

Behaviour of connections in steel structures and design of mechanical fasteners and welds according to Eurocode 3

Steel Design 3

Connections

J.W.B. Stark

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Colophon

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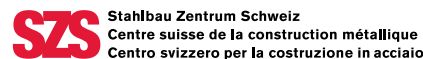
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Connections

Connections in steel structures

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Connections in steel structures

A steel structure cannot exist without bolted and welded connections between the elements of the steel structure itself or with other materials or components. This chapter discusses the role of connections in steel structures, identifies the various components, and provides examples of the most common connections in buildings. Thereafter, important aspects of the design and the detailing are discussed in general. An overview is given of the most important codes and regulations for connections in steel. Finally, the role of the computer within the design of steel structures will be discussed.

NA In general, the term 'connection' is used for the location at which two or more structural elements (like plates and sections) meet and are connected (e.g. by bolting or welding). The focus is on the transfer of the relevant internal forces and moments at the connection.

The term 'joint' is generally used for the zone where two or more structural members (like beams and columns) are interconnected. The focus is on the structural properties (like resistance, stiffness, and rotation capacity).

1.1 The function of connections

Steel structures usually comprise sections and plates that are processed and assembled into structural parts in a fabrication shop, for example, a truss or a part of a bridge deck. The dimensions of these elements are limited, because they usually have to be transported and, subsequently,

1.1 Use of large parts on the construction site.



assembled (with a crane). The maximum dimensions depend on the capacity of the steel fabricator, the corrosion protection, the storage options, the method and route of transport to the construction site, and the capacity of the crane. In general, it is desirable to aim for the largest possible elements. This results in a reduced number of connections on site (fig. 1.1), where the elements are assembled to form the steel structure. So, the steel structure contains connections that are manufactured in the fabrication shop as well as on site. The conditions under which these two types of connections are made strongly vary and this should be taken into account when designing the connections.

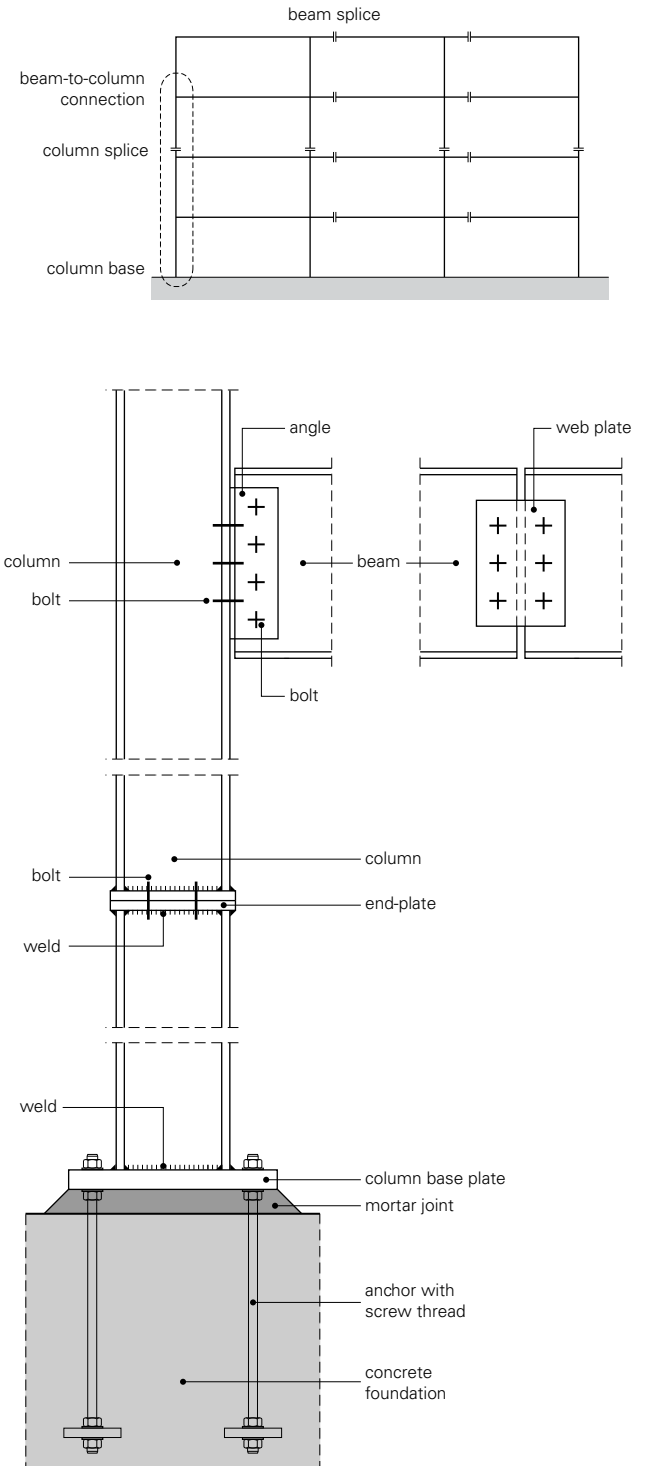
Generally the number of connections in a steel structure is rather large. They are needed in the following situations.

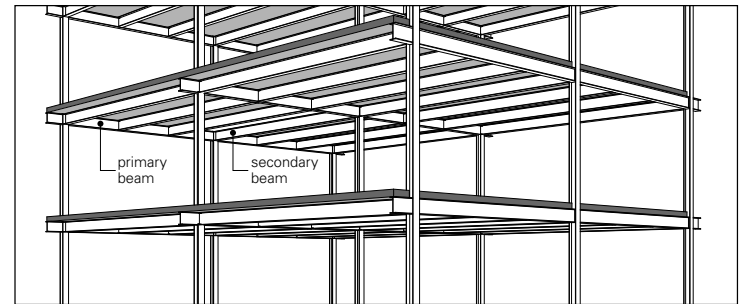
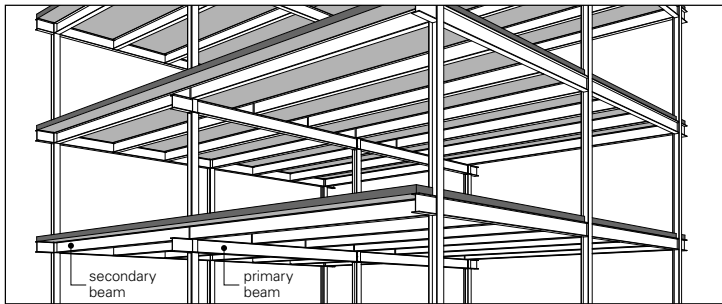
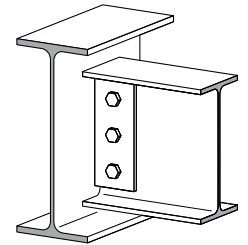
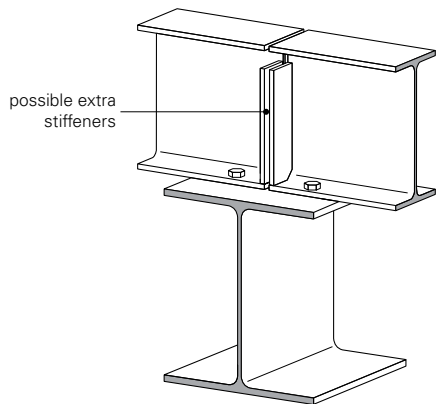
- When structural elements meet from different directions. For instance, this is the case for connections between beams and columns, secondary and primary beams, and members in a truss.
- Connections on site are needed in order to limit the dimensions of the components considering transport, storage, and assembly. As an example: the columns in a building may contain a splice every two or three storeys. When elements are galvanized thermally, the length of the galvanizing bath must be considered. For example, the maximum length of a galvanizing bath in a European galvanizing plant is approximately 15 m.
- When the steel structure has to be connected to structural elements formed from other materials, or connections between different types of steel element. For example, the connection of a steel column to a concrete foundation, the connection of a steel beam to a concrete body such as a lift core, or the connection of steel sheeting to steel trusses;
- When the cross-section of a component changes, for example when the column size reduces at higher stories.

Figure 1.2 shows an example of a steel frame containing four storeys, in which the first three situations apply.

Connections are an important part of any steel structure. The structural properties of the connection affect the strength, stiffness, and stability of the complete steel structure. The number and complexity of connections have a decisive influence on the effort - and therefore, on the cost - that is needed to carry out the design calculations and produce construction drawings. Also, the manufacturing and assembly costs significantly depend on the chosen connection type.

1.2 Locations in a framework where connections are necessary.





1.25 Floor system with beams in two layers and simply supported secondary beams.

1.26 Floor system with beams in one layer, in which the upper side of the secondary beams is lower than the upper side of the primary beams.

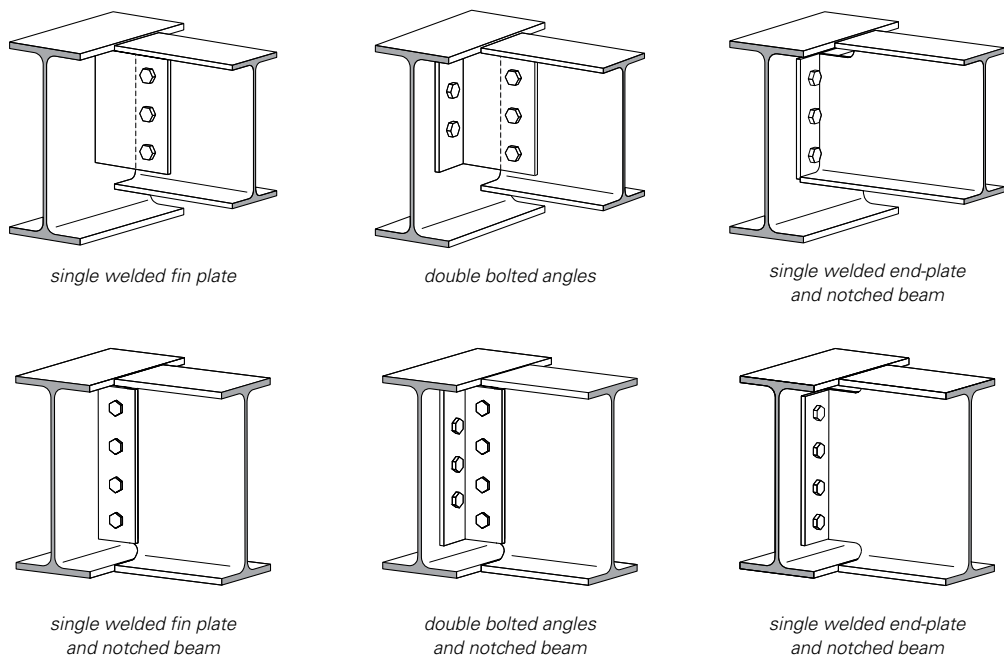
1.3.4 Beam-to-beam connection

In a floor system with beams in two (stacked) layers, the secondary beams can be simply supported by the primary beams (fig. 1.25). The structural design should indicate whether any local stiffeners are necessary.

In a floor system with beams in a single layer, the upper flanges of the secondary beams may be either lower than, or at the same level as, the upper flanges of the primary beams. Flanges at different levels may be used for composite floor slabs. A fin plate connection between the secondary beam and the primary beam is a suitable solution (fig. 1.26). This is a low-cost connection in terms of both fabrication and erection.

Different solutions for the upper flanges of the beams at the same level are shown in figure 1.27. The variants in which the upper flange of the secondary beam – or both flanges – is shortened (notched) are more expensive, but this is sometimes necessary in case of large shear forces. The characteristics of the various beam-to-beam connections shown in this figure correspond to the beam-to-column connections in table 1.18.

In general, non-notched connections with end-plates or angles will provide sufficient restraint against twist of the beam end. Fin plate connections and notched connections are often not rotationally stiff to provide full restraint against twist.



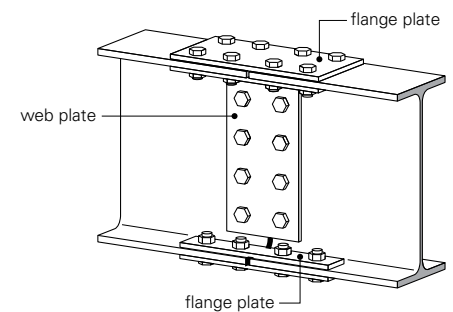
1.27 Floor system with beams in one layer, in which the upper side of the secondary beams is level with the upper side of the primary beams.



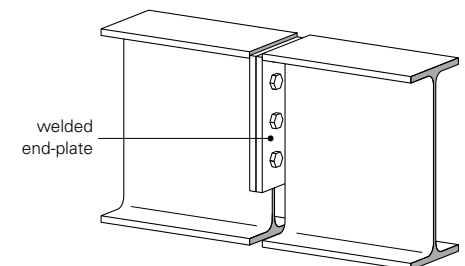
1.3.5 Beam splice

A beam splice is needed when the designed beam is longer than the maximum available (commercial) length, or when an assembly connection is necessary on site, for example due to transportation limitations. In the first case, the beam splice can be welded or bolted in a workshop, whereas a bolted connection is almost always chosen in the second case (for execution on the construction site). A welded beam splice is in principle the same as a welded column splice (see fig. 1.9).

When there is a moment and a shear force at the location of a bolted beam splice, the flanges are connected with flange plates and the webs with web plates (fig. 1.28). Flange plates can be applied either single-sided or double-sided. When in the frame design a hinge is assumed at the location of the splice – for example in a Gerber beam – only shear force needs to be transferred. In that case, only web plates, or beam end-plates, are needed (figure 1.29).



1.28 Beam splice for resisting a bending moment and shear force.



1.29 Beam splice for resisting a shear force alone.

Example 2.3

- **Given.** The web of a notched HEA 200 beam in steel grade S235 is connected with five M24 bolts (with $d_0 = 26$ mm) in two variations (a and b) for the positioning of the bolt group (fig. 2.37).
- **Question.** Determine for each variant the design value for the block tearing resistance $V_{\text{eff},2,\text{Rd}}$.
- **Answer.** In both variants the tearing pattern is un-symmetrical. In addition, the side of the tear pattern loaded in tension is eccentrically loaded. The design value of the block tearing resistance $V_{\text{eff},2,\text{Rd}}$ follows from equation (2.12). For variant a:

$$A_{\text{nt}} = t_w \left(e_2 - \frac{1}{2} d_0 \right) = 13 \cdot \left(45 - \frac{1}{2} \cdot 26 \right) = 416 \text{ mm}^2$$

$$A_{\text{nv}} = t_w \left(\ell_v + a_1 - n d_0 \right) = 13 \cdot (280 + 45 - 4,5 \cdot 26) = 2704 \text{ mm}^2$$

$$V_{\text{eff},2,\text{Rd}} = \frac{0,5 f_u A_{\text{nt}}}{\gamma_{\text{M2}}} + \frac{f_y A_{\text{nv}}}{\sqrt{3} \gamma_{\text{M0}}} = \frac{0,5 \cdot 360 \cdot 416 \cdot 10^{-3}}{1,25} + \frac{\frac{235 \cdot 2704 \cdot 10^{-3}}{\sqrt{3}}}{1,0} = 427 \text{ kN}$$

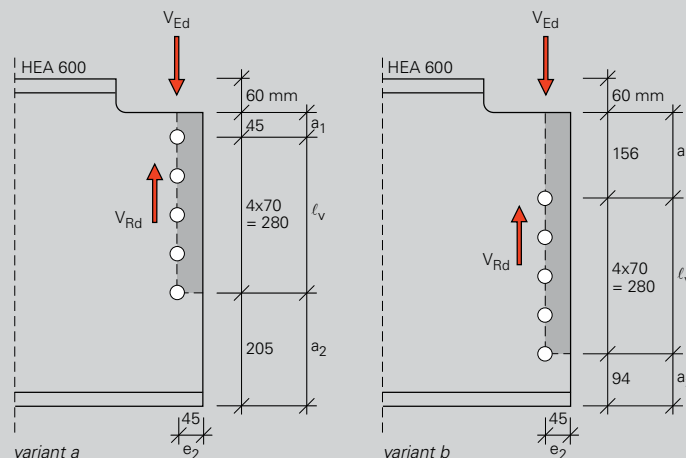
For variant b:

$$A_{\text{nt}} = t_w \left(e_2 - \frac{1}{2} d_0 \right) = 13 \cdot \left(45 - \frac{1}{2} \cdot 26 \right) = 416 \text{ mm}^2$$

$$A_{\text{nv}} = t_w \left(\ell_v + a_1 - n d_0 \right) = 13 \cdot (280 + 156 - 4,5 \cdot 26) = 4147 \text{ mm}^2$$

$$V_{\text{eff},2,\text{Rd}} = \frac{0,5 f_u A_{\text{nt}}}{\gamma_{\text{M2}}} + \frac{f_y A_{\text{nv}}}{\sqrt{3} \gamma_{\text{M0}}} = \frac{0,5 \cdot 360 \cdot 416 \cdot 10^{-3}}{1,25} + \frac{\frac{235 \cdot 4147 \cdot 10^{-3}}{\sqrt{3}}}{1,0} = 623 \text{ kN}$$

This illustrates that the block tearing resistance strongly depends on the position of the bolts.



2.37 Notched beam connections with two variants of bolt placements.

2.5 Resistance of bolts

The resistance of individual, non-preloaded bolts depends on the way the bolt is loaded, namely:

- bolts loaded in shear (section 2.5.1);
- bolts loaded in tension (section 2.5.2);
- bolts loaded in shear and tension (section 2.5.3).

Design checks are given in table 3.2 of EN 1993-1-8. For bolts with a countersunk head some of the checks are modified (section 2.5.4).

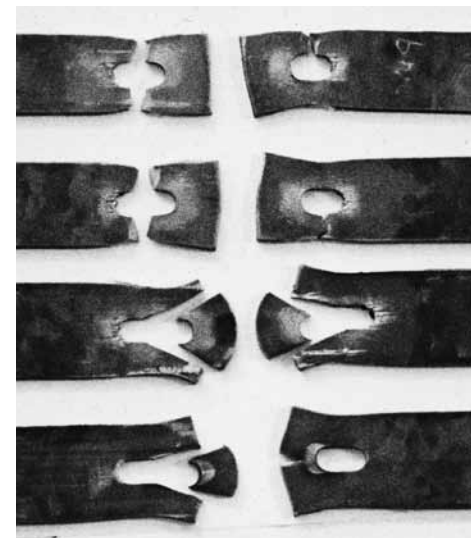
2.5.1 Bolts loaded in shear

A connection with non-preloaded bolts loaded in shear – see section 2.1.1 and figure 2.6 – will settle until the sides of the holes press against the bolt shafts. This settlement is reflected in the load/deformation behavior of the connection. The force distribution is schematically shown in figure 2.7a (category A). Different failure modes can determine the resistance of a connection, depending on the bolt diameter, the plate thickness, and the material characteristics of the bolts and plates (fig. 2.38 and 2.39).

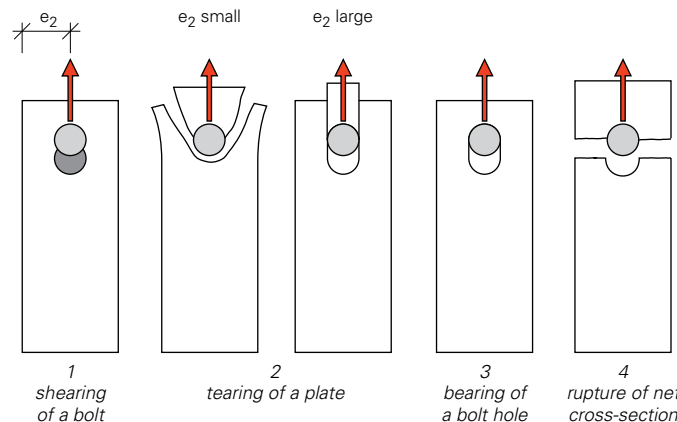
In principle, each possible failure mode must be checked individually; the failure mode with the lowest resistance is critical. However, this does not mean that it does not matter which failure mode governs. Shearing of a bolt (option 1 in fig. 2.39) occurs with relatively small deformation, while failure in bearing of a plate provides a larger deformation capacity. A design in which bearing is critical is preferable. EN 1993-1-8, cl. 3.6.1 combines tearing of a plate (option 2 in fig. 2.39) and bearing of a bolt hole (option 3 in fig. 2.39) in one failure mode. This combined failure mode is referred to as bearing resistance.

For non-preloaded bolts loaded in shear the following resistances are therefore important:

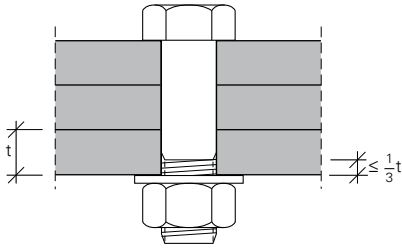
- shear resistance (shearing of a bolt);
- bearing resistance (tearing of a plate or bearing of a bolt hole).



2.38 Different failure modes of failed test pieces.



2.39 Possible failure modes of connections loaded in shear.



3.15 Maximum length of the thread for fit bolts.

In EN 1993-1-8, cl. 3.6.1, the following conditions are given for the use of fit bolts:

- paragraph (6): fit bolts must be designed as bolts in normal sized holes;
- paragraph (7): the shear plane must not pass through the thread of the bolt;
- paragraph (8): the length of the thread in bearing may not exceed 1/3 of the plate thickness (fig. 3.15);
- paragraph (9): the hole tolerance must comply with EN 1090-2, cl. 6.6.2, in which the tolerance class H111 is prescribed according to EN-ISO 286-2.

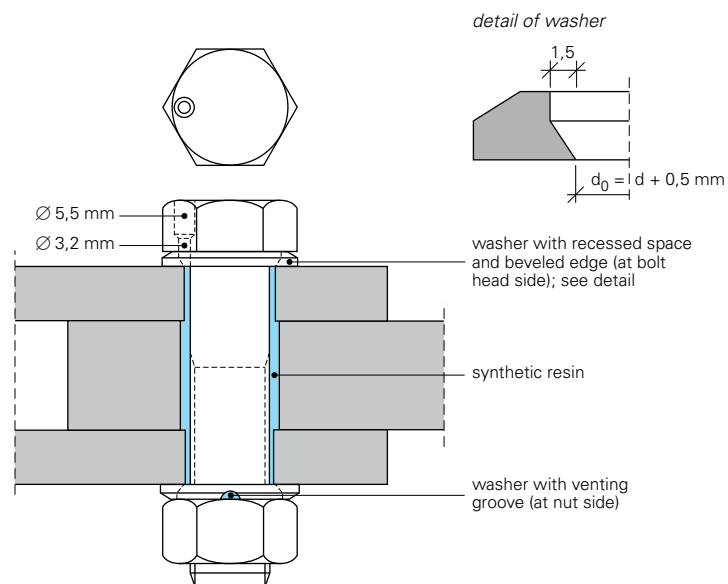
For the design of a connection with fit bolts the same rules as for 'normal' non-preloaded bolts apply, see *Connections 2* (Bolts in clearance holes), section 2.5 and equation (3.4) to (3.8), and (3.13) and (3.14).

3.2.2 Injection bolts

Injection bolts are bolts of class 8.8 or higher with an opening in the head of the bolt (fig. 3.16). A synthetic resin is injected through this opening into the space between the bolt shaft and the sides of the hole. In this way, a connection is created in which relative movement of the plates is prevented. Special washers under the head and the nut are necessary to be able to inject the resin. When the bolt and washers are assembled, attention must be paid to the correct positioning of the washers. The closing washer, which has a recessed space and a beveled edge, must be underneath the bolt head, and the washer with the venting groove must be underneath the nut. Appendix K of EN 1090-2 provides rules for the execution of injection bolts.



3.16 Non-preloaded injection bolt.



One disadvantage of injection bolts is that injection is not possible in unfavourable weather conditions, and at low temperatures. The bolts must be fully tightened before injection, and the sides of the hole must be clean and dry. The preparation of the synthetic resin and the injection process itself are rather simple, although due care is necessary.

Injection bolts can be preloaded, in which case a connection is created in which the load is not only transferred by friction between the contact surfaces but also by bearing of the synthetic resin and shear of the bolt shaft. Non-preloaded injection bolts are mainly used in maintenance and repair work, and in slip-resistant connections in which the slip factor of the contact surfaces is low. The latter is for example the case with hot-dipped galvanized or painted structural elements. Preloaded injection bolts are particularly suitable for dynamically loaded structures, or for structures with varying loads, such as traffic and railway bridges.

Non-preloaded injection bolts

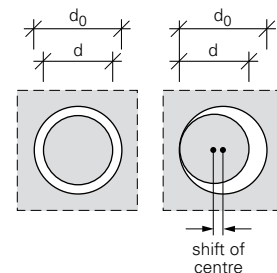
Load transfer for non-preloaded injection bolts is achieved through bearing and shear. Non-preloaded bolts belong to category A. The design values of the tension resistance $F_{t,Rd}$ and shear resistance $F_{v,Rd}$ are equal to those of non-preloaded bolts in normal holes, see *Connections 2*, section 2.5.1 and 2.5.2.

The design value of the bearing resistance $F_{b,Rd,resin}$ depends on the properties of the synthetic resin. Under the influence of excessive constant pressure creep would occur in the resin, meaning that in the long term the bolt would come to lie eccentrically in the bolt hole, with resulting slip (fig. 3.17). To prevent creep, the design value for the bearing resistance of the synthetic resin should be based on a long term bearing stress for which the slip does not exceed 0,3 mm.

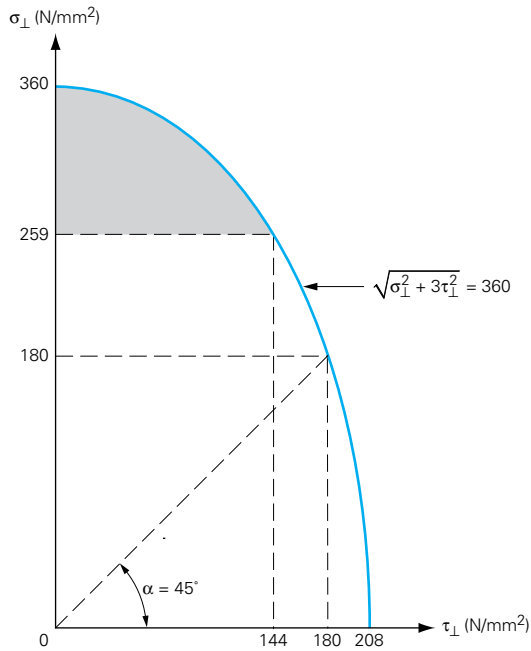
To determine the design value for the bearing resistance, the bearing stress must be known for each connected part. In the connection of figure 3.18, for example, the bearing stress σ_1 in the intermediate plate and the bearing stress σ_2 in the outer plates must both be known. The bearing stress σ_1 will be critical when $t_1/t_2 < 2$. The bearing stress increases with a decreasing ratio of t_1/t_2 and this effect is taken into account with a factor β . A similar situation occurs when four or more plates are connected, but this situation is not treated in EN 1993-1-8. Conservatively, one can assume that $\beta = 1$.

According to EN 1993-1-8, cl. 3.6.2.2(5), the design value for the bearing resistance of the synthetic resin $F_{b,Rd,resin}$ is determined as follows:

$$F_{b,Rd,resin} = \frac{k_t k_s d t_{b,resin} \beta f_{b,resin}}{\gamma_{M4}} \quad \text{with} \quad \gamma_{M4} = 1,0 \quad (3.18)$$



3.17 Shift of an injection bolt in the bolt hole due to creep of the synthetic resin.



4.36 Relationship between σ_{\perp} and τ_{\perp} , for $\tau_{\parallel} = 0$, for grade S235 steel.

Equation (4.2) is only critical when σ_{\perp} is clearly higher than τ_{\perp} . The requirement in equation (4.2) is necessary to avoid that the side area (see fig. 4.34) becomes more critical than the throat section. This requirement is not critical in normal cases for symmetric fillet welds. For asymmetric fillet welds and partial penetration butt welds, that can be designed as fillet welds, equation (4.2) may be critical. The factor 0,9 in equation (4.2) is based on fracture mechanics studies on partial penetration butt welds.

Figure 4.36 shows the relationship between the stresses σ_{\perp} and τ_{\perp} for grade S235 steel and for $\tau_{\parallel} = 0$. With $\gamma_{M2} = 1,25$, $\beta_w = 0,8$ and $f_u = 360 \text{ N/mm}^2$ equation (4.1) is:

$$\sqrt{\sigma_{\perp}^2 + 3\tau_{\perp}^2} \leq \frac{f_u}{\beta_w \gamma_{M2}} = \frac{360}{0,8 \cdot 1,25} = 360 \text{ N/mm}^2$$

Some typical values are:

$$\sigma_{\perp} = 0 \text{ N/mm}^2 \Rightarrow \sqrt{3\tau_{\perp}^2} = 360 \Rightarrow \tau_{\perp} = 208 \text{ N/mm}^2$$

$$\sigma_{\perp} = \tau_{\perp} \Rightarrow \sqrt{4\tau_{\perp}^2} = 360 \Rightarrow \sigma_{\perp} = \tau_{\perp} = 180 \text{ N/mm}^2$$

$$\sigma_{\perp} = \frac{0,9 f_u}{\gamma_{M2}} = \frac{0,9 \cdot 360}{1,25} = 259 \text{ N/mm}^2 \Rightarrow \sqrt{259^2 + 3\tau_{\perp}^2} = 360 \Rightarrow \tau_{\perp} = 144 \text{ N/mm}^2$$

In a similar way, these characteristic values can also be determined for other steel grades. Figure 4.37 gives practical formulae for calculating the stresses for some common load cases.

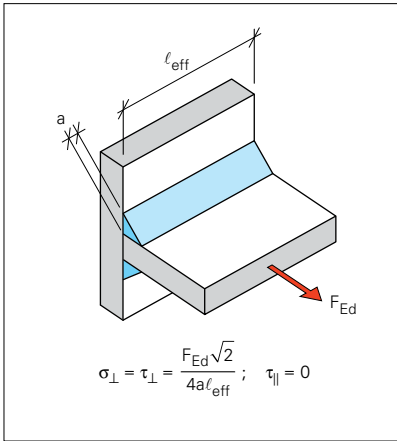
Simplified method

In the simplified method, according to EN 1993-1-8, cl. 4.5.3.3 a check is made to ensure that the mean stress – independent of the orientation of the plane of the throat section relative to the applied force – is smaller than the design value of the shear strength of the weld $f_{vw,d}$:

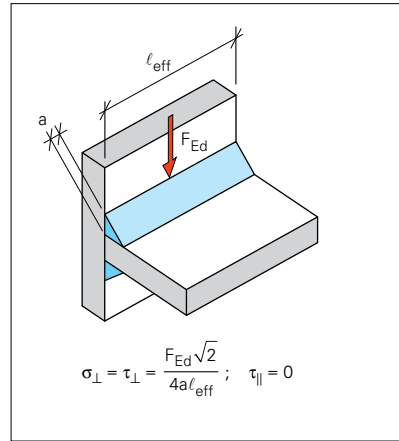
$$\frac{F_{w,Ed}}{a \Sigma \ell_{eff}} \leq f_{vw,d} = \frac{f_u}{\beta_w \gamma_{M2} \sqrt{3}} \quad (4.3)$$

Where $F_{w,Ed}$ is the design value of the applied force per unit length.

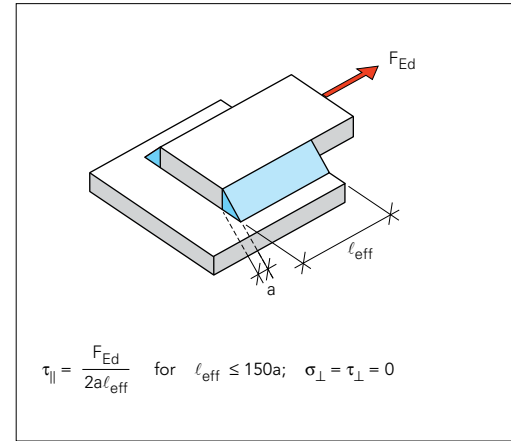
load case 1



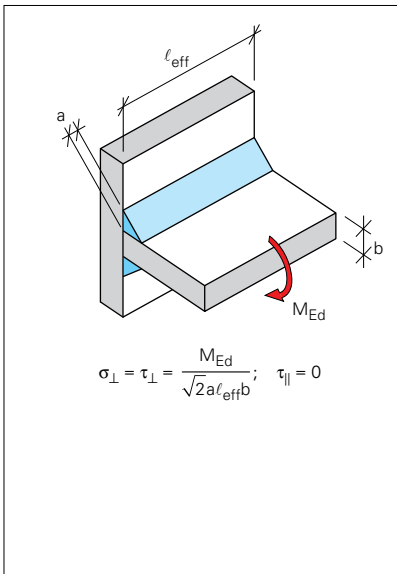
load case 2



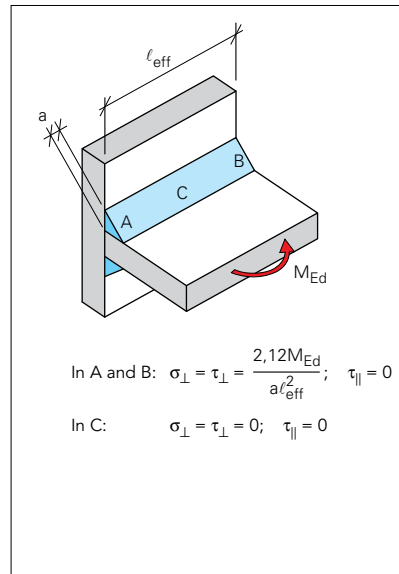
load case 3



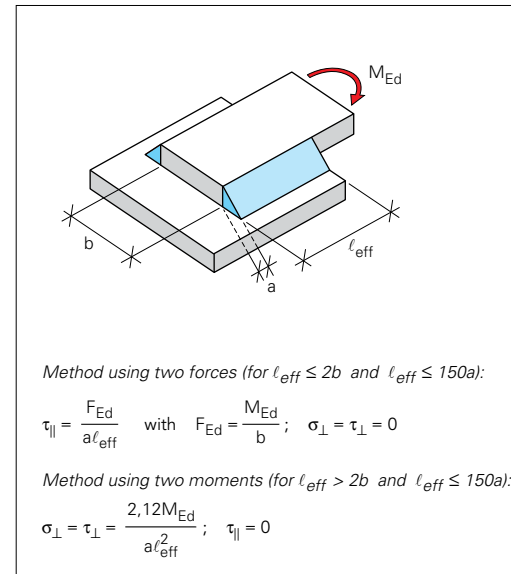
load case 4



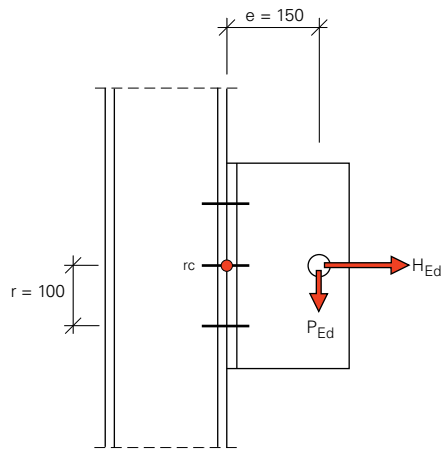
load case 5



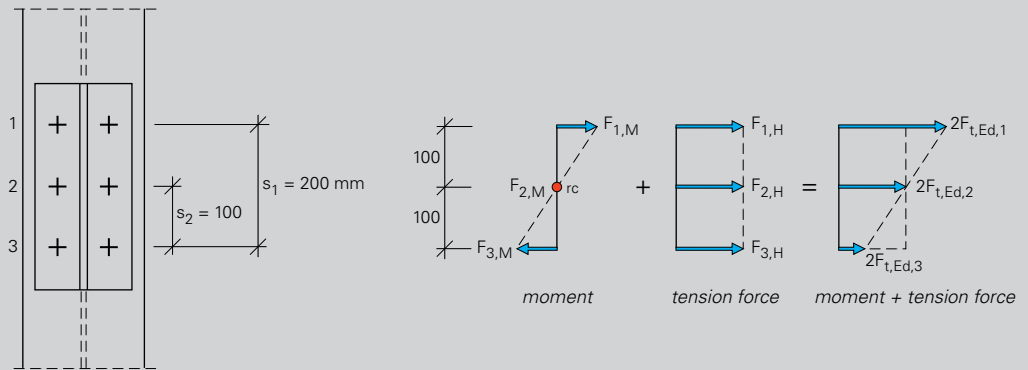
load case 6



4.37 Six common Load cases with the corresponding stresses in the weld.



5.21 Gusset plate bolted to a column with a free centre of rotation.



Example 5.4

- **Given.** A gusset plate is bolted to a column, loaded by a vertical force $P_{Ed} = 200$ kN at a distance $e = 150$ mm to the centre of the connection, and a horizontal force $H_{Ed} = 600$ kN applied at the centre of the bolt group (fig. 5.21).
- **Question.** Determine the maximum tension force $F_{t,Ed}$ in the bolts.
- **Answer.** The distance between the horizontal force and the lower bolt row is $r = 100$ mm. The bending moment on the bolt group due to P_{Ed} is $M_{Ed} = P_{Ed}e = 200 \cdot 150 \cdot 10^{-3} = 30,0$ kNm. From the given loads and geometry:

$$r + \frac{M_{Ed}}{H_{Ed}} = 100 + \frac{30,0 \cdot 10^3}{600} = 150 \text{ mm} < \frac{\sum s_i^2}{\sum s_i} = \frac{200^2 + 100^2}{200 + 100} = 167 \text{ mm}$$

This means that a free centre of rotation is created, see equation (5.15), located at the centre of gravity of the bolt group. Due to the moment M_{Ed} and the horizontal force H_{Ed} the following forces can occur in the bolt rows:

$$F_{1,M} = \frac{M_{Ed}}{s_1} = \frac{30,0 \cdot 10^3}{200} = 150 \text{ kN}; \quad F_{2,M} = 0 \text{ kN}; \quad F_{3,M} = -F_{1,M} = -150 \text{ kN}$$

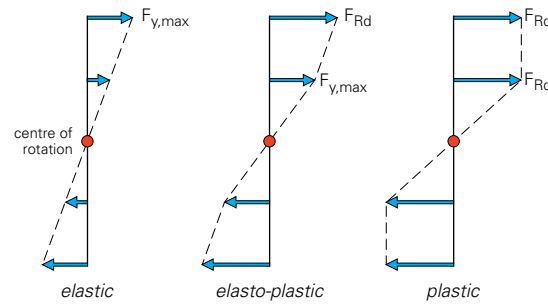
$$F_{1,H} = F_{2,H} = F_{3,H} = \frac{H_{Ed}}{n} = \frac{600}{3} = 200 \text{ kN}$$

The forces in bolts 1, 2, and 3 are:

$$F_{t,Ed,1} = \frac{F_{1,M} + F_{1,H}}{2} = \frac{150 + 200}{2} = 175 \text{ kN} \quad \text{and} \quad F_{v,Ed,1} = \frac{P_{Ed}}{6} = \frac{200}{6} = 33,3 \text{ kN}$$

$$F_{t,Ed,2} = \frac{F_{2,M} + F_{2,H}}{2} = \frac{0 + 200}{2} = 100 \text{ kN} \quad \text{and} \quad F_{v,Ed,2} = \frac{P_{Ed}}{6} = \frac{200}{6} = 33,3 \text{ kN}$$

$$F_{t,Ed,3} = \frac{F_{3,M} + F_{3,H}}{2} = \frac{-150 + 200}{2} = 25 \text{ kN} \quad \text{and} \quad F_{v,Ed,3} = \frac{P_{Ed}}{6} = \frac{200}{6} = 33,3 \text{ kN}$$



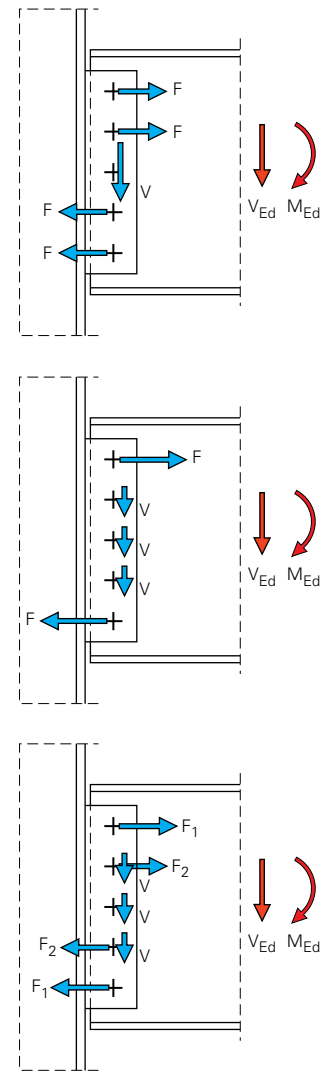
5.22 Elastic and plastic force distribution in a bolt group.

5.2.3 Non-linear distribution of bolt forces

So far in this chapter a linear ('elastic') force distribution has been assumed. However, when designing bolted connections plastic behaviour may also be assumed according to EN 1993-1-8, cl. 3.12(1). As the applied load is increased the force in the most heavily loaded bolt at some point reaches its limiting value F_{Rd} (fig. 5.22). As the applied load is increased further, the force in this bolt will remain constant, while the deformation of the connection increases. This deformation results in increasing forces in the bolts that are nearer to the centre of rotation. Eventually, these bolts will also reach their limiting load capacity. The resistance of the connection as a whole can increase until all bolts reach their limiting load capacity, provided the deformation capacity of the bolts is adequate.

In principle, with a plastic distribution of bolt forces, it may be assumed that each bolt force acts perpendicularly to a radius from the centre of rotation. A non-linear distribution of bolt forces can also occur when the design is based on the equilibrium method. Any force distribution that satisfies equilibrium – without the limiting load capacity of any bolt being exceeded and the deformation capacity is adequate – provides a lower bound for the ultimate load capacity of the connection (fig. 5.23). This allows some simplifying assumptions to be made, as explained below. When a bending moment and a shear force act on a bolt pattern it may be assumed that the bolts close to the centre of rotation contribute little towards the moment transfer because of their small lever arms. It is assumed that these bolts only transfer the (entire) shear force. The more distant bolts are assumed to only transfer the (entire) moment. These assumptions are only valid if the components of the connection have sufficient deformation capacity, as is the case when the plate material governs the resistance of the connection, rather than the bolts or welds. The deformation capacity of the plate material, either in bending or bearing, is much greater than that of the bolts. That is why EN 1993-1-8, cl. 3.12(2), indicates that a linear force distribution must be used when the shear resistance $F_{v,Rd}$ of the bolts is smaller than the bearing resistance $F_{b,Rd}$. When applying a plastic approach the following basic principles are important:

- choose a realistic internal force distribution that balances the external load, so that the three equilibrium conditions are met: $\Sigma V = 0$, $\Sigma H = 0$ and $\Sigma M = 0$;
- make sure that the bolts are strong enough, and use relatively large diameter bolts so that their shear resistance $F_{v,Rd}$ is larger than the bearing resistance $F_{b,Rd}$.



5.23 Possible force distributions over the bolts in a fin plate connection loaded in bending and shear.

Connections

This textbook covers the behaviour of connections in steel structures and the design of mechanical fasteners and welds according to EN 1993 (Eurocode 3).

- Chapter 1 is about the function of connections in a steel structure and describes the different components. It also discusses the most important aspects of design and detailing and the role of the computer in the analysis and design of a steel structure
- Chapter 2 deals with the most widely used mechanical fasteners in steel structures: bolts in clearance holes (normal holes, oversized holes and slotted holes). The characteristics of bolts are discussed as well as the design rules for checking connections in steel structures with these type of bolts.
- Chapter 3 deals with the design rules for checking mechanical fasteners when bolts in clearance holes are not allowed or not desirable. This concerns preloaded bolts, fit bolts, injection bolts and pins. The design rules for rivets are also briefly discussed.
- Chapter 4 gives an overview of the most important welding processes for steel structures and of the most commonly used weld shapes. It then discusses the welding procedures to execute a good welded joint and the methods for detecting and assessing any welding defects. Finally, the design principles of fillet welds and butt welds are covered.
- Finally, chapter 5 deals with the design of complete connections that are common in steel structures of buildings. A complete connection (mostly called a 'joint') comprises all the individual steel parts – such as end plates, stiffeners and angles – and the individual connectors, which together ensure the transfer of forces and moments between the connected elements. It is important how the forces on the individual elements of the connection can be determined from the external forces on the connection.

Connections is effective as a textbook for students and as a reference guide to Eurocode 3 for practising structural engineers.

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