6 MOMENT CONNECTIONS

Moment connections are designed to transfer bending moments, shear forces and sometimes normal forces. The design strength and stiffness of a moment connection are defined in relation to the strength and stiffness of the connected members. The design strength of a moment connection may be full- strength (i.e. the moment capacity of the connection is equal to or large than the capacity of the connected member) or partial-strength (i.e. the moment capacity of the connection is less than that of the connected member). Similarly the stiffness of a moment connection can be rigid or semi-rigid compared to the stiffness of the connected member.

The rotation capacity of the structure may be provided by either the connection or the connected member.

6.1 Development of the component method

In the past connections were designed as either pinned or rigid, full strength connections. Considerable work on connection behaviour has been completed and the concepts of semi-rigid design and partial strength design have been developed, which model more accurately the true behaviour of connections. These models have been developed into a comprehensive set of design rules which have been introduced into Eurocode 3 part 1.8 [prEN 1993-1-8, 2003]. These rules allow the designer to calculate the strength, stiffness and deformation capacity of moment connections. The following steps are required to design a moment connection:

- Determine the path of the forces through the connection. E.g. the tensile force in the top flange of the beam shown in Figure 6.1 must pass through the fillet welds of the connected end plate, the end plate in bending, the bolts in tension, the column flange in bending, the column web in tension, the column web in shear and the column web in compression, in order to balance the compression force coming from the beam compression flange. Furthermore the path of other forces (e.g. the shear force from the beam) has to be determined in a logical way. Of course the forces in the connection must be in equilibrium with the applied bending moment, shear force and /or the normal force acting on the connection.
- Once the path of the forces is determined, the strength of every part or component of the connection in this path must be calculated. The component in the chain with the lowest strength governs the strength of the connection (e.g. the tensile force in the top beam flange).
- The stiffness of the connection depends on the deformations of the components in the path that the forces follow. The deformation of each component gives the stiffness of the connection.
- The third important mechanical property of a connection is its deformation capacity. Deformation capacity is usually provided by plastic deformation of one or more components. If the strength of the connection clearly exceeds the strength of one of the connected members, the designer can rely on the deformation capacity coming from the connected member, e.g. the formation of a plastic hinge in the beam.

After the mechanical properties of all components have been determined, the various components can be assembled to determine the strength, stiffness and deformation capacity of the whole connection. The above method for the determination of the mechanical properties of the connection is called the <u>component method</u>. Rules for calculating the strength, stiffness and deformation capacity of each component are given in prEN 1993-1-8.

The principles of the component method are based on Zoetemeijer's work [Zoetemeijer, 1983b]. Later, other researchers worked on this method to determine the mechanical properties of more components and to refine the calculation methods, in order to obtain more accuracy in the description of the mechanical behaviour. Furthermore many tests were carried out to validate many different connection configurations. For further study see the references at the end of this publication.

The accuracy of the component method depends on the accuracy of the description of the basic components and on the quality of the assembling process. It is assumed that the component properties are independent. However, some components do not act independently, but influence others. For hand calculation this can only be accounted for in a simplified way, because the general approach results in a complicate iterative calculation procedure. This is not a problem for software. An example of the components for a bolted beam to column connection is given in Figure 6.1.

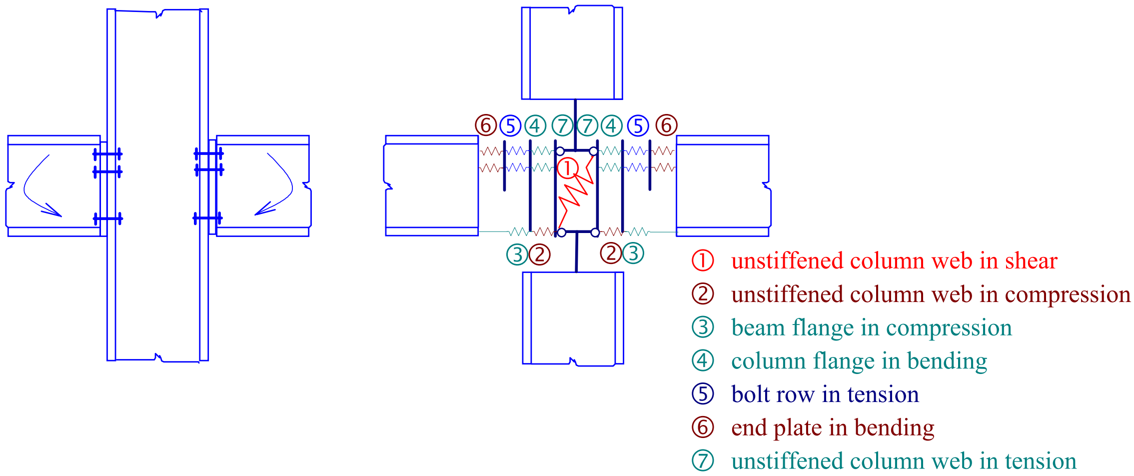


Figure 6.1 Components represented by springs in a bolted beam to column joint

6.2 Practical application of the component method

The component method allows the designer to analyse many different connection configurations. It enables the design of more economical connections than the traditional design methods based on tables (that could only be used for a limited number of connection configurations). The component method is complex and requires considerable effort even for the simplest moment connection. For practical application, therefore several computer programs have been developed, which can quickly determine a connection's mechanical properties. These computer programs can also be used to examine the effects that small changes have on a connection's strength, stiffness and deformation capacity. It is noted, however, that before using such programs, the designer must check that the software has been validated.

6.3 Determination of the connection's strength, stiffness and rotational capacity

As indicated above, the properties needed for the global analysis of a structure are the bending moment strength (resistance) $M_{j,Rd}$, the deformation / rotational stiffness S_j and the deformation / rotational capacity ϕ_{Cd}. Figure 6.2 gives an example of a beam to column connection and its moment rotational diagram. In tests, the first part of the moment rotational diagram (representing the stiffness), is usually linear. However, the linear elastic curve deviates from its straight line at low bending moments [Wald, Steenhuis, 1993]. This is due to local plasticity caused by stress concentrations and residual stresses. In prEN1993-1-8 it is assumed that up to 2/3 of the calculated moment resistance the behaviour is elastic, following the straight line determined by the calculated (initial) stiffness.

After reaching 2/3 of the moment resistance, the stiffness is reduced until the moment resistance $M_{j,Rd}$ is reached. Rules are given in prEN1993-1-8, see also the question 6Q&A1.

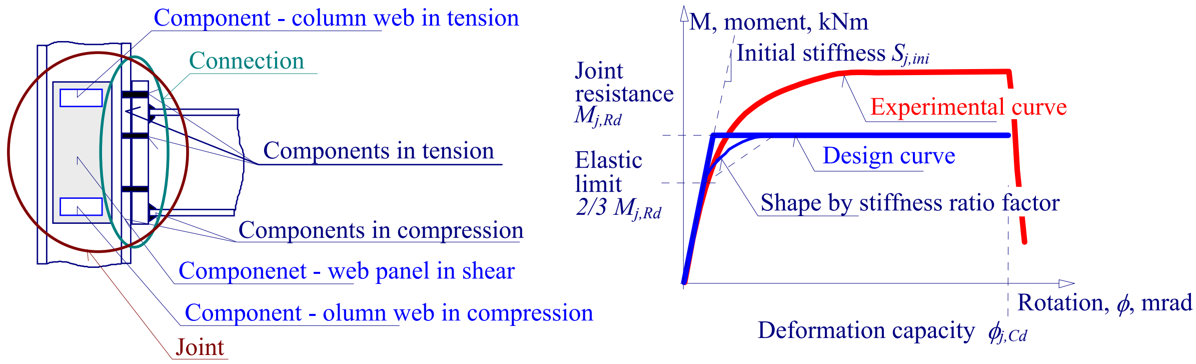


Figure 6.2 Basic components of a beam to column joint and the resulting moment – rotational diagram

The required deformation capacity (rotational capacity) of a joint depends on the type of the structure (e.g. statically determined or statically indeterminate) and the method of global analysis for the whole structure (elastic or plastic), but seldom this exceeds *60 mrad*, see Figure 6.3.

The determination of the rotational capacity is qualitative. In prEN1993-1-8, deemed to satisfy rules are given, e.g. if yielding of the column flange or end plate in bending or the column web panel in shear governs the strength, then it is assumed that sufficient rotational capacity is provided. The prediction of the available rotational capacity by the component method, based on the deformation capacity of each component, is under development.

The column flange and the end plate in bending and the column web panel in shear are ductile components. The bolts in tension and shear and welds are typical examples of brittle components. Therefore, both modes of failure should not govern the strength of the connection. Such design will lead to brittle connections and potentially unsafe structures.

In the evaluation of the rotational capacity, it is necessary to consider possible unfavourable differences between the calculated strength and the actual strength of the component that is to provide the rotational capacity.

Such deviations may be caused by a higher actual yield strength of the component material or a difference in the actual dimensions or due to the calculation model underestimating the actual strength. Therefore, in the analysis of the rotational capacity, the upper limit of the resistance of the component to provide the rotational capacity, needs to be taken (for the calculation of the moment resistance $M_{j.Rd}$, the lower limit must be taken).

The rules to assure adequate rotational capacity for bolted and welded connections are included in prEN 1993-1-8 as "deem to satisfy" criteria, e.g. the rules to limit brittle failure caused by rupture of the bolts (mode 2 – bolt failure and plastic hinge in the plate, mode 3 – bolt failure) and weld failure.

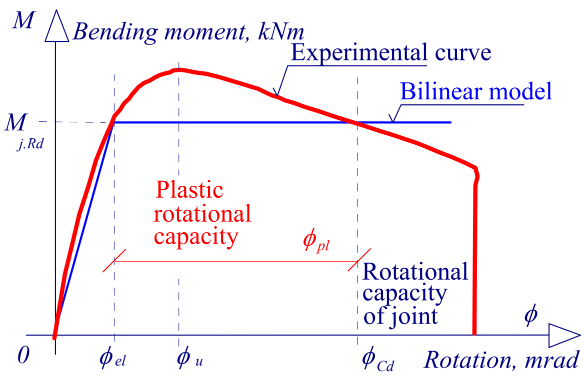


Figure 6.3 Rotational capacity, design limit of resistance by brittle collapse of the second bolt row of a beam to column end plate connection

Q&A 6.1 Stiffness Modification Coefficient η for End-Plate Connections

The values of the stiffness modification coefficient η given in prEN 1993-1-8 Table 5.2 do not cover the full range of different end-plate connections that can be used. For example, do the values allow for connections into the web of a column/beam, thin end plates vs. thick end plates, extended vs. flush end-plates etc.? Please provide the background to this table.

To make an elastic global analysis according to clause 5.1.2, you may take into account a stiffness, which is assumed to be the initial stiffness divided by the stiffness modification coefficient η, see Figure 6.4. The coefficient η is given in prEN 1993-1-8 Table 5.2.

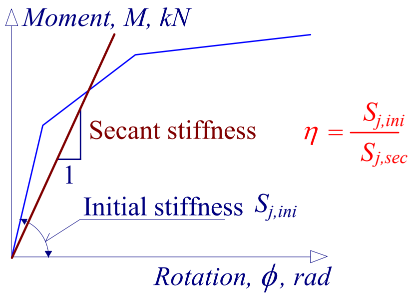
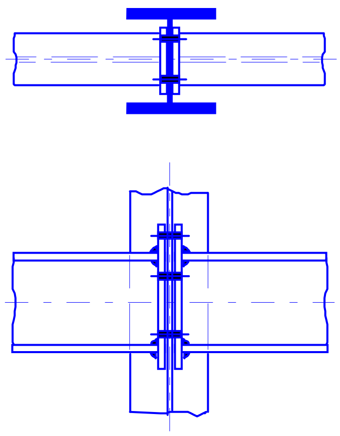


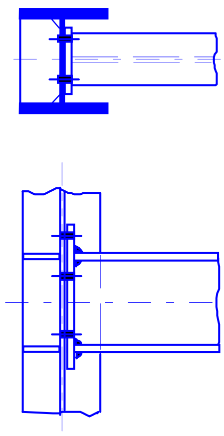
Figure 6.4 Stiffness for global elastic analysis

The thickness of the end plates influences the initial stiffness of the connection but not the stiffness modification coefficient η.

- For beams connected to the web of an unstiffened column or beam, the stiffness modification coefficient η is not relevant and these joints may be considered as hinges, see [Gomes, Jaspart, 1994] and [Gomes et al, 1994], for simplicity.
- For a continuous beam connected on both sides of the column web, see Figure 6.5a, the connection can be considered as a 'beam splice' with longer bolts.
- If the beam is connected to a stiffened column web, see Figure 6.5b, the stiffeners between the column flanges create a similar effect as a connection to the column flange.



a) beam splice



b) beam to stiffened column web

Figure 6.5 Beam to column minor axis joints

Q&A 6.2 Effective Length of Stiffened T-stub

Could you give the background information for the α curves used to calculate the effective length of a T-stub and equations for α and its dependence on λ_1 and λ_2?

The background to these rules is a mechanical model based on yield line theory. Details are given in a TU-Delft report written by Zoetemeijer [Zoetemeijer, 1990]. (Note the α values in figure 2.12 of the Zoetemeijer's publication need to be divided by *2* to compare with those given in the Eurocode).

The parallel parts of the curves in Figure 6.12 of prEN 1993-1-8 correspond to the basic equations in table 6.6. With the values of m_1, m_2 and e, the values for λ_1 and λ_2 can be determined which give the value of α. The parallel parts of the curves $L_{eff} = \alpha\, m_1$ correspond to the basic equation: $L_{eff} = 4\, m_1 + 1{,}25\, e$. In the study by Zoetemeijer, the value of α did not exceed $2\,\pi$. The curves for $\alpha = 7$ and 8 are added in prEN 1993-1-8, see Figure 6.6. The curves for a constant value α as illustrated in Figure 6.6, are given by the following equations:

$$\text{in case } \lambda_2 < \lambda_2^*:\quad \lambda_1 = \lambda_1^* + (1-\lambda_1^*)\left(\frac{\lambda_2^* - \lambda_2}{\lambda_2^*}\right)^{\frac{\alpha}{\sqrt{2}}} \tag{6.1}$$

$$\text{in case } \lambda_2 \geq \lambda_2^*:\quad \lambda_1 = \lambda_1^* \tag{6.2}$$

where

$$\lambda_1^* = \frac{1{,}25}{\alpha - 2{,}75}\,, \tag{6.3}$$

$$\lambda_2^* = \frac{\alpha\, \lambda_1^*}{2}\,. \tag{6.4}$$

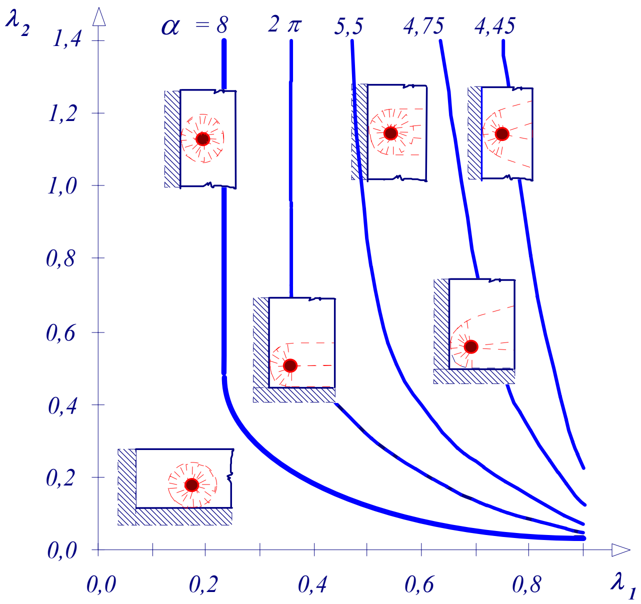


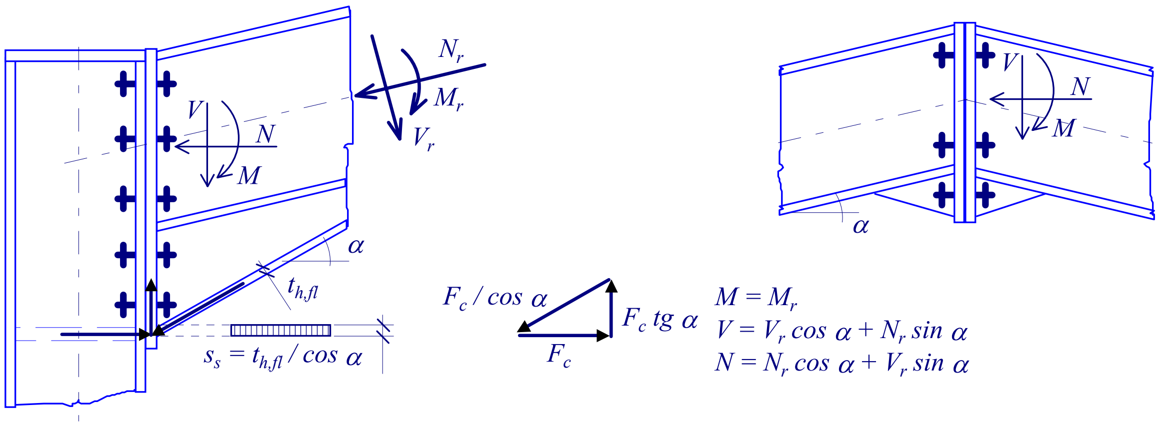
Figure 6.6 Values of α for stiffened column flanges and end plates

Q&A 6.3 Haunched Connections

The current version of prEN 1993-1-8 does not contain rules of the design of portal frame haunched connections. Could you recommend simple and safe rules or give a reference?

Two basic types can be distinguished: haunches designed to economise the rafter (inclination of about 10%) and haunches used to increase the bending moment resistance of the connection (by about 35%-40%). Similar questions arise in case of tapered built-up members. The component method in prEN 1993-1-8 can be used for all joints that can be decomposed into a set of basic components. This method can also be used for haunched connections.

There are two questions related to the component description and assembly of components in the case of haunched connections: the influence of the inclination of the beam on the internal forces and resistance of the beam flange and column web in compression, see Figure 6.7. The inclination needs to be taken into account for the evaluation of the component properties of the column web in compression and the end plate in bending (for welded connections, also of the column flange in bending and the column web in tension).



a) stiffened haunch with flange *b) unstiffened haunch without flange*

Figure 6.7 Typical haunched joints in portal frame

Precise details about haunches are given in Clause 6.2.4.7 of prEN 1993-1-8. If the height of the beam including the haunch exceeds *600 mm,* the contribution of the beam web to the compression resistance should be limited to 20%. Reinforcing haunches should be arranged with the following restrictions: the steel grade should match that of the member; the flange size and web thickness of the haunch should not be less than that of the member; the angle of the haunch flange to the flange of the member should not be greater than 45°; and the length of the stiff bearing s_s should be taken as equal to the thickness of the haunch flange parallel to the beam, see Figure 6.7.

Q&A 6.4 Diagonal and K-stiffeners

Does it matter whether a (diagonal) stiffener of a beam-to-column joint is loaded in tension or in compression?

There is a difference in the resistance calculation of the stiffener. For a stiffener loaded in tension, the cross section resistance should be checked. For stiffeners loaded in compression, plate-buckling verification is required, see Figure 6.8. As a simplification, it is possible to use the following rules:

- The plate thickness of the stiffener should be the same size as the flange of the beam.
- The *b/t* ratio of the stiffener should be at least a class 3 cross section.

K-stiffeners are loaded in tension and in compression. Both aspects have to be checked as described above.

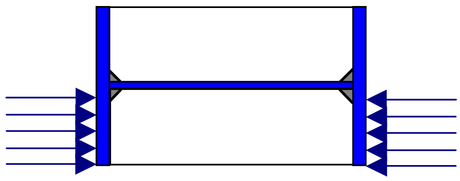
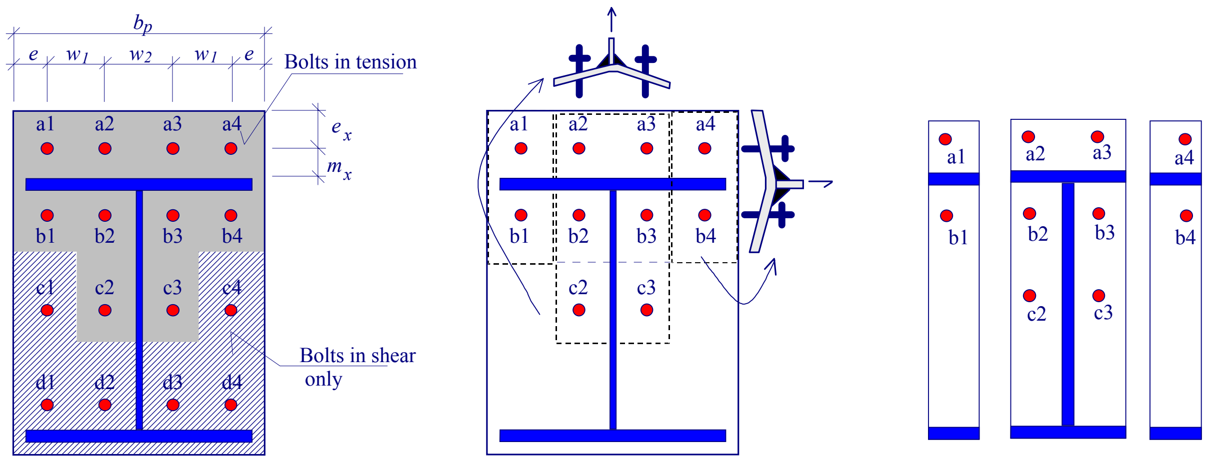


Figure 6.8 Plate buckling design of a web stiffener

Q&A 6.5 Yield line Patterns for End Plate Connection with Four Bolts in a Row
Can the method given in prEN 1993-1-8 be used for end plate connections with four bolts in a row?

The bolts a1-a4 and b1-b4 near the beam flange under tension can be taken for the bending moment resistance calculation, see Figure 6.9a. The bolts c2 and c3 could also be considered. However, the bolts c1 and c4 cannot be considered for the transfer of tension due to the limited stiffness of the end plate. These bolts together with the bolt row d can be used to transfer the applied shear forces.



a) separation into bolts in tension and in shear *b) separation into T stubs in tension* *c) division of top bolt rows into separate T stubs*

Figure 6.9 End-plate with 4 bolts in a row

Depending on the size of the end plate and bolt spacing, there are several possibilities for yield line patterns for the bolts in rows a and b. The most likely pattern for the extended part of the end plate (bolt row a) is given in Figure 6.10. Also bolts b1 and b4 are assumed to develop the same yield line pattern.

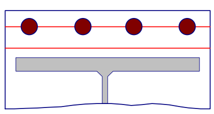


Figure 6.10 Assumed yield line pattern in the extended part of the end plate.

The logical approach using the T-stub schematisation is to divide the connection into T stubs as given in Figure 6.9c. For each T-stub, the effective length based on the yield line patterns given in prEN 1993-1-8, should be checked. Finally, the usual assembly of components can be applied to determine the bending moment resistance of the connection.

Q&A 6.6 Distribution of Forces in a Thick End Plate Connection

Is it permitted to use a plastic distribution of internal forces for a partial strength beam-to-column connection if very thick end plates are used? If not, are there any criteria for the thickness that is required for the elastic design?

The ratio of the strength of the end plate (column flange) to the strength of the bolts governs the collapse mode. PrEN 1993-1-8 gives the following three collapse modes:

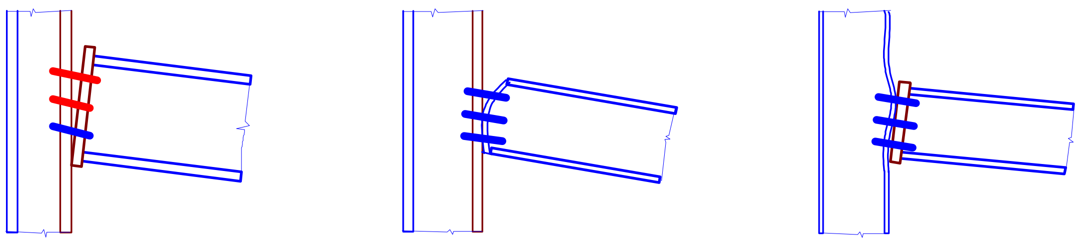
- Mode 1: Yielding of the end plate or column flange (4 plastic hinges in T-stub). This occurs if the bolts are strong compared to the strength of the end plate or column flange.
- Mode 2: Bolt failure after partly yielding of the end plate or column flange (2 plastic hinges in T-stub). This occurs if the bolts are weak compared to the strength of the end plate or column flange.
- Mode 3: Bolt failure without yielding of the end plate or column flange. This occurs if strong end plates or column flanges are applied and relatively weak bolts.

Because of the differences in the deformation capacity between bolt failure and plate yielding by bending, Mode 1 is ductile and Mode 3 is brittle (no rotation capacity), see Fig 6.2.2 and tab. 6.2 of prEN 1993-1-8.

Following the procedure given in prEN 1993-1-8, the designer will be warned if his design results in Mode 3. This brittle mode of failure is usually not allowed because the rules for having sufficient rotational capacity of the joint are not fulfilled.

However, if the moment capacity of the joint is designed to carry *1,2* times the moment capacity of the connected beam, the necessary rotational capacity may be assumed to be supplied by the formation of a plastic hinge in the beam.

The rotational capacity of the joint depends on the deformation capacity of the end plate as well as that of the column flange, see Figure 6.11. If the column flange is thin, it will deform and provide sufficient rotational capacity, see Figure 6.11c.



a) thick column flange and thick end plate relative to bolt strength;

- elastic distribution of bolt forces,
- no rotational capacity,
- design allowed provided:

$M_{j,Rd} > 1,2\, M_{pl,d,beam}$ then rotational capacity will come from the beam section (plastic hinge in beam).

b) thick column flange and thin end plate relative to bolt strength;

- plastic distribution of bolt forces,
- moment capacity determined by the end plate,
- rotational capacity comes from the end plate.

c) thin column flange and thick end plate relative to bolt strength;

- plastic distribution of bolt forces,
- moment capacity determined by the column flange,
- rotational capacity comes from the column flange.

Figure 6.11 Influence of column flange thickness and end plate thickness on rotational capacity of the joint

Q&A 6.7 Distribution of Shear Forces in a Bolted Connection

An end-plate connection is usually loaded by a moment and a shear force. How is the shear force distributed over the bolts?

Any distribution of shear force between the bolts is allowed, provided that the conditions for equilibrium and deformation capacity (ductility) are fulfilled. It is allowed to distribute the shear force equally over all bolts, see Figure 6.12. The bolts loaded in tension and shear should be checked using the following rules for combined tension and shear, see prEN 1993-1-8,

$$\frac{F_{v,Sd}}{F_{v,Rd}} + \frac{F_{t,Rd}}{1{,}4\ F_{t,Rd}} \le 1{,}0\ . \tag{6.5}$$

In general, however, the shear force is distributed over the bolts in the compression part of the joint. The number of bolts needed equals the shear force divided by the shear capacity of each bolt. The other bolts can be designed for tension only (in the tension zone).
Furthermore, to get sufficient deformation capacity, it is important that the capacity of the bolts in shear is higher than the bearing capacity of the bolts in the end plate or the column flange.

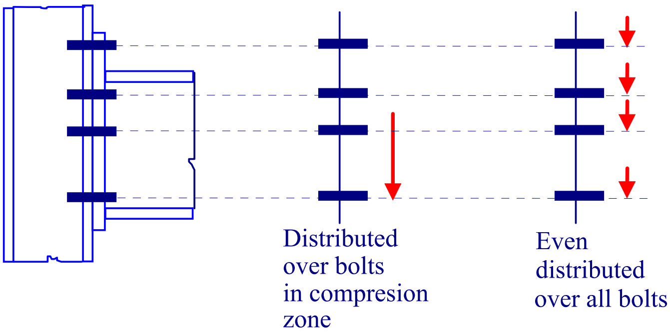


Figure 6.12 Example of distribution of shear forces in an end plate connection

Q&A 6.8 Prying Force of T-stub in Fatigue Design

The effect of prying of bolts is included in the formulae for the resistance of bolt rows. However, in the case of fatigue, the effect of prying forces in the bolts should be known in order to verify the bolts. How is this done?

In the case of fatigue, bolts should always be pre-tensioned. Transfer of the varying loads should pass directly through a stiff contact surface and not via the bolts. This is illustrated in Figure 6.13 and Figure 6.14.

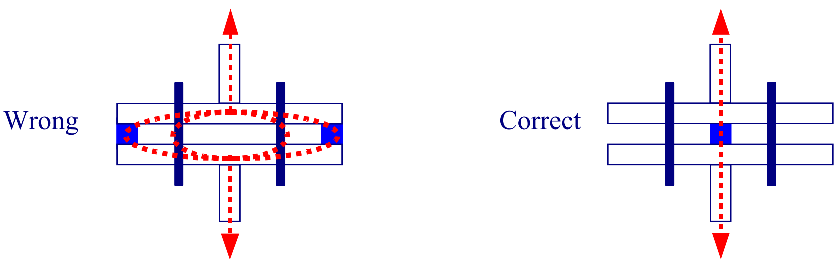


Figure 6.13 Wrong and correct detailing of a pre-tensioned joint;
the flow of the varying force through the joint is illustrated by dotted lines

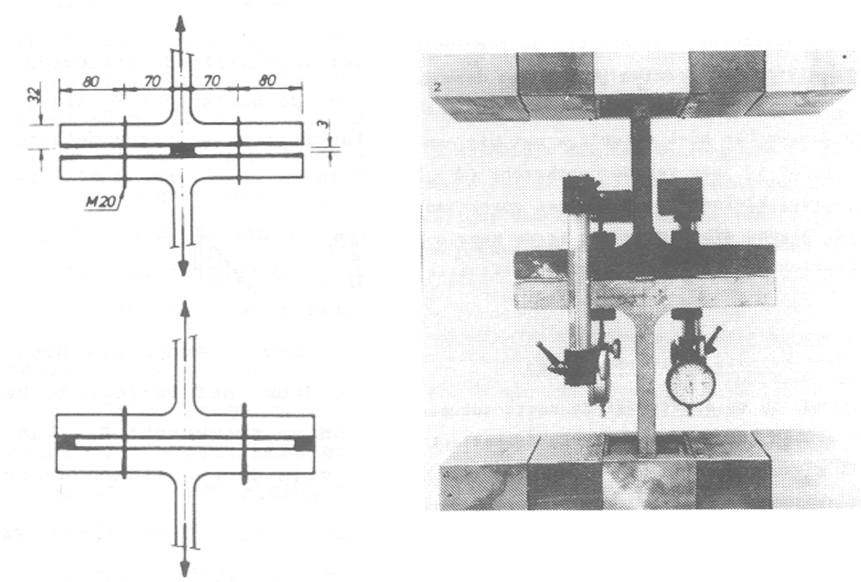
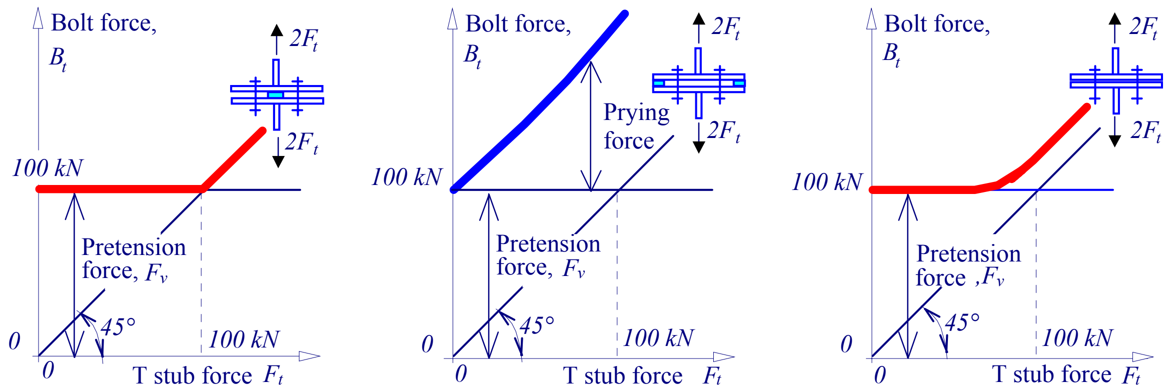


Figure 6.14 Test specimens and assembling of a T-stub



a) stud in centre *b) studs at edges* *c) due to hot rolling/welding the contact point is close to the centre*

Figure 6.15 Test results of a T-stub, force in bolts F_b, T-stub force F_t

In general, the force distribution through the joint is via the stiffest route. The force in bolt F_b can be divided into a contact force F_c and a tensile force F_t. In all three cases the joint has been pre-stressed with a force F_v. By introducing an external tensile force $2\,F_t$ in test set-up a) the contact force will be reduced by a force equal to F_t. As long as F_t is less than F_v, no cyclic load effect will occur in the bolt. In the test set-up as indicated in b) the contact force does not change when an external force is applied. This means that all cyclic load effects will appear in the bolt. Set-up c) shows the behaviour when two flanges are connected without a contact plate but with a contact area near the web due to the deformation of the plate caused by hot rolling or by the process of welding the end plate.

Q&A 6.9 Joints Loaded by Bending Moment and Axial Force

What approach should be used to design a haunched connection, which is loaded by axial force and bending moment?

There are two parts in this question, which will be discussed separately: What is the influence of an inclination of the beam on the design of the joint? What is the influence of an axial load on the moment resistance of a joint?

The rules in prEN 1993-1-8 only apply when the forces have been resolved in to their horizontal (parallel) and vertical (perpendicular) components. The inclination of the beam causes the geometry of the joint to change. The increase in level arm as a result of the inclination of the beam should be taken into account.

The moment resistance of a joint loaded with an axial load can be determined from a linear interaction line between M_{Sd} and N_{Sd}, as shown in Figure 6.16. The linear interaction line is found by determining the extreme values of the moment resistance (M_{Rd}) with no axial load and the axial load resistance (N_{Rd}) without bending moment. The design resistance of the joint should be checked using the following equation

$$\frac{N_{Sd}}{N_{Rd}} + \frac{M_{Sd}}{M_{Rd}} \leq 1 . \tag{6.6}$$

In some cases this is a conservative approach, especially for non-symmetrical joints. An alternative approach was developed, e.g. see [Jaspart et al, 1999], [Sokol et al, 2002]. In Figure 6.16 the point ① represents the maximum bending resistance; ② bending resistance in case of zero axial force; ③ maximum resistance in compression; ④ resistance in compression in case of zero bending moment; ⑤ negative bending in case of zero axial force; ⑥ maximum negative bending resistance; ⑦ point of activation of second bolt row; ⑧ resistance in axial tension; ⑨ point of activation of second bolt row.

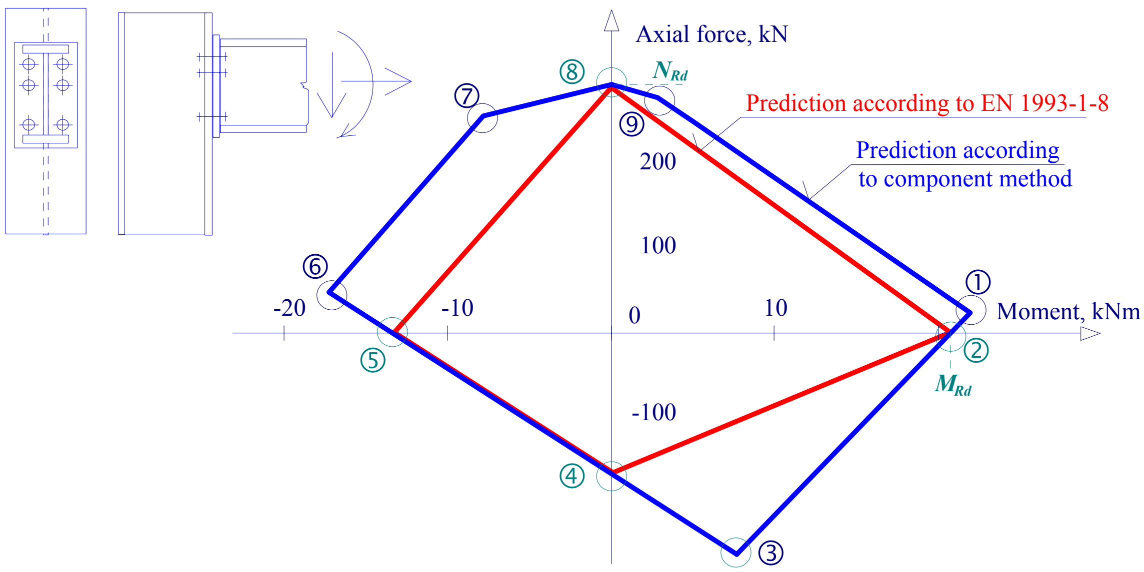
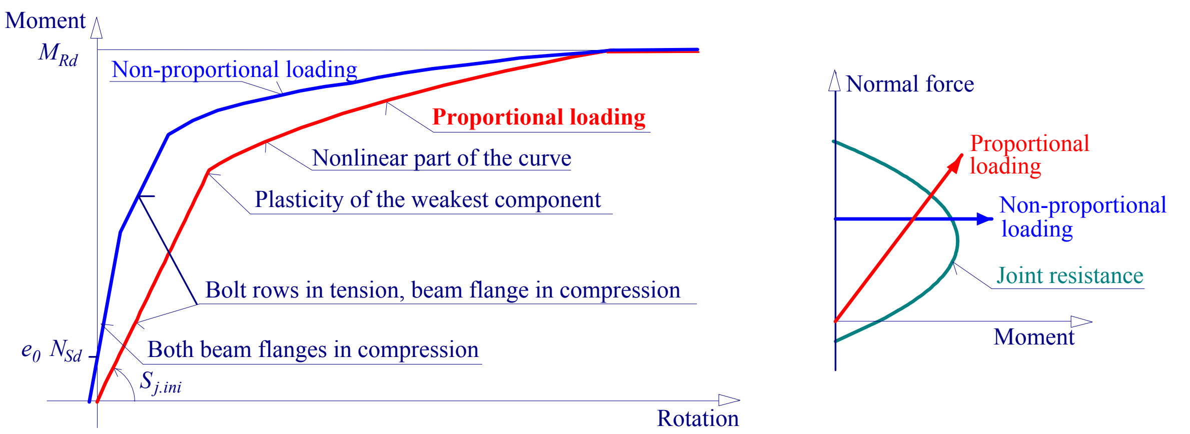


Figure 6.16 Moment – axial load interaction curve, prediction according to prEN 1993-1-8 is marked by dotted line; the component method is marked as a solid line

This approach is based on an extension of the component method used for base plates in prEN 1993-1-8. Properties of the components should be evaluated in the same way as for joints without axial force and are used in a modified assembly procedure to calculate the resistance and stiffness of the joint [Sokol et al, 2002].

Two typical loading paths may be distinguished, see Figure 6.17. In the case of non-proportional loading, the normal force is applied to the end plate connection in the first step, followed by application of the moment. In the case of proportional loading, the normal force and the bending moment are applied simultaneously at a constant ratio between the moment and normal force. In the case of non-proportional loading, the initial stiffness of the joint is higher than for non-proportional loading. This effect is caused by the presence of the normal force, which keeps the end plate in contact with the column flange at low bending moments. Therefore only the components in compression contribute to the deformation of the joint.



a) moment - rotational curve *b) loading paths on the moment - axial force interaction diagram*

Figure 6.17 Proportional and non-proportional loading

The size and shape of the contact area between the end plate and the column flange are based on the effective rigid area [Wald, 1995]. The position of the neutral axis can be evaluated from equilibrium equations, taking into account the resistance of the tension and compression parts $F_{t,Rd}$ and $F_{c,Rd}$ respectively, and the applied normal force N_{Sd} and bending moment M_{Sd}. Plastic distribution of internal forces is assumed for the calculation, see Figure 6.18.

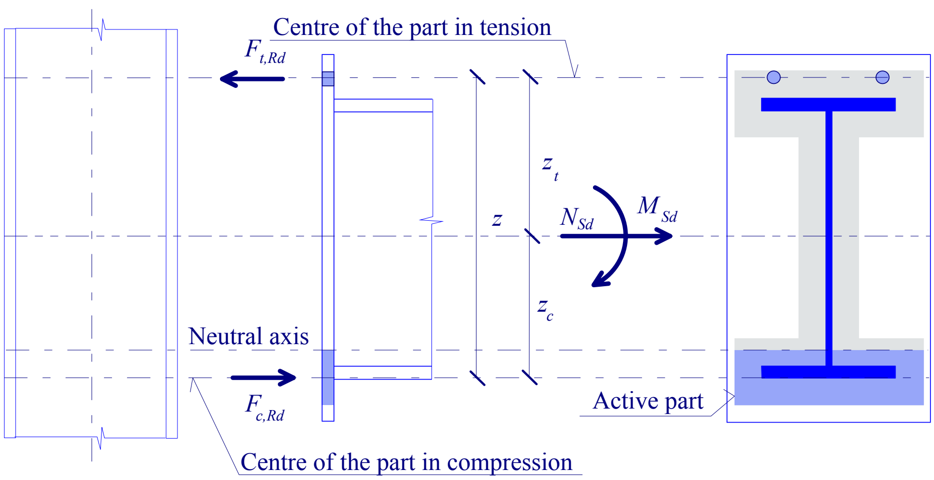
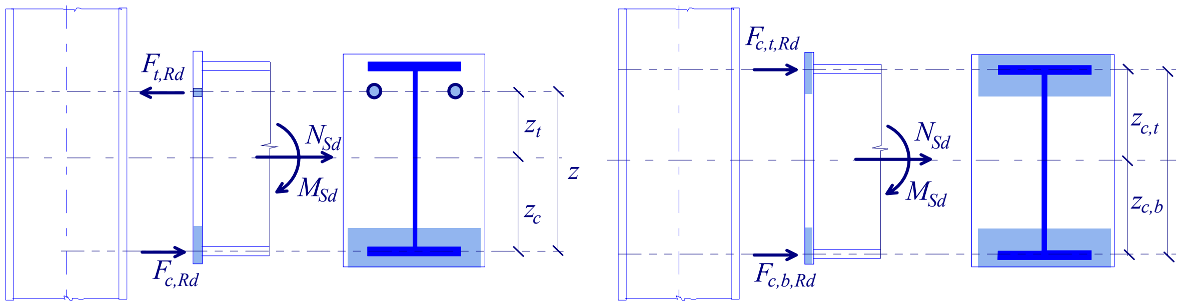


Figure 6.18 The equilibrium in the joint

The simplified model takes into account only the effective area at beam flanges [Steenhuis, 1998] and the effective area of the beam web is neglected, as shown in Figure 6.19. It is assumed that the compression force acts at the centre of the compression flange. The tension force is located in the bolt row in tension. In the case of two or more bolt rows in the tension part, the resistance of the part in tension is obtained as the resulting force of the active bolt rows.



a) one bolt row in tension *b) no bolts in tension*

Figure 6.19 The simplified model with the effective area at the flanges only

The forces represent resistances of the components in tension $F_{t,Rd}$, and in compression $F_{c,t,Rd}$, $F_{c,b,Rd}$. For simplicity, the model will be derived for proportional loading only. Using equilibrium equations, the following equations may be derived from Figure 6.20a assuming the eccentricity $e = M_{Sd} / N_{Sd} \leq -z_c$.

$$\frac{M_{Sd}}{z} + \frac{N_{Sd}\ z_c}{z} \leq F_t \tag{6.6}$$

and

$$\frac{M_{Sd}}{z} - \frac{N_{Sd}\ z_t}{z} \leq -F_c\,. \tag{6.7}$$

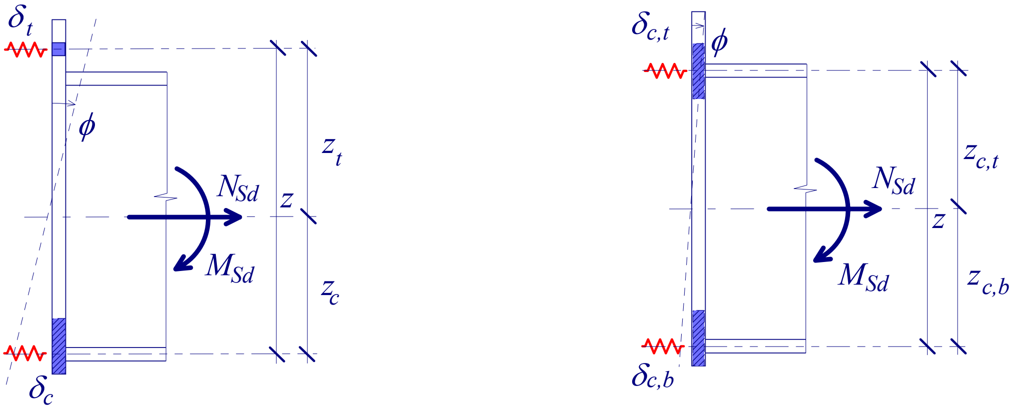
Since the eccentricity $e = \dfrac{M_{Sd}}{N_{Sd}} = \dfrac{M_{Rd}}{N_{Rd}} = const$ for proportional loading, equations (6.6) and (6.7) may be rearranged to

$$M_{j.Rd} = min\left\{\begin{matrix} \dfrac{F_{t,Rd}\ z}{\dfrac{z_c}{e}+1} \\ \\ \dfrac{F_{c,Rd}\ z}{1-\dfrac{z_{t,l}}{e}} \end{matrix}\right\}. \tag{6.8}$$

When the eccentricity $e = M_{Sd} / N_{Sd} \geq -z_c$, see Figure 6.20b, there is no tension force in the bolt row, but both parts of the connection are loaded in compression. In this case, equation (6.8) needs to be modified to give

$$M_{j,Rd} = min\left\{\begin{matrix} \dfrac{-F_{c,t,Rd}\ z}{\dfrac{z_{c,b}}{e}+1} \\ \\ \dfrac{-F_{c,b,Rd}\ z}{\dfrac{z_{c,t}}{e}-1} \end{matrix}\right\}. \tag{6.9}$$

The rotational stiffness of the connection is based on the deformation of the components.



a) bolts in tension *b) both flanges in compression*

Figure 6.20 Mechanical model of the end plate

The elastic deformation of the components in tension and compression parts, see Figure 6.20a, may be expressed as

$$\delta_t = \frac{\dfrac{M_{Sd}}{z} + \dfrac{N_{Sd}\ z_c}{z}}{E\ k_t} = \frac{M_{Sd} + N_{Sd}\ z_c}{E\ z\ k_t}\ , \tag{6.10}$$

$$\delta_c = \frac{\dfrac{M_{Sd}}{z} - \dfrac{N_{Sd}\ z_t}{z}}{E\ k_{c.r}} = \frac{M_{Sd} - N_{Sd}\ z_t}{E\ z\ k_c}\ , \tag{6.11}$$

and the joint rotation is calculated as follows using the deformation of these components

$$\phi = \frac{\delta_t + \delta_c}{z} = \frac{1}{E\ z^2}\left(\frac{M_{Sd} + N_{Sd}\ z_c}{k_t} + \frac{M_{Sd} - N_{Sd}\ z_t}{k_c}\right). \tag{6.12}$$

The rotational stiffness of the joint depends on the bending moment, which is induced by the normal force applied with constant eccentricity e

$$S_{j.ini} = \frac{M_{Sd}}{\phi}\ . \tag{6.13}$$

The stiffness is derived by substitution of the rotation of the joint (6.12) into equation (6.13)

$$S_{j.ini} = \frac{M_{Sd}}{M_{Sd} + N_{Sd}\ e_0}\ \frac{E\ z^2}{\left(\dfrac{1}{k_c} + \dfrac{1}{k_t}\right)} = \frac{e}{e + e_0}\ \frac{E\ z^2}{\sum \dfrac{1}{k}}\ , \tag{6.14}$$

where the eccentricity e_0 is defined as follows

$$e_0 = \frac{z_c\ k_c - z_t\ k_t}{k_c + k_t}\ . \tag{6.15}$$

The non-linear part of the moment-rotation curve may be modelled by introducing the stiffness ratio μ, which depends on the ratio γ of the acting forces and their capacities

$$\mu = (1{,}5\ \gamma)^{2,7} \geq 1\ . \tag{6.16}$$

Assuming the lever arms z_t and z_c of the components are approximately equal to $h/2$, i.e. one half of the height of the connected beam, the factor γ can be defined as

$$\gamma = \frac{M_{Sd} + 0{,}5\ h\ N_{Sd}}{M_{Rd} + 0{,}5\ h\ N_{Rd}}\ , \tag{6.17}$$

and by substituting for the eccentricity e, this can be simplified to

$$\gamma = \frac{e + \dfrac{h}{2}}{\left(\dfrac{M_{Rd}}{M_{Sd}}\right) e + \dfrac{h}{2}}\ . \tag{6.18}$$

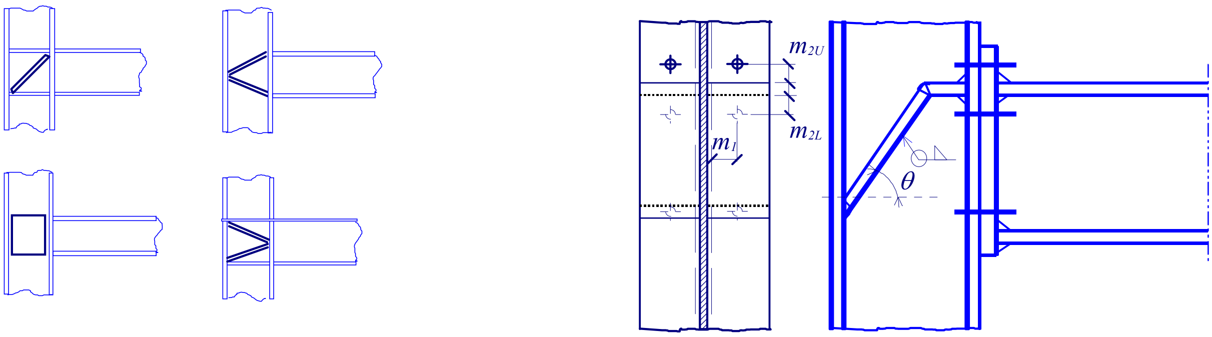
Using the factor μ the moment-rotation curve of a joint, which subject to proportional loading, may be expressed by following equation

$$S_j = \frac{e}{e + e_0}\ \frac{E\ z^2}{\mu \sum \dfrac{1}{k}}\ . \tag{6.19}$$

Q&A 6.10 Stiffening of the Column Web Panel with a Morris Stiffener

Can the general rules for transverse stiffeners given in prEN 1993-1-8 be applied to Morris stiffeners?

When the resistance of the web is inadequate, web plates, diagonal or "K" stiffeners can be used to increase its capacity. However, the Morris shear stiffener has been developed to resolve two problems simultaneously – the shear capacity of the column web and the distortion of the column flange.



a) diagonal stiffener, web plate, K stiffeners *b) Morris shear stiffener*

Figure 6.21 Shear stiffener of column web

Tests have been carried out to compare the Morris stiffening arrangement with traditional ways of stiffening and the experimental results show that the Morris shear stiffener is structurally efficient. It has better initial stiffness and post-yield performance when compared to 'K' stiffeners. Furthermore, it is economic to manufacture and it overcomes the difficulties of bolt access associated with the other stiffeners. It is particularly effective for use with UB sections used as columns, but it is difficult to accommodate in smaller UC sizes. The horizontal portion carries the same forces as a tension stiffener at the same location. The length should be sufficient to provide for bolt access (about *100 mm*). The diagonal portion should be designed as a diagonal stiffener as follows: The area of the stiffeners $A_{sg} = 2\, b_{sg}\, t_s$ is given for the thickness of the stiffener t_s and the width of the stiffener on each side b_{sg} by

$$A_{sg} \geq \frac{F_v - F_{v,w,Rd}}{f_{yd}\ cos\,\theta}, \tag{6.20}$$

where F_v is the applied shear force, $F_{v,w,Rd}$ is the resistance of the unstiffened column web panel, f_{yd} is the lower design strength of stiffener or column and θ is the angle of the stiffener from the horizontal.

The welds connecting the diagonal stiffeners to the column flange should be fill-in welds with a sealing run providing a combined throat thickness equal to the thickness of the stiffener. Welds connecting the horizontal part of the Morris stiffener to the column flange should be designed to provide a net throat thickness equal to $A_{sn} = 2\, b_{sn}\, t_s$ given by

$$A_{sn} \geq \frac{m_1}{f_{yd}} \left(\frac{F_{ri}}{m_1 + m_{2L}} + \frac{F_{rj}}{m_1 + m_{2U}} \right), \tag{6.21}$$

where m_1 is the distance from the centre of the bolt to the root radius, m_{2L} is the distance from the edge of the stiffener to the centre of the lower bolt row, m_{2U} is the distance from the edge of the stiffener to the centre of the upper bolt row, F_{ri} is the tension force in the upper bolt row, F_{rj} is the tension force in the lower bolt row and f_{yd} is the design strength of the stiffener or the column (the lesser of the two).