

NCCI: Design of roof trusses

This NCCI deals with some special cases that can occur when designing a roof truss. Such cases are for example how to treat eccentricity in the joints, non-nodal loads and reversed loadings

Contents

1. General	2
2. Design of upper chord	2
3. Design of lower chord	5
4. Design of web members	9
5. Eccentricity	10
6. Examples of truss to column connection and truss splice	11
7. References	12

1. General

The traditional analysis of a truss assumes that all loads are applied in the joints and that all joints in the truss are pinned. Even though this is generally not the case, since the upper and lower chords are normally continuous and the web members are often welded to the chords, it is still a common and acceptable procedure to determine the axial forces in the members. In a situation when the dimensions of the upper chord is very large and the overall depth of the truss is small the moments due to continuous upper chord has to be considered. However, this is rarely the case for roof trusses in industrial buildings that is treated here. Bending moments have to be considered in other cases, which will be explained here.

In this NCCI the considered roof trusses are assumed to be simply supported as in Figure 2.1, i.e. the truss to column connections are not fixed.

The analysis of a truss is rather simple using the assumptions in the previous paragraph and will not be discussed here but there are some special cases that complicate things:

- When for example the roof sheeting is attached directly to the truss or if purlins are used and they are not placed only in the truss joints, bending moment will be introduced into the upper chord.
- Due to eccentricity in the joints between chord and web members, moments might be introduced that need to be taken into account in the design.
- For low pitch roofs, reversed loadings due to the wind load can cause compression in the lower chord and it has to be designed for lateral buckling.

2. Design of upper chord

2.1 General

If all loads are introduced in the joints, only axial forces should be considered. When the upper chord is compressed, both buckling in and out of plane have to be considered if not constrained in the lateral direction. According to [Annex BB of EN 1993-1-1 \[1\]](#), the buckling length is equal to the distance between the truss joints for buckling in the plane of the truss. The buckling length out of plane is the distance between the purlins. If the upper chord is a built-up member it should be designed according to [§6.4 of EN1993-1-1](#) for buckling in the lateral direction.

The above statement that the buckling length out of plane is the distance between the purlins requires that the purlins act as lateral support which means that they cannot move in their axial direction. This means that the purlins should be restrained by bracings in the roof structure and by vertical bracings in the walls. It is also possible to use the roof sheeting, if it is stiff enough to work as a diaphragm and fulfils the requirements for structural class I or II according to EN 1993-1-3 [2], instead of bracings. In [SS050](#) examples of stabilising trusses and bracings are explained and shown.

If the roof sheeting is a deck placed directly on the roof truss, i.e. without purlins, the buckling out of plane of the upper chord is restrained. This requires that the roof sheeting is

stiff enough to work as a diaphragm and is in structural class I or II according to EN 1993-1-3 [2].

In the ideal case the roof will be attached to purlins that will be placed in the joints of the truss and only the axial forces need to be checked. This is not always the case, sometimes the roof sheeting is directly attached to the roof truss. In such cases the upper chords will be subjected not only to axial force but also to bending moment. In Figure 2.1 an example is shown with non-nodal loads applied to the roof truss.

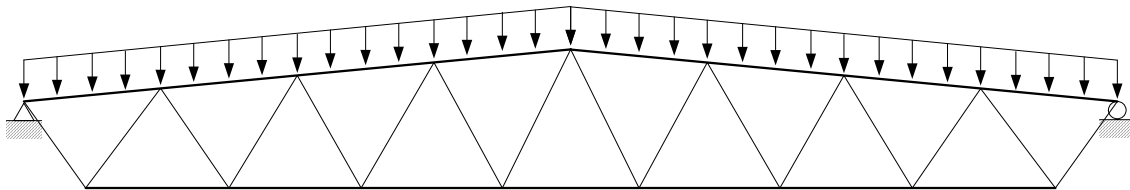


Figure 2.1 Example of a roof truss with roof sheeting attached directly on the truss, introducing non-nodal loads on the truss.

In such case the top chord should be treated as a continuous beam and the bending moment should be taken into account in the member design. A short schematic example on this topic can be found in the section below.

For different reasons eccentricity can be present due to the joints design between the top chord and the web members, see Figure 5.1. If this is the case the moments arising from this eccentricity have to be considered. This is further described in section 5.

2.2 Schematic example of non-nodal loads on upper chord

Consider the roof truss in Figure 2.1 subjected to a distributed load. For simplicity, only an interior section is cut out from the truss and treated here, see Figure 2.2.

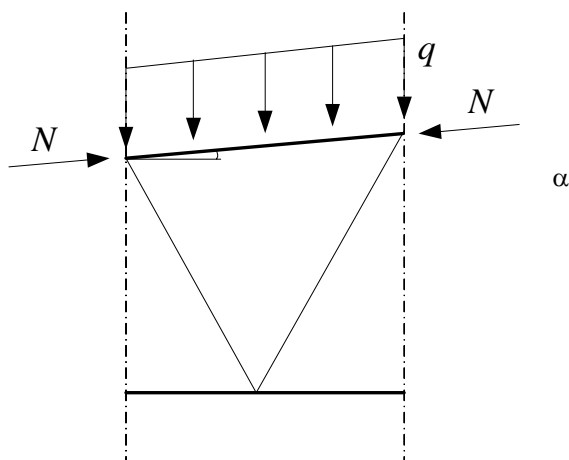


Figure 2.2 Section of the truss considered in the example.

The upper chord is continuous and can be treated as a beam clamped at both ends, see Figure 2.3. This is a simplification and of course it is more accurate to carry out a computer calculation of a continuous beam to evaluate the moments for a continuous chord.

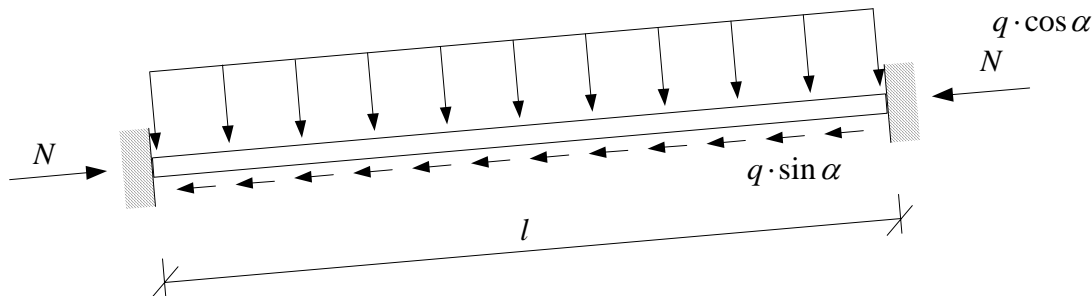


Figure 2.3 Upper chord treated as a beam clamped at both ends.

With the simplification shown in Figure 2.3 the moment distribution can be obtained as shown in Figure 2.4.

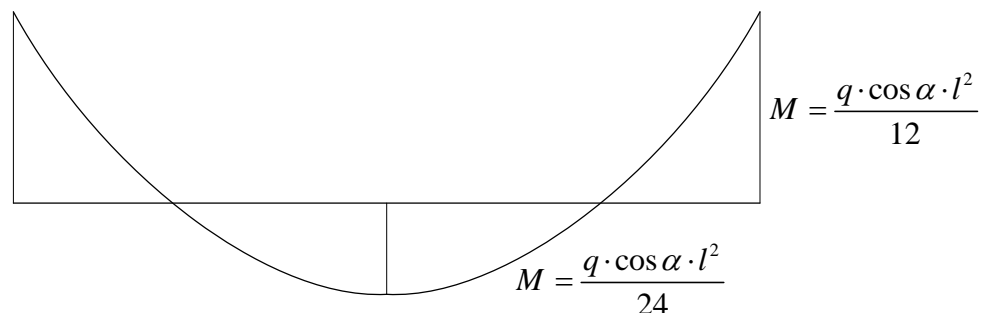


Figure 2.4 Moment distribution in the upper chord.

The upper chord should be verified for both bending moment and axial force. In this case, where the roof sheeting is attached to the upper chord acting as a lateral support for out of plane deformation, the member needs to be checked for flexural buckling in the plane of the truss according to §6.3 of EN 1993-1-1 [1]. The roof sheeting is assumed to be in structural class 1 or 2 according to EN 1993-1-3 [2]. If the roof sheeting is in structural class 3 it can not act as lateral support and the upper chord has to be checked for lateral buckling as well.

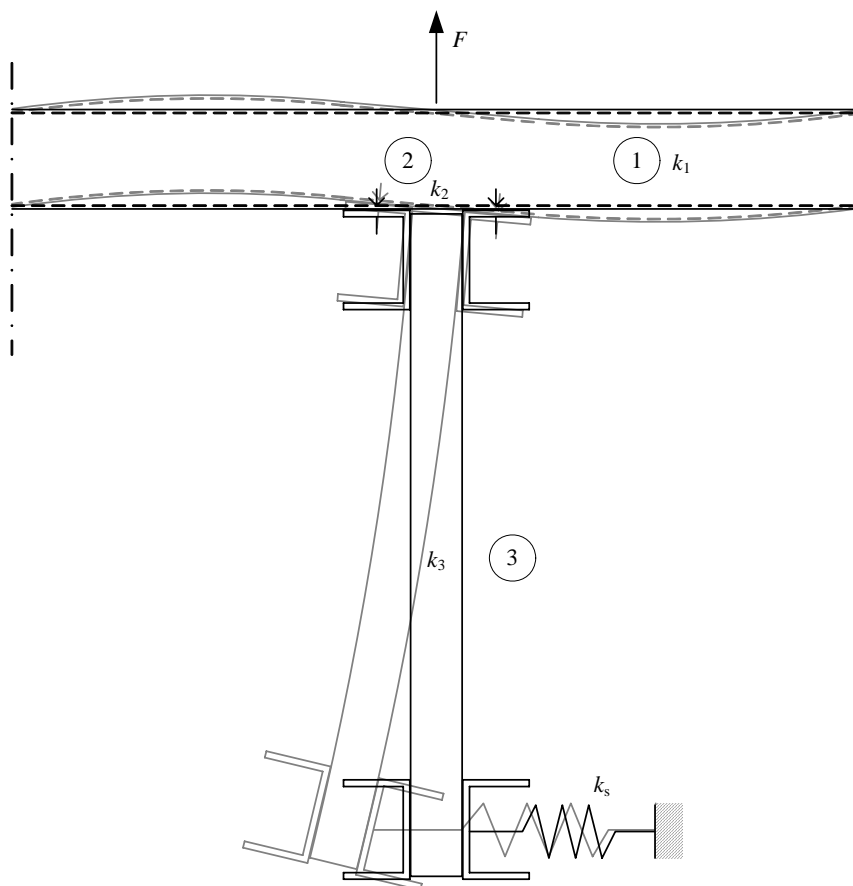
The design model above is also valid for purlin positions between nodes but the support conditions change with the positions and magnitudes of the loads on the upper chord. For a simple and safe sided design when the purlin positions are not known the bending moment may be taken as $wL^2/6$, where L is the length between the truss nodes and w is the sum of all point loads applied perpendicular to the chord between the purlins divided by the purlin distance.

3. Design of lower chord

If the load case including dead load and imposed load is considered, the lower chord are normally in tension and only the tension resistance needs to be checked for.

For different reasons eccentricity can be present due to the joints design between the lower chord and the web members. If this is the case the moments arising from this eccentricity have to be considered. This is further described in section 5.

However, wind loads causing external suction or sometimes also internal pressure in buildings with low pitch roofs have to be carefully considered when designing a roof truss. The parts of the truss that act as ties under dead and imposed load can be severely overstressed when subjected to compression, i.e. reversed loading is important to take into account in the design of a roof truss. When the bottom chord is loaded in compression, lateral buckling of the bottom chord can take place. It is often possible to confirm the strength of the bottom chord without bracing by taking into account the stiffness of the connected parts. Consider the example shown in Figure 3.1 with roof sheeting in structural class 1 or 2 according to EN 1993-1-3.



Key:

- 1 Roof sheeting, with stiffness, k_1 ,
- 2 Roof to truss connection, with stiffness, k_2 ,
- 3 Truss, with stiffness, k_3 .

Figure 3.1 Example of a part of a roof structure subjected to reversed loading

If the connection between web members and chords was designed with gusset plates, if for example other sections are used, the stiffness in those connections should be considered as well.

The fictitious spring holding the bottom chord has a spring stiffness, k_s , calculated as

$$k_s = \frac{1}{\frac{1}{k_1} + \frac{1}{k_2} + \frac{1}{k_3}} \quad (1)$$

where

k_1 is the stiffness of the roof sheeting

k_2 is the stiffness of the roof to truss connection, i.e. the screws

k_3 is the bending stiffness of the web members in the truss

All stiffnesses are expressed per unit length, i.e. they have dimension force per length square.

The roof sheeting stiffness, k_1 , is calculated as

$$k_1 = \frac{1}{\delta} \quad (2)$$

Assume a force of 1 per unit length, according to Figure 3.2 yielding a moment of

$$M = h \cdot 1 \quad (3)$$

and an angle, θ

$$\theta = \frac{M \cdot l_{\text{roof}}}{2 \cdot EI_{\text{roof}}} = \frac{h \cdot 1 \cdot l_{\text{roof}}}{2 \cdot EI_{\text{roof}}} \quad (4)$$

The rotation angle, θ , is calculated under the assumption that the trusses next to each other are buckling in opposite directions.

The bending stiffness of the sheeting is per unit width.

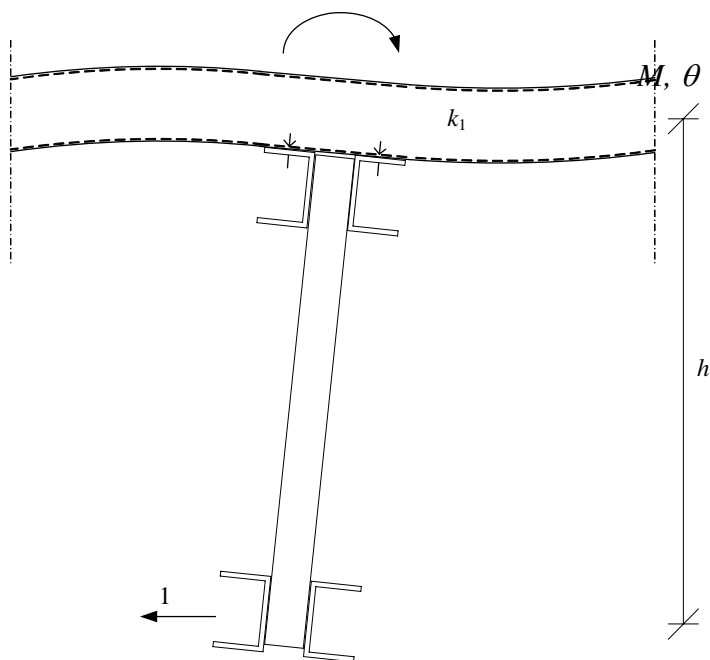


Figure 3.2 Explanation of how to calculate the stiffness of the sheeting, k_1 .

The displacement due to this unity force is determined as

$$\delta = h \cdot \theta = h \cdot \frac{h \cdot l_{\text{roof}}}{2 \cdot EI_{\text{roof}}} \quad (5)$$

and the stiffness, k_1 , is obtained as

$$k_1 = \frac{2 \cdot EI_{\text{roof}}}{h^2 \cdot l_{\text{roof}}} \quad (6)$$

The stiffness of the roof to truss connection, k_2 , is determined according to [§10.1.5 of EN1993-1-3](#). This is the largest contribution to the flexibility. The screws will be loaded in tension, which gives rather large deformations as the sheeting is loaded perpendicular to its plane. In order to get a reasonable stiffness the screws should be placed in zigzag and if this is not sufficient two screws per trough should be used, which double the stiffness.

The bending stiffness out of plane of the web members is calculated as

$$k_3 = \frac{1}{\delta} \quad (7)$$

where δ can be determined as the displacement due to a unity load, see Figure 3.3, according to equation (8).

$$\delta = \frac{1 \cdot l_1 \cdot l_d^3}{3 \cdot EI_d} \quad (8)$$

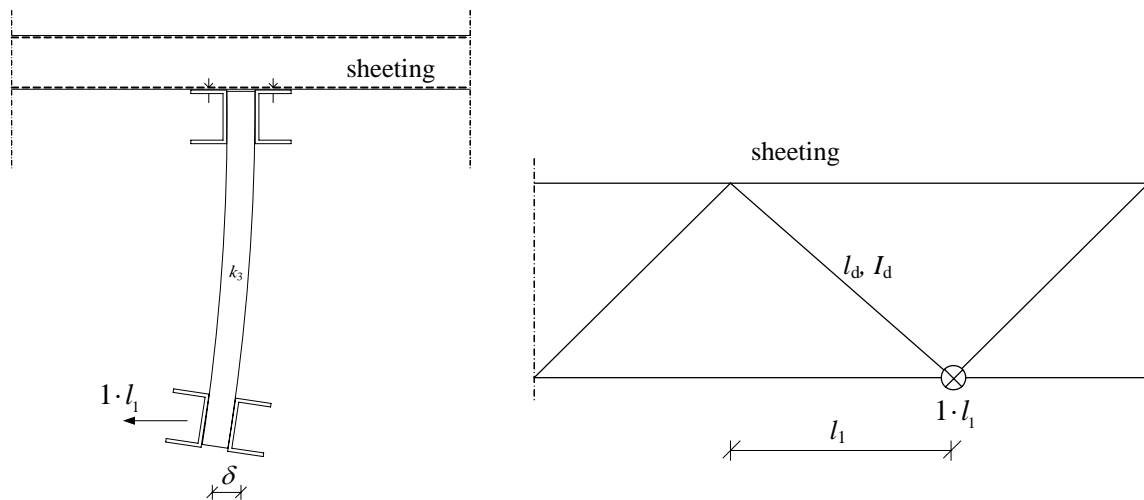


Figure 3.3 Explanation of how to calculate the stiffness of the web members, k_3 .

The stiffness, k_3 , can then be determined as

$$k_3 = \frac{3 \cdot EI_d}{l_1 \cdot l_d^3} \quad (9)$$

The critical buckling load, N_{cr} , is determined according to

$$N_{cr} = 2\sqrt{EI \cdot k_s} \quad (10)$$

assuming that the lower chord consists of one single profile. If the lower chord consists of two battened profiles a reduced stiffness should be used that considers the local bending deformations between the battens.

The following procedure is an extension of the design procedure in [§6.4 of EN 1993-1-1](#) to the case of a battened lower chord with continuous elastic lateral restraint. Notations follow [§6.4](#) and only the changes to the procedure are given here. It is assumed that the battens are welded to the chords and at least twice as long as their widths making them stiff enough to neglect their flexibility.

The buckling length, l_c , is determined according to

$$l_c = \pi \sqrt[4]{\frac{EI_{\text{eff}}}{k}} \quad (10)$$

and the shear stiffness with stiff battens

$$S_v = \frac{2\pi^2 EI_{\text{ch}}}{a^2} \quad (11)$$

where a is the centre distance between the battens.

The critical axial force can be calculated as:

$$N_{cr} = \sqrt{kEI_{eff}} \left[2 - \frac{\sqrt{kEI_{eff}}}{S_v} \right] \quad \text{if } S_v / \sqrt{kEI_{eff}} > 1 \quad (12)$$

$$N_{cr} = \frac{kEI_{eff}}{S_v} \quad \text{if } S_v / \sqrt{kEI_{eff}} \leq 1 \quad (13)$$

The formula for M_{Ed} in §6.4.1(6) of EN 1993-1-1 is changed to Equation (14) for this case.

$$M_{Ed} = \frac{N_{Ed}e_0 + M_{Ed}^I}{1 - \frac{N_{Ed}}{N_{cr}}} \quad (14)$$

where e_0 is a bow imperfection, $e_0 = \frac{L}{500}$, and M_{Ed}^I is the design value of the maximum moment in the middle of the built-up member according to first order theory.

From this the procedure in §6.4 of EN 1993-1-1 should follow.

The buckling resistance of the lower chord can then be evaluated according to §6.3 of EN 1993-1-1 [1].

If necessary some kind of bracing can be used to stabilize the truss and avoid this lateral buckling problem.

The battens and the individual chords should also be checked according to §6.4 of EN 1993-1-1. Due to the elastic deformations in the battens the members will be subjected to combinations of axial force and bending moment or shear force. The individual chords should be checked at the batten position and at mid-span of a panel with forces according to Figure 6.11 in EN 1993-1-1.

4. Design of web members

Web members in a truss are usually designed to resist only axial forces unless eccentricity in the joints is present, see the section 5 below. The compressed members should be checked for in and out of plane buckling. The resistance to in-plane buckling should be calculated with a buckling length, L_{cr} , equal to 90% of the system length, i.e. $0,9L$. For out of plane buckling the buckling length, L_{cr} , should be taken as the system length, L . This is further described in Annex BB of EN 1993-1-1 [1].

5. Eccentricity

The aim when designing a joint in a truss is to connect the members so that each member's centre of mass will pass through the joint centre. However, this is not always possible and this ends up in eccentricity loads that will give end moments in the members, assuming that the connection can transfer other loads than axial, e.g. a welded joint. In Figure 5.1 an example of such a joint is shown.

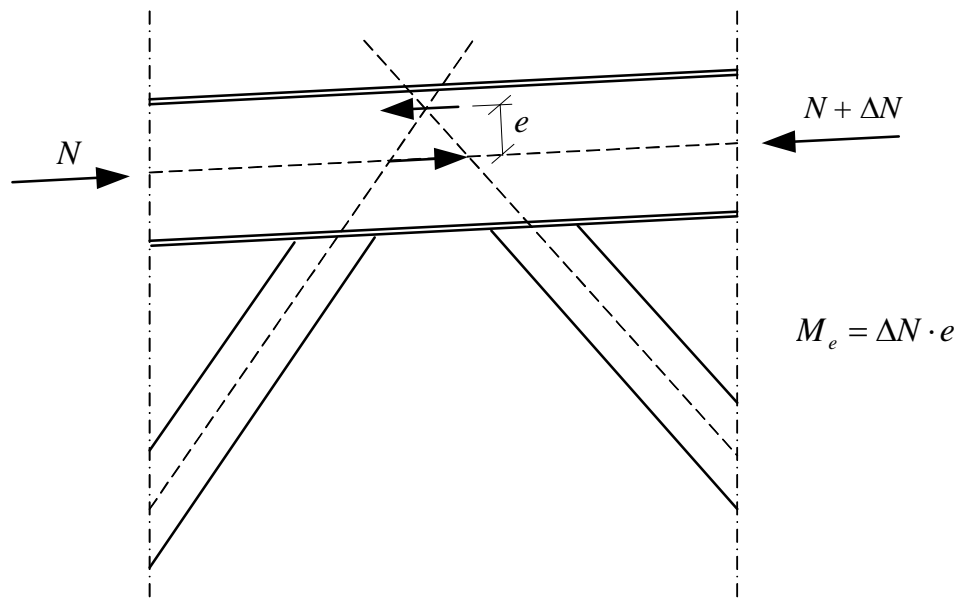


Figure 5.1 Example showing the eccentricity that might appear in a truss joint.

For such case the moment, M_e , due to the eccentricity should be divided equally to the compression chord members either side of the connection, i.e. for the web members only axial forces has to be considered.

6. Examples of truss to column connection and truss splice

The most common joint when connecting the roof truss to columns is a simple joint. It can for example be designed according to Figure 6.1.

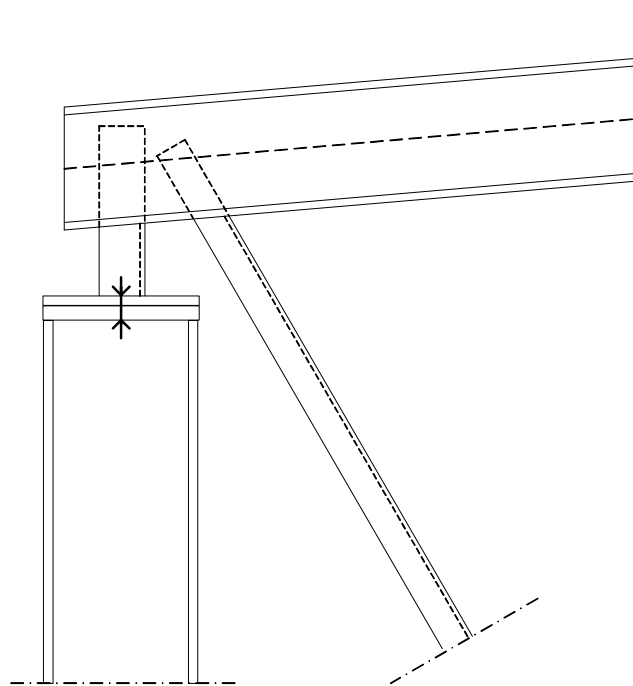


Figure 6.1 Example of a simple joint between truss and column.

The transport of a truss to the construction site can introduce limitations on the length of the truss. In such cases the truss may be divided into two or more parts that will be assembled on site. Figure 6.2 shows how a truss splice can be designed in the middle section of a truss.

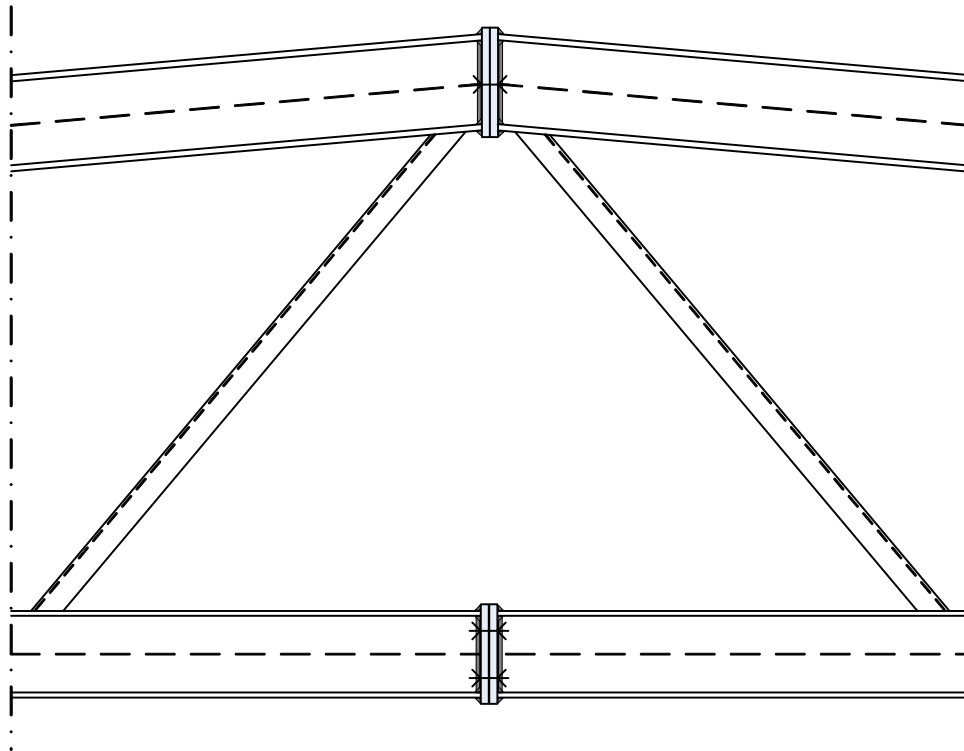


Figure 6.2 Example of a splice in the middle section of a truss.

7. References

- [1] EN 1993-1-1, Design of steel structures, General rules and rules for buildings
- [2] EN 1993-1-3, Design of steel structures, Supplementary rules for cold-formed members and sheeting

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