

## 5 SIMPLE CONNECTIONS

### 5.1 Design Philosophy

Connection design depends very much on the designer's decision regarding the method by which the structure is analysed. The latest draft of Eurocode 3 [prEN 1993-1-8: 2003] includes four approaches for the design of a structure in which the behaviour of the connection is fundamental. These design methods are defined as simple design, semi-continuous design, continuous design and experimental verification. Elastic, plastic and elastic-plastic methods of global analysis can be used with any of the first three approaches, see Chapter 4.

The joints are classified according to the method of global analysis and the type of joint model. This chapter is concerned with the design of simple joints where the method of global analysis may be elastic, rigid-plastic or elastic-plastic. Simple connections are defined as those connections that transmit end shear only and have negligible resistance to rotation and therefore do not transfer significant moments at the ultimate Limit State [BCSA 1996]. This definition underlies the design of the overall structure in which the beams are designed as simply supported and the columns are designed for axial load and the small moments induced by the end reactions from the beams. In practice, however, the connections do have a degree of fixity, which although not taken in to account in the design is often sufficient to allow erection to take place without the need for temporary bracing. The following four principal forms of simple connection are considered in this section:

- Double angle web cleats
- Flexible end-plates (header plates)
- Fin plates
- Column splices

To comply with the design assumptions, simple connections must allow adequate end rotation of the beam as it takes up its simply supported deflected profile and practical lack of fit. At the same time this rotation must not impair the shear and tying (for structural integrity – see below) capacities of the connection. In theory a 457 mm deep, simply supported beam spanning 6,0 m will develop an end rotation of 0,022 radians (1,26°) when carrying its maximum factored load. In practice this rotation will be considerably smaller because of the restraining action of the connection. When the beam rotates it is desirable to avoid the bottom flange of the beam bearing against the column as this can induce large forces in the connection. The usual way of achieving this is to ensure that the connection extends at least 10 mm beyond the end of the beam.

### 5.2 Structural Integrity

The partial collapse of Ronan Point in the UK in 1968 alerted the construction industry to the problem of progressive collapse arising from a lack of positive attachment between principal elements in a structure [BS 5950]. Structures are required to have a minimum robustness to resist accidental loading. One method of achieving this is by tying all the principal elements of a structure together. This means that the beam-to-column connections of a steel frame must be capable of transferring a horizontal tying force in order to preserve the integrity of the structure and prevent progressive collapse in the event of accidental damage.

### 5.3 Design Procedures

The design of these simple connections is based on the principles and procedures adopted in Eurocode 3 Part 1.8 [prEN 1993-1-8: 2003]. Typical practice in terms of type and nature of fixings and connection types varies between the member countries of the Community. This section deals with the general principles applicable to all types of simple connection. Detailed design procedures in the form of a check list are presented for flexible end plates and fin plates. The relevant formulae may be found in the Eurocode.

### 5.4 Beam-to-column connections

#### 5.4.1 Double angle web cleats

Typical bolted double angle cleat connections about both the major and minor axis of a column are shown in Figure 5.1. These types of connection are popular because they have the facility

to provide for minor site adjustments when using untorqued bolts in 2 mm clearance holes. Normally the cleats are used in pairs. Any simple equilibrium analysis is suitable for the design of this type of connection. The one recommended in this publication assumes that the line of action of shear transfer between the beam and the column is at the face of the column. Using this model the bolt group connecting the cleats to the beam web must be designed for the shear force and the moment produced by the product of the end shear and the eccentricity of the bolt group from the face of the column. The bolts connecting the cleats to the face of the column should be designed for the applied shear only. In practice the cleats to the column are rarely critical and the design is almost always governed by the bolts bearing on to the web of the beam. The rotational capacity of this connection is governed largely by the deformation capacity of the angles and the slip between the connected parts. Most of the rotation of the connections comes from the deformation of the angles while fastener deformation is very small. To minimise rotational resistance (and increase rotational capacity) the thickness of the angle should be kept to a minimum and the bolt cross-centres should be as large as is practically possible.

When connecting to the minor axis of a column it may be necessary to trim the flanges of the beam but this does not change the shear capacity of the beam. During erection the beam with the cleats attached is lowered down the column between the column flanges.

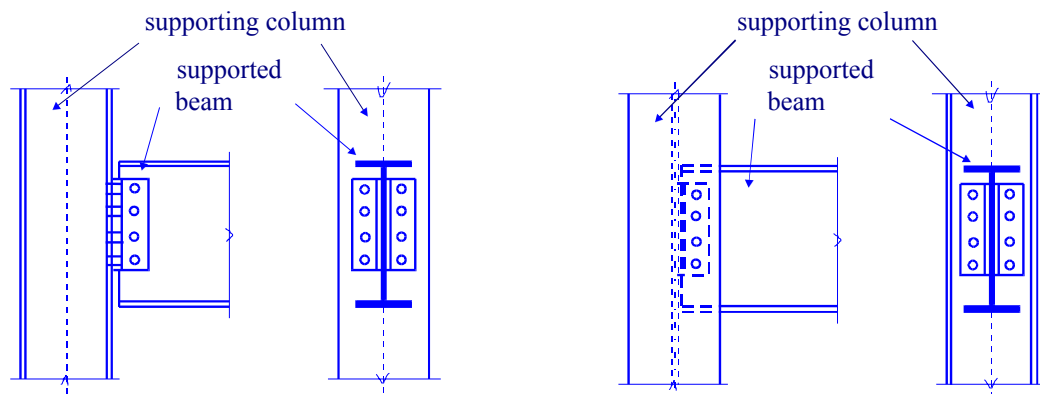


Figure 5.1 Typical major and minor axis double angle cleat connections

#### 5.4.2 Single angle web cleats

Single angle web cleats are normally only used for small connections or where access precludes the use of double angle or end-plate connections.

This type of connection is not desirable from an erector's point of view because of the tendency of the beam to twist during erection. Care should be taken when using this type of connection in areas where axial tension is high. The bolts connecting the cleat to the column must also be checked for the moment produced by the product of the end shear force and the distance between the bolts and the centre line of the beam.

#### 5.4.3 Flexible end-plates

Typical flexible end-plate connections about the major and minor axis of a column are shown in Figure 5.2. These connections consist of a single plate fillet welded to the end of the beam and site bolted to either a supporting column or beam. This connection is relatively inexpensive but has the disadvantage that there is no room for site adjustment. Overall beam lengths need to be fabricated within tight limits although packs can be used to compensate for fabrication and erection tolerances. The end-plate is often detailed to extend to the full depth of the beam but there is no need to weld the end-plate to the flanges of the beam.

Sometimes the end-plate is welded to the beam flanges to improve the stability of the frame during erection and avoid the need for temporary bracing. This type of connection derives its flexibility from the use of relatively thin end-plates combined with large bolt cross-centres. An 8 mm thick end-plate combined with 90 mm cross-centres is usually used for beams up to approximately 450 mm deep. For beams 533 mm deep and over a 10 mm thick end-plate combined with 140 mm

cross-centres is recommended. The local shear capacity of the web of the beam must be checked and, because of their lack of ductility, the welds between the end-plate and beam web must not be the weakest link.

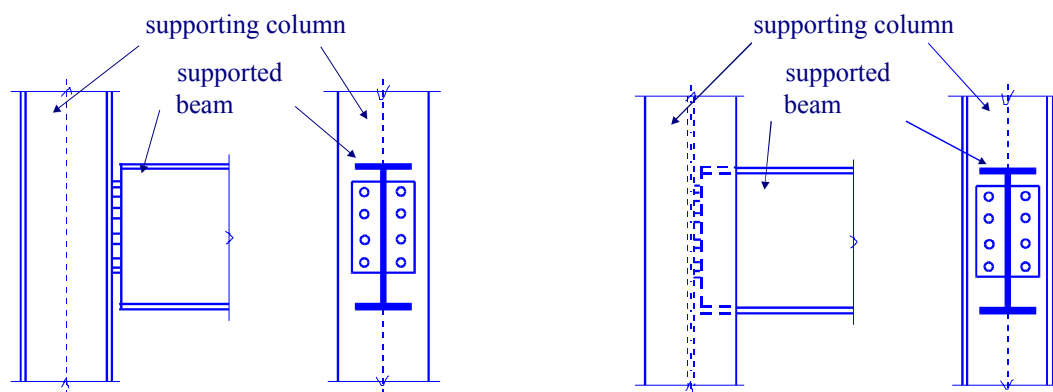


Figure 5.2 Typical major and minor axis flexible end-plate connections

### Detailed Design Procedure

#### 1. Shear capacity of bolt group

The shear capacity of the bolt group must be greater than the reaction at the end of the beam. The shear capacity of either the threaded or unthreaded portion of the bolt should be checked.

#### 2. Shear and bearing capacity of the end plate

The shear capacity of the end plate must be greater than half the reaction at the end of the beam. The bearing capacity of the end plate must be greater than half the reaction at the end of the beam.

#### 3. Shear capacity of the beam web

The shear capacity of the beam web connected to the end plate must be greater than the reaction at the end of the beam.

#### 4. Capacity of fillet welds connecting end plate to beam web

The capacity of the fillet weld must be greater than the reaction at the end of the beam.

#### 5. Local shear and bearing capacity of column web

The local shear capacity of the column web must be greater than half the sum of the beam end reactions either side of the column web. The bearing capacity of the column web must be greater than half the sum of the reactions either side of the column web divided by the number of bolt rows.

#### 6. Structural integrity requirements

The tension capacity of the end plate, beam web and bolt group must be greater than the tie force.

### 5.4.4 Fin plates

A more recent development, which follows both Australian and American practice has been the introduction of the fin plate connection. This type of connection is primarily used to transfer beam end reactions and is economical to fabricate and simple to erect. There is clearance between the ends of the supported beam and the supporting beam or column thus ensuring an easy fit. Figure 5.3 shows a typical bolted fin plate connection to the major and minor axes of a column. These connections comprise a single plate with either pre-punched or pre-drilled holes that is shop welded to the supporting column flange or web.

Considerable effort has been invested in trying to identify the appropriate line of action for the shear. There are two possibilities, either the shear acts at the face of the column or it acts along the centre of the bolt group connecting the fin plate to the beam web. For this reason all critical sections should be checked for a minimum moment taken as the product of the vertical shear and the distance between the face of the column and the centre of the bolt group. The critical sections are then checked for the resulting moment combined with the vertical shear. The validation of this and other design assumptions were checked against a series of tests on fin plate connections. The results of these tests concluded that the design approach was conservative and gave adequate predictions of strength. The tests also showed that fin plates with long projections had a tendency to twist and fail by lateral torsional buckling. Fin plate connections derive their in-plane rotational capacity from the bolt

deformation in shear, from the distortion of the bolt holes in bearing and from the out-of-plane bending of the fin plate.

### Detailed Design Procedure

#### 1. Capacity of the bolt group connecting the fin plate to the web of the supported beam

The bearing capacity per bolt must be greater than the resultant force on the outermost bolt due to direct shear and moment.

#### 2. Strength of the fin plate at the net section under bearing and shear

The shear capacity of the fin plate must be greater than the reaction at the end of the beam. The elastic modulus of the net section of the fin plate must be greater than the moment due to the end reaction and the projection of the fin plate.

#### 3. Strength of the supported beam at the net section

The shear capacity of the supported beam must be greater than the reaction at the end of the beam. For long fin plates the resistance of the net section must be greater than the applied moment.

#### 4. Strength of weld connecting fin plate to supporting column

The leg length of the fillet weld(s) must be greater than 0,8 times the thickness of the fin plate.

#### 5. Local shear check of column web

The local shear capacity of the column web must be greater than half the sum of the beam end reactions either side of the column web.

#### 6. Buckling resistance of long fin plates

The buckling resistance moment of the fin plate must be greater than the moment due to the end reaction and the projection of the fin plate.

#### 7. Structural integrity

The tension capacity of the fin plate and the beam web must be greater than the tie force. The bearing capacity of the beam web or fin plate must be greater than the tie force and the tying capacity of the column web must be greater than the tie force.

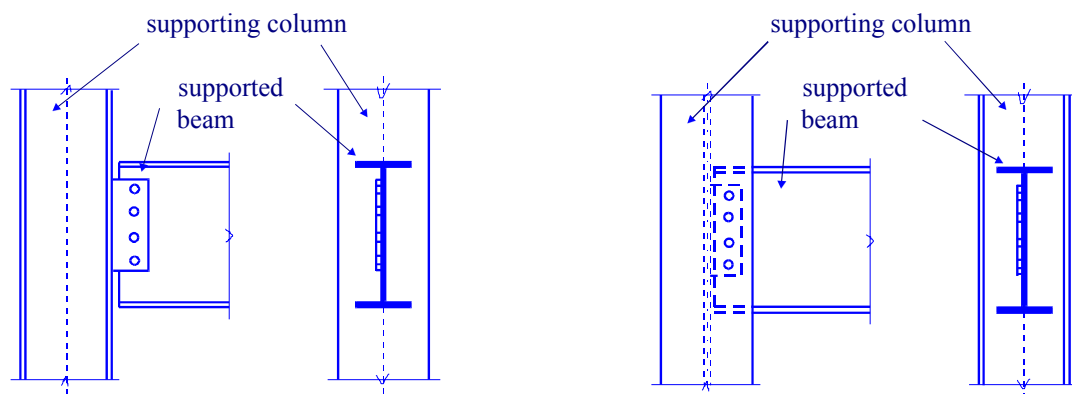


Figure 5.3 Typical major and minor axis fin-plate connections

## 5.5 Beam-to-beam connections

There are three forms of beam-to-beam connection, double angle web cleats, flexible end-plates and fin plates and the comments given in sections 5.4.1, 5.4.3 and 5.4.4 on similar beam-to-column connections will apply. The following sections highlight some of the additional items that need to be considered when designing and using beam-to-beam connections

### 5.5.1 Double angle web cleats

Figure 5.4 shows typical beam-to-beam double angle web cleat connections with single notched and double notched beams. Where the top flanges of the connected beams are at the same level, as in the case of the connection shown in Figure 5.4, the flange of the supported beam is notched and the web must be checked allowing for the effect of the notch. The top of the web of the notch, which is in compression, must be checked for local buckling of the unrestrained web. For

beams, which are not laterally restrained, a more detailed investigation is required on the overall stability of the beam with notched ends against lateral torsional buckling.

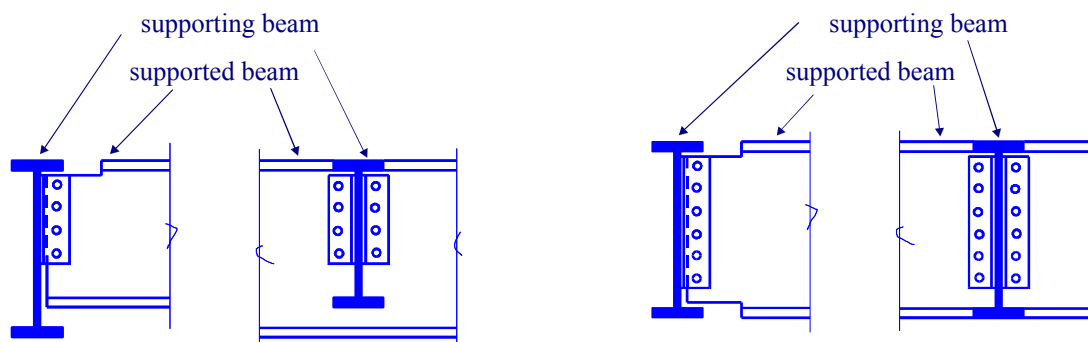


Figure 5.4 Single and double notched beam to beam connections

The web angle cleat can become cumbersome when used to connect unequal sized beams. In this case it is necessary to notch the bottom flange of the smaller beam to prevent fouling of the bolts. Alternatively the cleat of the larger beam could be extended and the bolts placed below the bottom of the smaller beam.

### 5.5.2 Flexible end-plates (header plates)

This type of connection is shown in Figure 5.4. Like the double angle cleat connection, the top flange of the supported beam is notched to allow it to fit to the web of the supporting beam. If both beams are of a similar depth both flanges are notched. In either case if the length of the notches exceed certain limits the unrestrained web and beam must be checked for lateral torsional buckling.

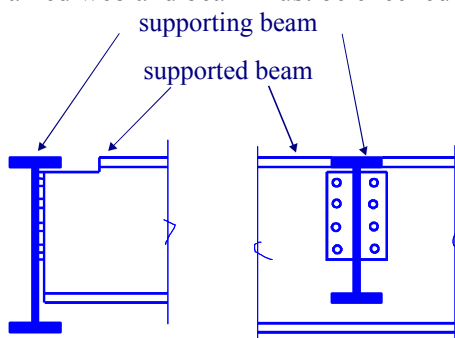


Figure 5.5 Typical beam to beam flexible end-plate connection

In practice the end-plate is often detailed to extend to the full depth of the notched beam and welded to the bottom flange. This makes the connection relatively stiffer than a partial depth end-plate but provided the end-plate is relatively thin and the bolt cross centres are large, the end-plate retains sufficient flexibility to be classified as a simple connection.

If the supporting beam is free to twist there will be adequate rotational capacity even with a thick end-plate. In the cases where the supporting beam is not free to twist, for example in a double sided connection, the rotational capacity must be provided by the connection itself. In such cases thick, full depth end-plates may lead to overstressing of the bolts and welds. Both partial and full depth end-plates derive their flexibility from the use of relatively thin end-plates combined with large bolt cross centres. Normally end-plates no more than 8 mm or 10 mm thick should be used.

### 5.5.3 Fin plate connections

Typical bolted fin plate connections are shown in Figure 5.6. The comments made in section 5.4.4 on beam-to-column fin plates apply to beam-to-beam fin plates. In addition, a beam-to-beam fin plate connection requires either a long fin plate as shown in Figure 5.6a or a notched beam as shown in Figure 5.6b. The designer must therefore choose between the reduced capacity of a long fin plate

and the reduced capacity of a notched beam. Another minor consideration is the torsion induced when fin plates are attached to one side of the supported beam web. However, tests have shown that in these cases the torsional moments are small and can be neglected.

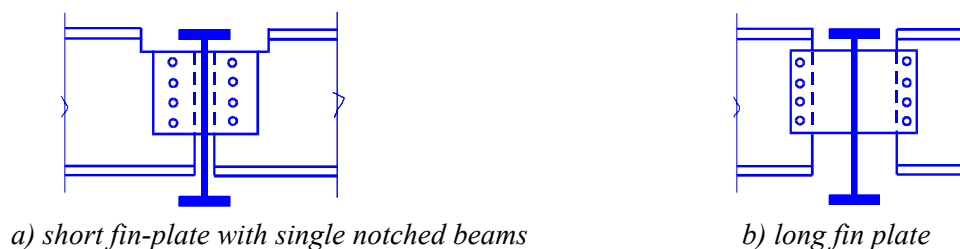


Figure 5.6 Beam to beam fin plate connections

## 5.6 Column splices

This section presents design requirements for column splices in braced multi-storey buildings. In this type of building column splices are required to provide continuity of both strength and stiffness about both axes of the columns. In general they are subject to both axial compression and moments resulting from the end reactions of the beams. If a splice is positioned near to a point of lateral restraint (i.e. within say 500 mm above the floor level), and the column is designed as pinned at that point the splice may simply be designed for the axial load and any applied moments. If, however, the splice is positioned away from a point of lateral restraint (i.e. more than 500 mm above the level of the floor), or end fixity or continuity has been assumed when calculating the effective length of the column, the additional moment that can be induced by strut action must be taken in to account.

Two types of splices are considered in this section, those where the ends of the members are prepared for contact in bearing, and those where ends of the members are not prepared for contact in bearing. In both cases the column splices should hold the connected members in line and wherever practicable the members should be arranged so that the centroidal axis of the splice material coincides with the centroidal axes of the column sections above and below the splice.

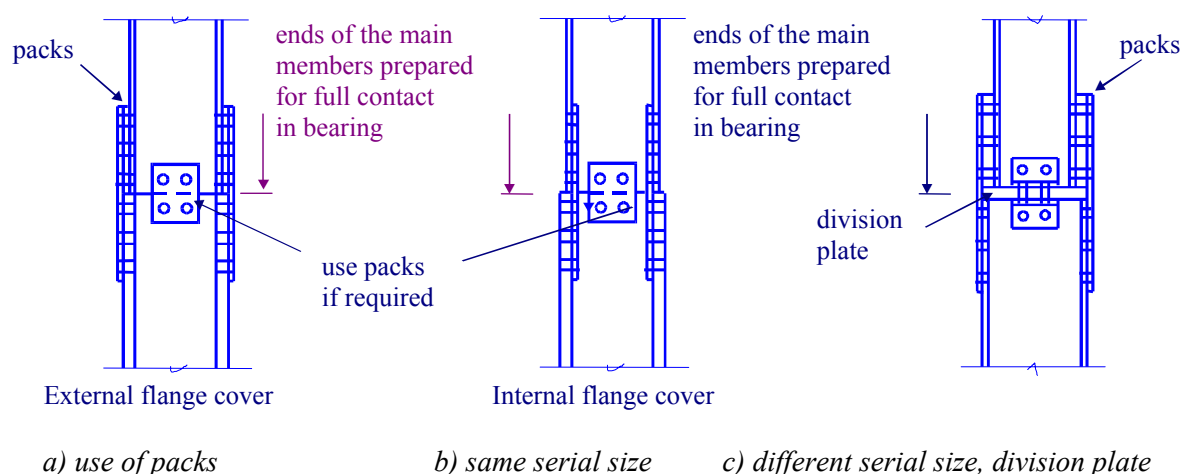


Figure 5.7 Column splice with ends prepared for bearing

### 5.6.1 Ends prepared for contact in bearing

Typical details for this type of column splice are shown in Figure 5.7. In all three cases the splice is constructed using web and flange cover plates and packs are used to make up any differences in the thickness of the web and the flanges. The flange cover plates may be placed on either the outside or the inside of the column. Placing the cover plates on the inside has the advantage of reducing the overall depth of the column. Each column splice must be designed to carry axial compressive forces, the tension (if any) resulting from the presence of bending moments and any horizontal shear forces.

The ends of the columns are usually prepared for full contact, in which case compressive forces may be transmitted in bearing. However, it is not necessary to achieve an absolutely perfect fit over the entire area of the column. Columns with saw cut ends are adequately smooth and flat for bearing and no machining is required. This is because after erection the ends of the column bed down as successive dead loads are applied to the structure. The cover plates provide continuity of stiffness and are designed to resist any tension where the presence of bending moments is sufficiently high to overcome the compressive forces in the column. Tension forces must be allowed for. The horizontal shear forces that arise from the moment gradient in the column are normally resisted by the friction across the bearing surfaces of the two columns and by the web cover plates. Wind forces on the external elevations of buildings are normally taken directly in the floor slab. It is rare for column splices in simple construction to transmit wind shears.

### 5.6.2 Ends not prepared for contact in bearing

Typical details for this type of column splice are shown in Figure 5.8. Both diagrams show that the column above and below the splice is the same serial size. In this case the splice is constructed using web and flange cover plates and where required packs are used to make up the differences in the web and flange thickness. Where columns of different serial size are to be connected multiple packs are necessary to take up the dimensional variations. For this type of splice all the forces and moments are carried by the cover plates and no load is transferred through direct bearing. The axial load in the column is normally shared between the flange and web cover plates in proportion to their areas while any bending moments are normally carried by the flange cover plates alone.

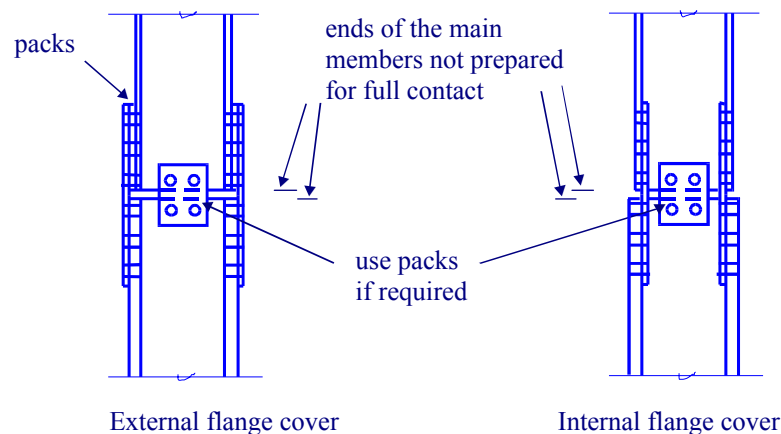


Figure 5.8 Column splice with ends not prepared for bearing

### Q&A 5.1 Bolt bearing resistance with respect to tolerances

Figure T.5.5 of the execution standard [ENV 1090-1] allows a tolerance of  $\Delta = \pm 5 \text{ mm}$  for the position of a group of boltholes. Should this variation be taken into account when calculating the bearing capacity of a bolt group? For example:

Bearing capacity assuming  $\Delta = \pm 0 \text{ mm}$

The bearing capacity of a single bolt is given by the following expression:

$$F_{b,Rd} = \frac{2,5 \alpha d t f_u}{\gamma_{Mb}}$$

Where for the above example:

$$d = 12 \text{ mm}$$

$$d_o = 13 \text{ mm}$$

$$e_1 = 1,2 d = 1,2 \times 13 = 15,6 \text{ mm}$$

$$\alpha = e_1 / 3d_o = 15,6 / (3 \times 13) = 0,4$$

### Bearing capacity assuming $\Delta = -5\text{mm}$

Where for the same example:

$$d = 12 \text{ mm}$$

$$d_o = 13 \text{ mm}$$

$$e_1 = 1,2 d - 5 = 1,2 \times 13 - 5 = 10,6 \text{ mm}$$

$$\alpha = e_1 / 3 d_o = 10,6 / (3 \times 13) = 0,272$$

Compared to the first example the second gives a capacity, which is 32% lower as a result of allowing for the tolerance. Should a reduction in edge and end distances etc. be taken when calculating the capacity of a connection?

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Tolerances are not usually taken into account in the design of the connection. It is assumed that the tolerances are small in comparison to the edge and end distances and that the reduction in capacity is also small and can be accommodated by the partial safety factor.

### **Q&A 5.2 Angles connected by one or two bolts**

How it is possible, that the tension resistance of the net section of an angle with one bolt is greater than that with two bolts? See the simple example below.

Input data  $A = 480 \text{ mm}^2$

L  $50 \times 5$   $t = 5 \text{ mm}$

$$d_o = 14 \text{ mm}$$

$$e_2 = 25 \text{ mm}$$

$$f_u = 510 \text{ MPa}$$

According to ENV 1993-1-1 (clause 6.2.3) the tension resistance of the net section is checked: for one bolt (See clause 3.10.3(2) and Table 3.8 in ENV 1993-1-8):

$$A_{eff} = 2(e_2 - 0,5 d_o) t = 2 \cdot (25 - 7) \cdot 5 = 180 \text{ mm}^2$$

$$N_{u.Rd} = \frac{A_{eff} f_u}{\gamma_{M2}} = \frac{180 \cdot 510}{1,25} = 73\,440 \text{ N}$$

for two bolts: (See clause 6.5.2.3 and Table 6.5.1 in ENV 1993-1-8):

$$\beta_{eff} = 0,4$$

$$A_{eff} = \beta_2 A_{net} = \beta_2 (A - d_o t) = 0,4 (480 - 14 \cdot 5) = 164 \text{ mm}^2$$

$$N_{u.Rd} = \frac{A_{eff} f_u}{\gamma_{M2}} = \frac{164 \cdot 510}{1,25} = 66\,912 \text{ N}$$



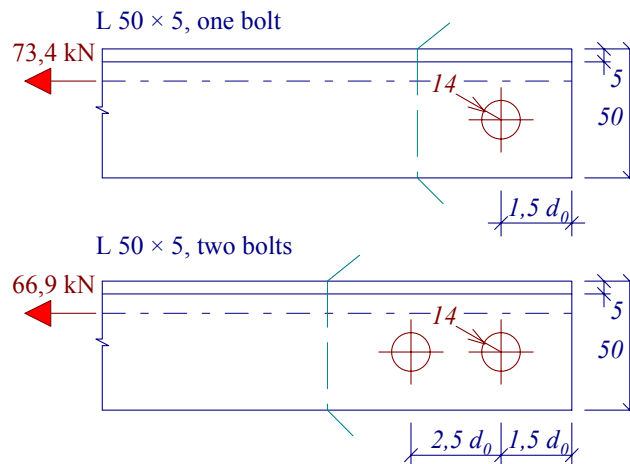


Figure 5.9 Angle connected with one and two bolts

Numerous tests have been carried out on angles connected by one and two bolts. These tests have shown that the resistance of an angle connected by a single bolt is higher than one connected by two or more bolts. The reasons for this are unclear but is possible due to the additional moments that are attracted by connection with more than one bolt.

### Q&A 5.3 Rotation capacity

How do simple connections derive their rotational capacity?

Cleated connections derive their ductility from having reactively thin cleats (8 mm or 10 mm thick) and bolts in the supporting member, which are at reasonable cross centres (100 mm + beam web thickness).

Similarly, it is usual to use relatively thin end-plates (8 mm or 10 mm) and bolt cross centres of 90 mm or 140 mm to ensure that an end-plate connection has adequate flexibility and ductility to be classified as a 'simple connection'.

Ductility in a fin plate is achieved by ensuring a bearing mode of failure in either the fin plate or beam web. This is usually achieved by adopting the following detailing requirements:

- the thickness of the fin plate or beam web is  
 $\leq 0,42 d$  (for S355 steel)  
 $\leq 0,50 d$  (for S275 steel), where  $d$  is bolt diameter,
- all end and edge distances on the plate and the beam web are at least  $2 d$
- 8.8 bolts are used, un-torqued and in clearance holes
- the fillet weld leg length is at least 0,8 times the thickness of the fin plate.

The rotational capacity of the beam to column connection may be very roughly assumed as the rotation limited by thatching of the lower flange of the beam to the column.

### Q&A 5.4 Structural Integrity

What method should be used to determine the tying capacity of simple connections and what is the background to this method?

Reference [SCI, 2002] gives a method for calculating the tying capacity of double angle web cleat connection and flexible end-plate connections. The method for double angle web cleat connections is based on the large displacement analysis of the cleats in tension [Carril et al, 1994] using the deformed shape shown in Figure 5.10. The main features of this approach are:

- The potential magnitude of the displacement  $\delta$ , ignoring second order effects. Displacement  $\delta$  defines the displaced geometry of the web cleats.
- These displacements reduce the eccentricities on the web cleats. Part of the tying force is carried by tension in the legs of the cleats.
- There are four critical sections in each cleat, which are subject to high plastic stains under the combined action of shear, tension and moment. Two are located at, or near, the bolt centrelines; two are located at the tips of the radiused portion of the heel of the cleat.

A programme of tests was carried out to verify this approach the details of which are given in the reference. Comparison between the method and the test results showed that the method gives an adequate margin of safety.

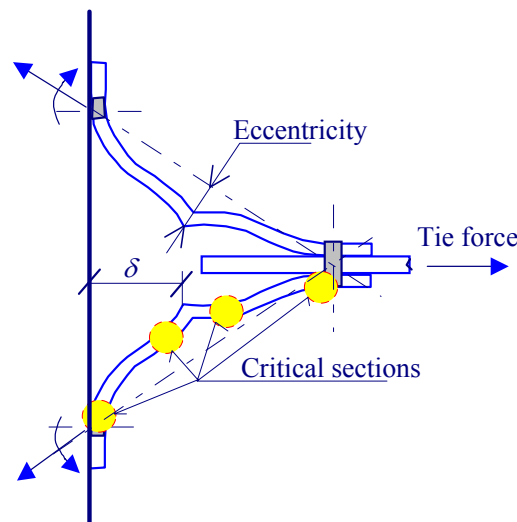


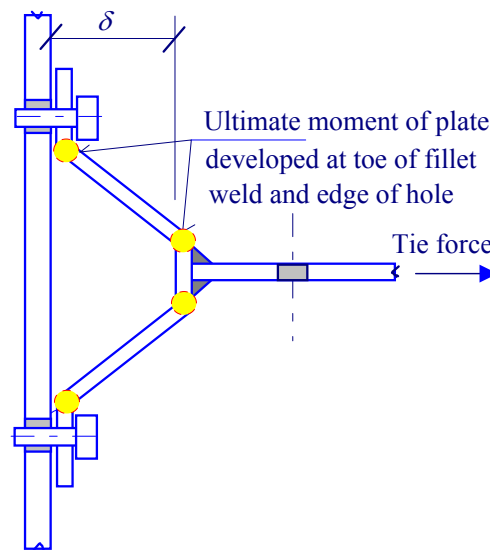
Figure 5.10 Double angle web cleats under tension

Generally, it will be found that the tying capacity of a web cleat connection is adequate. This is because of its ability to accept large deformations before failure. However, if the connection is unable to carry large tying forces, then extra capacity can be achieved by increasing the thickness of the cleat and/or by reducing the distance between the bolt cross-centres in the support. In both cases consideration should be given to the increase in the rotational stiffness of the connection.

Similarly the method for flexible end-plate connections is based on a large displacement analysis of the end-plate in tension. The deformed shape of the end-plate is shown in Figure 5.11. The features of this approach are:

- Considerable deformation can arise but only if there is rotation of the hinges at the toes of the welds. These regions may well have suffered some embrittlement from fast cooling after welding.
- These displacements do reduce the eccentricities within the connection but they do offer an alternative membrane path for some of the tying force. However, this membrane restraint is only available if the end plate is bolted to a more substantial plate or flange. For general solution this membrane action has been ignored.

- There are four critical sections in the plate. Since membrane action is being discounted these need only be considered under moment. (Moment/shear interaction will exist but need not be considered because the applied shear is only a small proportion of its shear capacity.)



*Figure 5.11 End-plate under tension*

Once again a programme of tests was carried out to verify this approach. Details of the tests are given in reference [Jarrett, 1990]. Although the ratio of experimental capacity to calculated capacity varied considerable the tests proved that the method is conservative. The reason for the variability is not clear but may be due to the unquantified and variable membrane action.

The tying capacity of an end plate is generally lower than that of a web cleat connection or a fin plate. The tying capacity can be as low as 35% of the connection's shear capacity. For connections to I-section columns the critical mode of failure will be the tension capacity of the end-plate. To increase the capacity the solution will generally involve either increasing the thickness of the end-plate or reducing the cross centre distance between the bolts.

In both methods because the analysis relies on gross deformations the prying forces developed in the bolts are generally higher than methods based on the more traditional approaches. Thus a simple design check is introduced for both connections that ensures that the nominal tensile stress of bolts does not exceed its tensile resistance in interaction with shear acting forces. The tying capacity of a fin plate connection can be determined by simple structural analysis.