + 0,75 + \dots + 0,17 + 0,75 + 0,72 + 0,68 + 0,66 + 0,68 + 0,66 + 0,5)$$

$$v = 58,4 \text{ mm}$$

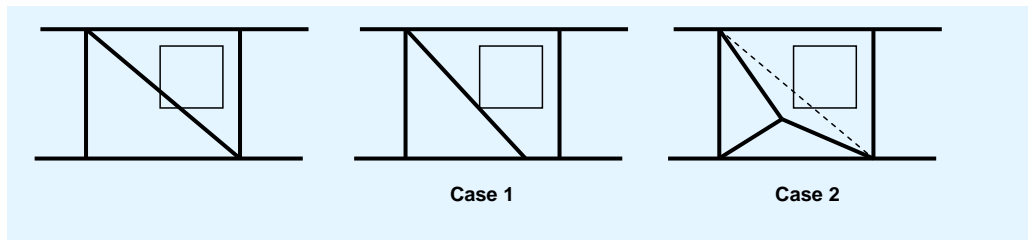
This is a significant additional deflection, compared with the deflection due to the ULS combination (127 mm).

### 3.7 Modification of a truss for the passage of equipment

It frequently occurs that it is necessary to modify the form of a truss in order to allow equipment to pass (a large section duct for example).

Several solutions can be provided (Figure 3.9):

- Either to increase the passage area available by an eccentricity in the connection of one of the chords (case 1)
- Or “break” the straight form of a diagonal, by triangulating the breakage point (case 2).



**Figure 3.9** Passage of a duct – Local modification of the truss

In case 1, the secondary moments which result from the introduction of an eccentricity increase in relation to the size of the eccentricity. If there is a choice, it is always preferable to introduce an eccentricity in the least stressed chords.

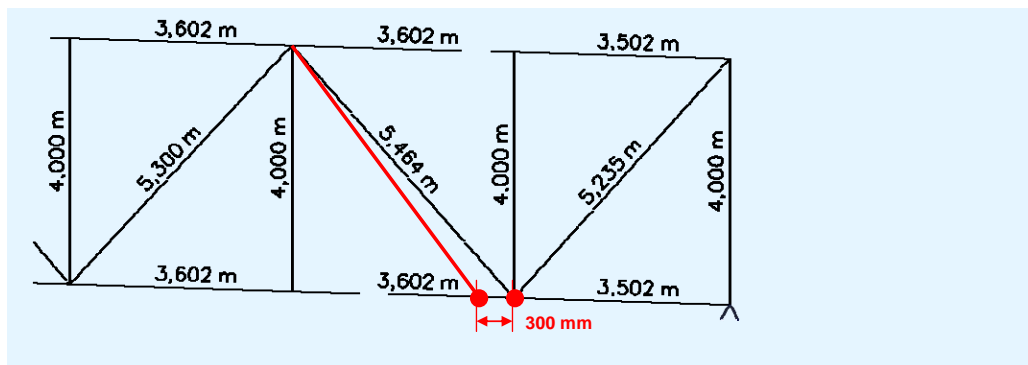
In case 2, care must be taken with several phenomena:

- The axial force can increase significantly in certain chords situated in the immediate proximity of the modified panel (as a result of modification to the position of the members).
- “Secondary” moments appear as a result of the lack of stiffness in a broken diagonal compared with a straight diagonal, even if the breakage point is triangulated.
- The breakage point must of course be triangulated in the plane of the truss; it must also be restrained out-of-plane (where three members meet) if the broken diagonal is in compression.

These two phenomena (case 1 and case 2) are illustrated using the worked example.

### 3.7.1 Introduction of an eccentricity axis in a diagonal (case 1)

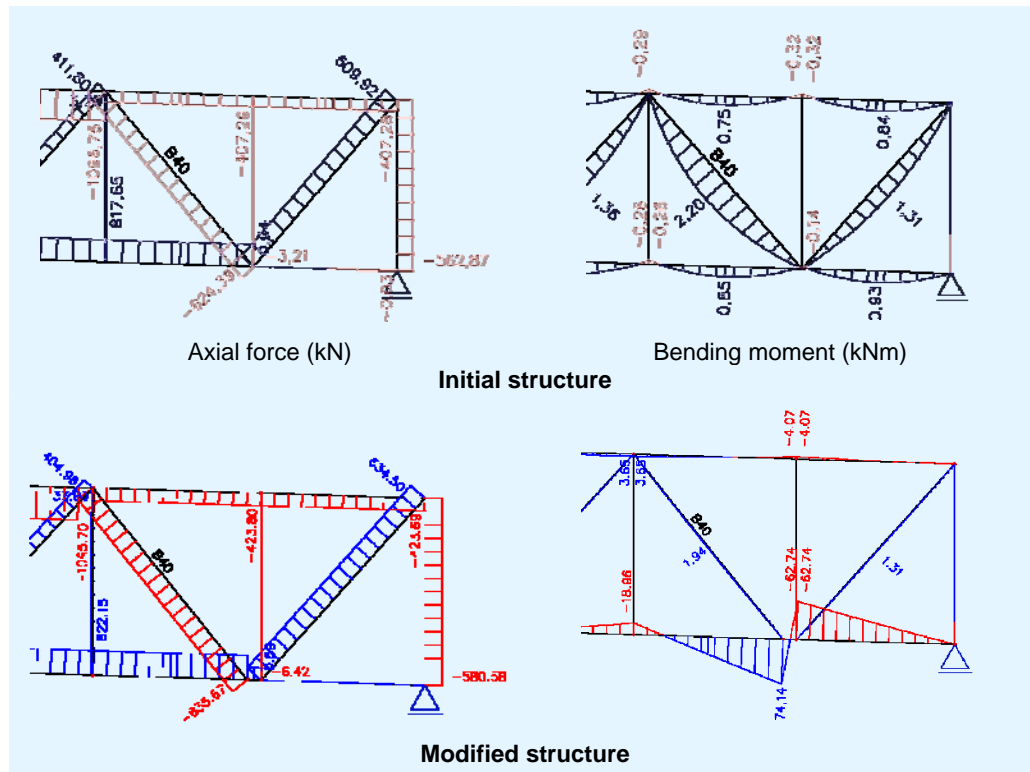
The truss panel through which the passage of equipment is required is the second panel from the support on the right. Figure 3.10 shows a part of the truss, with the eccentricity of a diagonal.



**Figure 3.10** Passage of a duct – Eccentricity of a diagonal

Changes in axial forces in the modified area are represented on the Figure 3.11.





**Figure 3.11** Effects of the eccentricity of diagonal under ULS gravity loading

The 300 mm eccentricity makes the triangulation imperfect.

The main consequence of this arrangement is a significant increase in the bending moments in the lower chord that receives the eccentric diagonal. A 74,15 kNm moment is calculated in the second chord member from the right hand support, a 62,72 kNm moment in the first chord member, much higher than in the initial structure without eccentricity.

The elastic moment resistance of an IPE 330 horizontal section is:

$$69,2 \times 0,355 = 24,57 \text{ kNm}$$

The bending capacity is therefore greatly exceeded, apart from any other interactions. Reinforcement of the lower chord member will therefore be required in order to support the axis eccentricity introduced.

### 3.7.2 “Broken” diagonal (example 2)

The panel of the penetration equipment is the same as in 3.6.1. Figure 3.12 is a diagram of the diagonal “breakage”.

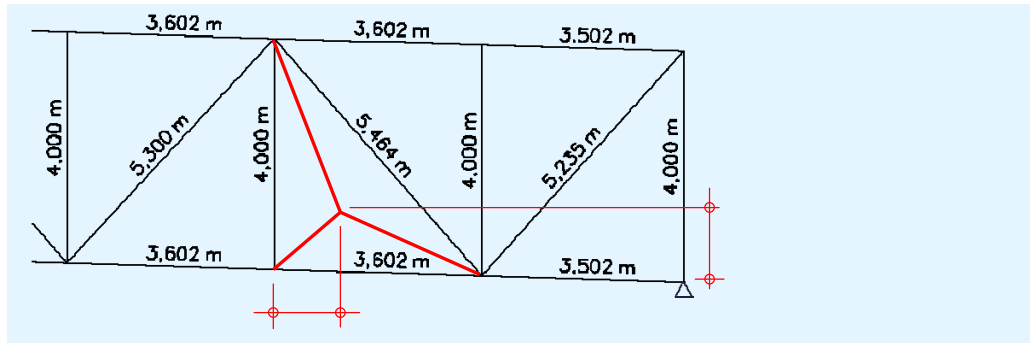


Figure 3.12 Passage of a duct – Broken diagonal

Development of stress in the modified area is represented on the section diagrams in Figure 3.13.

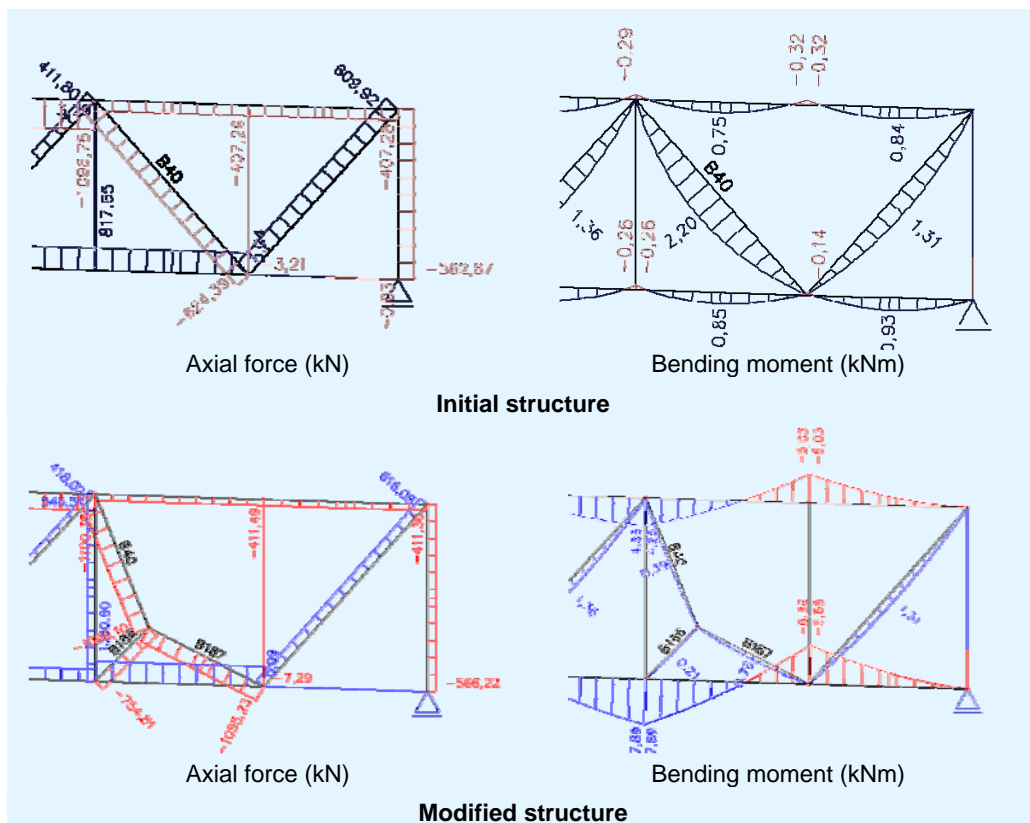


Figure 3.13 Effects of a broken diagonal under ULS gravity loading

The effects of modification on the calculated stresses are mainly:

- A noticeable increase is observed in the axial force in the second lower chord member from the right hand support (in the panel with the broken diagonal): the tension calculated increases from 818 to 1350 kN.
- A significant increase is also observed in the compression force in the broken diagonal compared with the rectilinear diagonal of the initial structure: compression increases from 624 to 1090 kN.
- As far as the additional triangulation member is concerned, this supports a normal compression force of 755 kN.
- In the lower chord, as well as an increase in the normal tension force, an increase in “secondary” moments is also observed on the three right panels

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The modification to the structure (broken diagonal) therefore has a significant effect on the size of the members.

## 4 VERIFICATION OF MEMBERS

As seen in the preceding section, which dealt with the global analysis, the members are mainly subjected to axial forces.

It was also observed that, in many cases, members are also subject to stress by bending moments, i.e. secondary moments.

### 4.1 Verification of members under compression

The resistance of a member to compression is evaluated by taking into account the different modes of instability:

- Local buckling of the section is controlled using section classification, and when necessary, effective section properties (class 4)
- Buckling of the member is controlled by applying a reduction coefficient in the calculation of resistance.

For a compression member, several buckling modes must be considered. In most truss members, only flexural buckling of the compressed members in the plane of the truss structure and out of the plane of the truss structure need be evaluated.

For each buckling mode, the buckling resistance is obtained from EN 1993-1-1<sup>[3]</sup> by applying a reduction to the resistance of the cross-section. This reduction factor is obtained from the slenderness of the member, which depends on the elastic critical force.

For the diagonals and the verticals stressed in uniform compression. the elastic critical force is determined from the buckling length of the member in accordance with EN 1993-1-1, 6.3.1.3. The following can be observed, according to Annex BB §BB.1 of EN 1993-1-1:

- For buckling in the plane of the truss beam: the buckling length is taken equal to 90% of the system length (distance between nodes), when the truss member is connected at each end with at least two bolts, or by welding (EN 1993-1-1 §BB.1.1(4)).

(An exception is made by Annex BB for angle truss members, for which a different evaluation is given; it is not specified in this annex if the particular rule also concerns members made up to two pairs of angles: by way of simplification, it is recommended that a buckling length of 0,9 times the length of the axis be retained.)

- For buckling out of plane of the truss beam, the buckling length is taken equal to the system length.

For buckling in the plane of the truss of the chord members in uniform compression, the buckling length may be taken as 90% of its system length (distance between nodes).

For buckling out of plane of the truss, it can be more difficult to determine the elastic critical force for the following reasons:

- There is not necessarily a lateral support at each node of the truss
- The lateral support points are not necessarily effectively rigid.

When there is no lateral support at each node along the chord, the segment located between support points is subject to variable compression between bays. In these circumstances:

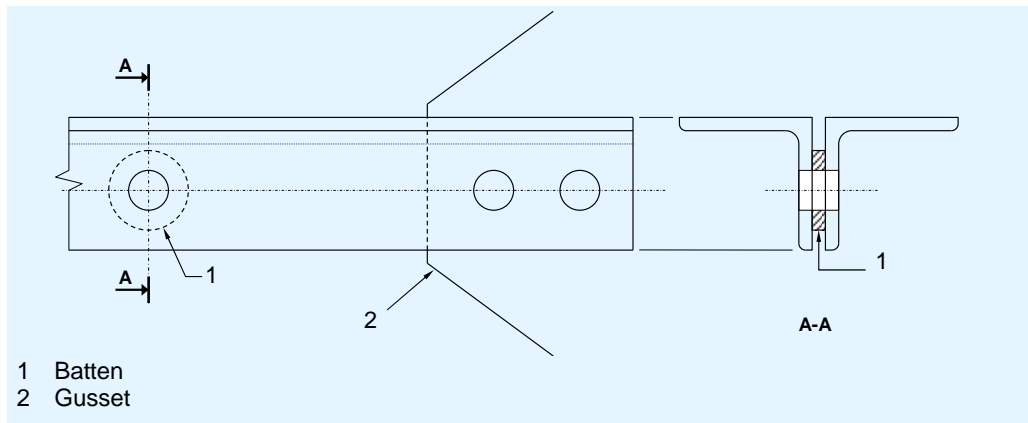
- A conservative approach would be to use the normal compression force at its maximum value and to take the buckling length as the distance between supports but this can lead to an under-estimate of the chord resistance.
- Refined methods can be adopted by investigating an equivalent buckling length under constant compression.

In the worked example, where the truss supports a roof, with purlins at the level of the upper chord of the truss:

- All the purlins connected to a roof bracing can be considered as lateral rigid support points.
- Intermediate purlins can also be considered as a rigid point of support. Insofar as a diaphragm role has been attributed to the roof (class 2 construction according to EN 1993-1-3).
- With regard to the lower chord, these lateral support points are provided by additional vertical bracing elements between trusses (see the braces under the truss purlins in Figure 2.2).

Another point to note, which is very common, concerning determination of the compression resistance, is the case of pairs of members. It is quite common, as was stated, to make up members from a truss structure using two angles, or two channels (UPE).

In order to ensure that such built-up members will behave as sole members in the flexural buckling mode, the two components are connected by small battens (Figure 4.1). Since the role of these members is to prevent relative slip of one component compared with the other, they must be connected without slack. The gap between the angles, and the thickness of the battens, should be the same as the thickness of the gusset to which the built-up member is connected.



**Figure 4.1** Members composed of two angles

The maximum spacing of the connections between members is limited by EN 1993-1-1 to 15 times the minimum radius of gyration of the isolated component. Otherwise a more complex verification needs to be carried out, by taking into account the shear stiffness of the composed member. This limitation is very restrictive. By way of example, in order to link two  $50 \times 50 \times 5$  angles by respecting the spacing limit, it would be necessary to provide a batten every 15 cm.

In order to illustrate the different principles stated above, justifying calculations are developed in the following sections for the different types of compressed members in the worked example truss structure. The results are taken from the basic worked example:

- IPE 330 chords with horizontal web
- Web members are assumed to be hinged at both ends
- Chords are assumed to be continuous.

#### 4.1.1 Upper chord in compression

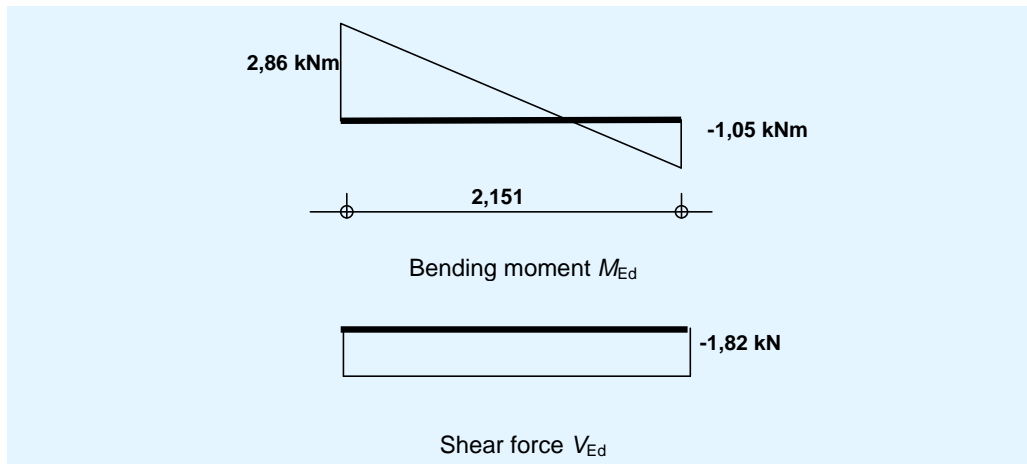
The verifications set out below, concern the upper chord member adjacent to mid span (element B107 in Figure 3.1), in which the normal compression force calculated under gravity ULS loads is greatest and equal to:

$$N_{Ed} = -1477 \text{ kN}$$

The checks take into account the coincident bending moments.

Note that the verification should also be carried out on the first member from the mid span, which is not restrained by the secondary truss: axial force of lesser compression, but with increased buckling length in the plane of the truss. Since the calculation is identical, it is not set out formally below. If this verification indicated a lack of resistance, the reinforcement solution would of course consist of extending the installation of the secondary truss.

The shear force and the bending moments are given in Figure 4.2.



**Figure 4.2** Bending moment and shear force in the upper chord

### Cross-section properties

For an IPE 330 with horizontal web (steel grade S355)

$$A = 62,6 \text{ cm}^2$$

$$I_y = 11770 \text{ cm}^4$$

$$I_z = 788 \text{ cm}^4$$

$$W_{el,z} = 98,5 \text{ cm}^3$$

### Class of the cross-section

The material parameter is:

$$\varepsilon = 0,81$$

As simplification, the cross-section can be classified in uniform compression, even if it is subjected to combined axial force and bending moment.

The compressed flanges are classified as outstand flanges (EN 1993-1-1 Table 5.2, Sheet 2):

$$\frac{c}{t} = \frac{58,25}{11,5} = 5,1 < 9\varepsilon = 7,29$$

The flange is Class 1.

The web is classified as an internal compressed part (EN 1993-1-1 Table 5.2, Sheet 1):

$$\frac{c}{t} = \frac{271}{7,5} = 36,1 > 42\varepsilon = 34,02$$

The web is Class 4.

### Effective properties of the cross-section

The effective area  $A_{\text{eff}}$  is calculated for pure compression.

The flanges are Class 1, so fully effective.

The effective width of the web is evaluated according to EN 1993-1-5 (Table 4.1):

$$\psi = 1 \Rightarrow k_{\sigma} = 4$$

$$\bar{\lambda}_p = \frac{\frac{b}{t}}{28,4\epsilon\sqrt{k_{\sigma}}} = \frac{\frac{271}{7,5}}{28,4 \times 0,81 \times \sqrt{4}} = 0,782 > 0,673$$

$$\psi = 1 \Rightarrow k_{\sigma} = 4 \Rightarrow \bar{\lambda}_p = \frac{\frac{b}{t}}{28,4\epsilon\sqrt{k_{\sigma}}} = \frac{\frac{271}{7,5}}{28,4 \times 0,81 \times \sqrt{4}} = 0,782 > 0,673$$

$$\rho = \frac{\bar{\lambda}_p - 0,055(3 + \psi)}{\bar{\lambda}_p^2} = 0,919 \Rightarrow b_{\text{eff}} = 0,919 \times 271 = 249 \text{ mm}$$

$$b_{e1} = b_{e2} = 0,5b_{\text{eff}} = 124,5 \text{ mm}$$

$$b_{\text{eff}} = 0,919 \times 271 = 249 \text{ mm}$$

$$b_{e1} = b_{e2} = 0,5 b_{\text{eff}} = 124,5 \text{ mm}$$

The effective area of the section is:

$$A_{\text{eff}} = 6260 - (271 - 249) \times 7,5 = 6095 \text{ mm}^2$$

The effective elastic modulus about the weak axis ( $W_{\text{eff},z}$ ) is calculated for pure bending.

In simple bending in the plane of the truss, about the weak axis, the flanges are inevitably Class 1, whilst the web is not stressed. Then the section is fully effective:

$$W_{\text{eff},z} = W_{\text{el},z} = 98,5 \text{ cm}^3$$

### Resistance of cross-section

In compression (EN 1993-1-1 §6.2.4):

$$N_{\text{c,Rd}} = \frac{A_{\text{eff}} f_y}{\gamma_{\text{M0}}} = \frac{6095 \times 0,355}{1,0} = 2164 \text{ kN}$$

$$\frac{N_{\text{Ed}}}{N_{\text{c,Rd}}} = \frac{1477}{2164} = 0,683 < 1 \text{ OK}$$

In bending in the plane of the truss (EN 1993-1-1 §6.2.5):

$$M_{\text{z,Rd}} = \frac{W_{\text{eff},z} f_y}{\gamma_{\text{M0}}} = \frac{98,5 \times 0,355}{1,0} = 34,97 \text{ kNm}$$

$$\frac{M_{\text{Ed}}}{M_{\text{z,Rd}}} = \frac{2,86}{34,97} = 0,082 < 1 \text{ OK}$$

In shear (EN 1993-1-1 §6.2.6):

$$A_{\text{v},y} = 2 \times 160 \times 11,5 = 3680 \text{ mm}^2$$



$$V_{pl,Rd} = \frac{A_{v,y} \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{3680 \times \frac{0,355}{\sqrt{3}}}{1,0} = 754 \text{ kN}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{1,82}{754} = 0,002 < 1 \text{ OK}$$

Since  $V_{Ed}/V_{pl,Rd}$  is less than 0,5, there is no influence of the shear force on the resistance of the cross-section under bending moment and axial force.

M-N interaction (EN 1993-1-1 §6.2.93):

The M-N interaction is taken into account using the following criterion:

$$0,683 + 0,082 = 0,765 < 1 \quad \text{OK}$$

### Buckling resistance of member

**Buckling resistance in the plane of the truss, i.e. about the weak axis of the cross-section** (EN 1993-1-1 § 6.3.1)

The buckling length of the upper chord member is equal to 90% of the system length (EN 1993-1-1 §B.B.1.1):

$$L_{cr,z} = 0,9 \times 2151 = 1936 \text{ mm}$$

The elastic critical force is:

$$N_{cr,z} = \frac{\pi^2 EI_z}{l_z^2} = \frac{\pi^2 \times 21000 \times 788}{193,6^2} = 4357 \text{ kN}$$

The slenderness is given by:

$$\bar{\lambda}_z = \sqrt{\frac{A_{eff} f_y}{N_{cr,z}}} = \sqrt{\frac{6095 \times 0,355}{4357}} = 0,705$$

The buckling curve to use is curve **b** (EN 1993-1-1 Table 6.2), and the imperfection factor is:

$$\alpha = 0,34$$

$$\Phi_z = 0,5 \times (1 + \alpha(\bar{\lambda}_z - 0,2) + \bar{\lambda}_z^2) = 0,8344$$

$$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \bar{\lambda}_z^2}} = \frac{1}{0,8344 + \sqrt{0,8344^2 - 0,705^2}} = 0,781$$

The design buckling resistance is then:

$$N_{b,z,Rd} = \frac{\chi_z A_{eff} f_y}{\gamma_{M1}} = \frac{0,781 \times 6095 \times 0,355}{1,0} = 1690 \text{ kN}$$

$$N_{Ed} / N_{b,z,Rd} = 1477/1690 = 0,874 \text{ OK}$$

**Buckling resistance out of the plane of the truss, i.e. about the strong axis of the cross-section (EN 1993-1-1 § 6.3.1)**

The lateral supports of the upper chord are composed of truss purlins at 8504 mm intervals.

The normal compression force is almost constant between lateral supports (see 3.2).

There is therefore no need to use a method which allows for non-uniform force.

The elastic critical force is:

$$N_{cr,y} = \frac{\pi^2 EI_y}{l_y^2} = \frac{\pi^2 \times 21000 \times 11770}{850,4^2} = 3373 \text{ kN}$$

The slenderness is given as:

$$\bar{\lambda}_y = \sqrt{\frac{A_{eff} f_y}{N_{cr,y}}} = \sqrt{\frac{6095 \times 0,355}{3373}} = 0,8009$$

The buckling curve is curve **a** (EN 1993-1-1 Table 6.2), and the imperfection factor is:

$$\alpha = 0,21$$

$$\Phi_y = 0,5(1 + \alpha(\bar{\lambda}_y - 0,2) + \bar{\lambda}_y^2) = 0,8838$$

$$\chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \bar{\lambda}_y^2}} = \frac{1}{0,8838 + \sqrt{0,8838^2 - 0,8009^2}} = 0,7952$$

And so the compression resistance is therefore:

$$N_{b,y,Rd} = \frac{\chi_y A_{eff} f_y}{\gamma_{M1}} = \frac{0,7952 \times 6095 \times 0,355}{1,0} = 1720 \text{ kN}$$

$$N_{Ed} / N_{b,y,Rd} = 1477 / 1720 = 0,859 \quad \text{OK}$$

**M-N interaction (EN 1993-1-1 §6.3.3):**

There is no effect of lateral torsional buckling to consider for a member in bending about its weak axis (no bending about the strong axis). The criteria are:

$$\frac{N_{Ed}}{\chi_y A_{eff} f_y / \gamma_{M1}} + k_{yz} \frac{M_{z,Ed}}{W_{eff,z} f_y / \gamma_{M1}} \leq 1 \quad (\text{Eq. 6.61 in EN 1993-1-1})$$

$$\frac{N_{Ed}}{\chi_z A_{eff} f_y / \gamma_{M1}} + k_{zz} \frac{M_{z,Ed}}{W_{eff,z} f_y / \gamma_{M1}} \leq 1 \quad (\text{Eq. 6.62 in EN 1993-1-1})$$

Using resistances already calculated, these criteria can also be written as:

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$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yz} \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1$$

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zz} \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1$$

The interaction factors  $k_{yz}$  and  $k_{zz}$  are calculated according to Annex A of EN 1993-1-1, for a Class 4 section:

$$k_{yz} = C_{mz} \frac{\mu_y}{1 - \frac{N_{Ed}}{N_{cr,z}}}$$

where:

$$C_{mz} = 0,79 + 0,21\psi + 0,36(\psi - 0,33) \frac{N_{Ed}}{N_{cr,z}}$$

$$\psi = \frac{-1,05}{2,86} = -0,367$$

$$C_{mz} = 0,628$$

$$\mu_y = \frac{1 - \frac{N_{Ed}}{N_{cr,y}}}{1 - \chi_y \frac{N_{Ed}}{N_{cr,y}}} = \frac{1 - \frac{1477}{3373}}{1 - 0,7952 \frac{1477}{3373}} = 0,8624$$

$$k_{yz} = 0,628 \times \frac{0,8624}{1 - \frac{1477}{4357}} = 0,819$$

First interaction criterion (eq. 6.61)

$$\frac{1477}{1720} + 0,819 \times \frac{2,86}{34,97} = 0,926 \leq 1 \text{ OK}$$

$$k_{zz} = C_{mz} \frac{\mu_z}{1 - \frac{N_{Ed}}{N_{cr,z}}}$$

where:

$$C_{mz} = 0,628$$

$$\mu_z = \frac{1 - \frac{N_{Ed}}{N_{cr,z}}}{1 - \chi_z \frac{N_{Ed}}{N_{cr,z}}} = \frac{1 - \frac{1477}{4357}}{1 - 0,781 \frac{1477}{4357}} = 0,899$$

Then, the factor  $k_{zz}$  can be calculated:

$$k_{zz} = 0,628 \times \frac{0,899}{1 - \frac{1477}{4357}} = 0,854$$

Second interaction criterion (eq. 6.62)

$$\frac{1477}{1690} + 0,854 \times \frac{2,86}{34,97} = 0,944 \leq 1 \text{ OK}$$

#### **Note on secondary trusses**

The presence of secondary trusses in the central part of the truss (see diagram 2.3) permitted the reduction by half of the buckling length of the upper chord in the plane of the truss.

The secondary truss is sized in order to support a buckling restraint load whose value depends on the compression force in the supported chord and on its slenderness ratio (see EN 1993-3-1 on subject of design of pylons in annex H4).

#### **4.1.2 Lower chord in compression**

With respect to the complete design of the structure, it is also of course essential to check the lower chord, subject to the lower compression force, but without support from a secondary truss.

Verification of the lower chord in compression is similar to that described for the upper chord in compression, in 4.1.1.

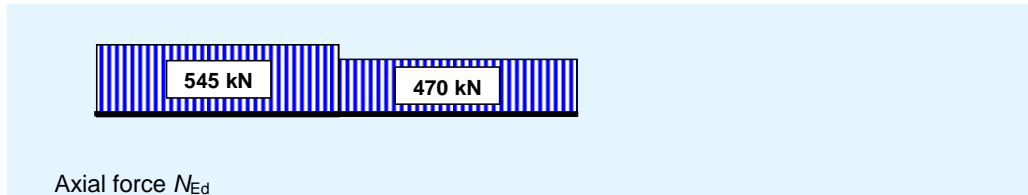
Lateral restraint of the lower chord is provided at each purlin (Figure 2.2).

The only specific point which would be interesting to develop is an analysis of the buckling out of plane of the truss.

Buckling of the lower chord is to be considered similarly to that of the upper chord, for a length equal to the distance between truss panels, thanks to the presence of sub-panel braces (See Figure 2.3).

The difference is that the axial force in the lower chord varies along the buckling length, in two panels, whereas the force was constant along the buckling length for the upper chord.

It should also be noted here that, for the chord member with the greatest bending moment, the variation in axial force is very small; in a real design, the small reduction in buckling length due to variation of normal axial force can safely be ignored.



**Figure 4.3** Axial force in the lower chord

### 4.1.3 Diagonal in compression

The diagonal, whose resistance is calculated here, by way of example, is the second diagonal from the right support (element B40 in Figure 3.1), under ULS gravity loading.

The compression force is:

$$N_{Ed} = -624,4 \text{ kN}$$

Initially, as in common practice, the bending moment due to the self weight of the member is ignored.

The effect of this moment will be evaluated later.

#### Cross-section properties of a single angle

For a  $150 \times 150 \times 15$  L

$$A = 43 \text{ cm}^2$$

$$z_G = y_G = 4,25 \text{ cm}$$

$$I_y = I_z = 898,1 \text{ cm}^4$$

$$I_v = 369 \text{ cm}^4$$

For a pair of angles

Section area:

$$A = 2 \times 43 = 86 \text{ cm}^2$$

Second moment of area out of plane of the truss (the section is assumed to be homogeneous), assuming the gap between the angles is 10 mm:

$$I_y = 2 \times 898,1 + 2 \times 43 \times (4,25 + 1,0/2)^2 = 3737 \text{ cm}^4.$$

Second moment of area in the plane of the truss:

$$I_z = 2 \times 898,1 = 1796 \text{ cm}^4$$

#### Class of section in uniform compression

Material parameter for  $f_y = 355 \text{ N/mm}^2$ :  $\varepsilon = 0,81$

For an angle (EN 1993-1-1 Table 5.2 (Sheet 3)):

$$\frac{h}{t} = \frac{150}{15} = 10 < 15\varepsilon = 12,15$$

$$\frac{h+b}{2t} = \frac{2 \times 150}{2 \times 15} = 10 > 11,5\varepsilon = 9,31$$

The section is a Class 4 and it is therefore not fully effective in uniform compression. The effective area of the cross-section should be calculated with reference to EN 1993-1-5. Such a calculation leads to a fully effective area:

$$A_{\text{eff}} = A = 86 \text{ cm}^2$$

### Resistance of the cross-section

The resistance of the section in uniform compression is therefore given by:

$$N_{\text{c,Rd}} = \frac{Af_y}{\gamma_{\text{M0}}} = \frac{8600 \times 0,355}{1,0} = 3053 \text{ kN}$$

### Buckling resistance of member

#### Buckling resistance in the plane of the truss

The buckling length is equal to:

$$0,9 \times 5,464 = 4,918 \text{ m}$$

The elastic critical force is:

$$N_{\text{cr,z}} = \frac{\pi^2 EI_z}{l_y^2} = \frac{\pi^2 \times 21000 \times 1796}{491,8^2} = 1539 \text{ kN}$$

The slenderness is given by:

$$\bar{\lambda}_z = \sqrt{\frac{Af_y}{N_{\text{cr,z}}}} = \sqrt{\frac{8600 \times 0,355}{1539}} = 1,408$$

The buckling curve is curve b (EN 1993-1-1 Table 6.2), and the imperfection factor is:

$$\alpha = 0,34$$

$$\Phi_z = 0,5 \times (1 + \alpha(\bar{\lambda}_z - 0,2) + \bar{\lambda}_z^2) = 1,697$$

$$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \bar{\lambda}_z^2}} = \frac{1}{1,697 + \sqrt{1,697^2 - 1,408^2}} = 0,378$$

And the buckling resistance is then:

$$N_{\text{b,z,Rd}} = \frac{\chi_z Af_y}{\gamma_{\text{M1}}} = \frac{0,378 \times 8600 \times 0,355}{1,0} = 1154 \text{ kN}$$

#### Buckling resistance out of plane of the truss

The buckling length is equal to the system length:  $L_{\text{cr,y}} = 5,464 \text{ m}$ .

The critical axial force is:

$$N_{\text{cr,y}} = \frac{\pi^2 EI_y}{l_y^2} = \frac{\pi^2 \times 21000 \times 3737}{546,5^2} = 2594 \text{ kN}$$

The slenderness is given by:

$$\bar{\lambda}_y = \sqrt{\frac{Af_y}{N_{cr,y}}} = \sqrt{\frac{8600 \times 0,355}{2594}} = 1,085$$

The buckling curve to use is curve b (see EN 1993-1-1, table 6.2), and the imperfection factor is:

$$\alpha = 0,34$$

$$\Phi_y = 0,5 \times (1 + \alpha(\bar{\lambda}_y - 0,2) + \bar{\lambda}_y^2) = 1,239$$

$$\chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \bar{\lambda}_y^2}} = \frac{1}{1,239 + \sqrt{1,239^2 - 1,085^2}} = 0,544$$

The design buckling resistance is:

$$N_{b,y,Rd} = \frac{\chi_y Af_y}{\gamma_{M1}} = \frac{0,544 \times 8600 \times 0,355}{1,0} = 1661 \text{ kN}$$

The buckling resistance in the plane of the truss is less and the verification is:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{624,4}{1154} = 0,541 < 1,0 \text{ OK}$$

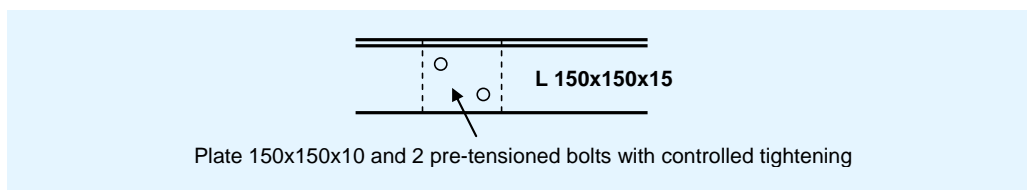
The resistance of the diagonal is adequate; its section could be optimised.

### Connection battens

The diagonal is composed of two angles linked by battens. The calculation of the resistance previously undertaken assumed the section is homogenous (for the buckling out of plane of the truss).

In order to support this hypothesis, EN 1993-1-1 requires the placing of connection bars spread out at no more than 15 times the minimum radius of gyration of the isolated angle; for an angle  $150 \times 150 \times 15$  that is a distance of  $15 \times 29,3 = 440$  mm.

In view of the resistance reserves, it is recommended that the connection bars be spaced further apart (the costs of fabrication and installation are not negligible). Instead of the 12 connection battens per diagonal which the above condition lead to, consider only 3 bars be placed, 1366 mm apart.



**Figure 4.4 Connection batten**

In order for the battens to be effective, they must be arranged as illustrated here. This results in a buckling length about the principal axis equal to  $0,7 \times 1366 = 956$  mm.

For this type of buckling the elastic critical force is:

$$N_{cr,v} = \frac{\pi^2 EI_v}{l_v^2} = \frac{\pi^2 \times 210000 \times 369 \times 10^4}{956^2} \times 10^{-3} = 8368 \text{ kN}$$

The slenderness for a single angle is:

$$\bar{\lambda}_v = \sqrt{\frac{Af_y}{N_{cr,v}}} = \sqrt{\frac{4300 \times 355}{8368000}} = 0,427$$

The buckling curve to use is curve **b** and the imperfection factor is:  $\alpha = 0,34$

$$\Phi_v = 0,5 \times (1 + 0,34 \times (\bar{\lambda}_v - 0,2) + \bar{\lambda}_v^2) = 0,630$$

$$\chi_v = \frac{1}{\Phi_v + \sqrt{\Phi_v^2 - \bar{\lambda}_v^2}} = \frac{1}{0,630 + \sqrt{0,630^2 - 0,427^2}} = 0,915$$

Conservatively, the resistance to the compression may be evaluated calculating the reduction factor as the product of that for the whole member and that for an individual angle between battens:

$$\chi = \text{Min}(\chi_y ; \chi_z) \times \chi_v = 0,378 \times 0,915 = 0,346$$

The design buckling resistance of the diagonal is:

$$N_{b,Rd} = \frac{\chi Af_y}{\gamma_{M1}} = \frac{0,346 \times 8600 \times 355}{1,0} \times 10^{-3} = 1056 \text{ kN}$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{624,4}{1056} = 0,591 < 1,0$$

The compression resistance is adequate.

### Local verification of the section to the right of the gusset plate connection

This verification carried out in Appendix B

### Effect of bending moment due to self weight of the diagonal

The bending moment is:

$$M_{y,Ed} = 2,20 \text{ kNm} \quad (\text{see 3.2 above}).$$

The elastic modulus of the cross-section for bending in the plane of the truss is:

$$W_{el,z} = 167 \text{ cm}^3.$$

Interaction criteria are given in EN 1993-1-1 §6.3.3:

$$\frac{N_{Ed}}{\chi_y Af_y / \gamma_{M1}} + k_{yz} \frac{M_{z,Ed}}{W_{el,z} f_y / \gamma_{M1}} \leq 1$$

$$\frac{N_{Ed}}{\chi_z Af_y / \gamma_{M1}} + k_{zz} \frac{M_{z,Ed}}{W_{el,z} f_y / \gamma_{M1}} \leq 1$$



where:

The  $k_{yz}$  factor is:

$$k_{yz} = C_{mz} \frac{\mu_y}{1 - \frac{N_{Ed}}{N_{cr,z}}}$$

$$\mu_y = \frac{1 - \frac{N_{Ed}}{N_{cr,y}}}{1 - \chi_v \chi_y \frac{N_{Ed}}{N_{cr,y}}} = \frac{1 - \frac{624,4}{2594}}{1 - 0,915 \times 0,544 \times \frac{624,4}{2594}} = 0,863$$

$$C_{mz} = 1 + 0,03 \frac{N_{Ed}}{N_{cr,z}} = 1 + 0,03 \frac{624,4}{1539} = 1,012$$

$$k_{yz} = 1,012 \times \frac{0,863}{1 - \frac{624,4}{1539}} = 1,47$$

The  $k_{zz}$  factor is:

$$k_{zz} = C_{mz} \frac{\mu_z}{1 - \frac{N_{Ed}}{N_{cr,z}}}$$

$$\mu_z = \frac{1 - \frac{N_{Ed}}{N_{cr,z}}}{1 - \chi_v \chi_z \frac{N_{Ed}}{N_{cr,z}}} = \frac{1 - \frac{624,4}{1539}}{1 - 0,915 \times 0,378 \times \frac{624,4}{1539}} = 0,691$$

$$k_{zz} = 1,012 \times \frac{0,691}{1 - \frac{624,4}{1539}} = 1,18$$

From which:

$$\frac{624400}{0,915 \times 0,544 \times 8600 \times 355 / 1,0} + 1,47 \times \frac{2,20 \times 10^6}{167000 \times 355 / 1,0} = 0,465 \leq 1$$

$$\frac{624400}{0,915 \times 0,378 \times 8600 \times 355 / 1,0} + 1,18 \times \frac{2,20 \times 10^6}{167000 \times 355 / 1,0} = 0,635 \leq 1$$

When the bending moment due to self weight of the diagonal is taken into account, the resistance criterion increases from 0,591 to 0,635: that is an increase of 7%.

## 4.2 Verification of members in tension

A particular feature when checking the resistance of tension members is the existence of criteria which bring into play the net section of the member. This is explored for the worked example.

#### 4.2.1 Lower chord in tension (flat IPE 330)

The lower chord in tension is verified for calculated forces near the mid-span. Given the results shown in 3.2 above:

$$N_{Ed} = 1582 \text{ kN}$$

$$M_{Ed} = 1,69 \text{ kNm}$$

The tension resistance of the section is determined by two conditions, one in a “gross” section and the other in a “net” section :

##### Gross section

$$A = 6260 \text{ mm}^2$$

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{6260 \times 0,355}{1,0} = 2222 \text{ kN}$$

##### Net section

$$A_{net} = 6260 - (4 \times 24 \times 11,5) - (3 \times 22 \times 7,5) = 4661 \text{ mm}^2$$

$$N_{u,Rd} = \frac{0,9 A_{net} f_u}{\gamma_{M0}} = \frac{0,9 \times 4661 \times 0,51}{1,25} = 1711 \text{ kN}$$

Tension resistance is given by:

$$N_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) = 1711 \text{ kN}$$

In simple bending, in the truss plane (EN 1993-1-1 (6.2.5)), class 1 of the section allows the plastic modulus to be mobilised:

$$W_{pl} = \frac{2 \times 1,15 \times 16^2}{4} = 147,2 \text{ cm}^3$$

$$M_{pl,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} = \frac{147,2 \times 0,355}{1,0} = 52,3 \text{ kNm}$$

The verification is:

$$\frac{N_{Ed}}{N_{t,Rd}} = \frac{1582}{1711} = 0,93$$

$$\frac{M_{Ed}}{M_{Rd}} = \frac{1,69}{52,3} = 0,03$$

$$N\text{-}M \text{ Interaction: } 0,93 + 0,03 = 0,96 < 1$$

#### 4.2.2 Diagonal in tension (double angles L120 × 120 × 12)

Checking is done for the diagonal at the left hand support, under gravity loads. Given the results shown in 3.2 above:

$$N_{Ed} = 616,3 \text{ kN}$$

$$M_{Ed} = 1,36 \text{ kNm}$$

### Tension resistance

The tension resistance of the section is determined by two conditions, one in gross section and the other in net section:

#### Gross section

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{5510 \times 0,355}{1,0} = 1956 \text{ kN}$$

**Net section** (See arrangements described in Annex 2)

$$A_{net} = 5510 - (2 \times 26 \times 12) = 4886 \text{ mm}^2$$

For angles connected by a single leg, EN 1993-1-8 gives an additional requirement for the effect of eccentricity of the tension force in the angle (distance between the neutral axis and the gauge marking) on the forces (appearance of secondary moments).

This method involves the application of an ultimate resistance reduction factor for the angle (EN 1993-1-8 Clause 3.10.3(2))

$$N_{u,Rd} = \frac{\beta_3 A_{net} f_u}{\gamma_{M2}}$$

The reduction factor  $\beta_3$  depends on the distance between axes  $p_1$ .

$$\text{For, } p_1 = 2,5 \text{ } d_0 = 65 \text{ mm: } \beta_3 = 0,5 \text{ (EN 1993-1-8 Table 3.8)}$$

**N.B.:** The reduction factors  $\beta$  are only provided for a simple angle; the method is conservative for a “double angle”. It is recommended that, within the connection, the behaviour of the two simple diagonals is considered with respect to these local phenomena.

$$N_{u,Rd} = \frac{0,5 A_{net} f_u}{\gamma_{M0}} = \frac{0,5 \times 4886 \times 0,51}{1,25} = 997 \text{ kN}$$

Then:

$$N_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) = 997 \text{ kN}$$

### Bending resistance

In simple bending in the truss plane (EN 1993-1-1 (6.2.5)):

$$W_{el} = 85,46 \text{ cm}^3$$

$$M_{el,Rd} = \frac{W_{el} f_y}{\gamma_{M0}} = \frac{85,46 \times 0,355}{1,0} = 30,3 \text{ kNm}$$

Verification:

*Part 5: Detailed Design of Trusses*

$$\frac{N_{Ed}}{N_{t,Rd}} = \frac{616,3}{997} = 0,62 < 1$$

$$\frac{M_{Ed}}{M_{Rd}} = \frac{1,36}{30,3} = 0,05$$

And the M-N interaction criterion is:  $0,62 + 0,05 = 0,67 < 1$

## 5 VERIFICATION OF CONNECTIONS

### 5.1 Characteristics of the truss post connection

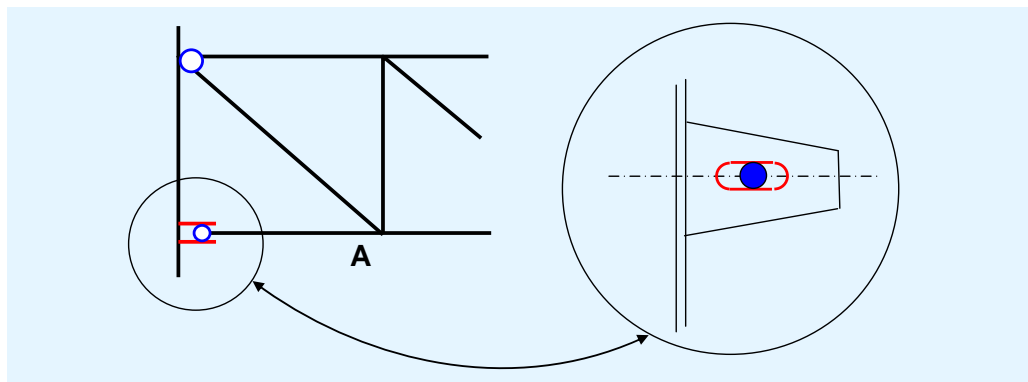
#### 5.1.1 General

It is essential to connect the truss and post according to the assumptions in the modelling.

In particular, the choice between a fixed connection and a pinned connection must be respected. The difference between these two types of connection is that the pinned connection allows a rotation independent deflection of the truss and the post. The outcome in terms of loading is that the hinge does not transmit any bending moment from the truss to the post, whereas a fixed connection does.

The rotation at the support of a truss is manifested by a differential horizontal displacement between the original node of the upper chord and the original node of the lower chord.

In order to permit global rotation, it is therefore necessary to allow the horizontal displacement of the end of one of the chords in relation to the post: usually, the displacement of the chord which does not receive the diagonal on support is released.



**Figure 5.1** Elongated hole on the bottom chord of the truss

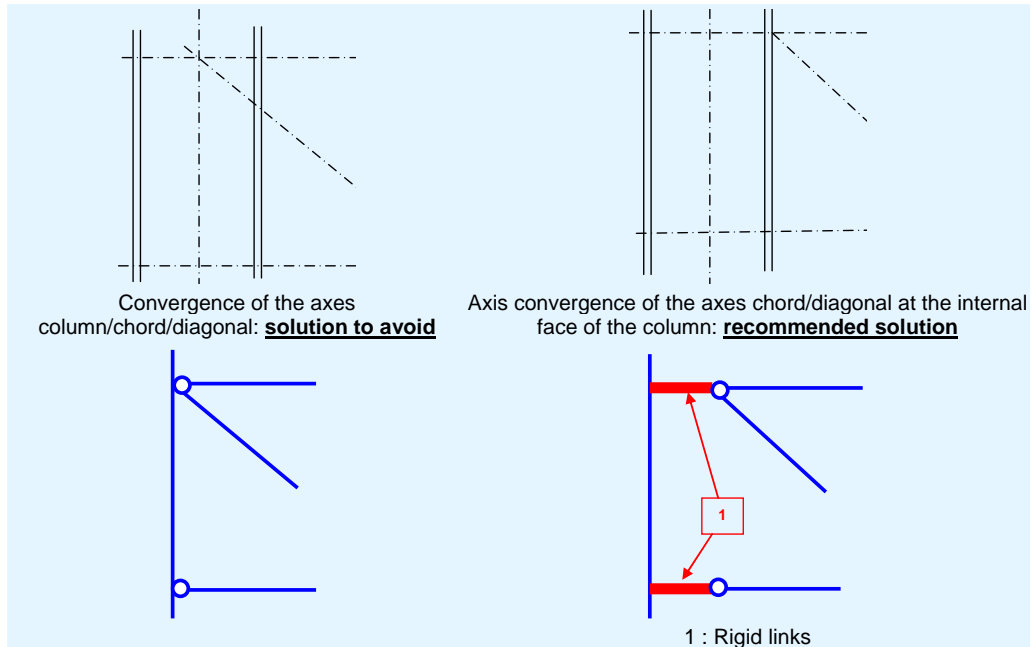
With such an arrangement, the axial force is zero in the lower chord in the first panel. The lower chord of the first truss node could therefore be stopped short (A in the diagram); nevertheless it is preferable to lengthen the lower chord and to connect it to the post in order to provide lateral stability of the truss at the level of the lower chord.

An application of this type of hinge action in the worked example is given in 5.1.2 below.

By contrast, in order to carry out a rigid truss-column connection, it is necessary to make a connection without slack from each of the chords of the truss to the column.

### 5.1.2 Convergence of the axes at the truss-column connection

Another question to be asked when carrying out the connection of a truss on a post is that of convergence of the axes of the connected members and of its effect on the modelling. The choices are illustrated in Figure 5.2.



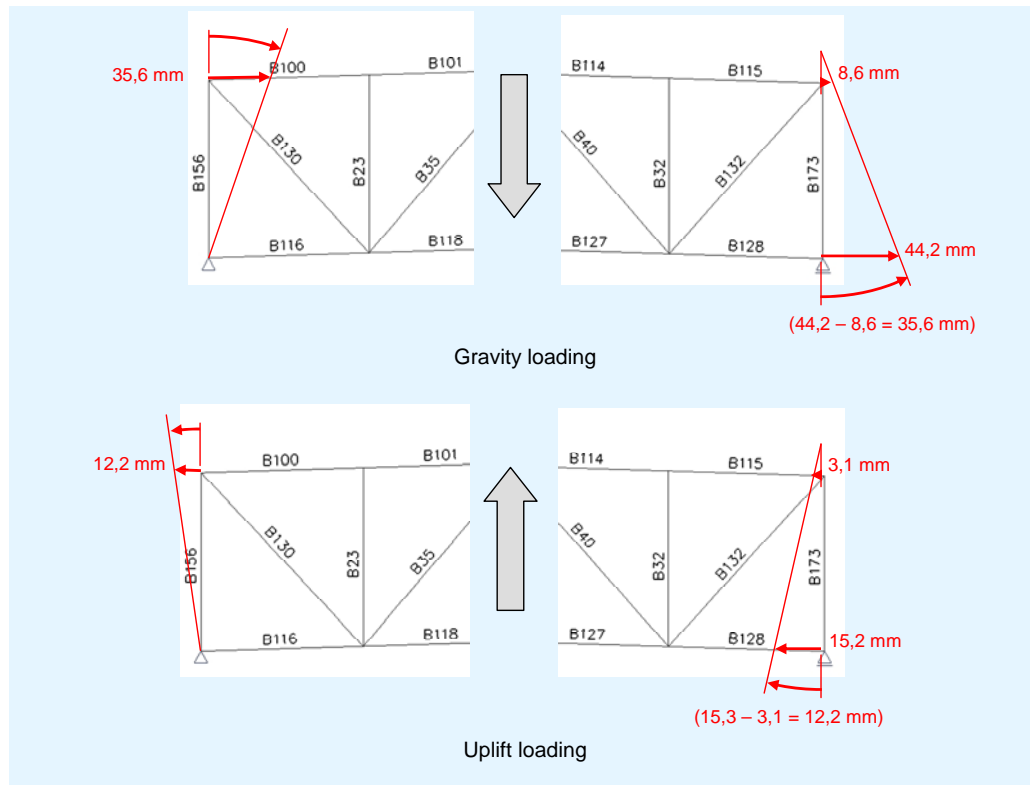
**Figure 5.2 Rigid truss-column connection**

In the first example, the actual physical connection and the model are not consistent: there is a risk of causing significant secondary moments in the diagonal and the chord. In the second example, the consistency is much greater; the eccentric moment is clearly supported by the post, which has a higher bending resistance than the chord or the diagonal, particularly when the truss is hinged at the post.

Note that this not the case in the worked example in which the posts have their web perpendicular to the plane of the truss: the convergence of the three axes happens then without causing secondary moments.

### 5.1.3 Worked example: detailing a pinned joint

The Figure 5.3 represents horizontal displacements of the lower and upper nodes of the two support sections, for cases of ULS gravity load combinations and for cases of ULS uplift load combinations. We can observe that, when the structure is symmetric or symmetrically loaded, each load case produces equal global rotations in the two support sections.



**Figure 5.3 Rotations at truss supports**

In order for the global rotations at the supports to be free (assumption for truss with pinned connections to the column), the elongated holes introduced into the column on lower chord connection must allow a 35,6 mm movement towards the outside and 12,2 mm towards the inside. It is of course prudent to allow for a certain safety margin on the sizing of the elongated holes (say 50 mm), and to check after erection that, under self weight, the freedom of movement remains adequate in both directions.

## 5.2 Chord continuity

It is often necessary to deliver large span truss beams to site in several sections; it is therefore necessary to provide continuous chord joints between these sections. Generally, the preferred method is to make such connections on site by bolting rather than by welding.

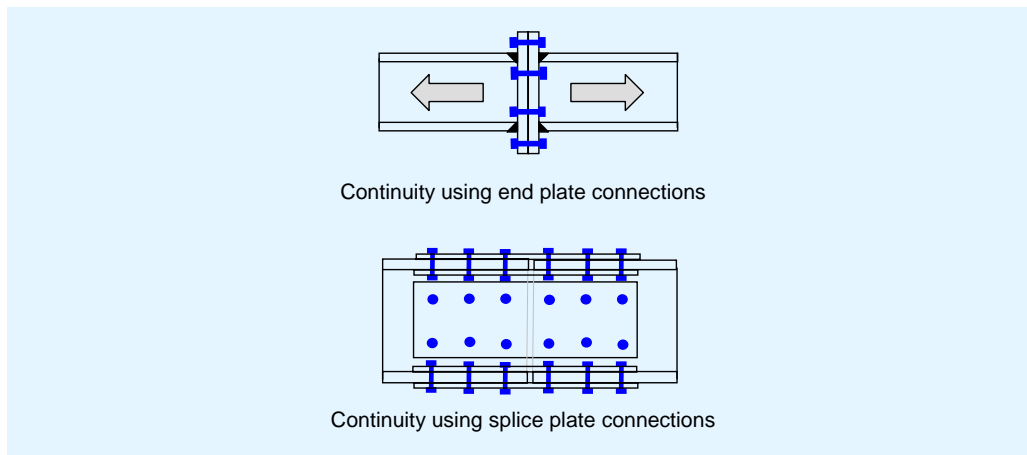
The design of these bolted connections depends on the type of chord section to be connected. However, we can distinguish between two types of such connections:

- Those in which the bolts are mainly loaded in tension : these use end plates
- Those in which bolts are loaded perpendicular to their shank: these use splice plates.

When the chords are made of a single profile/section in I or H, either of the connections can be used.

When the chords are made of two double angle or channel sections, splice connections are generally used.

When the chords are made of hollow sections end plate connections are generally used (use of hollow sections is outside the scope of this guide).



**Figure 5.4 Chord continuity**

The splice plate connection shown Figure 5.4 has double cover splice plates on the web and flanges (giving two interfaces for shear forces). If the force in the splice is low, single external spliced plates can be used, although double plates are normally used on the web, to preserve symmetry in the transmission of the axial force.

The resistance of the splice connections of truss chords must be verified under dominant load with secondary bending moment in the truss plane, according to EN 1993-1-8, by adapting the components method developed for beam-post connections. Software is freely available for this verification (see the SteelBizFrance.com website developed by CTICM). Verification of this type of connection, for the worked example, is given in Appendix A.

As well as verifying the resistance, it is essential to ensure the stiffness of the continuous chord connections. Generally, when the resistance of a beam-beam connection using end plates is selected, it can be considered as rigid.

Spliced plate connections are only effectively rigid when the slack is controlled (see Section 3.6 for evaluation example of the effect of slack in the bolted connections of the truss in the worked example). For splice connections, it is therefore recommended that one of the following options is selected:

- Use preloaded bolts with controlled tightening, allowing transmission of loads by friction (non-slip)
- Use fit bolts, preferably loaded on the shank in order to avoid slip under load by distortion of the thread of the connected pieces.

### 5.3 Connection of diagonals to chords

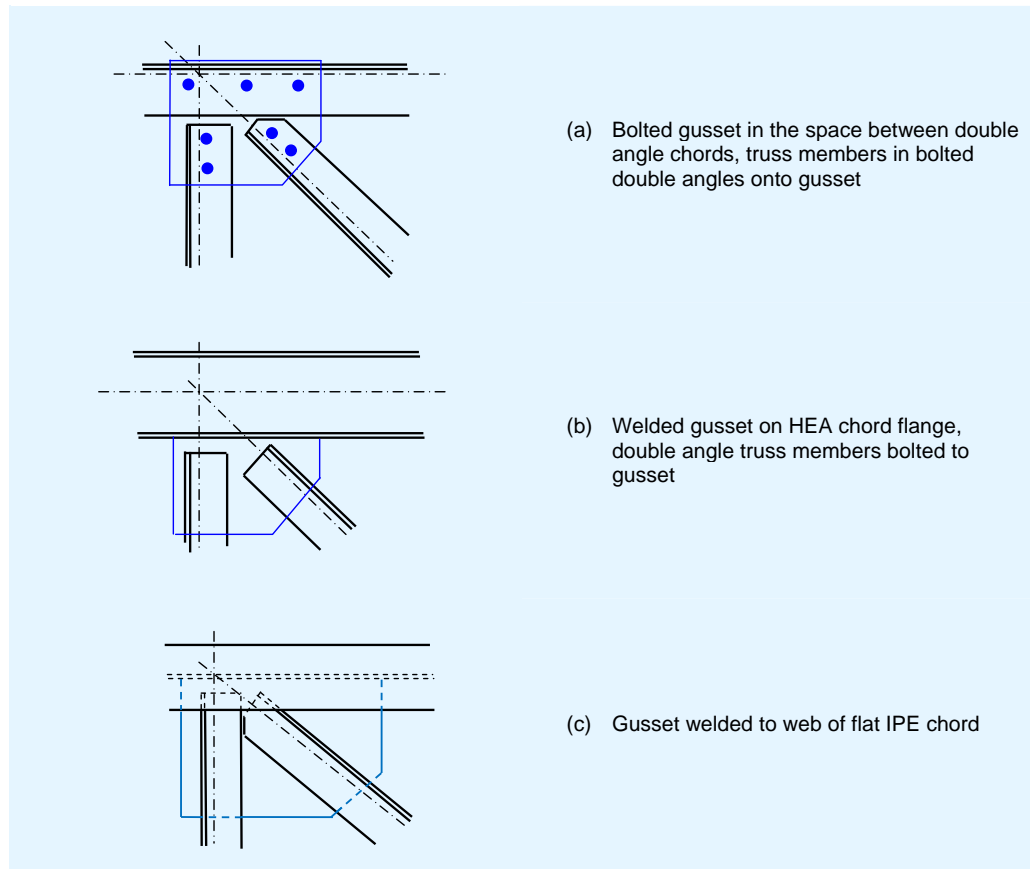
Connection of diagonals and posts to chords can be made in different ways, according to the type of sections to be connected.

When the chords are made of double members (two angles or two UPE sections), common practice is to insert gusset plates between the two



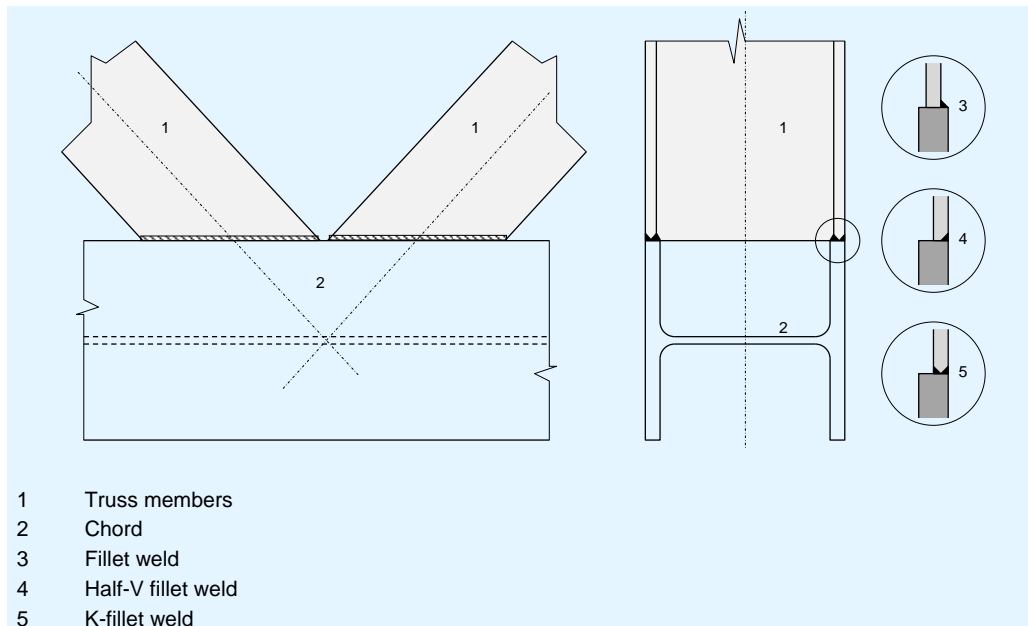
component members of the chord. The gussets are, therefore, either welded or bolted on the chords. The diagonals and posts are connected to the gussets, usually by bolting.

When the chords are made of IPE or HEA/HEB sections, the most common connection method is also to use a welded gusset plate on the chord. The gusset plate is attached to the flange when the section is upright (vertical web), and to the web when the section is flat (horizontal web).



**Figure 5.5 Truss connections on chord**

When the chord sections are flat, it is also common to use IPE or HEA truss members with the same depth as the chords and to connect them by double gussets, one on each flange. An alternative solution is to design a welded connection without gussets, as shown in Figure 5.6.



**Figure 5.6 Welded connection between truss members and chord**

When the chords are hollow sections (outside the scope of this guide), the connection using a gusset welded on the chord is also used. Direct welding of the diagonals and posts to the chords is also used; this requires profiling for connections to circular section chords.

In the gusset connections described above, verification of the resistance of the bolted or welded connection clearly defined in EN 1993-1-8. However, verification of the resistance of the gusset plate is not. Verification of a gusset plate connection for the worked example is given in Appendix B.

Special attention must be given to checking of the gussets, particularly those which have a large non stiffened part: many truss problems have been caused local buckling of the gusset plate. For example, in the connections in Figure 5.5(c), if the height of the flat chord web is insufficient for the angles making up the truss members to be connected near the web, the unstiffened part of the gusset and its stability must be examined carefully.

Although hollow section trusses are not the subject of the present guide, note that EN 1993-1-8 devotes a Section to the design of welded connections of hollow sections.

In the connections to the chords, slip must also be controlled (as indicated for continuous chords), in order to control displacements of the structural components, and, as a result, the distribution of forces if the structure is hyperstatic.

## **REFERENCES**

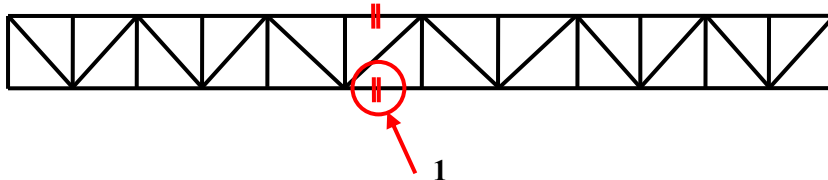
- 1 Single-Storey Steel Buildings. Part 7: Fire engineering.
- 2 EN 1993-1-8:2005 Eurocode 3: Design of steel structures. Part 1.8 Design of joints.
- 3 EN 1993-1-1: 2005, Eurocode 3: Design of steel structures. Part 1.1 General rules and rules for buildings.



**APPENDIX A**  
**Worked Example – Design of a continuous chord**  
**connection using splice plate connections**

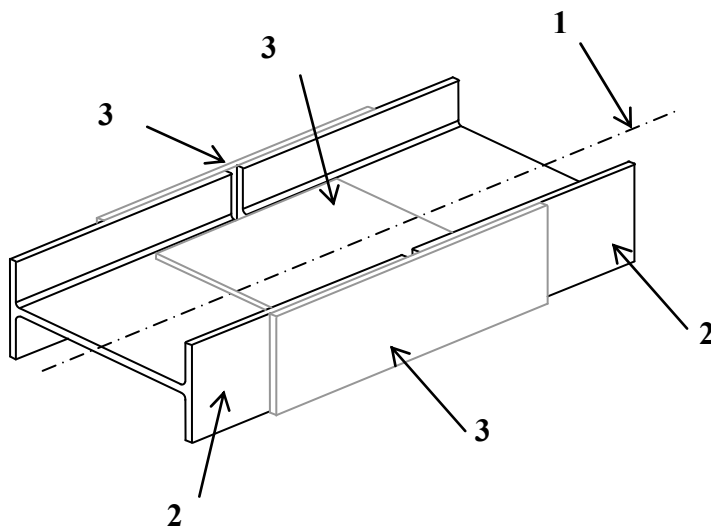
## 1. Splice joint using bolted cover plates

This calculation sheet refers to the splice plate connection located on the Figure A.1. This connection has double spliced plates on the web and single external spliced plate on the flanges (see Figure A.2).



- 1 Splice plate connection studied

**Figure A.1 Location of the splice plate connections**



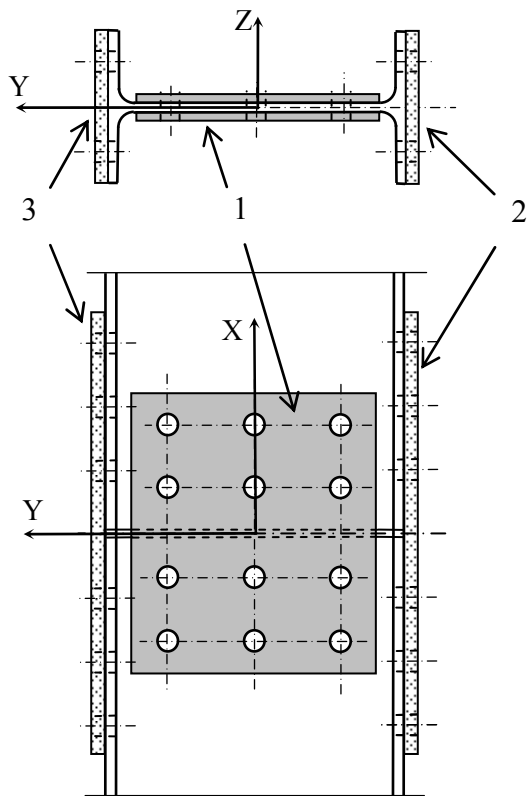
- 1 Longitudinal axis  
 2 Lower chords to assembly (IPE 330)  
 3 Splice plate connection

**Figure A.2 Chord continuity by splice plate connections**

The resistance of this connection must be verified under tension axial force with secondary moment in the plane of the truss.

Four bolted cover plates must be verified (See Figure A.3)

It is also essential to ensure the stiffness of the continuous chord connection. A slip resistant connection is required.



- 1 cover plates of web chord
- 2 cover plate of flange 1 (on the right-hand side)
- 3 cover plate of flange 2 (on the left-hand side)

**Figure A.3 Cover plates**

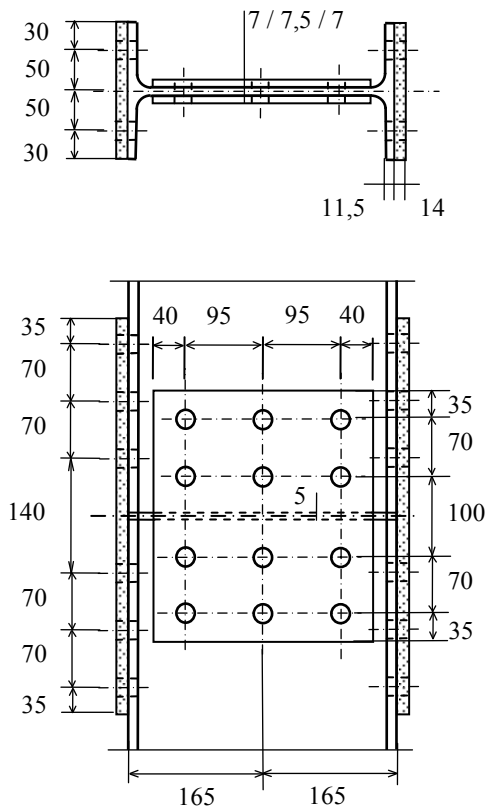
The global coordinates system is such as:

The XOZ plane is that of the truss plane

The XOY plane is that of the web chord

## 2. Basic data

The sizes of the cover-plates and the positioning of holes are shown on the Figure A.4.



**Figure A.4** Sizes (in mm) and positioning

#### Material data (except bolts)

The I-profile and the cover-plates are grade S355 to EN 10025-2.

Steel grade	S355
Yield strength	$f_y = 355 \text{ N/mm}^2$
Ultimate tensile strength	$f_u = 510 \text{ N/mm}^2$

#### I Beam data

Depth	$h = 330 \text{ mm}$
Flange width	$b = 160 \text{ mm}$
Web thickness	$t_w = 7,5 \text{ mm}$
Flange thickness	$t_f = 11,5 \text{ mm}$
Radius of root fillet	$r = 18 \text{ mm}$
Cross-section area	$A = 62,61 \text{ cm}^2$
Second moment of area	$I_y = 788,1 \text{ cm}^4$
Plastic modulus	$W_{pl,y} = 153,7 \text{ cm}^3$

EN 1993-1-1  
Table 3.1





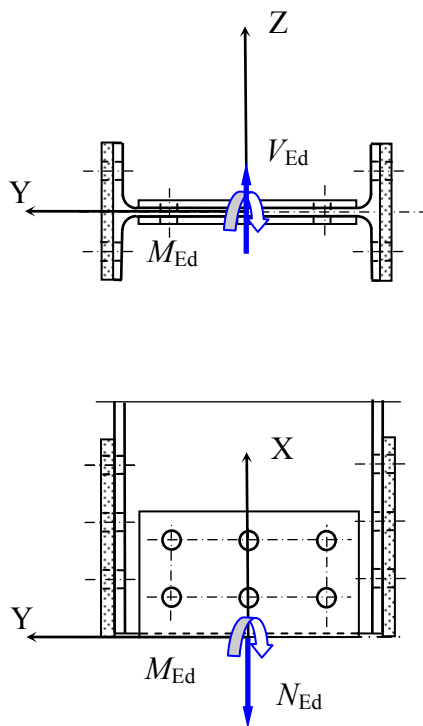


Figure A.5 Internal forces and moment

### 3. Classification of cross-section chord

For the classification of the cross-section, it's necessary to know the distribution of the normal stresses.

For the web we consider a uniform stress equal to:

$$\sigma_w = \frac{N_{Ed}}{A} = -250,34 \text{ N/mm}^2$$

For the flanges we have:

$$\sigma_i = \frac{N_{Ed}}{A} - \frac{M_{Ed}}{I_{yy} / v_i}$$

Where  $v_i$  is the position of the considered fibre.

And for the upper part ( $Z > 0$ ) of the flange:

$$v_1 = b_f / 2 \text{ and } v_2 = t_w / 2 + r$$

$$\sigma_1 = 180,91 \text{ N/mm}^2, \sigma_2 = 245,62 \text{ N/mm}^2$$

And for the inner part ( $Z < 0$ ) of the flange:

$$v_1 = -(b_f / 2) \text{ and } v_2 = -(t_w / 2 + r)$$

$$\sigma_1 = 319,78 \text{ N/mm}^2, \sigma_2 = 255,06 \text{ N/mm}^2$$

In view of these results, the cross-section being all over in tension is considered of class 1.

Title	APPENDIX A Worked Example: Design of a continuous chord connection using splice plate connections	6 of 24
<p><b>4. Global checking of the cross-section chord</b></p> <p><b>4.1. Effect of the shear force</b></p> <p>Determination of <math>\frac{V_{Ed}}{V_{pl,Rd}}</math></p> <p>With: <math>A_v = A - h_w t_w = 3959 \text{ mm}^2</math></p> $V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}} = 811,3 \text{ kN}$ <p>From where <math>\frac{V_{Ed}}{V_{pl,Rd}} = 0,002 &lt; 0,5</math></p> <p>So, no reduction due to the shear force needs to be taken into account.</p> <p><b>4.2. Combination M + N – Effect of the axial force</b></p> $N_{Ed} = 1567,4 > \frac{h_w t_w f_y}{\gamma_{M0}} = 817,4 \text{ kN}$ <p>Allowance has to be made for the effect of the axial force.</p> <p><b>4.3. Combination M + N – Consideration of fastener holes</b></p> <p><b>Axial force</b></p> <p>Under tension axial force, the fastener holes should be considered.</p> <p>Category C connection <math>\Rightarrow</math> the design tension resistance is:</p> $N_{t,Rd} = N_{net,Rd} = \frac{A_{net} f_y}{\gamma_{M0}}$ <p>For the net cross-section, we consider 7 holes for fastener (2 by flange and 3 for the web).</p> <p>The net area is: <math>A_{net} = 4707 \text{ mm}^2</math></p> <p>Therefore: <math>N_{net,Rd} = 1671 \text{ kN}</math></p> <p><b>Bending moment</b></p> <p>With <math>A_f = b t_f</math> and <math>A_{f,net} = A_f - 2 d_{0,f} t_f</math></p> <p>For each flange in tension, one checks:</p> $\frac{A_{f,net} 0,9 f_u}{\gamma_{M2}} = 473 < \frac{A_f f_y}{\gamma_{M0}} = 653,2 \text{ kN}$ <p>So, the holes for fasteners in the flange should be considered.</p>		
		EN 1993-1-1 6.2.10
		EN 1993-1-1 6.2.6(2)
		EN 1993-1-1 6.2.10 (2)
		EN 1993-1-1 6.2.9.1
		EN 1993-1-1 6.2.9.1 (5)
		EN 1993-1-1 6.2.3(4)
		EN 1993-1-1 6.2.5 (4)

Title	APPENDIX A Worked Example: Design of a continuous chord connection using splice plate connections	7 of 24
<p>With <math>A_{\text{net}} = A - 4d_{0,f}t_f - 3d_{0,w}t_w</math></p> <p>For the full tension area, one checks:</p> $\frac{A_{\text{net}} 0,9 f_u}{\gamma_{M2}} = 1728,4 < \frac{A f_y}{\gamma_{M0}} = 2222,7 \text{ kN}$ <p>So, the holes for fasteners in the web should be considered.</p> <p><b>Design resistance for bending</b></p> <p>With for a IPE 330: <math>W_{pl,y} = 153,7 \text{ cm}^3</math></p> <p><math>d_z = 50 \text{ mm}</math> = distance from centre of holes of flange to z-z axis</p> $W_{pl,y,\text{holes}} = 4(d_{0,f}t_f d_z) = 55,2 \text{ cm}^3$ <p>The design plastic moment resistance of the net section is:</p> $M_{pl,Rd} = \frac{(W_{pl,y} - W_{pl,y,\text{holes}})f_y}{\gamma_{M0}} = 34,967 \text{ kNm}$ <p><b>4.4. Combination M + N – Verification</b></p> <p>The following criterion should be verified:</p> $M_{Ed} \leq M_{N,Rd}$ <p>With: <math>n = \frac{N_{Ed}}{N_{\text{net},Rd}} = 0,938</math></p> $a = \min \left\{ (A - 2b t_f) / A; 0,5 \right\} = 0,412$ <p>We obtain :</p> $M_{N,Rd} = M_{pl,Rd} \left[ 1 - \left( \frac{n-a}{1-a} \right)^2 \right] = 6,99 \text{ kNm}$ $M_{Ed} = 1,71 < M_{N,Rd} = 6,99 \text{ kNm} \quad \text{OK}$ <p><b>5. Distribution of the internal forces</b></p> <p>Note that the web is in the horizontal plane.</p> <p><b>5.1. Axial force</b></p> <p>The axial force is distributed between the web and the flanges. This distribution is based on the ratio of the gross cross-section of the web and the flanges. The fillets are appointed to the flange.</p> <p>So, with:</p> $A_w = (h - 2t_f)t_w = 2302,5 \text{ mm}^2$ $A_f = (A - A_w) / 2 = 3958,5 \text{ mm}^2 \text{ (per flange)}$ <p>Then:</p> $N_{N,w} = N_{Ed} A_w / A = 576,4 \text{ kN}$ $N_{N,f} = (N_{Ed} - N_{N,w}) / 2 = 495,5 \text{ kN}$		<p>EN 1993-1-1 6.2.5 (5)</p> <p>EN 1993-1-1 6.2.5(2)</p> <p>EN 1993-1-1 6.2.9.1(1)</p> <p>EN 1993-1-1 6.2.9.1(3)</p> <p>EN 1993-1-8 2.5</p>

## 5.2. Shear force

The shear force is fully transferred by the flanges.

So:  $V_{V,f} = V_{Ed} / 2$  (per flange)

## 5.3. Bending moment

The bending moment about the weak axis is fully transferred by the flanges:

$M_{M,f} = 0,855$  kNm for each flanges

## 6. Internal forces in each connected parts

### 6.1. Connection of the webs

The cover plate of webs (and its bolts) is only subjected to an axial force:

$N_{N,w} = 576,4$  kN

### 6.2. Connection of the flanges

Each of cover plates of flanges (and its bolts) is subjected to:

- An axial force  $N_{N,f} = 495,49$  kN,
- A shear force  $V_{V,f} = 0,85$  kN
- A bending moment  $M_{M,f} = 0,855$  kNm

The moment due to the eccentricity of the shear force against the centroid of the joint (see Figure A.6):

$M_{V,f} = V_{V,f} e_v$

With:  $e_v = 140$  mm  $M_{V,f} = 0,119$  kNm

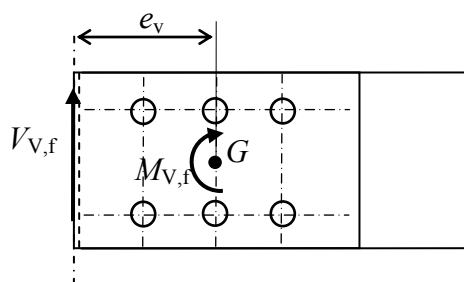


Figure A.6 Moment due to the eccentricity of the shear force

### 6.3. Summary of the internal forces and moments

In the web:  $N_w = 576,42 \text{ kN}$

In one flange:  $N_f = 495,49 \text{ kN}$

$V_f = 0,85 \text{ kN}$

$M_f = 0,97 \text{ kN}$

## 7. Verification of the web connection

The connection of the webs is a double lap joint.

The web component will be verified and by symmetry only one plate component.

### 7.1. Design details

It is assumed that the structure is not exposed to the weather or other corrosive influences.

The design details are verified for the web component and for the plate component in the tables below

**Table A.1 Connection of the webs – Web component – Design details**

Distance or spacing	Min. value	Design value	Max. value
$e_1$	24	47,5	
$e_2$	24	1)	
$p_1$	44	70	105
$p_2$	48	95	105

<sup>1)</sup> Not applicable because of the proximity of the flange

**Table A.2 Connection of the webs – Plate component – Design details**

Distance or spacing	Min. value	Design value	Max. value
$e_1$	24	35	
$e_2$	24	40	
$p_1$	44	70	98
$p_2$	48	95	98

### 7.2. Design shear force $F_{V,Ed}$ for each bolt

$F_{V,Ed,w} = \frac{N_w}{6} = 96,07 \text{ kN}$  for the component web

$F_{V,Ed,p} = \frac{N_w / 2}{6} = 48,03 \text{ kN}$  for each component plate

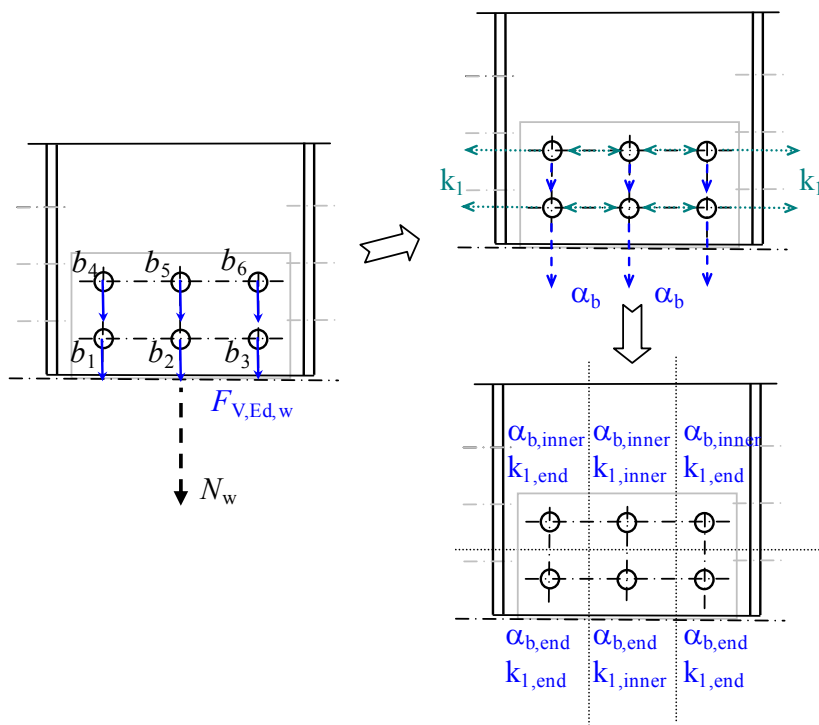
### 7.3. Design slip resistance $F_{S,Rd}$

By considering: Bolts in normal holes  $\Rightarrow k_s = 1,0$

EN 1993-1-8  
Table 3.3

EN 1993-1-8  
3.12 (3)

Title	APPENDIX A Worked Example: Design of a continuous chord connection using splice plate connections	10 of 24
	<p>Class friction surfaces = Class A <math>\Rightarrow \mu = 0,5</math></p> <p>And with: <math>A_{s,w} = 192 \text{ mm}^2</math> tensile stress area of the bolt</p> <p><math>F_{p,c} = 0,7 f_{ub} A_{s,w} = 134,4 \text{ kN}</math> pretension force</p> <p><math>n</math> number of the friction surfaces</p> <p><math>n_w = 2</math> relatively to the web component</p> <p><math>n_p = 1</math> relatively to the plate component</p> <p>Then: <math>F_{s,Rd,w} = \frac{k_s n_w \mu}{\gamma_{M3}} F_{p,c} = 107,52 \text{ kN}</math></p> <p><math>F_{s,Rd,p} = \frac{k_s n_p \mu}{\gamma_{M3}} F_{p,c} = 53,76 \text{ kN}</math></p> <p><b>7.4. Design bearing resistance <math>F_{b,Rd}</math> for each bolt</b></p> <p>Table 3.4 of EN 1993-1-8 gives the expressions of the design bearing resistance. In these expressions, the coefficients <math>\alpha_b</math> and <math>k_1</math> depend on the orientation of the loading, the position compared with the ends of the component and also the position of the other bolts.</p> <p>The general expression for the design bearing resistance is:</p> $F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}}$ <p>According to Table 3.4 of the Eurocode 1993-1-8, the coefficients <math>\alpha_b</math> and <math>k_1</math> are determined from:</p> <p>For end bolts <math>\alpha_{b,end} = \min \left\{ \frac{e_1}{3d_0}; \frac{f_{ub}}{f_u}; 1, 0 \right\}</math></p> $k_{1,end} = \min \left\{ 1, 4 \frac{p_2}{d_0} - 1, 7; 2, 8 \frac{e_2}{d_0} - 1, 7; 2, 5 \right\}$ <p>For inner bolts <math>\alpha_{b,inner} = \min \left\{ \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1, 0 \right\}</math></p> $k_{1,inner} = \min \left\{ 1, 4 \frac{p_2}{d_0} - 1, 7; 2, 5 \right\}$ <p><b>Web component</b></p> <p>Figure A.7 shows how it is processed for the determination of the coefficients <math>\alpha_b</math> and <math>k_1</math>.</p>	<p>EN 1993-1-8 3.9.1 (1)</p> <p>EN 1993-1-8 Table 3.4</p> <p>EN 1993-1-8 Table 3.4</p>



**Figure A.7 Connection of the webs – Web component – Determination of type of bolts**

The determination of coefficients  $k_1$  is carried out perpendicularly to the direction of load transfer. But two directions are conceivable for this perpendicular and it is difficult for some bolts ( $b_1$ ,  $b_4$ ,  $b_3$ , and  $b_6$ ) to determine if they are end or inner bolts.

In these cases we consider the minimum value of  $k_{1,inner}$  and  $k_{1,end}$ . And by noticing that  $\min\{k_{1,inner}; k_{1,end}\} = k_{1,end}$ , these bolts are considered as end bolts.

In addition, for the web component, it is reminded that the edge distance  $e_2$  is not applicable because of the proximity of the flange. So, the expressions of  $k_{1,inner}$  and  $k_{1,end}$  are identical.

As the design shear force is identical for each bolt and furthermore:

$$k_{1,inner} = k_{1,end} = 2,50$$

So only one row of bolts is considered, for example the bolts  $b_1$  and  $b_4$ .

Then, for the bolt  $b_1$ :

$$\alpha_{b,b1} = \alpha_{b,b1,end} = 0,79$$

$$F_{b,b1,Rd,w} = 109,01\text{kN}$$

And for the bolt  $b_4$ :

$$\alpha_{b,b4} = \alpha_{b,b4,inner} = 0,92$$

$$F_{b,b4,Rd,w} = 126,23\text{kN}$$



Therefore, in the end for the web component,

$$F_{b,Rd,w} = 109,01\text{kN}$$

### Plate component

Compared with the web component, for the plate it can be noticed that the bolts  $b_1, b_2, b_3$  become inner bolts and the bolts  $b_4, b_5, b_6$  become end bolts (see Figure A.8).

Then, for the bolt  $b_1$ :

$$\alpha_{b,b1} = \alpha_{b,b1,inner} = 0,92$$

$$F_{b,b1,Rd,p} = 117,81\text{kN}$$

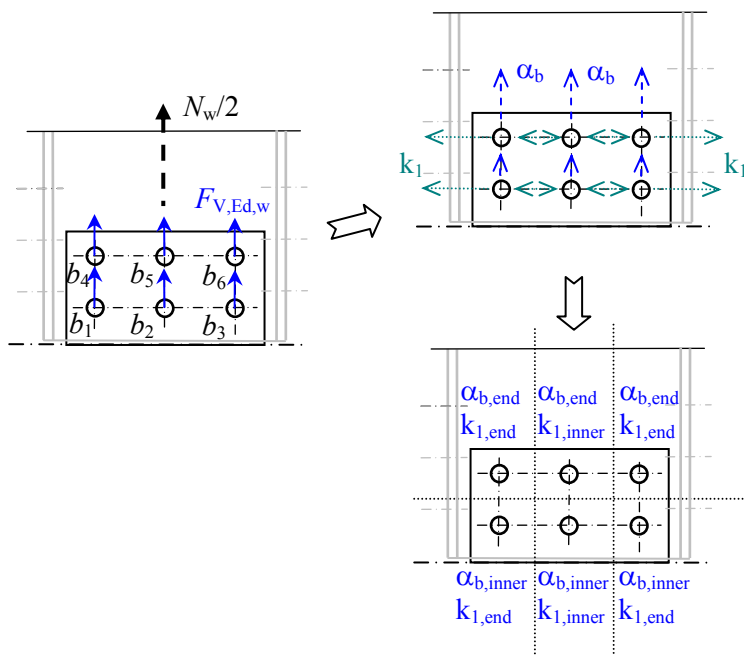
And for the bolt  $b_4$ :

$$\alpha_{b,b4} = \alpha_{b,b4,end} = 0,58$$

$$F_{b,b4,Rd,p} = 74,97\text{kN}$$

In the end, for the plate component, it should retained:

$$F_{b,Rd,p} = 74,97\text{kN}$$



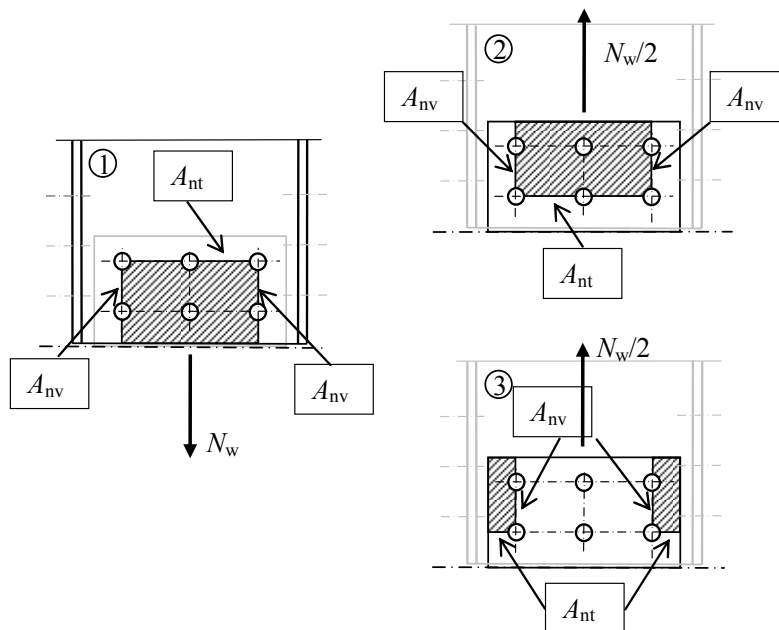
**Figure A.8** Connection of the webs – Plate component – Determination of type of bolts

Title	APPENDIX A Worked Example: Design of a continuous chord connection using splice plate connections	13 of 24
<p><b>7.5. Checking bolts</b></p> <p><b>7.5.1. With regard to the web component</b></p> <p><b>Individual checking</b></p> <p>Design bearing resistance <math>F_{V,Ed,w} = 96,07 &lt; F_{b,Rd,w} = 109,01 \text{ kN}</math></p> <p>Design slip resistance <math>F_{V,Ed,w} = 96,07 &lt; F_{s,Rd,w} = 107,52 \text{ kN}</math></p> <p><b>Group of fasteners</b></p> <p>The shear resistance per shear plane <math>F_{v,Rd}</math> is taken as:</p> $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$ <p>By considering that the shear plane does not pass through the threaded portion of the bolt in normal holes:</p> <p><math>\Rightarrow \alpha_v = 0,6</math></p> <p><math>\Rightarrow A = 254,47 \text{ mm}^2</math> (gross cross-section of the bolt)</p> <p>Then: <math>F_{v,Rd} = 122,15 \text{ kN}</math></p> <p>Since <math>F_{v,Rd} &gt; F_{b,Rd,w}</math> for only three bolts as a result the design of our group of fasteners:</p> $F_{gr,b,Rd,w} = n_{bi} \times \min \{ F_{b,bi,Rd,w} \} = 6 \times 109,01 = 654,06 \text{ kN}$ <p>Then: <math> N_w  = 576,42 &lt; F_{gr,b,Rd,w} = 654,06 \text{ kN}</math></p> <p><b>7.5.2. With regard to the plate component</b></p> <p><b>Individual checking</b></p> <p>Design bearing resistance <math>F_{V,Ed,p} = 48,03 &lt; F_{b,Rd,p} = 74,97 \text{ kN}</math></p> <p>Design slip resistance <math>F_{V,Ed,p} = 48,03 &lt; F_{s,Rd,p} = 53,76 \text{ kN}</math></p> <p><b>Group of fasteners</b></p> <p>The shear resistance per shear plane <b>Error! Objects cannot be created from editing field codes.</b> is equal to:</p> $F_{v,Rd} = 122,15 \text{ kN}$ <p>Since <b>Error! Objects cannot be created from editing field codes.</b> for each of the bolts as a result the design of our group of fasteners:</p> $F_{gr,b,h,Rd} = \sum_1^{n_{bi}} F_{b,bi,h,Rd} = 3 \times 117,81 + 3 \times 74,97 = 578,34 \text{ kN}$ <p>Then: <math> N_w / 2  = 228,21 &lt; F_{gr,b,Rd} = 578,34 \text{ kN}</math></p>		

Title	APPENDIX A Worked Example: Design of a continuous chord connection using splice plate connections	14 of 24
<p><b>7.6. Design of net cross-section</b></p> <p>For a connection in tension, the design plastic resistance of the net cross-section at bolt holes should be verified:</p> $\sum_1^{n_b} F_{V,Ed} \leq N_{net,Rd}$ <p>where <math>n_b</math> is the number of bolts located in the considered net cross-section.</p> <p><b>7.6.1. Web component</b></p> <p>The net cross-section is taken as <math>A_{w,net} = A_w - 3d_{0,w}t_w = 1852,5 \text{ mm}^2</math></p> <p>The design resistance is: <math>N_{w,net,Rd} = \frac{A_{w,net} f_y}{\gamma_{M0}} = 657,64 \text{ kN}</math></p> <p>Then: <math>N_{w,net,Rd} = 657,64 &gt; \sum_1^3 F_{V,Ed,w} = 3 \times 96,07 = 288,21 \text{ kN}</math></p> <p><b>7.6.2. Plate component</b></p> <p>The net cross-section is taken as <math>A_{p,net} = A_p - 3d_{0,w}t_p = 1470 \text{ mm}^2</math></p> <p>The design resistance is: <math>N_{p,net,Rd} = \frac{A_{p,net} f_y}{\gamma_{M0}} = 521,85 \text{ kN}</math></p> <p>Then: <math>N_{w,net,Rd} = 521,85 &gt; \sum_1^3 F_{V,Ed,w} = 3 \times 48,03 = 144,10 \text{ kN}</math></p> <p><b>7.7. Design for block tearing</b></p> <p>The Figure A.9 shows the block tearing for the web and for the plate.</p> <p><b>7.7.1. Web component</b></p> <p>The bolt group is subjected to concentric loading.</p> <p>And with: <math>A_{nt} = (2p_2 - 2d_0)t_w = 1125 \text{ mm}^2</math></p> $A_{nv} = 2(e_1 + p_1 - 1,5d_0)t_w = 1312,5 \text{ mm}^2$ <p>Then: <math>V_{eff,1,Rd} = 728,01 \text{ kN}</math></p> $V_{eff,1,Rd} = 728,01 > N_w = 576,42 \text{ kN}$ <p><b>7.7.2. Plate component</b></p> <p>Two block tearing are defined. For the both, the shear area is the same, so the case giving the minimum area subjected in tension is considered. The bolt group is subjected to concentric loading.</p> <p>And with: <math>A_{nt} = (2e_2 - d_0)t_p = 420 \text{ mm}^2</math></p> $A_{nv} = 2(e_1 + p_1 - 1,5d_0)t_p = 1050 \text{ mm}^2$		
		EN 1993-1-8 Table 3.2
		EN 1993-1-8 3.10.2 (1)
		EN 1993-1-8 3.10.2 (2)
		EN 1993-1-8 3.10.2 (2)

$$V_{\text{eff},1,\text{Rd}} = 386,57 \text{ kN}$$

So:  $V_{\text{eff},1,\text{Rd}} = 386,57 > N_w / 2 = 288,21 \text{ kN}$



- 1 Block tearing for web component (concentric loading)
- 2 First block tearing for plate component (concentric loading)
- 3 Second block tearing for plate component (concentric loading)

**Figure A.9** Block tearing for connection of the webs

## 8. Checking of connection of the flanges

The connection of the flanges is a single lap joint.

The flange component and the plate component will be verified.

In general rule in the presence of a combination of loads, we obtain for each bolt a design shear force not parallel to the edge of the components. In this case, the Eurocode states that the bearing resistance can be verified separately for the bolt load components parallel and normal to the end of components.

$$F_{V,\text{bi},\text{h},\text{Ed}} \leq F_{\text{b},\text{bi},\text{h},\text{Rd}}$$

$$F_{V,\text{bi},\text{v},\text{Ed}} \leq F_{\text{b},\text{bi},\text{v},\text{Rd}}$$

In the ECCS publication P126 (European recommendations for the Design of Simple Joints in Steel Structures – 2009), an additional check is proposed, based on an interaction expression:

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

The load components will be performed in a basis  $\{h, v\}$  located at the centre of gravity of the joint and oriented with the principal directions of the flange (See Figure A.10).

EN 1993-1-8  
Table 3.4<sup>3)</sup>

## 8.1. Design details

It is assumed that the truss is not exposed to weather or other corrosive influences.

The design details should be verified in both directions of loading. By taking into consideration the limits specified in Table 3.3 of EN 1993-1-8, the following requirements have to be fulfilled:

$$\min\{e_1; e_2\} \geq 1,2d_0$$

$$\min\{p_1; p_2\} \geq 2,2d_0$$

$$\max\{p_1; p_2\} \leq \min\{14t; 200 \text{ mm}\}$$

The tables below check the design details for each component.

**Table A.3 Connection of the flanges – Plate component – Design details**

Distance or spacing	Min. value	Design value	Max. value
$\min\{e_1; e_2\}$	28,8	30	
$\min\{p_1; p_2\}$	52,8	70	
$\max\{p_1; p_2\}$		100	161

**Table A.4 Connection of the flanges – Plate component – Design details**

Distance or spacing	Min. value	Design value	Max. value
$\min\{e_1; e_2\}$	28,8	30	
$\min\{p_1; p_2\}$	52,8	70	
$\max\{p_1; p_2\}$		100	196

## 8.2. Design shear force $F_{V,Ed}$ for each bolt

### With regard to the flange component

The components of the design shear force are calculated in the basis  $\{h, v\}$  (see Figure A.10). The group of bolts is subjected to an axial force  $N_f$ , a shear force  $V_f$  and a bending moment  $M_f$  (see 6.2)

The axial force  $N_f$  generates a horizontal shear force:

$$F_{N,bi,h} = \frac{N_f}{6} = -82,58 \text{ kN} \quad \text{for each bolt}$$

The shear force  $V_f$  generates a vertical shear force:

$$F_{V,bi,v} = \frac{V_f}{6} = 0,14 \text{ kN} \quad \text{for each bolt}$$

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Table 3.3

The moment  $M_f$  is divided out the bolts according to the distance  $r_i$  between the centre of bolts  $b_i$  and the centre of gravity of the group of bolts

$$F_{M,bi} = \frac{M_f r_i}{\sum_1^6 r_i^2}$$

This shear force  $F_{M,bi}$  resolved in the basis  $\{h, v\}$  gives:

$$F_{M,bi,h} = \frac{M_f v_i}{\sum_1^6 r_i^2} \text{ a horizontal component for the bolt } b_i.$$

$$F_{M,bi,v'} = \frac{M_f h_i}{\sum_1^6 r_i^2} \text{ a vertical component for the bolt } b_i.$$

With  $h_i$  and  $v_i$  coordinates of centre of bolt  $b_i$ .

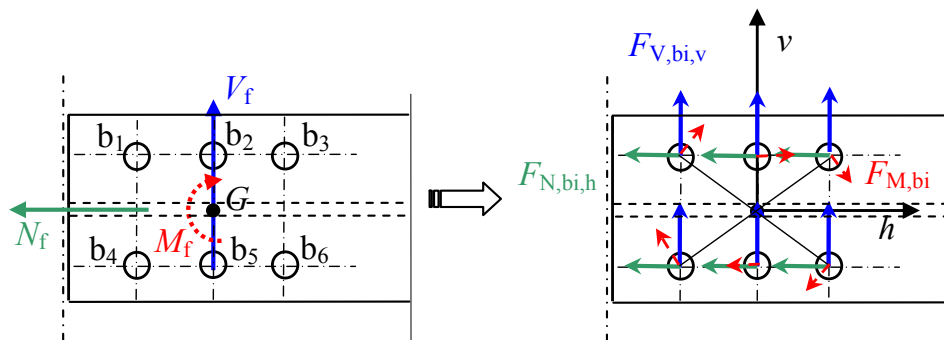
In the end, for each bolt:

$$F_{V,bi,h,Ed} = F_{N,bi,h} + F_{M,bi,h} \quad \text{Horizontal design shear force}$$

$$F_{V,bi,v,Ed} = F_{V,bi,v} + F_{M,bi,v} \quad \text{Vertical design shear force}$$

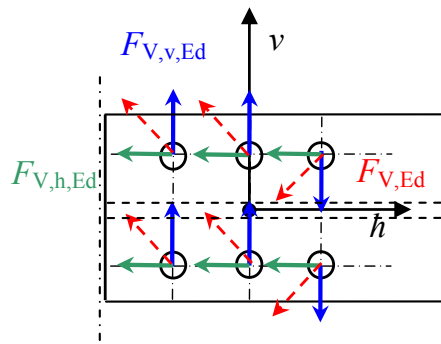
$$F_{V,bi,Ed} = \sqrt{F_{V,bi,h,Ed}^2 + F_{V,bi,v,Ed}^2} \quad \text{Resulting design shear force}$$

The Figure A.10 shows the distribution of the internal forces.



**Figure A.10 Distribution of the internal forces for the flange component.**

The Figure A.11 shows the directions of the resulting force and its components.



**Figure A.11** Directions of the design shear force

Table A.5 sums up the determination of the design shear forces.

The vertical component of the load can be neglected. We will confine to the horizontal direction for the design bearing resistance checking.

In addition, if we had not considered the shear force  $V_{Ed}$  and the moment  $M_{Ed}$ , the unique horizontal design shear force would be:

$$F_{V,bi,h,Ed} = F_{N,bi,h} = -82,58 \text{ kN}$$

That is a difference of  $\pm 2\%$

So the value of 84,02 kN can be retained (= maximum value obtained for  $F_{V,bi,Ed}$ ) for the design shear force:  $F_{V,Ed} = 84,02 \text{ kN}$ .

**Table A.5 Connection of the flanges – Flange component – Design shear forces in kN in the basis  $\{h, v\}$ .**

Bolt	$b_1$	$b_2$	$b_3$	$b_4$	$b_5$	$b_6$
$h_i$	-70	0	70	-70	0	70
$v_i$	50	50	50	-50	-50	-50
$r_i$	86,02	50	86,02	86,02	50	76,02
$F_{M,bi}$	2,42	1,41	2,42	2,42	1,41	2,42
$F_{M,bi,h}$	1,41	1,41	1,41	-1,41	-1,41	-1,41
$F_{M,bi,v}$	1,97	0	-1,97	1,97	0	-1,97
$F_{N,bi,h}$	-82,58	-82,58	-82,58	-82,58	-82,58	-82,58
$F_{V,bi,v}$	0,14	0,14	0,14	0,14	0,14	0,14
$F_{V,bi,Ed}$	81,20	81,17	81,20	84,02	83,99	84,01
$F_{V,bi,h,Ed}$	-81,17	-81,17	-81,77	-83,99	-83,99	-83,99
$F_{V,bi,v,Ed}$	2,11	0,14	-1,83	2,11	0,14	-1,83

#### With regard to the plate component

The connection of the flanges is a single lap joint so the design shear forces for each bolt with regard to the plate component are directly deduced from the previous results.

The value of 84,02 kN can be retained.

### 8.3. Design slip resistance $F_{s,Rd}$

By considering: Bolts in normal holes  $\Rightarrow k_s = 1,0$

Class friction surfaces = Class A  $\Rightarrow \mu = 0,5$

And with:  $A_{s,f} = 303 \text{ mm}^2$  tensile stress area of the bolt

$F_{p,c} = 0,7 f_{ub} A_{s,f} = 212,1 \text{ kN}$  pretension force

$n$  number of the friction surfaces

Single lap joint  $\Rightarrow n = 1$  for each component

Then:  $F_{s,Rd,f} = F_{s,Rd,p} = \frac{k_s n \mu}{\gamma_{M3}} F_{p,c} = 84,54 \text{ kN}$

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3.9.1

### 8.4. Design bearing resistance $F_{b,Rd}$ for each bolt

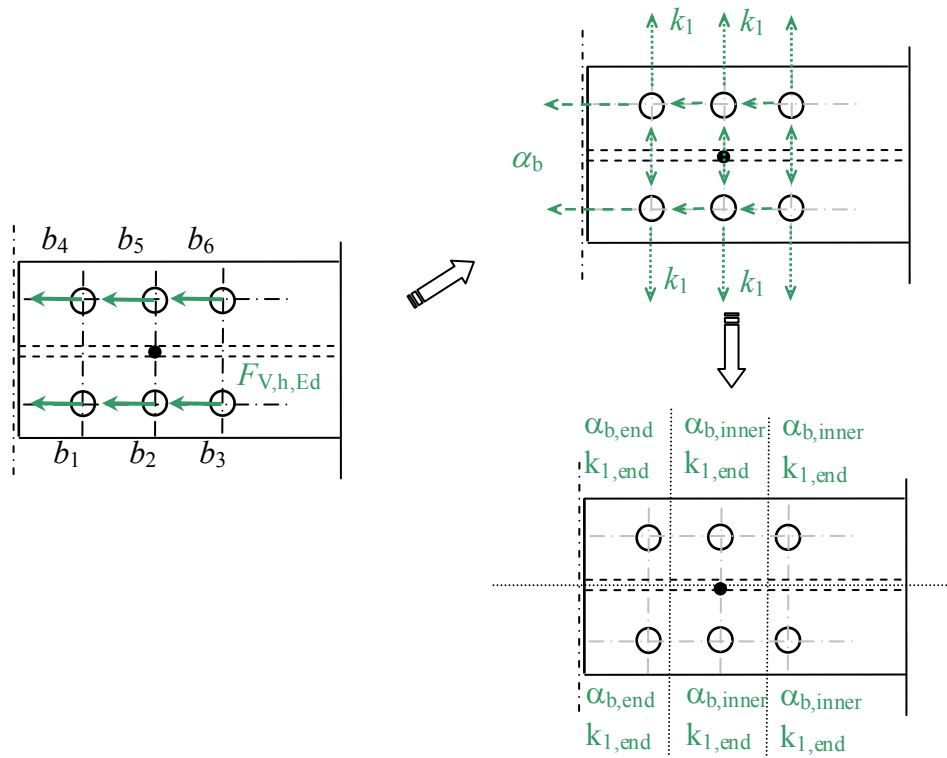
We confine to the horizontal direction for the determination of the design bearing resistance (see 8.2).

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Table 3.4



### Flange component

Figure A.12 shows for each bolt how the factors  $\alpha_b$  and  $k_1$  are determined.



**Figure A.12 Connection of the flanges – Flange component – Determination of type of bolts**

For all the bolts:  $k_{1,end} = 1,80$ .

For the bolt  $b_1$  and  $b_4$ :  $\alpha_{b,end} = 0,94$

$$F_{b,Rd,f} = 174,19 \text{ kN}$$

For the other bolts:  $\alpha_{b,inner} = 0,72$

$$F_{b,Rd,f} = 134,19 \text{ kN}$$

In the end for the flange component, the minimum value is retained:

$$F_{b,Rd,f} = 134,19 \text{ kN}$$

### Plate component

For all the bolts,  $k_{1,end} = 1,80$ .

For the bolt  $b_3$  and  $b_6$ :  $\alpha_{b,end} = 0,49$

$$F_{b,Rd,p} = 90,32 \text{ kN}$$

Title	APPENDIX A Worked Example: Design of a continuous chord connection using splice plate connections	21 of 24
<p>For the other bolts: <math>\alpha_{b,inner} = 0,72</math></p> $F_{b,Rd,p} = 134,19 \text{ kN}$ <p>In the end for the plate component, the minimum value is retained:</p> $F_{b,Rd,p} = 90,32 \text{ kN}$ <p><b>8.5. Verification of the bolts</b></p> <p><b>8.5.1. With regard to the flange component</b></p> <p><b>Individual checking</b></p> <p>Design bearing resistance <math>F_{V,Ed,w} = 84,02 &lt; F_{b,Rd,w} = 134,19 \text{ kN}</math></p> <p>Design slip resistance <math>F_{V,Ed,w} = 84,02 &lt; F_{s,Rd,w} = 84,54 \text{ kN}</math></p> <p><b>Group of fasteners</b></p> <p>The design shear resistance per shear plane <math>F_{v,Rd}</math> is taken as:</p> $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$ <p>By considering that the shear plane does not pass through the threaded portion of the bolt in normal holes:</p> $\Rightarrow \alpha_v = 0,6$ $\Rightarrow A = 380,13 \text{ mm}^2 \text{ (gross cross-section of the bolt)}$ <p>Then: <math>F_{v,Rd} = 182,46 \text{ kN}</math></p> <p>Since <math>F_{v,Rd} &gt; F_{b,Rd,w}</math> for all the bolts, the design resistance of our group of fasteners is equal to:</p> $F_{gr,b,Rd,w} = \sum_1^{n_{bi}} F_{b,bi,Rd,f} = 2 \times 174,19 + 4 \times 134,19 = 885,15 \text{ kN}$ <p>Then: <math> N_f  = 495,49 &lt; F_{gr,b,Rd,f} = 885,15 \text{ kN}</math></p> <p><b>8.5.2. With regard to the plate component</b></p> <p><b>Individual checking</b></p> <p>Design bearing resistance: <math>F_{V,Ed,p} = 84,02 &lt; F_{b,Rd,p} = 90,32 \text{ kN}</math></p> <p>Design slip resistance: <math>F_{V,Ed,p} = 84,02 &lt; F_{s,Rd,p} = 84,54 \text{ kN}</math></p> <p><b>Group of fasteners</b></p> <p>The shear resistance per shear plane <math>F_{v,Rd}</math> is equal to:</p> $F_{v,Rd} = 182,46 \text{ kN}$		

EN 1993-1-8  
Table 3.2

EN 1993-1-8  
Table 3.4

EN 1993-1-8  
3.7

EN 1993-1-8  
Table 3.4

Title	APPENDIX A Worked Example: Design of a continuous chord connection using splice plate connections	22 of 24	
<p>Since <math>F_{v,Rd} &gt; F_{b,Rd,w}</math> for all the bolts, the design of our group of fasteners is equal to:</p> $F_{gr,b,Rd,p} = \sum_1^{n_{bi}} F_{b,bi,Rd,p} = 2 \times 90,32 + 4 \times 134,19 = 717,40 \text{ kN}$ <p>Then: <math> N_p  =  N_f  = 495,49 &lt; F_{gr,b,Rd,p} = 717,40 \text{ kN}</math></p> <h3>8.6. Design of net cross-section</h3> <p>For a connection in tension, the design plastic resistance of the net cross-section at bolt holes should be verified:</p> $\sum_1^{n_b} F_{V,Ed} \leq N_{net,Rd}$ <p>Where <math>n_b</math> is the number of bolts located in the considered net cross-section.</p> <h4>8.6.1. Flange component</h4> <p>The net section area is: <math>A_{f,net} = A_f - 2 d_{0,f} t_f = 1427,25 \text{ mm}^2</math></p> <p>And: <math>N_{f,net,Rd} = \frac{A_{f,net} f_y}{\gamma_{M0}} = 506,67 \text{ kN}</math></p> <p>Then: <math>N_{f,net,Rd} = 506,67 &gt; \sum_1^2 F_{V,Ed,f} = 2 \times 84,02 = 168,04 \text{ kN}</math></p> <h4>8.6.2. Plate component</h4> <p>The net cross-section is taken as <math>A_{p,net} = A_p - 2 d_{0,w} t_p = 1568 \text{ mm}^2</math></p> <p>From where <math>N_{p,net,Rd} = \frac{A_{p,net} f_y}{\gamma_{M0}} = 556,64 \text{ kN}</math></p> <p>Then: <math>N_{p,net,Rd} = 556,64 &gt; \sum_1^2 F_{V,Ed,p} = 2 \times 84,02 = 168,04 \text{ kN}</math></p> <p><b>Note:</b> The global cross-section of the beam has been verified accounting for the holes for fastener and the combination of the internal forces (see 4).</p> <p>The net cross-section of the plate component should also be verified under this combination of internal forces.</p> <p>Assuming a uniform distribution of the load in the section, it is proposed that:</p> $\sigma_{max} = \sqrt{\sigma^2 + 3\tau^2} \leq f_y$ <p>Where: <math>\sigma = \frac{N_p}{A_{p,net}} \pm \frac{M_p}{I_{p,net} / v}</math> and <math>\tau = \frac{V_p}{A_{p,net}}</math></p>			<p>EN 1993-1-8 3.7</p> <p>EN 1993-1-8 Table 3.2</p>

Assuming a uniform distribution of the shear stresses, this leads to a conservative situation.

With  $A_{p,net} = 1568 \text{ mm}^2$

$$I_{p,net} = I_{p,gross} - I_{p,holes} = 477,87 - 171,23 = 306,64 \text{ cm}^4$$

Then:  $\sigma = 316 \text{ N/mm}^2$  and  $\tau = 25,31 \text{ N/mm}^2$

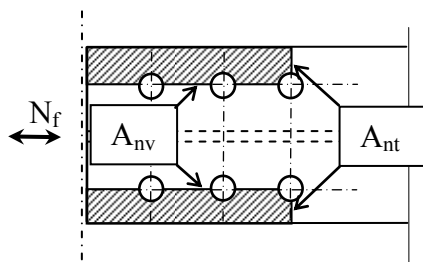
Finally:  $\sigma_{max} = 341,31 \text{ N/mm}^2 \leq f_y = 355 \text{ N/mm}^2$

## 8.7. Design for block tearing

### 8.7.1. Web component

The bolt group is subjected to a concentric loading  $N_f$  and an eccentric loading  $V_f$  but considering the presence of the web we only consider the case with a concentric loading.

The Figure A.13 shows the block tearing for the flange component



**Figure A.13 Connection of the flanges – Block tearing for flange component**

With:  $A_{nt} = 2(e_2 - 0,5d_0)t_f = 414 \text{ mm}^2$

$$A_{nv} = 2(e_1 + 2p_1 - 2,5d_0)t_f = 3392,5 \text{ mm}^2$$

Then:  $V_{eff,1,Rd} = 826,24 \text{ kN}$

And:  $V_{eff,1,Rd} = 826,24 > N_w = 495,49 \text{ kN}$

### 8.7.2. Plate component

The bolt group is subjected to a concentric loading  $N_p$  and an eccentric loading  $V_p$ .

The Figure A.14 shows the block tearing for the plate component

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3.10.2

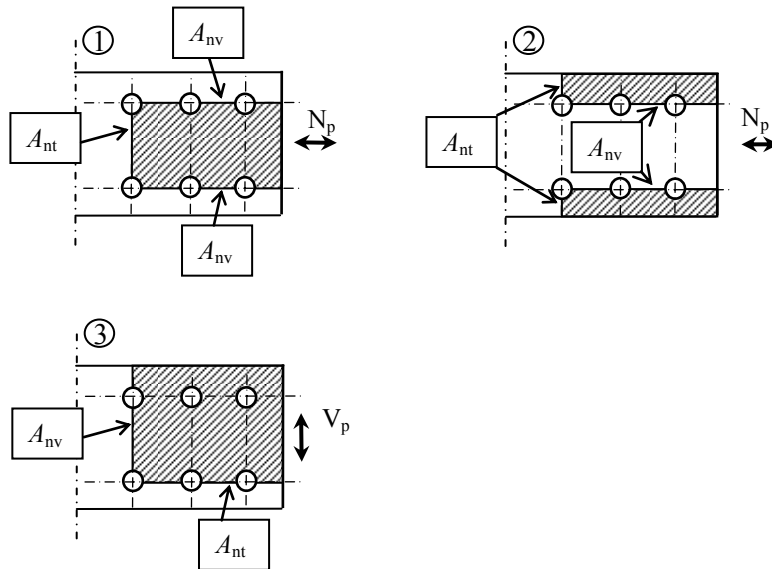
For the cases with a concentric loading, only the case giving the minimum area in tension is considered:

$$\text{With : } A_{nt} = \min \{ (p_2 - d_0); 2(e_2 - 0,5d_0) \} t_p = 504 \text{ mm}^2$$

$$A_{nv} = 2(e_1 + 2p_1 - 2,5d_0) t_p = 3220 \text{ mm}^2$$

$$\text{Then: } V_{\text{eff},1,\text{Rd}} = 865,60 \text{ kN}$$

$$\text{And: } V_{\text{eff},1,\text{Rd}} = 865,60 > N_f = 495,49 \text{ kN}$$



- 1 First block tearing with concentric loading
- 2 Second block tearing with concentric loading
- 3 Block tearing with eccentric loading

**Figure A.14 Connection of the flanges - Block tearing for plate component**

For the case with an eccentric loading, with:

$$A_{nt} = (e_1 + 2p_1 - 2,5d_0) t_p = 1610 \text{ mm}^2$$

$$A_{nv} = (e_2 + p_2 - 1,5d_0) t_p = 1316 \text{ mm}^2$$

$$\text{Then: } V_{\text{eff},2,\text{Rd}} = 598,17 \text{ kN}$$

$$\text{And: } V_{\text{eff},2,\text{Rd}} = 598,17 > V_p = 0,85 \text{ kN}$$

So we have just verified successively the bolt group according to the two loadings. An additional requirement based on an interactive expression should be fulfilled:

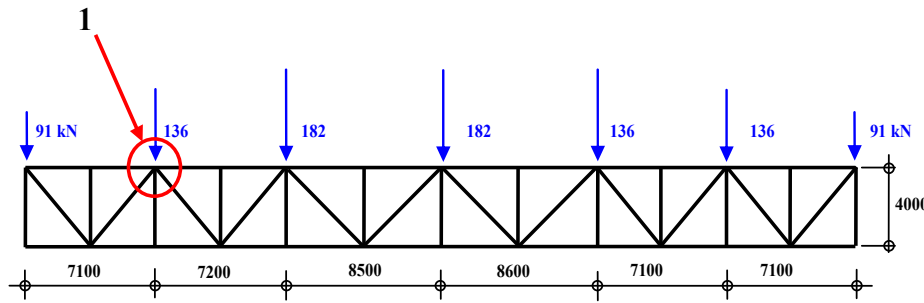
$$\frac{N_p}{\min \{ V_{\text{eff},1,\text{Rd},\text{block 1}}; V_{\text{eff},1,\text{Rd},\text{block 2}} \}} + \frac{V_p}{V_{\text{eff},2,\text{Rd},\text{block 3}}} \leq 1,0$$

$$\text{Then: } \frac{495,49}{865,60} + \frac{0,85}{598,17} = 0,57 \leq 1,0 \text{ OK}$$



**APPENDIX B**  
**Worked example – Design of a truss node with gusset**

The truss includes several types of joints: splice joints by bolted cover plates, T joints and KT joints. This Appendix gives the detailed design of a KT joint located on the upper chord, as shown in Figure B.1.

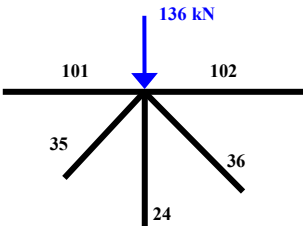


1 KT joint

**Figure B.1 Location of the KT joint**

The values of the internal forces in the truss members (see Table B.1) result from a gravity load case. This load case corresponds to a ULS combination of actions, determined according to EN 1990.

**Table B.1 KT joint – Internal forces in the truss members**

Member	N (kN)	V (kN)	M (kNm)	
	Diagonal 35	-609,4	-1,27	0
	Diagonal 24	406,9	1,03	0
	Post 36	2,6	0	0
	Chord 101	-413,8	1,25	-0,46
	Chord 102	-1084	1,26	-0,09

## 1. General presentation of KT joint

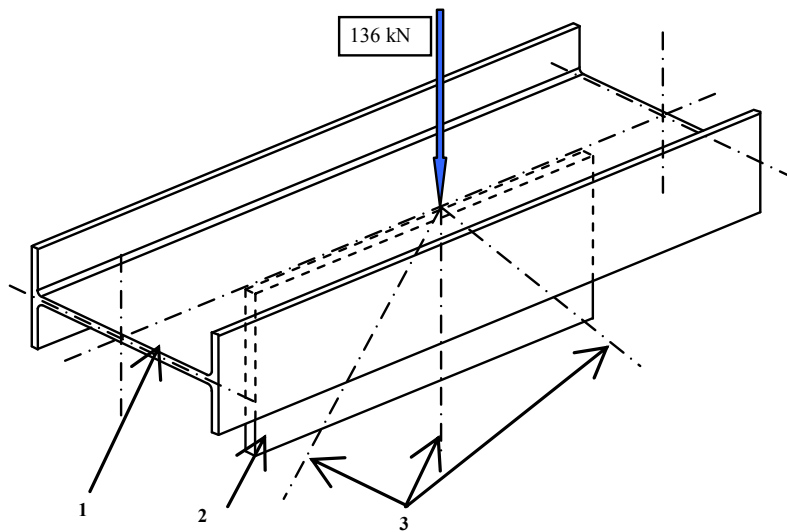
The KT joint studied consists of the following connections: the gusset to web chord welded connection and the angles to gusset bolted connection (see Figure B.2 and Figure B.3). Both connections should be verified according to the rules from EN 1993-1-1 and EN 1993-1-8.

The gusset to web chord welded connection is a plate welded perpendicular to the web of the chord by two fillets welds (See Figure B.7).

The angles to gusset bolted connection is composed of two back-to-back double-angle diagonal members (See Figure B.4) and a single angle post member (See Figure B.5).

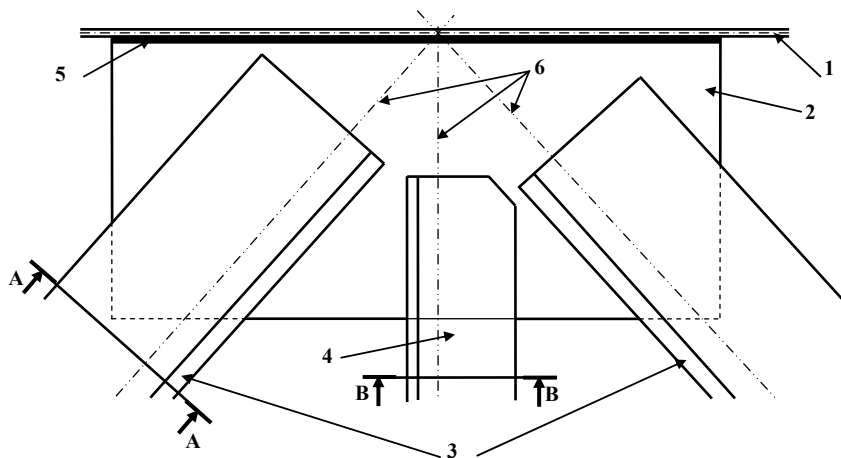
There are three shear connections to be designed as Category C.





- 1 Chord (IPE 330)
- 2 Gusset plate
- 3 Axes of the web members

**Figure B.2** General presentation of KT joint



- 1 Web of the chord (IPE 330)
- 2 Gusset plate 580×260×15
- 3 Angles L150×150×15
- 4 Angle L100×100×10
- 5 Fillet weld
- 6 Axes of the web members

**Figure B.3** KT joint

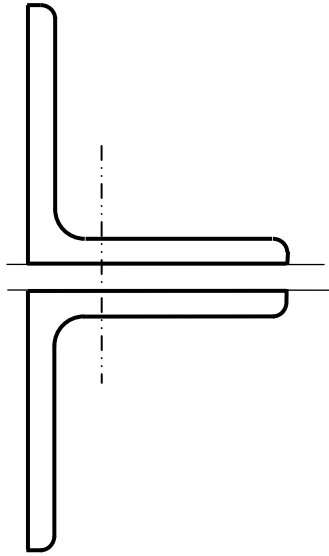


Figure B.4 KT Joint – Section AA

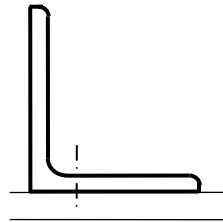


Figure B.5 KT Joint- Section BB

## 2. Gusset plate to web chord welded connection

This connection is a welded plate perpendicular to the web of the chord, see Figure B.6. The two fillet welds are identical. The design of the gusset plate and its weld to the chord takes into account the axial forces in all three angle members connected to it.

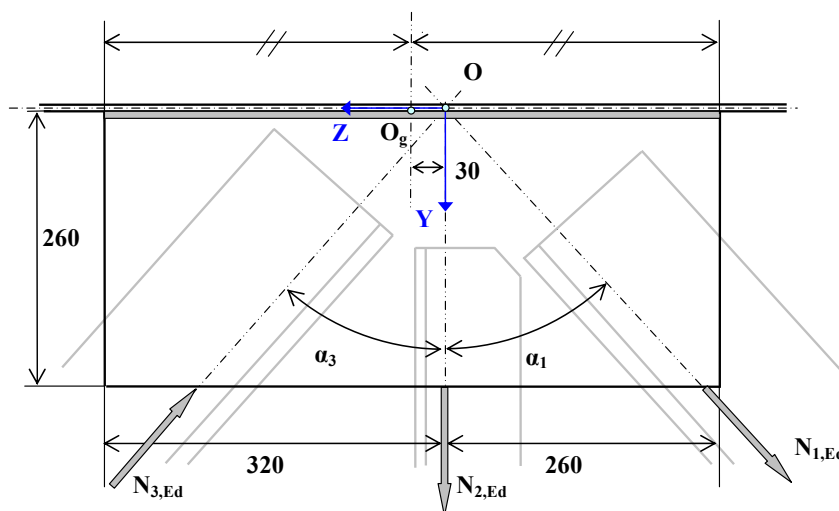
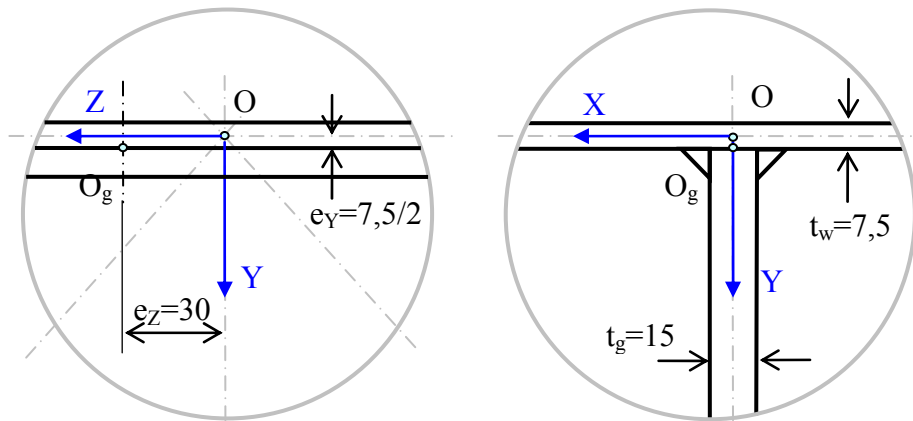


Figure B.6 Gusset plate to web chord welded connection

The longitudinal axes of all three angle members intersect on the chord axis at the point O in the web.

The gusset plane is not positioned symmetrically about the normal OY to the web plane (see Figure B.6 and Figure B.7). The moment resulting from the eccentricity  $e_z$  should be taken into account.

The moment resulting from the eccentricity  $e_y = t_w/2$  can be neglected.



**Figure B.7 Gusset plate to web chord – Details**

The basic assumption is that gusset plate transfers axial forces acting in its plane and in the direction of the member axes.

## 2.1. Data

### Global coordinates system (see Figure B.6 and Figure B.7)

The YOZ Plane is that of the gusset plate

The XOZ Plane is that of the chord web

### Geometric data

Gusset plate thickness	$t_g = 15 \text{ mm}$
Web thickness	$t_w = 7,5 \text{ mm}$
Angle between gusset and web	$\alpha_a = 90^\circ$
Number of fillet welds	$n_a = 2$
Effective throat thickness	$a = \text{Value to be defined}$
Length of welds	$L_w = 560 \text{ mm}$

### Material data

Steel grade:	S355
Yield strength:	$f_y = 355 \text{ N/mm}^2$
Ultimate tensile strength:	$f_u = 510 \text{ N/mm}^2$

**Note:** The specified yield strength and ultimate tensile strength of the filler metal are required to be at least equivalent to those specified for the parent material.

EN 1993-1-1  
Table 3.1

EN 1993-1-8  
4.2(2)

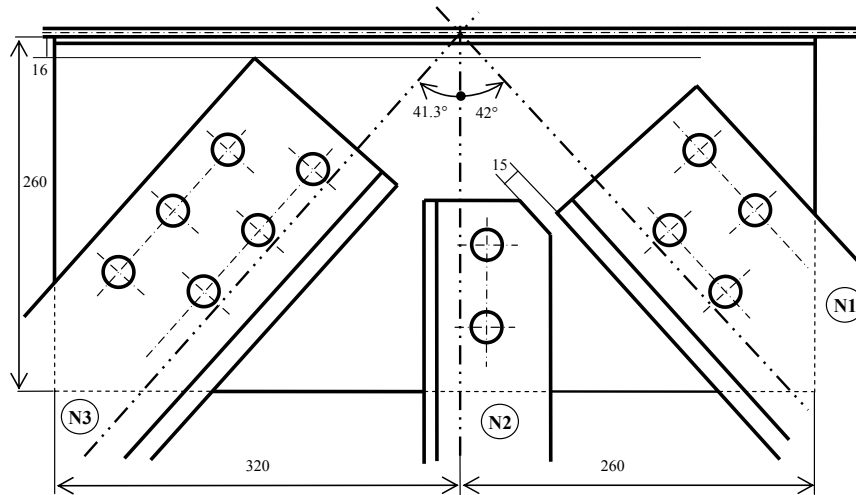
Title	Appendix B Worked Example: Design of a truss node with gusset	5 of 44
<p><b>Partial Factor</b></p> <p>Resistance of weld: <math>\gamma_{M2} = 1,25</math> (recommended value)</p> <p><b>Internal forces in the truss members (see Figure B.6)</b></p> <p>All axial forces are applied in the gusset plate XOZ plane:</p> <p>Tension axial force at an angle to normal OY of <math>\alpha_1 = 42^\circ</math>:</p> $N_{1,Ed} = 406,9 \text{ kN}$ <p>Tension axial force on the normal OY so <math>\alpha_2 = 0^\circ</math></p> $N_{2,Ed} = 2,6 \text{ kN}$ <p>Compression axial force at an angle to normal OY of <math>\alpha_3 = -41,3^\circ</math></p> $N_{3,Ed} = -609,4 \text{ kN}$ <p><b>2.2. Stresses in the gusset cross-section in front of welds</b></p> <p>The approach is based on a linear-elastic analysis that leads to a safe estimation of the resistance of the welded joint.</p> <p><b>2.2.1. Design forces in the gusset plate at the chord web face</b></p> <p>The effects of the small eccentricity <math>e_Y</math> from the chord axis will be neglected. The gusset plate section is verified for the following forces:</p> <p><math>N_{g,Ed}</math> Axial force at an eccentricity of <math>e_Z = 30 \text{ mm}</math> to the centreline of the gusset plate</p> <p><math>V_{g,Ed}</math> shear force</p> <p>With: <math display="block">N_{g,Ed} = \sum_{i=1}^3 N_i \cos(\alpha_i)</math></p> $V_{g,Ed} = \sum_{i=1}^3 N_i \sin(\alpha_i)$ <p>and <math>M_{g,Ed}</math>, the moment resulting from the eccentricity, <math>M_{g,Ed} = e_Z N_{g,Ed}</math></p> <p>Then: <math>N_{g,Ed} = -152,83 \text{ kN}</math></p> $V_{g,Ed} = 674,47 \text{ kN}$ $M_{g,Ed} = 4,585 \text{ kNm}$ <p><b>Note:</b> the high axial force component <math>N_{g,Ed}</math> is due to the local point load at the joint and the self weight of the truss.</p> <p><b>2.2.2. Normal stress</b></p> <p>Assuming a uniform distribution of the load in the section, the normal stress is:</p> $\sigma_{g,max} = \frac{N_{g,Ed}}{A_g} + \frac{M_{g,Ed}}{I_g / v}$		<p>EN 1993-1-8 Table 2.1 NOTE</p> <p>EN 1993-1-8 2.4(2)</p>

Title	Appendix B Worked Example: Design of a truss node with gusset	6 of 44
<p>Where: <math>A_g</math> is the cross-section area  <math>I_g</math> is the second moment of cross-section  <math>v</math> is the position of the end fibre</p> <p>With: <math>A_g = t_g L_w = 15 \times 580 = 8700 \text{ mm}^2</math>  <math>I_g = \frac{t_g L_w^3}{12} = 243,89.10^6 \text{ mm}^4</math>  <math>v = 290 \text{ mm}</math></p> <p>Then: <math>\sigma_{g,\max} = -23,02 \text{ N/mm}^2</math></p> <p><b>2.2.3. Shear stress</b></p> <p>The shear mean stress is:</p> $\tau_g = \frac{V_{g,Ed}}{A_g}$ <p>Then: <math>\tau_g = 77,53 \text{ N/mm}^2</math></p> <p>One usually checks the combination of axial and shear stresses in the gusset plate section using the Von Mises criterion.</p> <p><b>2.3. Design resistance of the fillet weld</b></p> <p>The design resistance of a fillet weld should be determined using either the directional method or the simplified method.</p> <p>The directional method is based on the comparison between the design tensile strength and the applied stress in the most severely loaded part of the weld throat. The applied stress, being determined from a Von Mises formulation, accounts for the influence on the weld strength of the inclination of the resultant force per unit length to the weld axis and plane.</p> <p>The simplified method is based on the design shear strength of the weld to which is compared directly to an applied weld throat shear stress obtained by dividing the resultant force per unit of length by the weld throat size. The simplified method is always safe compared to the directional method.</p> <p>Here, the directional method is applied.</p> <p><b>2.3.1. Directional method</b></p> <p><b>Note:</b> a uniform distribution of stress is assumed in the throat section of the weld.</p> <p>With: <math>\sigma_{\perp}</math> the normal stress to the throat plane  <math>\tau_{\perp}</math> the shear stress (in the plane of throat) perpendicular to the axis of the weld  <math>\tau_{//}</math> the shear stress (in the plane of throat) parallel to the axis of the weld</p>		
		<p>EN 1993-1-8 4.5.3.1(1)</p> <p>EN 1993-1-8 4.5.3.2</p> <p>EN 1993-1-8 4.5.3.2(4)</p>

Title	Appendix B Worked Example: Design of a truss node with gusset	7 of 44
<p><b>Note:</b> the normal stress <math>\sigma_{//}</math> in the weld needs not to be considered.</p> <p>On the throat section of the weld, the force per unit length are:</p> $a \sigma_{\perp} = \frac{\sigma_{g,\max} e_g}{n_a} \sin(\alpha_a / 2) = -122,08 \text{ N/mm.mm}$ $a \tau_{\perp} = \frac{\sigma_{g,\max} e_g}{n_a} \cos(\alpha_a / 2) = -122,08 \text{ N/mm.mm}$ $a \tau_{//} = \frac{\tau_g e_g}{n_a} = 581,44 \text{ N/mm.mm}$ <p>The design resistance of the fillet weld will be sufficient if the following conditions are both fulfilled:</p> $\sigma_w = [\sigma_{\perp}^2 + 3 (\tau_{\perp}^2 + \tau_{//}^2)]^{0,5} \leq f_u / (\beta_w \gamma_{M2})$ $\sigma_{\perp} \leq 0,9 f_u / \gamma_{M2}$ <p>Where: <math>\beta_w</math> is the correlation factor for fillet weld</p> $\beta_w = 0,8$ <p>These conditions can be rewritten in the following forms:</p> $(a \sigma_w) / a \leq f_u / (\beta_w \gamma_{M2})$ $(a \tau_{\perp}) / a \leq 0,9 f_u / \gamma_{M2}$ <p>From these conditions, a minimum value for the effective throat thickness is derived.</p> $a_{1,\min} = a \sigma_w / [f_u / (\beta_w \gamma_{M2})] = 2,03 \text{ mm}$ $a_{2,\min} = a \sigma_{\perp} / (0,9 f_u / \gamma_{M2}) = 0,33 \text{ mm}$ $a_{\min} = \max(a_{1,\min} ; a_{2,\min}) = 2,03 \text{ mm}$ <p>The following requirements must be satisfied:</p> $a \geq 3 \text{ mm}$ $l_{\text{eff}} \geq \max(30 \text{ mm} ; 6 a) \text{ with } l_{\text{eff}} = L_w - 2 a$ <p>An effective throat thickness of 4 mm is then sufficient.</p>		<p>EN 1993-1-8 4.5.3.2(5)</p> <p>EN 1993-1-8 4.5.3.2(6)</p> <p>EN 1993-1-8 Table 4.1</p> <p>EN 1993-1-8 4.5.2(2) 4.5.2(1)</p>

### 3. Angles to gusset bolted connection

Three shear connections are designed as Category C. These connections are shown in Figure B.8.



**Figure B.8 Angles to gusset bolted connections**

This connection is composed of two back-to-back double-angle diagonal members (N1 and N3) and a single angle post member (N2).

The internal forces in the truss members are:

$$N_{1,Ed} = 406,9 \text{ kN} \quad \text{tension axial force}$$

$$N_{2,Ed} = 2,6 \text{ kN} \quad \text{tension axial force}$$

$$N_{3,Ed} = -609,4 \text{ kN} \quad \text{compression axial force}$$

#### 3.1. Basic Data

##### Material data (except bolts)

Steel grade S355

Yield strength  $f_y = 355 \text{ N/mm}^2$

Ultimate tensile strength  $f_u = 510 \text{ N/mm}^2$

EN 1993-1-1  
Table 3.1

##### Gusset plate

Thickness  $t_g = 15 \text{ mm}$

Length  $L_g = 580 \text{ mm}$

Width  $H_g = 260 \text{ mm}$

##### Angle members

N1 two equal-leg angles L150×150×15

N2 one equal-leg angle L100×100×10

N3 two equal-leg angles L150×150×15

**Bolted connections data**

Category of bolted connections	Category C
Bolt Class	Class 10.9
Yield strength	$f_{yb} = 900 \text{ N/mm}^2$
Ultimate tensile strength	$f_{ub} = 1000 \text{ N/mm}^2$
Nominal bolt diameter	$d = 24 \text{ mm}$
Hole diameter	$d_0 = 26 \text{ mm}$

EN 1993-1-8  
Table 3.1**Partial Factors (Recommended values)**

Structural steel	$\gamma_{M0} = 1,00$
Structural steel	$\gamma_{M1} = 1,00$
Structural steel	$\gamma_{M2} = 1,25$
Bolts	$\gamma_{M2} = 1,25$
Bolts	$\gamma_{M3} = 1,25$

EN 1993-1-1  
6.1 NOTE 2BEN 1993-1-8  
2.2 NOTE**3.2. Global checking of gross cross-sections of the gusset plate**

The gross cross-sections of the gusset plates to check are located on the Figure B.9.

**Note:** The gross cross-sections of the angles are verified afterward.

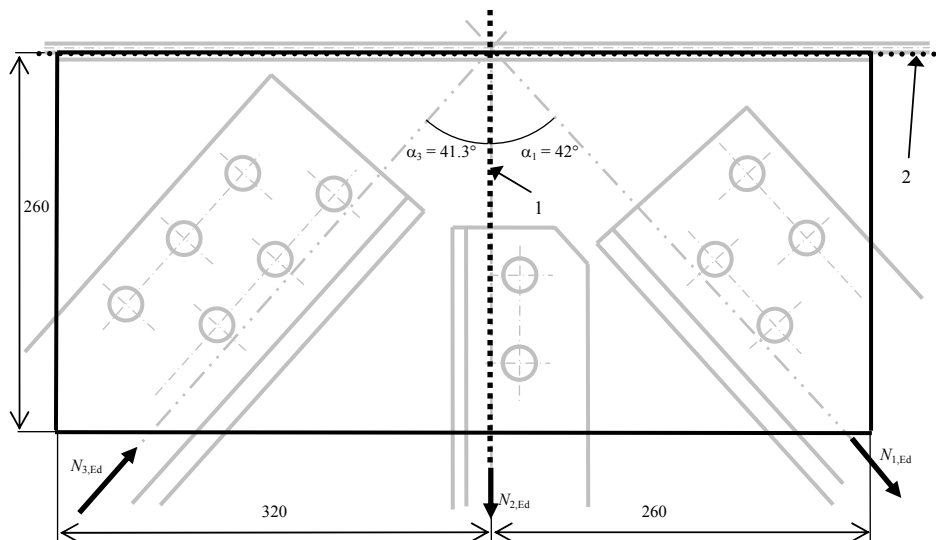


Figure B.9 Location of the gross cross-sections of the gusset plate

**Checking of gross cross-section 1**

With  $A_{g1}$  cross-sectional area 1  $A_{g1} = H_g t_g = 3900 \text{ mm}^2$



**Shear resistance**

$$V_{g1,Ed} = \max(N_{1,Ed} \cos \alpha_1; N_{2,Ed} \cos \alpha_2) = 457,82 \text{ kN}$$

$$V_{g1,pl,Rd} = A_{g1} f_y / (\gamma_{M0} \sqrt{3}) = 799,34 \text{ kN}$$

$$V_{g1,Ed} < V_{g1,pl,Rd} \Rightarrow \text{OK}$$

**Axial force resistance**

$$N_{g1,Ed} = \sum_{i=1}^3 N_{i,Ed} \sin(\alpha_i) = 674,47 \text{ kN}$$

$$N_{g1,pl,Rd} = A_{g1} f_y / \gamma_{M0} = 1384,50 \text{ kN}$$

$$N_{g1,Ed} < N_{g1,pl,Rd} \Rightarrow \text{OK}$$

**Checking of gross cross-section 2**

With  $A_{g2}$  cross-sectional area 2  $A_{g2} = L_g t_g = 8700 \text{ mm}^2$

**Shear resistance**

$$V_{g2,Ed} = \sum_{i=1}^3 N_{i,Ed} \sin(\alpha_i) = 674,47 \text{ kN}$$

$$V_{g2,pl,Rd} = A_{g2} f_y / (\gamma_{M0} \sqrt{3}) = 1783,15 \text{ kN}$$

$$V_{g2,Ed} < V_{g2,pl,Rd} \Rightarrow \text{OK}$$

**Axial force resistance**

$$N_{g2,Ed} = \sum_{i=1}^3 N_{i,Ed} \cos(\alpha_i) = 152,83 \text{ kN}$$

$$N_{g2,pl,Rd} = A_{g2} f_y / \gamma_{M0} = 3088,5 \text{ kN}$$

$$N_{g2,Ed} < N_{g2,pl,Rd} \Rightarrow \text{OK}$$

**3.3. Connection N3 – Back-to-back double-angle diagonal member N3 to gusset bolted connection**

The shear connection in compression is designed as Category C.

The sizes of the components and the positioning of the holes are shown on the Figure B.10 and Figure B.11.

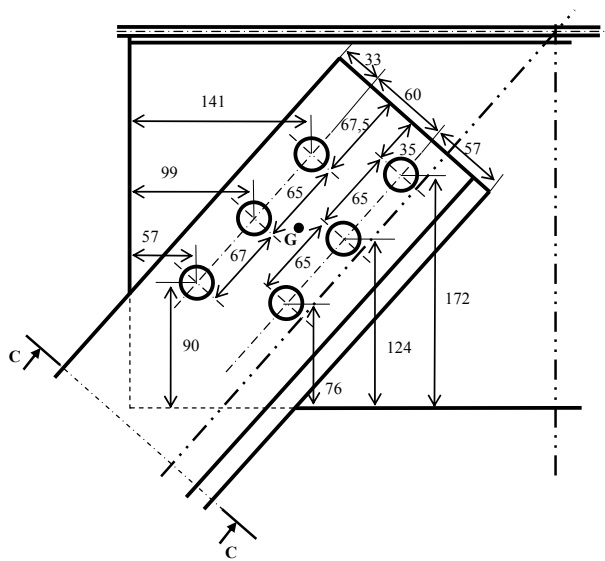
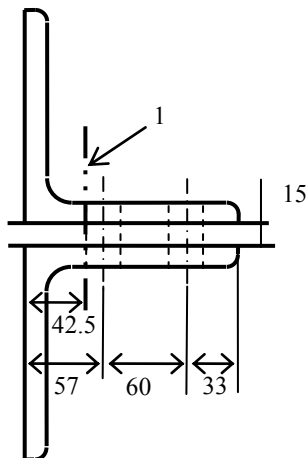


Figure B.10 Connection N3 – Sizes (in mm) and positioning



1 Angles neutral axis

Figure B.11 Connection N3 – Section CC

### 3.3.1. Connection N3 – Design forces

With:  $N_{3,Ed}$  Axial compression force at an eccentricity of  $e_{N3}$  to the centre of gravity of the joint

$M_{3,N,Ed}$  Bending moment resulting from the eccentricity,  $M_{3,N,Ed} = e_{N3} N_{3,Ed}$ .

For the gusset:

$$N_{3,g,Ed} = 609,4 \text{ kN}$$

$$e_{N3} = 44,5 \text{ mm}$$

$$M_{3,g,Ed} = e_{N3} N_{3,g,Ed} = 27,12 \text{ kNm}$$

For each angle:

$$N_{3,a,Ed} = 304,7 \text{ kN}$$

$$M_{3,a,Ed} = 13,56 \text{ kNm}$$

### 3.3.2. Connection N3 – Checking of angle

#### Resistance of gross cross-section

##### Longitudinal stress

Assuming a uniform distribution of the load in the section, the longitudinal stress is:

$$\sigma_i = \frac{N_{3,a,Ed}}{A_{3,a}} + \frac{M_{3,a,Ed}}{I_{3,a}} \cdot v$$

Where:  $A_{3,a}$  is the section area of the angle

$$A_{3,a} = 4302 \text{ mm}^2$$

$I_{3,a}$  is the second moment of area of angle

$$I_{3,a} = 8,981 \cdot 10^6 \text{ mm}^4$$

$v$  position of considered end fibre (see Figure B.12)

$$v_1 = 87 \text{ mm}$$

$$v_2 = 63 \text{ mm}$$

Then the normal stresses are:

$$\sigma_1 = 202,18 \text{ N/mm}^2 \text{ (compression)}$$

$$\sigma_2 = -24,29 \text{ N/mm}^2 \text{ (tension)}$$

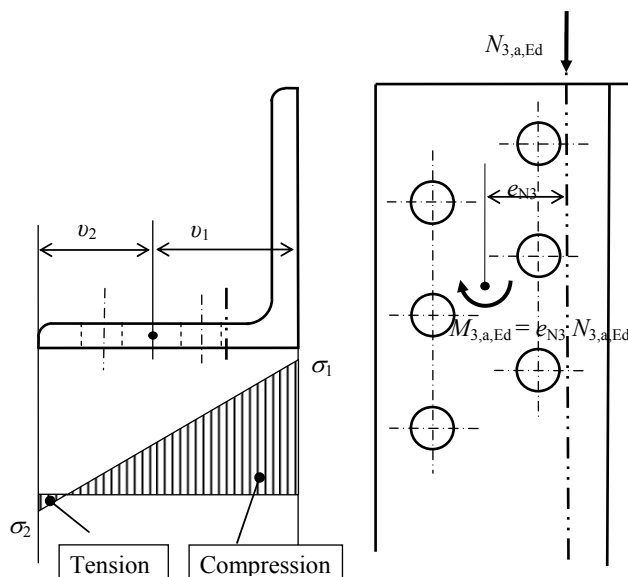


Figure B.12 Stresses in the angle N3

Title	Appendix B Worked Example: Design of a truss node with gusset	13 of 44
<p><b>Class of section</b></p> <p><math>h/t = 10 &lt; 15\varepsilon = 12,20</math></p> <p><math>(b+h)/2t = 10 &gt; 11,5\varepsilon = 9,36</math></p> <p><math>\Rightarrow</math> class 4</p> <p><math>c/t = 7,93 &lt; 10\varepsilon/\alpha = 10\varepsilon/1 = 8,14</math></p> <p><math>\Rightarrow</math> class 2</p> <p><math>\Rightarrow</math> Class of angle = class 4</p> <p><b>Combination M + N</b></p> <p>Criterion to satisfy: <math>\sigma_{x,Ed} = \frac{N_{3,a,Ed}}{A_{3,a,eff}} + \frac{M_{3,a,Ed}}{W_{3,a,eff}} \leq \frac{f_y}{\gamma_{M0}}</math></p> <p>with: <math>A_{3,a,eff}</math> effective area of cross-section</p> <p><math>A_{3,a,eff} = A_{3,a,eff,leg1} + A_{3,a,eff,leg2}</math></p> <p>where <math>A_{3,a,eff,leg1}</math> effective area relative to the “free” leg</p> <p><math>A_{3,a,eff,leg2}</math> effective area relative to the “connected” leg</p> <p>determination of the effective area of cross-section <math>A_{3,a,eff,leg1}</math></p> <p><math>\psi = \sigma_1 / \sigma_1 = 1,0</math></p> <p>buckling factor <math>k_\sigma = 0,43</math></p> <p><math>\bar{\lambda}_p = 0,660 \Rightarrow \rho = 1</math> no reduction</p> <p>determination of the effective area of cross-section <math>A_{3,a,eff,leg2}</math></p> <p><math>\psi = \sigma_2 / \sigma_1 = -0,120</math></p> <p>buckling factor <math>k_\sigma = 2,55</math></p> <p><math>\bar{\lambda}_p = 0,271 \Rightarrow \rho = 1</math> no reduction</p> <p>Verification</p> <p><math>A_{3,a,eff} = A_{3,a}</math> (no reduction)</p> <p><math>\sigma_{x,Ed} = \max(\sigma_1 ; \sigma_2) = 202,18 \leq \frac{f_y}{\gamma_{M0}} = 355 \text{ N/mm}^2</math></p> <p><math>\Rightarrow</math> criterion satisfied</p> <p><b>Resistance of net cross-section</b></p> <p>From 6.2.5 (5) of EN 1993-1-1, the fastener holes in tension zone need not be allowed for, provided that the following limit is satisfied for the complete tension zone:</p> <p><math>\frac{A_{t,net}}{\gamma_{M2}} \geq \frac{A_t f_y}{\gamma_{M0}}</math></p>		<p>EN 1993-1-1 Table 5.2 Sheet 3 of 3</p> <p>Table 5.2 Sheet 2 of 3</p> <p>EN 1993-1-1 6.2.9.3</p> <p>EN 1993-1-5 Table 4.2</p> <p>EN 1993-1-5 4.4 (2)</p> <p>EN 1993-1-5 Table 4.2</p> <p>EN 1993-1-5 4.4 (2)</p> <p>EN 1993-1-1 6.2.5 (5)</p>

Here, the holes are in the tension zone (see Figure B.12).

Accounting for  $A_{3,a,eff} = A_{3,a}$ , the following criterion should be fulfilled:

$$N_{3,a,Ed} \leq N_{3,a,c,Rd} = \frac{A_{3,a} f_y}{\gamma_{M0}}$$

With  $A_{3,a} = 4302 \text{ mm}^2$ :

$$N_{3,a,Ed} = 304,7 < N_{3,a,c,Rd} = 1527,2 \text{ kN}$$

### Buckling resistance

A compression member should be verified against buckling.

This condition has been verified in the section dealt with the verification of the members (see § 4 of this document).

### 3.3.3. Connection N3 – Checking of gusset plate

#### Resistance of cross-section

For the determination of the gross cross-section of gusset plate, a diffusion of  $45^\circ$  of the axial force  $N_{g,Ed}$  is assumed (see Figure B.13).

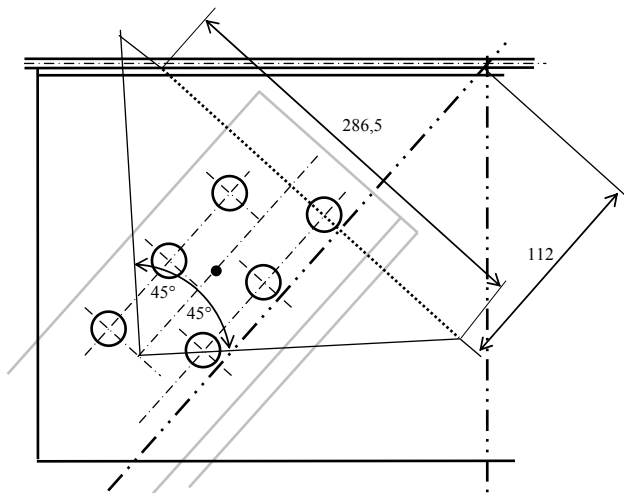


Figure B.13 Connection N3 – Diffusion by  $45^\circ$  of the axial force

The following criteria must be satisfied:

$$\sigma_{x,Ed} = \frac{N_{3,g,Ed}}{A_{3,g}} \pm \frac{M_{3,g,Ed}}{I_{3,g}/v} \leq \frac{f_y}{\gamma_{M0}}$$

$$\text{with: } A_{3,g} = 286,5 \times t_g = 4297,5 \text{ mm}^2$$

$$I_{3,g} = t_g \times 286,5^3 / 12 = 29395706 \text{ mm}^4$$

$$v = +325 / 2 \text{ mm}$$

$$\text{Then: } \sigma_{x,Ed} = 141,80 + 149,92 = 291,72 \leq \frac{f_y}{\gamma_{M0}} = 355 \text{ N/mm}^2$$

Title	Appendix B Worked Example: Design of a truss node with gusset	15 of 44
<p><b>Buckling resistance</b></p> <p>The gusset is made similar to an embedded column of characteristics:</p> <p>Area <math>A_{3,g} = 4297,5 \text{ mm}^2</math></p> <p>Height <math>h_c = 112 \text{ mm}</math> (see Figure B.13)</p> <p>Second moment of area <math>I_{c,zz} = 80578 \text{ mm}^2</math></p> <p>We should satisfy:</p> $N_{3,g,Ed} \leq N_{3,g,b,Rd} = \frac{\chi A_{3,g} f_y}{\gamma_{M1}}$ <p>Where <math>\chi</math> is the reduction factor for the relevant buckling curve</p> <p>With a buckling length of <math>2h_c</math>, the slenderness is given by:</p> $\bar{\lambda} = \sqrt{\frac{4h_c^2 A_c f_y}{\pi^2 EI_c}} = 0,677$ <p>The buckling curve to use is curve c and the imperfection is:</p> $\alpha = 0,49$ $\Phi = 0,5 \left[ 1 + \alpha (\bar{\lambda} - 0,2) + \bar{\lambda}^2 \right] = 0,846$ $\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = 0,739$ <p>Then: <math>N_{3,g,Ed} = 609,4 &lt; N_{3,g,b,Rd} = 1127 \text{ kN}</math></p> <p><b>3.3.4. Connection N3 – Checking of bolts with regard to the gusset component</b></p> <p><b>Design shear force <math>F_{V,Ed}</math> for each bolt</b></p> <p>Due to the orientation of the axial force <math>N_{3,Ed}</math>, the load on each bolt is not parallel to the edge of gusset. Also, the components of the design shear load will be performed in a suitable basis.</p> <p>In first the components are calculated in the basis <math>\{h', v'\}</math> located at the centre of gravity of the joint and oriented in agreement with the principal directions of the fasteners which are also the principal directions of the angles (See Figure B.14).</p> <p>Then a change of basis is performed from the initial <math>\{h', v'\}</math> to the basis <math>\{h, v\}</math> (see Figure B.15).</p> <p>In the basis <math>\{h', v'\}</math> the normal force <math>N_{3,g,Ed}</math> causes a horizontal shear load for each bolt <math>b_i</math>:</p> $F_{N,bi,h'} = \frac{N_{3,g,Ed}}{5} = 101,57 \text{ kN}$		<p>EN 1993-1-1 6.3.1.1</p> <p>Table 6.1</p> <p>EN 1993-1-1 6.3.1.2</p> <p>EN 1993-1-8 Table 3.4<sup>3)</sup></p>

The moment due to eccentricity is divided out according to the distance  $r'_i$  between the centre of bolts  $b_i$  and the centre of gravity of the joint:

$$F_{M,bi} = \frac{M_{1,a,Ed} r'_i}{\sum_1^5 r'_i{}^2}$$

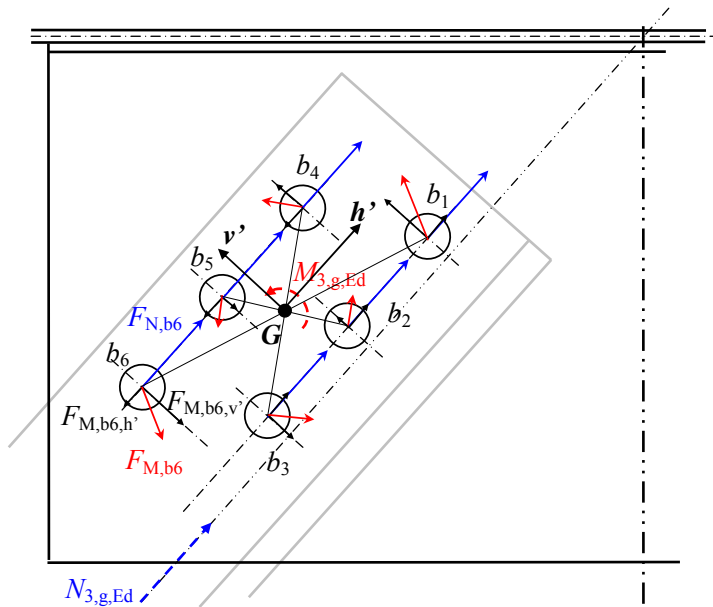


Figure B.14 Connection N3 – Gusset component – Locations

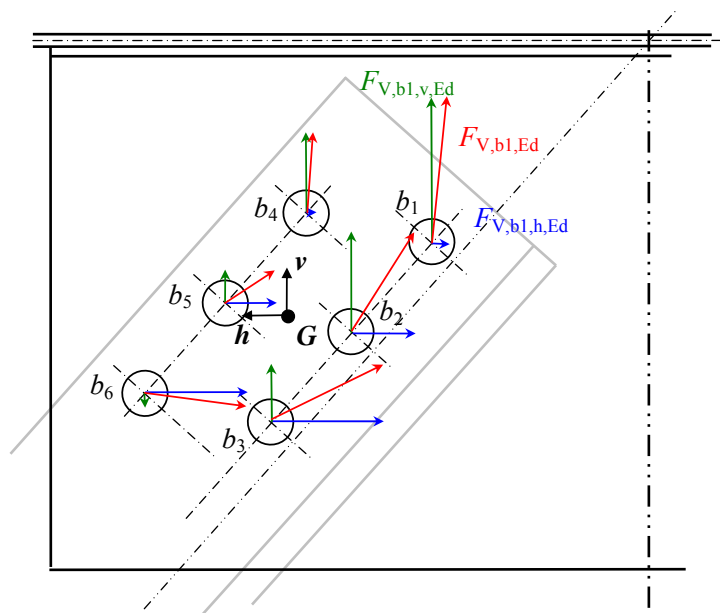


Figure B.15 Connection N3 – Gusset component – Loadings

This shear load  $F_{M,bi}$  is resolved in the basis  $\{h', v'\}$ :

$$F_{M,bi,h'} = \frac{M_{1,a,Ed} v'_i}{\sum_1^5 r_i'^2} \quad \text{horizontal component}$$

$$F_{M,bi,v'} = \frac{M_{1,a,Ed} h'_i}{\sum_1^5 r_i'^2} \quad \text{vertical component}$$

With  $h'_i$  and  $v'_i$  coordinates of centre of bolt  $b_i$ .

And we obtain (see Table B.2):

$$F_{V,bi,h',Ed} = F_{N,bi,h'} + F_{M,bi,h'} \quad \text{Horizontal shear force,}$$

$$F_{V,bi,v',Ed} = F_{M,bi,v'} \quad \text{Transverse shear force,}$$

$$F_{V,bi,Ed} = \sqrt{F_{V,bi,h',Ed}^2 + F_{V,bi,v',Ed}^2} \quad \text{Resulting shear force}$$

**Table B.2 Connection N3 – Gusset component – Design shear forces in kN in the basis  $\{h', v'\}$ .**

Bolt	$b_1$	$b_2$	$b_3$	$b_4$	$b_5$	$b_6$
$h'_i$	81,25	16,25	-48,75	48,75	-16,25	-81,25
$v'_i$	-30	-30	-30	30	30	30
$r'_i$	86,61	34,12	57,24	57,24	34,12	86,61
$F_{M,bi}$	-98,34	-38,74	-64,99	-64,99	-38,74	-98,34
$F_{M,bi,h'}$	34,06	34,06	34,06	-34,06	-34,06	-34,06
$F_{M,bi,v'}$	92,25	18,45	-55,35	55,35	-18,45	-92,25
$F_{N,bi}$	101,57	101,57	101,57	101,57	101,57	101,57
$F_{V,bi,Ed}$	164,03	136,88	146,49	87,30	69,98	114,31
$F_{V,bi,h',Ed}$	135,63	135,63	135,63	67,50	67,50	67,50
$F_{V,bi,v',Ed}$	92,25	18,45	-55,35	55,35	-18,45	-92,25

The change of basis is performed with:

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

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

Where  $\alpha_3 = 41,3^\circ$  (See Figure B.6)

Table B.3 gives the results.



Title	Appendix B Worked Example: Design of a truss node with gusset	18 of 44																												
<p><b>Table B.3 Connection N3 – Gusset component – Design shear loads in kN in the <math>\{h, v\}</math> reference system</b></p>																														
<table border="1"> <thead> <tr> <th>Bolt</th> <th><math>b_1</math></th> <th><math>b_2</math></th> <th><math>b_3</math></th> <th><math>b_4</math></th> <th><math>b_5</math></th> <th><math>b_6</math></th> </tr> </thead> <tbody> <tr> <td><math>F_{V,bi,Ed}</math></td> <td>164,03</td> <td>136,88</td> <td>146,49</td> <td>87,30</td> <td>69,98</td> <td>114,31</td> </tr> <tr> <td><math>F_{V,bi,h,Ed}</math></td> <td>-20,21</td> <td>-75,65</td> <td>-131,10</td> <td>-2,97</td> <td>-58,41</td> <td>-113,86</td> </tr> <tr> <td><math>F_{V,bi,v,Ed}</math></td> <td>162,78</td> <td>114,07</td> <td>65,36</td> <td>87,25</td> <td>38,54</td> <td>-10,17</td> </tr> </tbody> </table>			Bolt	$b_1$	$b_2$	$b_3$	$b_4$	$b_5$	$b_6$	$F_{V,bi,Ed}$	164,03	136,88	146,49	87,30	69,98	114,31	$F_{V,bi,h,Ed}$	-20,21	-75,65	-131,10	-2,97	-58,41	-113,86	$F_{V,bi,v,Ed}$	162,78	114,07	65,36	87,25	38,54	-10,17
Bolt	$b_1$	$b_2$	$b_3$	$b_4$	$b_5$	$b_6$																								
$F_{V,bi,Ed}$	164,03	136,88	146,49	87,30	69,98	114,31																								
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<p><b>Design details</b></p>																														
<p>The structure is not exposed to the weather or other corrosive influences.</p>																														
<p>We have to verify the design details in the two directions of the components of loading. By considering the limits specified in Table 3.3 of EN 1993-1-8, we have to satisfy the following checks:</p>																														
<p><math>\min\{e_1; e_2\} \geq 1,2d_0</math></p>																														
<p><math>\min\{p_1; p_2\} \geq 2,2d_0</math> or <math>\min\{p_1; p_2\} \geq 1,2d_0</math> if <math>L \geq 2,4d_0</math></p>																														
<p><math>\max\{p_1; p_2\} \leq \min\{14t; 200\text{ mm}\}</math></p>																														
<p>For <math>e_1</math> and <math>e_2</math> observe the minimum end and edge distances according to the directions <math>Gh</math> and <math>Gv</math>. And For <math>p_1</math> and <math>p_2</math> consider the spacing according to the directions <math>Gh'</math> and <math>Gv'</math>.</p>																														
<p>The design details are verified in the table below.</p>																														
<p><b>Table B.4 Connection N3 – Gusset component – Design details</b></p>																														
<table border="1"> <thead> <tr> <th>Distance or spacing</th> <th>Minimum value</th> <th>Design value</th> <th>Maximum value</th> </tr> </thead> <tbody> <tr> <td><math>\min\{e_1; e_2\}</math></td> <td>31,2</td> <td>57</td> <td></td> </tr> <tr> <td><math>\min\{p_1; p_2\}</math></td> <td>31,2</td> <td>60</td> <td></td> </tr> <tr> <td><math>\max\{p_1; p_2\}</math></td> <td></td> <td>65</td> <td>200</td> </tr> </tbody> </table>			Distance or spacing	Minimum value	Design value	Maximum value	$\min\{e_1; e_2\}$	31,2	57		$\min\{p_1; p_2\}$	31,2	60		$\max\{p_1; p_2\}$		65	200												
Distance or spacing	Minimum value	Design value	Maximum value																											
$\min\{e_1; e_2\}$	31,2	57																												
$\min\{p_1; p_2\}$	31,2	60																												
$\max\{p_1; p_2\}$		65	200																											
<p><b>Design bearing resistance <math>F_{b,Rd}</math> for each bolt</b></p>																														
<p>Table 3.4 of EN 1993-1-8 gives the expressions for the determination of the design bearing resistance. These expressions bring into play two coefficients <math>\alpha_b</math> and <math>k_1</math>.</p>																														
<p>For each bolt the value of these coefficients depend on the orientation of its loading, its location compared with the ends of the gusset but also with the location of the other bolts.</p>																														
<p>So we are considering successively the horizontal loading (loads in the direction <math>Gh</math>) and the vertical loading (loads in the direction <math>Gv</math>).</p>																														
<p>EN 1993-1-8 3.5 (1) and Table 3.3</p> <p>EN 1993-1-8 Table 3.3<sup>5)</sup></p> <p>EN 1993-1-8 Table 3.4</p>																														

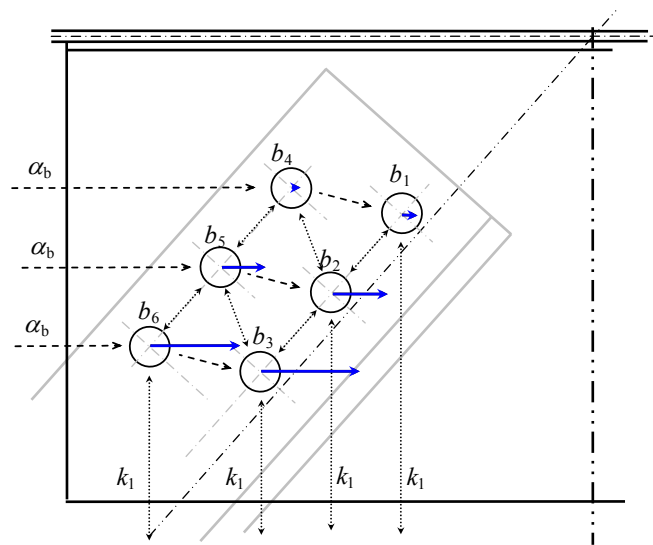
### Horizontal loading

The horizontal loading coming from the results of Table 3 is shown on the Figure B.16.

On this figure we indicate for each bolt how we are processing for the determination of its coefficients  $\alpha_b$  and  $k_1$ . So, we can specify for each bolt:

the end and edge distances ( $e_1$  and  $e_2$ ) and the spacing ( $p_1$ ,  $p_2$  and  $L$ ) to consider

the type; end or inner, or end and inner



**Figure B.16 Connection N3 – Gusset component – Horizontal loading**

The general expression for the design bearing resistance is:

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

According to Table 3.4 of the Eurocode 1993-1-8, the coefficients  $\alpha_b$  and  $k_1$  are determined from:

$$\text{For end bolts} \quad \alpha_{b,end} = \min \left\{ \frac{e_1}{3d_0}; \frac{f_{ub}}{f_u}; 1, 0 \right\}$$

$$k_{1,end} = \min \left\{ 1, 4 \frac{p_2}{d_0} - 1, 7; 2, 8 \frac{e_2}{d_0} - 1, 7; 2, 5 \right\}$$

$$\text{For inner bolts} \quad \alpha_{b,inner} = \min \left\{ \frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1, 0 \right\}$$

$$k_{1,inner} = \min \left\{ 1, 4 \frac{p_2}{d_0} - 1, 7; 2, 5 \right\}$$

Table B.6 gives the value of the horizontal component of the design bearing resistances  $F_{b,bi,h,Rd}$ .

EN 1993-1-8  
Table 3.4

**Table B.5 Connection N3 – Gusset component – Horizontal component of the design bearing resistances in kN**

Bolt	$b_1$	$b_2$	$b_3$	$b_4$	$b_5$	$b_6$
$e_1$						
$e_2$	172	124	76			90
$p_1$ <sup>1)</sup>	68,24	68,24	68,24	68,24	68,24	68,24
$p_2$	65	65 <sup>2)</sup>	65 <sup>2)</sup>	65 <sup>2)</sup>	65 <sup>2)</sup>	65
$\alpha_b$	$\alpha_{b,inner}$	$\alpha_{b,inner}$	$\alpha_{b,inner}$	$\alpha_{b,inner}$	$\alpha_{b,inner}$	$\alpha_{b,inner}$
	0,62	0,62	0,62	0,62	0,62	0,62
$k_1$	$k_{1,min}$ <sup>3)</sup>	$k_{1,min}$ <sup>3)</sup>	$k_{1,min}$ <sup>3)</sup>	$k_{1,inner}$	$k_{1,inner}$	$k_{1,min}$ <sup>3)</sup>
	1,80	1,80	1,80	1,80	1,80	1,80
$F_{b,bi,h,Rd}$	165,19	165,19	165,19	165,19	165,19	165,19

<sup>1)</sup> the distance  $L$  have been retained

<sup>2)</sup>  $\min\{65; L\}$

<sup>3)</sup>  $k_{1,min} = \min\{k_{1,inner}; k_{1,end}\}$

### Vertical loading

The vertical loading coming from the results of Table 3 is shown on the Figure B.17

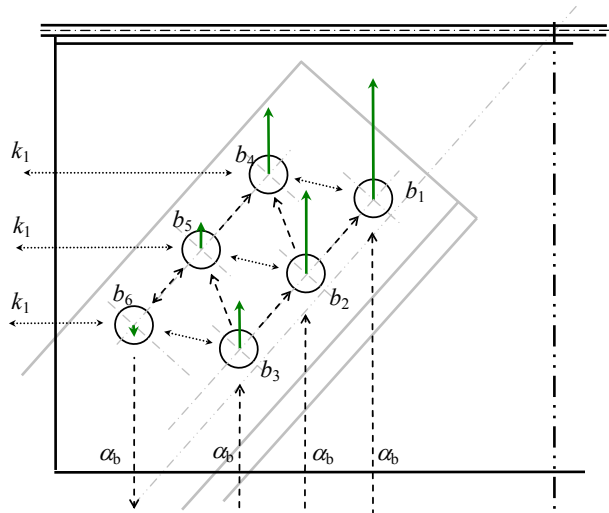
**Figure B.17 Connection N3 – Gusset component – Vertical loading**

Table B.6 gives the value of the vertical component of the design bearing resistances  $F_{b,bi,v,Rd}$ .

**Table B.6 Connection N3 – Gusset component – Vertical component of the design bearing resistances in kN**

Bolt	$b_1$	$b_2$	$b_3$	$b_4$	$b_5$	$b_6$
$e_1$						90
$e_2$				141	99	57
$p_1$	65	65 <sup>1)</sup>	65 <sup>1)</sup>	65 <sup>1)</sup>	65 <sup>1)</sup>	
$p_2$ <sup>2)</sup>	68,24	68,24	68,24	68,24	68,24	68,24
$\alpha_b$	$\alpha_{b,inner}$ 0,58	$\alpha_{b,inner}$ 0,58	$\alpha_{b,inner}$ 0,58	$\alpha_{b,inner}$ 0,58	$\alpha_{b,inner}$ 0,58	$\alpha_{b,end}$ 1,00
$k_1$	$k_{1,inner}$ 1,97	$k_{1,inner}$ 1,97	$k_{1,inner}$ 1,97	$k_{1,min}$ <sup>3)</sup> 1,97	$k_{1,min}$ <sup>3)</sup> 1,97	$k_{1,min}$ <sup>3)</sup> 1,97
$F_{b,bi,v,Rd}$	169,16	169,16	169,16	169,16	169,16	289,98

<sup>1)</sup>  $\min\{65; L\}$

<sup>2)</sup> the distance  $L$  have been retained

<sup>3)</sup>  $k_{1,min} = \min\{k_{1,inner}; k_{1,end}\}$

### Design slip resistance $F_{S,Rd}$

With:  $A_s = 353 \text{ mm}^2$  tensile stress area of the bolt

$F_{p,C} = 0,7 f_{ub} A_s = 247,1 \text{ kN}$  pretension force

$n = 2$  number of the friction surfaces relatively to the gusset

And by considering:

Bolts in normal holes  $\Rightarrow k_s = 1,0$

Class of friction surfaces = Class A  $\Rightarrow \mu = 0,5$

Then:  $F_{S,Rd} = \frac{k_s n \mu}{\gamma_{M3}} F_{p,C} = 197,68 \text{ kN}$

### Checking bolts – Individual checking

The criteria to satisfy are:

In relation to the design slip resistance

$$F_{V,bi,Ed} \leq F_{S,Rd}$$

In relation to the design bearing resistance

$$F_{V,bi,h,Ed} \leq F_{b,bi,h,Rd}$$

$$F_{V,bi,v,Ed} \leq F_{b,bi,v,Rd}$$

**Note:** an additional check based on an interactive expression is proposed:

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

EN 1993-1-8  
3.9

EN 1993-1-8  
3.9.1 (2)

EN 1993-1-8  
Table 3.6  
Table 3.7

EN 1993-1-8  
3.9.1 (1)

EN 1993-1-8  
Table 3.2

EN 1993-1-8  
Table 3.2 and  
Table 3.4<sup>3)</sup>

Each bolt has to be verified. The highest values of resistance do not necessary correspond with the bolt the most loaded.

Table B.7 summarizes only the checks for the bolt  $b_1$ .

**Table B.7 Connection N3 – Gusset component – Checking bolt  $b_1$**

Design values	Resistance values		
$ F_{V,b1,Ed} $	164,03	197,68	$F_{S,Rd}$
$ F_{V,b1,h,Ed} $	20,21	165,19	$F_{b,b1,h,Rd}$
$ F_{V,b1,v,Ed} $	162,78	169,16	$F_{b,b1,v,Rd}$
$\left(\frac{F_{V,b1,h,Ed}}{F_{b,b1,h,Rd}}\right)^2 + \left(\frac{F_{V,b1,v,Ed}}{F_{b,b1,v,Rd}}\right)^2$	0,94	1	

### Checking bolts – Group of fasteners

From the Eurocode, the design resistance of a group of fasteners may be taken as: EN 1993-1-8 3.7

$$F_{gr,b,Rd} = \sum_1^{n_{bi}} F_{b,bi,Rd} \quad \text{if for each bolt } b_i \text{ we have } F_{v,Rd} \geq F_{b,bi,Rd}$$

$$\text{else } F_{gr,b,Rd} = n_{bi} \times \min\{F_{b,bi,Rd}\}$$

Where  $F_{v,Rd}$ , the shear resistance per shear plane, is taken as:

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

By considering that the shear plane passes through the threaded portion of the bolt in normal holes:

$$\alpha_v = 0,5$$

$$A = A_s = 353 \text{ mm}^2 \text{ (tensile stress area)}$$

$$\text{Then: } F_{v,Rd} = 141,12 \text{ kN}$$

Finally for the design resistance we obtain:

$$F_{gr,b,h,Rd} = 991,17 \text{ kN for the horizontal components}$$

$$F_{gr,b,v,Rd} = 1014,94 \text{ kN for the vertical components}$$

And we verify that:

$$N_{3,g,Ed} \sin(\alpha_3) = 402,21 < F_{gr,b,h,Rd} = 991,17 \text{ kN}$$

$$N_{3,g,Ed} \cos(\alpha_3) = 457,82 < F_{gr,b,v,Rd} = 1014,94 \text{ kN}$$

### 3.3.5. Connection N3 – Checking bolts with regard to the angle component

#### Determination of the design ultimate shear load $F_{V,Ed}$ for each bolts

Table B.8 gives the results of the design ultimate shear load  $F_{V,bi,Ed}$  and its components  $F_{V,bi,h,Ed}$  and  $F_{V,bi,v,Ed}$  (See Figure B.18).

These results are deduced from the results obtained for the gusset in the basis  $\{h', v'\}$ .

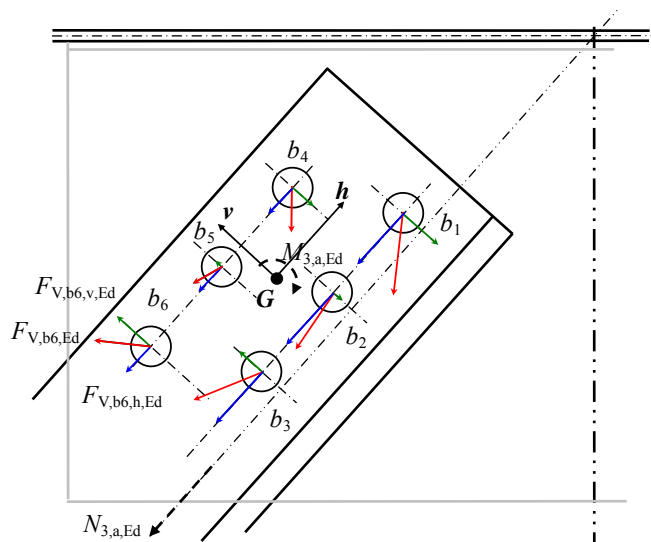


Figure B.18 Connection N3 – Angle component – Loading

Table B.8 Connection N3 – Angle component – Design shear loads in kN

Bolt	$b_1$	$b_2$	$b_3$	$b_4$	$b_5$	$b_6$
$F_{V,bi,Ed}$	82,01	68,44	73,24	43,65	34,99	57,16
$F_{V,bi,h,Ed}$	-67,81	-67,81	-67,81	-33,75	-33,75	-33,75
$F_{V,bi,v,Ed}$	-46,13	-9,23	27,68	-27,68	9,23	46,13

#### Design details

The design details are verified in the table below.

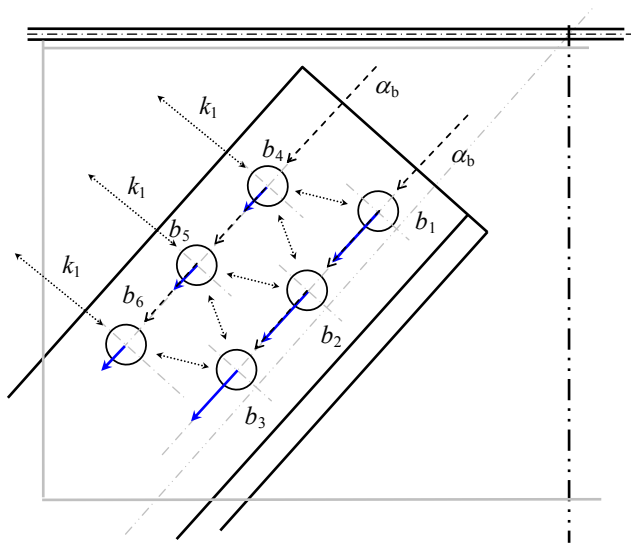
Table B.9 Connection N3 – Angle component – Design details

Distance or spacing	Minimum value	Design value	Maximum value
$\min\{e_1; e_2\}$	31,2	33	
$\min\{p_1; p_2\}$	31,2	60	
$\max\{p_1; p_2\}$		65	200

#### Determination of the design bearing resistance $F_{b,Rd}$ for each bolts

##### Horizontal loading

The horizontal loading coming from the results of Table B.8 is shown on the Figure B.19



**Figure B.19 Connection N3 – Angle component – Horizontal loadings**

Table B.10 gives the value of the horizontal component of the design bearing resistances  $F_{b,bi,h,Rd}$ .

**Table B.10 Connection N3 – Angle component – Horizontal component of the design bearing resistances in kN**

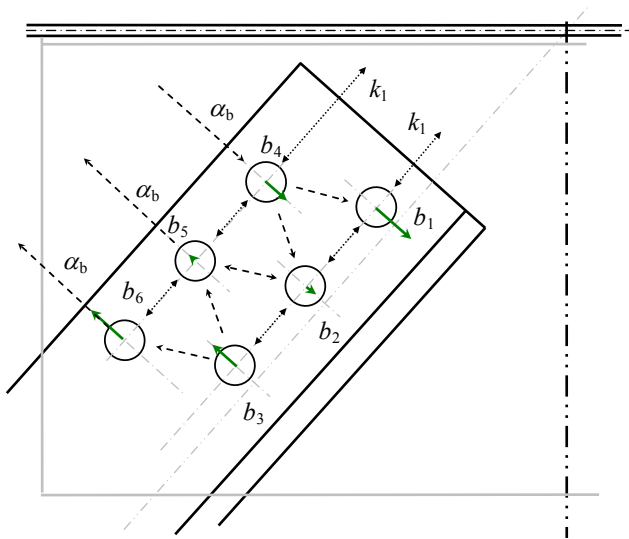
Bolt	$b_1$	$b_2$	$b_3$	$b_4$	$b_5$	$b_6$
$e_1$						
$e_2$				33	33	33
$\rho_1$	65	65	65	65	65	65
$\rho_2$ <sup>1)</sup>	68,24	68,24	68,24	68,24	68,24	68,24
$\alpha_b$	$\alpha_{b,inner}$	$\alpha_{b,inner}$	$\alpha_{b,inner}$	$\alpha_{b,inner}$	$\alpha_{b,inner}$	$\alpha_{b,inner}$
	0,58	0,58	0,58	0,58	0,58	0,58
$k_1$	$k_{1,inner}$	$k_{1,inner}$	$k_{1,inner}$	$k_{1,min}$ <sup>2)</sup>	$k_{1,min}$ <sup>2)</sup>	$k_{1,min}$ <sup>2)</sup>
	1,97	1,97	1,97	1,85	1,85	1,85
$F_{b,bi,h,Rd}$	169,16	169,16	169,16	158,84	158,84	158,84

<sup>1)</sup> the distance  $L$  have been retained

<sup>2)</sup>  $k_{1,min} = \min\{k_{1,inner}; k_{1,end}\}$

### Vertical loading

The vertical loading coming from the results of Table B.8 is shown on the Figure B.20



**Figure B.20 Connection N3 – Angle component – Vertical loading**

Table B.11 gives the value of the vertical component of the design bearing resistances  $F_{b,bi,v,Rd}$ .

**Table B.11 Connection N3 – Angle component – Vertical component of the design bearing resistances in kN**

Bolt	$b_1$	$b_2$	$b_3$	$b_4$	$b_5$	$b_6$
$e_1$					33	33
$e_2$	35			67,5		
$p_1^{1)}$	68,24	68,24	68,24	68,24	68,24	68,24
$p_2$	65	65	65	65	65	65
$\alpha_b$	$\alpha_{b,inner}$ 0,62	$\alpha_{b,inner}$ 0,62	$\alpha_{b,inner}$ 0,62	$\alpha_{b,inner}$ 0,62	$\alpha_{b,end}$ 0,42	$\alpha_{b,end}$ 0,42
$k_1$	$k_{1,min}^{2)}$ 1,80	$k_{1,inner}$ 1,80	$k_{1,inner}$ 1,80	$k_{1,min}^{2)}$ 1,80	$k_{1,inner}$ 1,80	$k_{1,inner}$ 1,80
$F_{b,bi,v,Rd}$	165,19	165,19	165,19	165,19	111,85	111,85

<sup>1)</sup> the distance  $L$  have been retained

<sup>2)</sup>  $k_{1,min} = \min\{k_{1,inner}; k_{1,end}\}$

### Determination of the design slip resistance $F_{s,Rd}$

For the angle component, the number of the friction surfaces is equal to 1.

So with  $n = 1$  we obtain:

$$F_{s,Rd} = \frac{k_s n \mu}{\gamma_{M3}} F_{p,C} = 98,84 \text{ kN}$$

### Checking bolts – Individual checking

Each bolt has to be verified.

Table B.12 summarizes only the checks for the bolt  $b_1$ .

EN 1993-1-8  
3.9

EN 1993-1-8  
3.9.1 (2)



**Table B.12 Connection N3 – Gusset component – Checking bolt  $b_1$** 

Design values		Resistance values	
$ F_{V,b1,Ed} $	82,01	98,84	$F_{S,Rd}$
$ F_{V,b1,h,Ed} $	67,81	169,16	$F_{b,b1,h,Rd}$
$ F_{V,b1,v,Ed} $	46,13	165,19	$F_{b,b1,v,Rd}$
$\left(\frac{F_{V,b1,h,Ed}}{F_{b,b1,h,Rd}}\right)^2 + \left(\frac{F_{V,b1,v,Ed}}{F_{b,b1,v,Rd}}\right)^2$	0,24	1	

**Checking bolts - Group of fasteners**

For the angle we can consider only the horizontal component. In this case:

$$F_{gr,b,h,Rd} = 991,17 \text{ kN}$$

And we verify that:

$$N_{3,a,Ed} = 304,70 < F_{gr,b,h,Rd} = 953,03 \text{ kN}$$

**3.3.6. Connection N3 – Design of net cross-section**

For a connection in tension, the design plastic resistance of the net cross-section at bolt holes should be verified only for a connection in tension.

EN 1993-1-8  
3.4.1 (1) c)

**3.3.7. Connection N3 – Design of block tearing**

Given that this connection is in compression it is not necessary to execute the design for block tearing.

**3.4. Connection N1 – Back-to-back double-angle diagonal member N1 to gusset bolted connection**

We have a shear connection in tension to be designed as Category C.

The sizes of the components of this connection and the positioning of the holes are shown on the Figure B.21. The section DD is identical to the section CC of the connection N3 (See Figure B.11).

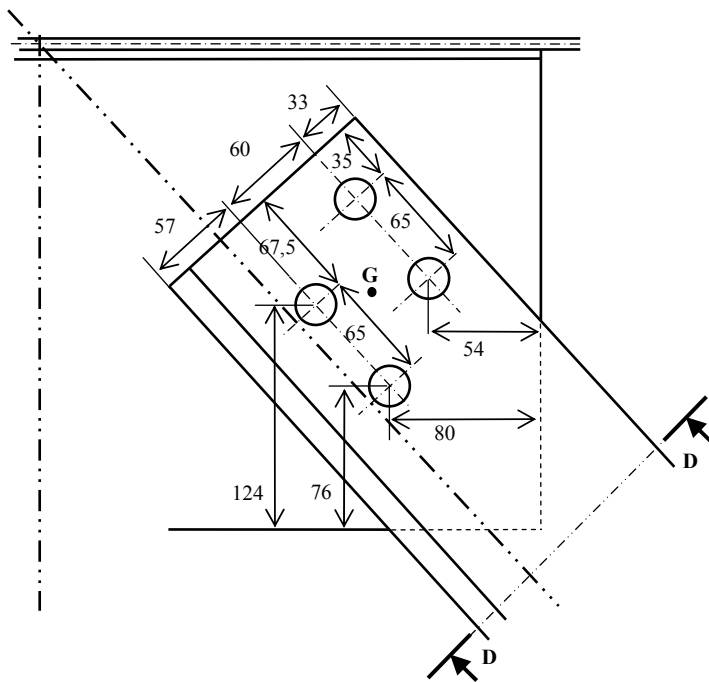


Figure B.21 Connection N1 – Sizes (in mm) and positioning

### 3.4.1. Connection N1 – Design forces

With:  $N_{1,Ed}$  the normal tension force at an eccentricity of  $e_{N1}$ , to the centre of gravity of the joint

$M_{1,N,Ed}$  the moment resulting from the eccentricity,  $M_{1,N,Ed} = e_{N1} N_{1,Ed}$ .

We have for the gusset:

$$N_{1,g,Ed} = 406,9 \text{ kN}$$

$$e_{N1} = 44,5 \text{ mm}$$

$$M_{1,g,Ed} = e_{N1} N_{1,g,Ed} = 18,11 \text{ kNm}$$

And for each angle:

$$N_{1,a,Ed} = 203,45 \text{ kN}$$

$$M_{1,a,Ed} = 9,05 \text{ kNm}$$

### 3.4.2. Connection N1 – Checking of angle

#### Resistance of gross cross-section

##### Longitudinal stress

Assuming an uniform distribution of the load on the section, the longitudinal stress is:

$$\sigma_i = \frac{N_{1,a,Ed}}{A_{1,a}} + \frac{M_{1,a,Ed}}{I_{1,a} / v}$$

Where:  $A_{1,a}$  cross-sectional area of angle  
 $I_{1,a}$  second moment of cross-section of angle  
 $v$  position of considered end fibre

With:  $A_{1,a} = 4302 \text{ mm}^2$   
 $I_{1,a} = 8,981 \cdot 10^6 \text{ mm}^4$   
 $v_1 = 87 \text{ mm}$  and  $v_2 = 63 \text{ mm}$  (see Figure B.22)

We obtain (with compression positive):

$$\sigma_1 = -134,99 \text{ N/mm}^2$$

$$\sigma_2 = 16,22 \text{ N/mm}^2$$

### Class of section

$$h/t = 10 < 15 \varepsilon = 12,20$$

$$(b+h)/2t = 10 > 11,5 \varepsilon = 9,36$$

⇒ class 4

$$c/t = 7,93 < 10 \varepsilon / \alpha = 10 \varepsilon / 1 = 8,14$$

⇒ class 2

⇒ Class of angle = class 4

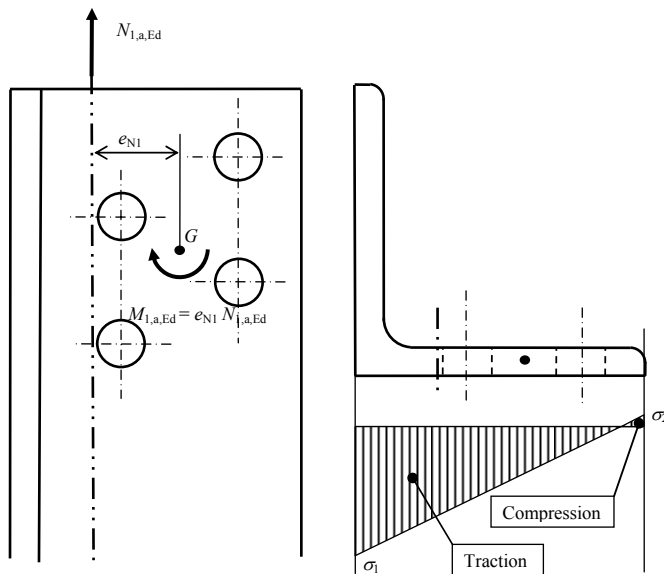


Figure B.22 Stresses in the angle N1

EN 1993-1-1  
Table 5.2  
Sheet 3 of 3

Table 5.2  
Sheet 2 of 3

**Combination M + N**

$$\text{Criterion to satisfy: } \sigma_{x,Ed} = \frac{N_{1,a,Ed}}{A_{1,a,eff}} + \frac{M_{1,a,Ed}}{W_{1,a,eff}} \leq \frac{f_y}{\gamma_{M0}}$$

with:  $A_{1,a,eff}$  effective area of cross-section

$$A_{1,a,eff} = A_{1,a,eff,leg1} + A_{1,a,eff,leg2}$$

where  $A_{1,a,eff,leg1}$  effective area relative to the “free” leg

$A_{1,a,eff,leg2}$  effective area relative to the “connected” leg

determination of the effective area of cross-section  $A_{3,a,eff,leg1}$

No reduction because “free” leg in traction

determination of the effective area of cross-section  $A_{3,a,eff,leg2}$

$$\psi = \sigma_2 / \sigma_1 = -0,120$$

buckling factor  $k_\sigma = 2,55$

$$\bar{\lambda}_p = 0,271 \Rightarrow \rho = 1 \quad \text{no reduction}$$

Verification

$$A_{1,a,eff} = A_{1,a} \text{ (no reduction)}$$

$$\sigma_{x,Ed} = \max(\sigma_1 ; \sigma_2) = 134,99 \leq \frac{f_y}{\gamma_{M0}} = 355$$

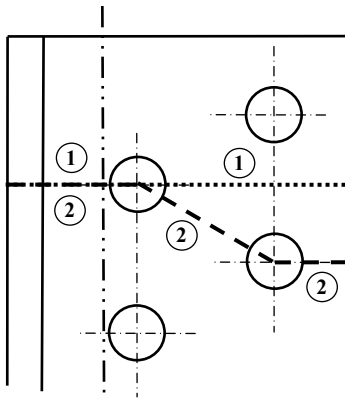
$\Rightarrow$  criterion satisfied

**Resistance of net cross-section**

We should satisfy:

$$N_{1,a,Ed} \leq N_{1,a,net,Rd} = \frac{A_{1,a,net} f_y}{\gamma_{M0}}$$

The net cross-sections considered are shown on the Figure B.23



**Figure B.23** Net cross-sections of angle N1

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6.2.9.3

EN 1993-1-5  
Table 4.2

EN 1993-1-5  
4.4 (2)

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6.2.3. (1) and (4)

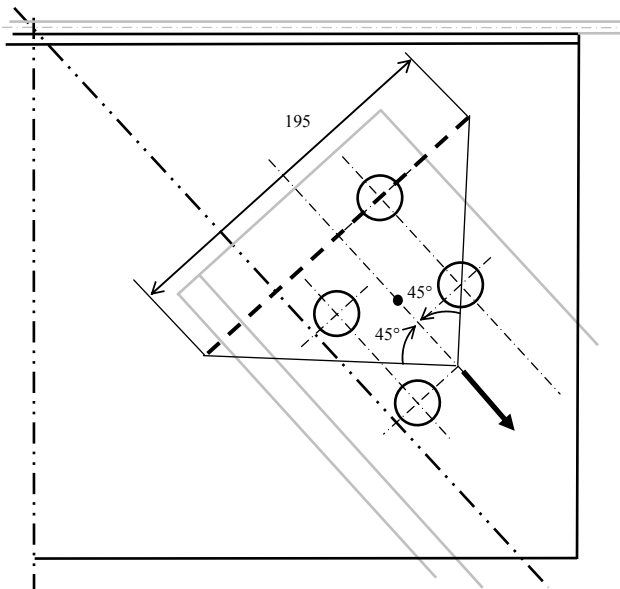
With:  $A_{1,a,net} = \min(A_{1,a,net1}; A_{1,a,net2}) = \min(3912; 3588) = 3588 \text{ mm}^2$   
we satisfy:

$$N_{1,a,Ed} = 203,45 < N_{1,a,net,Rd} = 1317,52 \text{ kN}$$

### 3.4.3. Checking of gusset

#### Resistance of cross-section

For the determination of the gross cross-section of gusset, we use an approach based on a diffusion of  $45^\circ$  of the internal force  $N_{g,Ed}$  (see Figure B.24).



**Figure B.24 Connection N1 – Diffusion by  $45^\circ$  of the internal force**

The following criteria must be satisfied:

$$\sigma_{x,Ed} = \frac{N_{1,g,Ed}}{A_{1,g}} \pm \frac{M_{1,g,Ed}}{I_{1,g} / v} \leq \frac{f_y}{\gamma_{M0}}$$

with:  $A_{1,g} = 195 \times t_g = 2925 \text{ mm}^2$

$$I_{3,g} = t_g \times 195^3 / 12 = 9268594 \text{ mm}^4$$

$$v = +195 / 2 \text{ mm}$$

We obtain:  $\sigma_{x,Ed} = 139,11 + 190,51 = 329,62 \leq \frac{f_y}{\gamma_{M0}} = 355 \text{ N/mm}^2$

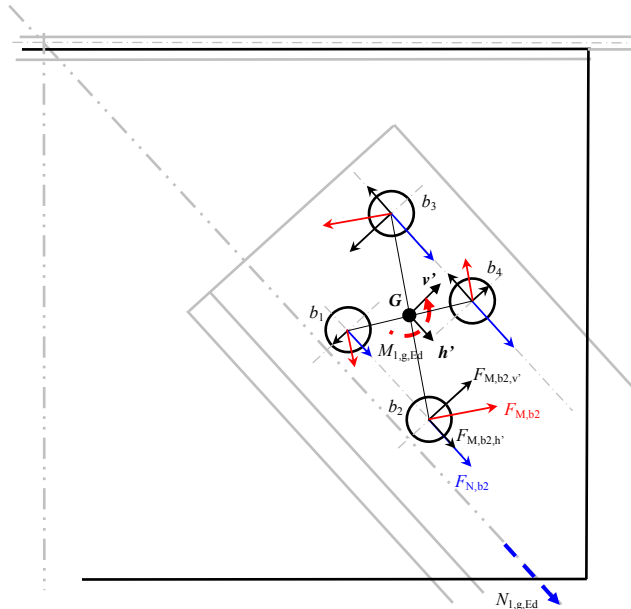
### 3.4.4. Connection N1 – Checking of bolts with regard to the gusset component

#### Determination of the design ultimate shear load $F_{V,Ed}$ for each bolts

Due to the orientation of the normal force  $N_{1,Ed}$ , the load on each bolt is not parallel to the edge of gusset. By consequent the components of the design shear load parallel and normal to the end will be performed.

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Table 3.4<sup>3)</sup>

The calculation of the components is performed in the same way as for connection N3 (see 3.3.4). We calculate the components in the basis  $\{h', v'\}$  (see Figure B.25.) then in the basis  $\{h, v\}$  (see Figure B.26).

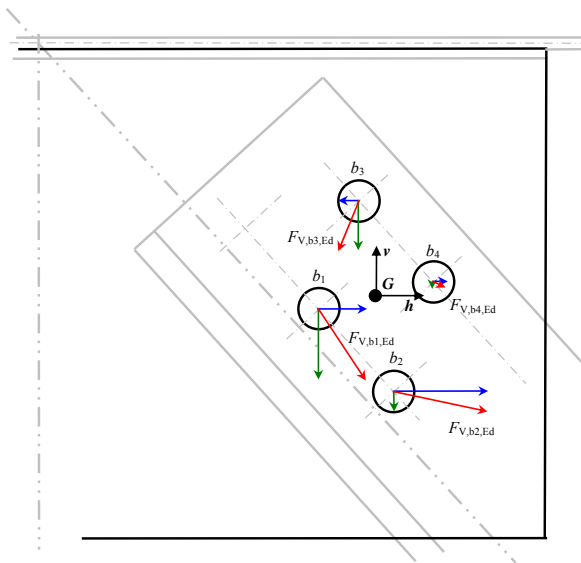


**Figure B.25 Connection N1 – Gusset component – Locations**

Table B.13 gives the calculations and the results of the design ultimate shear load  $F_{V,bi,Ed}$  and its two components  $F_{V,bi,h',Ed}$  and  $F_{V,bi,v',Ed}$  for each bolt  $b_i$  in the  $\{h', v'\}$  reference system.

**Table B.13 Connection N1 – Gusset component – Design shear loads in kN in the  $\{h', v'\}$  reference system.**

Bolt	b <sub>1</sub>	b <sub>2</sub>	b <sub>3</sub>	b <sub>4</sub>
$h'_i$	-16,25	48,75	-48,75	16,25
$v'_i$	-30	-30	30	30
$r'_i$	34,12	57,24	57,24	34,12
$F_{M,bi}$	69,56	116,70	116,70	69,56
$F_{M,bi,h'}$	61,16	61,16	-61,16	-61,16
$F_{M,bi,v'}$	-33,13	99,39	-99,39	33,13
$F_{N,bi}$	101,73	101,73	101,73	101,73
$F_{V,bi,Ed}$	166,22	190,82	107,35	52,37
$F_{V,bi,h',Ed}$	162,89	162,89	40,56	40,56
$F_{V,bi,v',Ed}$	-33,13	99,39	-99,39	33,13



**Figure B.26 Connection N1 – Gusset component – Loadings**

The change of basis is performed with:

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

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

Where  $\alpha_1 = 42^\circ$  (See Figure B.6)

Table B.14 gives the results.

**Table B.14 Connection N1 – Gusset component – Design shear loads in kN in the  $\{h, v\}$  reference system.**

Bolt	$b_1$	$b_2$	$b_3$	$b_4$
$F_{V,bi,Ed}$	166,22	190,82	107,35	52,37
$F_{V,bi,h,Ed}$	84,37	182,86	-46,72	51,76
$F_{V,bi,v,Ed}$	-143,22	-54,54	-96,65	-7,97

### Design details

The design details are verified in the table below.

For  $e_1$  and  $e_2$  we observe the minimums end and edge distances according to the appropriate direction ( $Gh$  or  $Gv$ ). For  $p_1$  and  $p_2$  we consider the spacing according to the principal direction of the joint ( $Gh'$  or  $Gv'$ ).

**Table B.15 Connection N1 – Gusset component – Design details**

Distance or spacing	Minimum value	Design value	Maximum value
$\min\{e_1; e_2\}$	31,2	54	
$\min\{p_1; p_2\}$	31,2	60	
$\max\{p_1; p_2\}$		65	200

### Determination of the design bearing resistance $F_{b,Rd}$ for each bolts Horizontal loading

The horizontal loading coming from the results of Table B.14 is shown on the Figure B.27

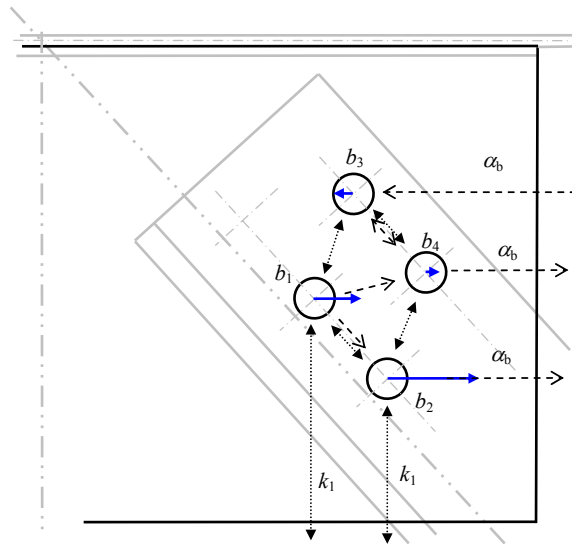


Figure B.27 Connection N1 – Gusset component – Horizontal loading

Table B.16 gives the value of the horizontal component of the design bearing resistances  $F_{b,bi,h,Rd}$ .

Table B.16 Connection N1 – Gusset component – Horizontal component of the design bearing resistances in kN

Bolt	$b_1$	$b_2$	$b_3$	$b_4$
$e_1$		80		54
$e_2$	124	76		
$\rho_1$	65 <sup>1)</sup>		65	
$\rho_2$	65 <sup>1)</sup>	65 <sup>1)</sup>	65 <sup>1)</sup>	65 <sup>1)</sup>
$\alpha_b$	$\alpha_{b,inner}$	$\alpha_{b,end}$	$\alpha_{b,inner}$	$\alpha_{b,end}$
	0,58	1,00	0,58	0,69
$k_1$	$k_{1,min}$ <sup>3)</sup>	$k_{1,min}$ <sup>3)</sup>	$k_{1,inner}$	$k_{1,inner}$
	1,80	1,80	1,80	1,80
$F_{b,bi,h,Rd}$	154,22	264,38	154,22	183,04

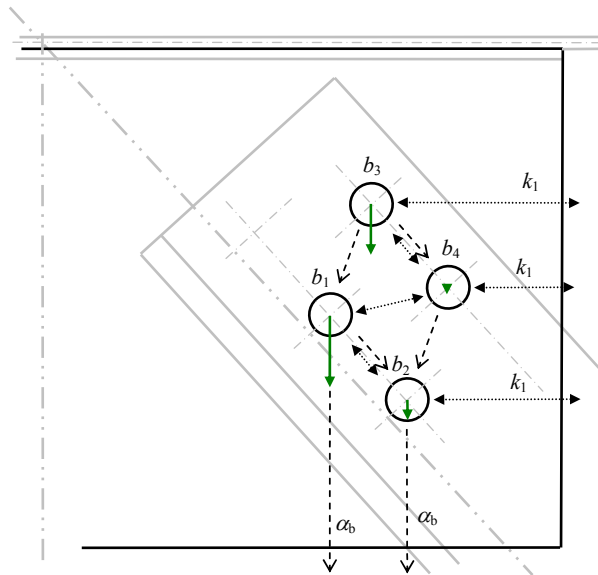
<sup>1)</sup>  $\min\{65; L\}$

<sup>2)</sup>  $k_{1,min} = \min\{k_{1,inner}; k_{1,end}\}$



### Vertical loading

The vertical loading coming from the results of Table B.14 is shown on the Figure B.28.



**Figure B.28 Connection N1 – Gusset component – Vertical loading**

Table B.17 gives the value of the vertical component of the design bearing resistances  $F_{b,bi,v,Rd}$ .

**Table B.17 Connection N1 – Gusset component – Vertical component of the design bearing resistances in kN**

Bolt	$b_1$	$b_2$	$b_3$	$b_4$
$e_1$	124	76		
$e_2$		80	98	54
$\rho_1$			65 <sup>1)</sup>	65 <sup>1)</sup>
$\rho_2$	65 <sup>1)</sup>	65	65	65 <sup>1)</sup>
$\alpha_b$	$\alpha_{b,end}$	$\alpha_{b,end}$	$\alpha_{b,inner}$	$\alpha_{b,inner}$
	1,00	0,97	0,58	0,58
$k_1$	$k_{1,inner}$	$k_{1,min}^{2)}$	$k_{1,min}^{2)}$	$k_{1,min}^{2)}$
	1,80	1,80	1,80	1,80
$F_{b,bi,v,Rd}$	264,38	257,60	154,22	154,22

<sup>1)</sup>  $\min\{65; L\}$

<sup>2)</sup>  $k_{1,min} = \min\{k_{1,inner}; k_{1,end}\}$

**Determination of the design slip resistance  $F_{s,Rd}$** 

With  $n = 2$ , the number of the friction surfaces relatively to the gusset, we obtain:

$$F_{s,Rd} = \frac{k_s n \mu}{\gamma_{M3}} F_{p,C} = 197,68 \text{ kN}$$

**Checking bolts – Individual checking**

Each bolt has to be verified.

Table B.18 and Table B.19 summarize only the checks for the bolt  $b_1$  and  $b_2$ .

**Table B.18 Connection N1 – Gusset component – Checking bolt  $b_1$** 

Design values		Resistance values	
$ F_{V,b1,Ed} $	166,22	197,68	$F_{s,Rd}$
$ F_{V,b1,h,Ed} $	84,37	154,22	$F_{b,b1,h,Rd}$
$ F_{V,b1,v,Ed} $	143,22	264,38	$F_{b,b1,v,Rd}$
$\left(\frac{F_{V,b1,h,Ed}}{F_{b,b1,h,Rd}}\right)^2 + \left(\frac{F_{V,b1,v,Ed}}{F_{b,b1,v,Rd}}\right)^2$	0,59	1	

**Table B.19 Connection N1 – Gusset component – Checking bolt  $b_2$** 

Design values		Resistance values	
$ F_{V,b1,Ed} $	190,82	197,68	$F_{s,Rd}$
$ F_{V,b1,h,Ed} $	182,86	264,38	$F_{b,b1,h,Rd}$
$ F_{V,b1,v,Ed} $	54,54	257,60	$F_{b,b1,v,Rd}$
$\left(\frac{F_{V,b1,h,Ed}}{F_{b,b1,h,Rd}}\right)^2 + \left(\frac{F_{V,b1,v,Ed}}{F_{b,b1,v,Rd}}\right)^2$	0,52	1	

**Checking bolts – Group of fasteners**

By considering that the shear plane passes through the threaded portion of the bolt in normal holes:

$$\alpha_v = 0,5$$

$$A = A_s = 353 \text{ mm}^2 \text{ (tensile stress area)}$$

We obtain:

$$F_{v,Rd} = 141,12 \text{ kN}$$

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3.9.1 (1)

And for the design resistance:

$$F_{gr,b,h,Rd} = 616,90 \text{ kN for the horizontal components}$$

$$F_{gr,b,v,Rd} = 616,90 \text{ kN for the vertical components}$$

And we verify that:

$$N_{1,g,Ed} \sin(\alpha_1) = 272,27 < F_{gr,b,h,Rd} = 616,90 \text{ kN}$$

$$N_{1,g,Ed} \cos(\alpha_1) = 302,39 < F_{gr,b,h,Rd} = 616,90 \text{ kN}$$

### 3.4.5. Connection N1 – Checking bolts with regard to the angle component

#### Determination of the design ultimate shear load $F_{V,Ed}$ for each bolts

Table B.20 gives the results of the design ultimate shear load  $F_{V,bi,Ed}$  and its components  $F_{V,bi,h,Ed}$  and  $F_{V,bi,v,Ed}$  (See Figure B.29).

These results are deduced from the results obtained for the gusset in the basis  $\{h', v'\}$ .

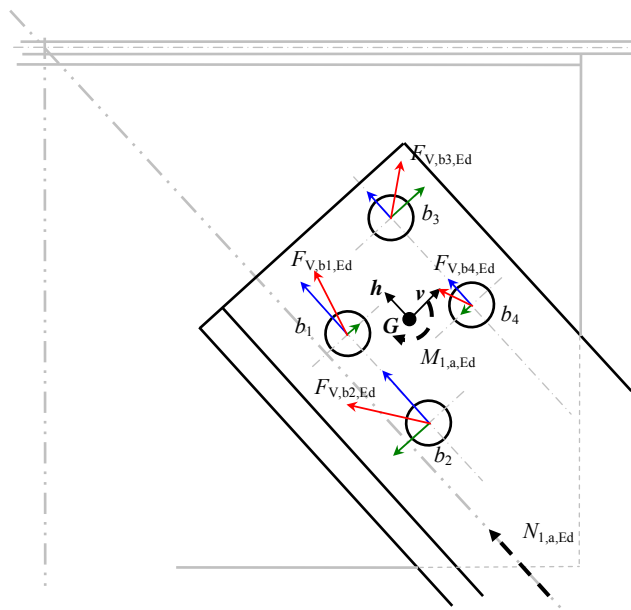


Figure B.29 Connection N1 – Angle component – Loading

Table B.20 Connection N1 – Angle component – Design shear loads in kN

Bolt	$b_1$	$b_2$	$b_3$	$b_4$
$F_{V,bi,Ed}$	83,11	95,41	53,67	26,19
$F_{V,bi,h,Ed}$	81,44	81,44	20,28	20,28
$F_{V,bi,v,Ed}$	16,57	-49,70	49,70	-16,57

#### Design details

The design details are verified in the table below.

**Table B.21 Connection N1 – Angle component – Horizontal loading – Design details**

Distance or spacing	Minimum value	Design value	Maximum value
$\min \{e_1 ; e_2 \}$	31,2	33	
$\min \{p_1 ; p_2 \}$	57,2	60	200
$\max \{p_1 ; p_2 \}$		65	200

### Determination of the design bearing resistance $F_{b,Rd}$ for each bolts Horizontal loading

The horizontal loading coming from the results of Table B.20 is shown on the Figure B.30

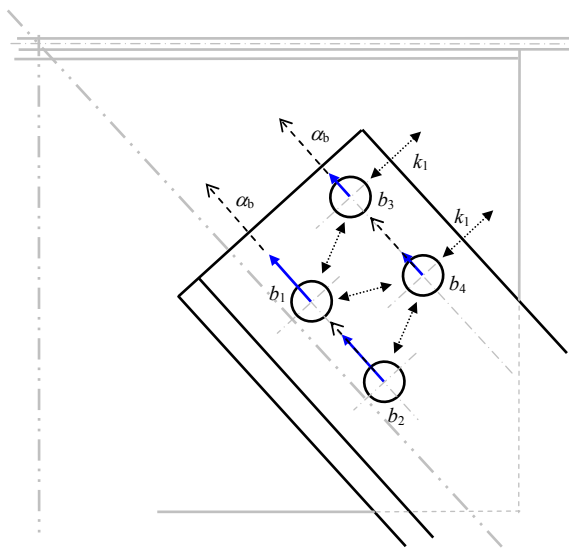
**Figure B.30 Connection N1 – Angle component – Horizontal loadings**

Table B.22 gives the value of the horizontal component of the design bearing resistances  $F_{b,bi,h,Rd}$ .

**Table B.22 Connection N1 – Angle component – Horizontal component of the design bearing resistances in kN**

Bolt	$b_1$	$b_2$	$b_3$	$b_4$
$e_1$	67,5		35	
$e_2$			33	33
$p_1$		65		65
$p_2$ <sup>1)</sup>	68,24	68,24	68,24	68,24
$\alpha_b$	$\alpha_{b,end}$ 0,87	$\alpha_{b,inner}$ 0,58	$\alpha_{b,end}$ 0,45	$\alpha_{b,inner}$ 0,58
$k_1$	$k_{1,inner}$ 1,97	$k_{1,inner}$ 1,97	$k_{1,min}$ <sup>2)</sup> 1,85	$k_{1,min}$ <sup>2)</sup> 1,85
$F_{b,bi,h,Rd}$	250,95	169,16	122,18	158,84

<sup>1)</sup> the distance  $L$  have been retained

<sup>2)</sup>  $k_{1,min} = \min\{k_{1,inner}; k_{1,end}\}$

### Vertical loading

The vertical loading coming from the results of Table 20 is shown on the Figure B.31

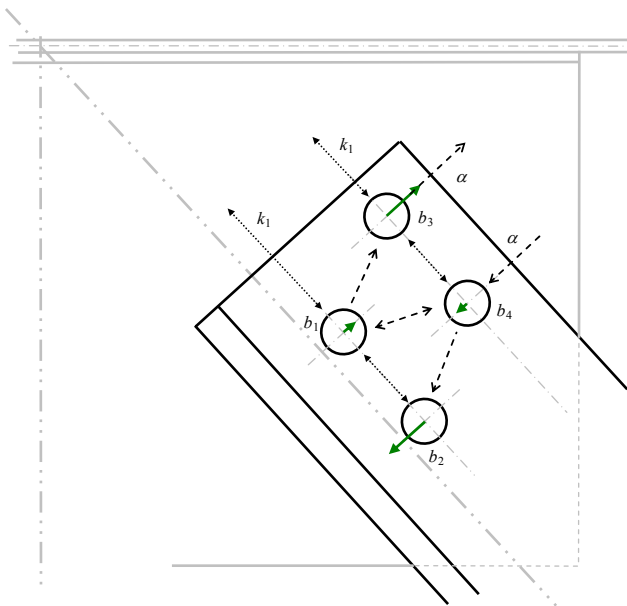
**Figure B.31 Connection N1 – Angle component – Vertical loading**

Table B.23 gives the value of the vertical component of the design bearing resistances  $F_{b,bi,v,Rd}$ .

**Table B.23 Connection N1 – Angle component – Vertical component of the design bearing resistances in kN**

Bolt	$b_1$	$b_2$	$b_3$	$b_4$
$e_1$			33	
$e_2$	67,5		35	
$p_1$ <sup>1)</sup>	68,24	68,24		68,24
$p_2$	65	65	65	65
$\alpha_b$	$\alpha_{b,inner}$ 0,62	$\alpha_{b,inner}$ 0,62	$\alpha_{b,end}$ 0,42	$\alpha_{b,inner}$ 0,62
$k_1$	$k_{1,min}$ <sup>2)</sup> 1,80	$k_{1,inner}$ 1,80	$k_{1,min}$ <sup>2)</sup> 1,80	$k_{1,inner}$ 1,80
$F_{b,bi,h,Rd}$	165,19	165,19	111,85	165,19

<sup>1)</sup> the distance  $L$  have been retained

<sup>2)</sup>  $k_{1,min} = \min\{k_{1,inner}; k_{1,end}\}$

### Determination of the design slip resistance $F_{S,Rd}$

For the angle component, the number of the friction surfaces is equal to 1.

So with  $n = 1$  we obtain:

$$F_{S,Rd} = \frac{k_s n \mu}{\gamma_{M3}} F_{p,C} = 98,84 \text{ kN}$$

### Checking bolts – Individual checking

Each bolt has to be verified. Table B.24 summarizes only the checks for the bolt  $b_2$ .

**Table B.24 Connection N1 – Angle component – Checking bolt  $b_2$** 

Design values	Resistance values
$ F_{V,b1,Ed} $ 95,41	98,84 $F_{S,Rd}$
$ F_{V,b1,h,Ed} $ 81,44	169,16 $F_{b,b1,h,Rd}$
$ F_{V,b1,v,Ed} $ 49,70	165,19 $F_{b,b1,v,Rd}$
$\left(\frac{F_{V,b1,h,Ed}}{F_{b,b1,h,Rd}}\right)^2 + \left(\frac{F_{V,b1,v,Ed}}{F_{b,b1,v,Rd}}\right)^2$ 0,32	1

### Checking bolts – Group of fasteners

For the angle we can consider only the horizontal component:

$$F_{gr,b,h,Rd} = 488,73 \text{ kN}$$

And we verify that:

$$N_{1,a,Ed} = 203,45 < F_{gr,b,h,Rd} = 488,73 \text{ kN}$$

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3.9.1 (2)

### 3.4.6. Connection N1 – Design of net cross-section

#### Gusset component

For a connection in tension, the design of the net cross-sections have to be verified.

Verify on the net cross-section marked 1 on the Figure B.32. For this section, we have to satisfy:

$$n_b \frac{N_{1,g,Ed}}{n_{bt}} \leq \frac{A_{net1} f_y}{\gamma_{M0}}$$

Where  $n_b = 2$  number of bolts relative to the cross-section

$n_{bt} = 4$  total number of the connection

With  $A_{net1} = 2194 \text{ mm}^2$

We satisfy:  $n_b \frac{N_{1,g,Ed}}{n_{bt}} = 203,4 < \frac{A_{net1} f_y}{\gamma_{M0}} = 778 \text{ kN}$

#### Angle component

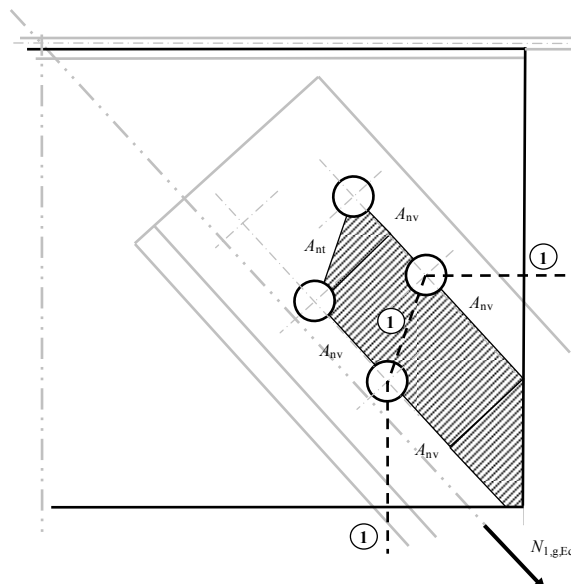
We have been already verified the net cross-section (see 3.4.2).

Moreover these checking have been realised with  $N_{Ed}$  in loco  $n_b F_{V,Ed}$ .

### 3.4.7. Connection N1 – Design for block tearing

#### Gusset component

The Figure B.32 shows the block tearing for the gusset.



**Figure B.32 Connection N1 – Block tearing for gusset**

Our bolt group is subjected to eccentric loading and we have to satisfy:

$$N_{1,g,Ed} \leq V_{eff,2,Rd}$$

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3.4.1 (1) c) and  
Table 3.2

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3.10.2

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3.10.2 (3)

Where 
$$V_{\text{eff},2,\text{Rd}} = \frac{0,5 f_u A_{\text{nt}}}{\gamma_{\text{M}2}} + \frac{1}{\sqrt{3}} \frac{f_y A_{\text{nv}}}{\gamma_{\text{M}0}}$$

With 
$$A_{\text{nt}} = 633,6 \text{ mm}^2$$
  

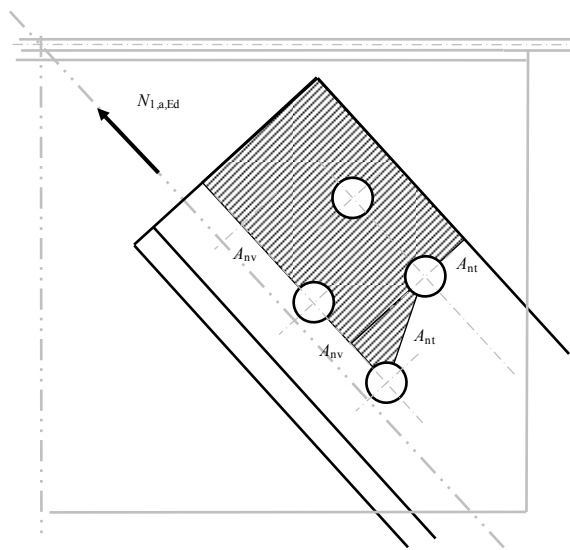
$$A_{\text{nv}} = 3533,1 \text{ mm}^2$$

We satisfy:

$$N_{1,\text{g},\text{Ed}} = 406,9 \leq V_{\text{eff},2,\text{Rd}} = 853,4 \text{ kN}$$

### Angle component

The Figure B.33 shows the block tearing for the gusset.



**Figure B.33 Connection N1 – Block tearing for angle**

Our bolt group is subjected to eccentric loading and we have to satisfy:

$$N_{1,\text{a},\text{Ed}} \leq V_{\text{eff},2,\text{Rd}}$$

With 
$$A_{\text{nt}} = 933,6 \text{ mm}^2$$
  

$$A_{\text{nv}} = 1402,5 \text{ mm}^2$$

We satisfy:

$$N_{1,\text{g},\text{Ed}} = 203,45 \leq V_{\text{eff},2,\text{Rd}} = 407,91 \text{ kN}$$

### 3.5. Connection N2 – Single angle post member N2 to gusset bolted connection

We have a shear connection in tension to be designed as Category C.

Given that the loading is low, the checking of this connection is not carry out. Otherwise the procedure stays the same with in addition the following point.

EN 1993-1-8  
3.10.2 (3)



Title	Appendix B Worked Example: Design of a truss node with gusset	42 of 44
<p>We are dealing with a single angle in tension by a single row of bolts in one leg. During the checking of the net cross-section of this angle, the design ultimate resistance should be determined as follows:</p> $N_{u,Rd} = \frac{\beta_2 A_{net} f_u}{\gamma_{M2}}$ <p>With <math>\beta_2 = 0,4</math> (<math>p_1 = 65 = 2,5 d_0</math>)</p> <p><b>3.6. Influences of the eccentricity and other parameters</b></p> <p>We consider only the bolts with regard to the gusset component.</p> <p><b>3.6.1. Connection N3 – Moment due to eccentricity</b></p> <p>The effects of the eccentricity depend of the locations of the bolts comparatively with the neutral axis but also to each other.</p> <p>Lets the moment due to the eccentricity equal to 0. In this case and whatever the bolt we obtain in the basis <math>\{h, v\}</math>:</p> $ F_{V,b,Ed}  = 101,57 \text{ kN (value without moment due to eccentricity)}$ $ F_{V,b,h,Ed}  = 67,03 \text{ kN (value without moment due to eccentricity)}$ $ F_{V,b,v,Ed}  = 76,30 \text{ kN (value without moment due to eccentricity)}$ <p>Values to compare at the results obtained for the bolt <math>b_1</math>:</p> $ F_{V,b,Ed}  = 164,03 \text{ kN (value with moment due to eccentricity)}$ $ F_{V,b,h,Ed}  = 20,21 \text{ kN (value with moment due to eccentricity)}$ $ F_{V,b,v,Ed}  = 162,78 \text{ kN (value with moment due to eccentricity)}$ <p><b>3.6.2. Connection N3 – Influence of number of bolts and spacing <math>p_1</math></b></p> <p>Reduce the number of bolts from 6 to 5 by suppression of bolt marked <math>b_6</math> (see Figure B.14). This modification modifies the location of the centre of gravity of the bolt group. Even if the moment due to eccentricity decrease, the design shear loads per bolt increase. And two bolts (<math>b_1</math> and <math>b_3</math>) do not again satisfy to the criteria relative to the design bearing resistances (see tables below).</p>		<p>EN 1993-1-8 3.10.3 (2) and Table 3.8</p>

**Table B.25 Connection N3 – Gusset component – Bolt  $b_1$  – Reduction of total number of bolts**

Design values		Resistance values	
Total number of bolts	6	5	
$ F_{V,b1,Ed} $	164,03	189,76	197,68 $F_{S,Rd}$
$ F_{V,b1,h,Ed} $	20,21	28,43	165,19 $F_{b,b1,h,Rd}$
$ F_{V,b1,v,Ed} $	162,78	<b>187,62</b>	<b>169,16</b> $F_{b,b1,v,Rd}$

**Table B.26 Connection N3 – Gusset component – Bolt  $b_3$  – Reduction of total number of bolts**

Design values		Resistance values	
Total number of bolts	6	5	
$ F_{V,b1,Ed} $	146,49	189,76	197,68 $F_{S,Rd}$
$ F_{V,b1,h,Ed} $	131,10	<b>182,40</b>	<b>165,19</b> $F_{b,b1,h,Rd}$
$ F_{V,b1,v,Ed} $	65,36	52,36	169,16 $F_{b,b1,v,Rd}$

At this stage, increase the value of the spacing  $p_1$  from 65 to 75 mm. So all the bolts satisfy the criteria. Look for example the results for bolt  $b_1$ .

**Table B.27 Connection N3 – Gusset component – Bolt  $b_1$  – Increasing of spacing  $p_1$  to 75 mm**

Design values	Resistance values	
$ F_{V,b1,Ed} $	180,06	197,68 $F_{S,Rd}$
$ F_{V,b1,h,Ed} $	28,74	225,70 $F_{b,b1,h,Rd}$
$ F_{V,b1,v,Ed} $	177,75	220,50 $F_{b,b1,v,Rd}$

### 3.6.3. Connection N1 – Influence of number of bolts

Reduce the number of bolts from 4 to 3 by suppression of bolt marked  $b_3$  (see Figure B.25). The moment due to eccentricity decrease whereas the design shear loads per bolt increase. And two bolts ( $b_1$  and  $b_2$ ) do not again satisfy to the criteria relative to the design bearing resistances (see tables below).

**Table B.28 Connection N1 – Gusset component – Bolt  $b_1$  – Reduction of total number of bolts**

Design values			Resistance values	
Total number of bolts	4	3		
$ F_{V,b1,Ed} $	166,22	<b>222,19</b>	<b>197,68</b>	$F_{S,Rd}$
$ F_{V,b1,h,Ed} $	84,37	57,25	154,22	$F_{b,b1,h,Rd}$
$ F_{V,b1,v,Ed} $	143,22	214,69	264,38	$F_{b,b1,v,Rd}$

**Table B.29 Connection N1 – Gusset component – Bolt  $b_2$  – Reduction of total number of bolts**

Design values			Resistance values	
Total number of bolts	4	3		
$ F_{V,b1,Ed} $	190,82	<b>222,19</b>	<b>197,68</b>	$F_{S,Rd}$
$ F_{V,b1,h,Ed} $	182,86	207,52	264,38	$F_{b,b1,h,Rd}$
$ F_{V,b1,v,Ed} $	54,54	79,38	257,60	$F_{b,b1,v,Rd}$

In order to satisfy the criteria we need to increase the value of the spacing  $p_1$  from 65 to a minimum of 101 mm. Look for example the results for bolt  $b_1$ .

**Table B.30 Connection N3 – Gusset component – Bolt  $b_1$  – Increasing of spacing  $p_1$  to 101 mm**

Design values		Resistance values	
$ F_{V,b1,Ed} $	197,33	197,68	$F_{S,Rd}$