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Equaljoints PLUS
Volume with pre-normative design recommendations for seismically qualified steel joints

1st Edition, 2018

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EQUALJOINT PLUS
Volume with pre-normative design recommendations for seismically qualified steel joints

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1. INTRODUCTION

The current document has been developed within the European RFCS project Equaljoints PLUS (754048 — EQUALJOINTS-PLUS — RFCS-2016/RFCS-2016).
Equaljoint-PLUS is a 24 month RFCS project devoted to disseminate the knowledge achieved within the previous RCFS 36 months-project EQUALJOINTS
Within the previous RFCS project EQUALJOINTS (RFSR-CT-2013-00021), European seismic prequalification criteria of a set of steel been-to-column joints have been developed.
Equaljoint-PLUS aims at the valorisation, the dissemination and the extension of the developed prequalification criteria for practical applications to a wide audience (i.e. academic institutions, Engineers and architects, construction companies, steel producers).
The main objectives of the Equaljoints PLUS can be summarized as follows:

- To collect and organize informative material concerning the prequalified joint typologies: informative documents have been prepared in 12 languages (English, Spanish, French, German, Italian, Dutch, Portuguese, Czech, Bulgarian, Romanian, Greek, and Slovenian).
- To develop pre-normative design recommendations of seismically qualified joints based on results from Equaljoints project in 12 languages.
- To develop design guidelines to design steel structures accounting for the type of joints and their relevant non-linear response.
- To develop a software and an app for mobile to predict the inelastic response of joints.
- To organize seminars and workshops for disseminating the gained knowledge over EU and internationally.
- To create a web site with free access to the users to promote the obtained results.
- To create a You-Tube channel to make available the videos of the experimental tests and simulations to show the evolution of damage pattern.

The Equaljoints PLUS project is coordinated by the University of Naples Federico II. The Consortium consists of 15 partners, 7 of which already involved in the former Equaljoints project. All the partners involved are listed in the following table:

Equaljoints PLUS Consortium	
Coordinator	Università degli Studi di Napoli Federico II (UNINA)
Partners	Arcelormittal Belval & Differdange SA (AM)
	Universite de Liege (Ulg)
	Universitatea Politehnica Timisoara (UPT)
	Universidade de Coimbra (UC)
	Convention Europeenne de la Construction Métallique (ECCS)
	Universita degli Studi di Salerno (UNISA)
	Imperial College of Science Technology and Medicine (IC)
	Centre Technique Industriel de la Construction Métallique (CTICM)
	National Technical University of Athens (NTUA)
	Ceske Vysoke Uceni Technicke V Praze (CVUT)
	Technische Universiteit Delft (TUD)
	Univerza V Ljubljani (UL)
	Universitet Po Architektura Stroitelstvo I Geodezija (UASG)
	Universitat Politecnica de Catalunya (UPC)
	Rheinisch-Westfaelische Technische Hochschule Aachen (RWTHA)

The present document summarizes design guidelines for three types of bolted joints prequalified within Equaljoints Project (i.e. (i) haunched, (ii) stiffened extended end-plate and (iii) unstiffened extended end-plate beam-to-column joints) as well as welded dog bone joints.
In detail, the following information is provided:

– technological requirements;
– description of the joint configurations;
– list of systems for which the joints are prequalified;
– list of limit values for prequalified data;
– design procedure.

2. TECHNOLOGICAL REQUIREMENTS

The construction of a structure undergoes several stages, each of which must be thoroughly thought. In structures that may be subject to seismic actions at some point of their use life, these considerations are especially significant. Joints between steel elements in this type of structures should always be designed, fabricated and erected such that fragile failure is avoided and a ductile mode of failure governs the collapse.

Designers must always bear in mind design requirements set by the relevant design standards. In Europe, EN1998 must be observed for the seismic design of structures, with significant reference to EN1993 for the design of steel structures and EN1993-1-8 in particular for the design of steel joints.

EN1993-1-8 defines all parameters relevant to the design of connections with respect to their strength and stiffness. Connections may be welded, bolted or combinations of bolts and welds may be used.

Bolted connections must be designed in accordance with EN1993-1-8, Section 3. Table 3.1 of the standard defines the bolt classes and their nominal yield and ultimate stresses. Table 3.3 of the same document sets out the minimum and maximum pitch, end and edge distances in order to ensure enough bearing capacity. Connections are designed following the component resistance method. When the capacities of each component are calculated, a ductile failure mode (such as bolt bearing, bearing on the supporting element or on the plates) must be the governing criterion.

The design criteria for welded joints are described in EN1993-1-8, Section 4. In seismic design, welds are usually designed to be full strength and thus avoid weld failure (fragile failure mode).

When specifying the materials and dimensions, the engineer should always consider the standard available element dimensions and characteristics of the raw elements. For example, the fabricator can source standard plates of 10 or 12 mm thickness, designers should not specify 11 mm thick plates, in order to avoid unnecessary machining as far as possible.

Material toughness and through-thickness properties are given in EN 1993-1-10. EN 1993-1-10 contains design guidance for the selection of steel for fracture toughness and for through-thickness properties of welded elements where there is a significant risk of lamellar tearing during fabrication, for constructions executed in accordance with EN 1090-2.

The guidance given in Section 2 of EN 1993-1-10 shall be used for the selection of material for new construction. The rules shall be used to select a suitable steel grade from the European Standards for steel products listed in EN 1993-1-1.

The choice of Quality Class shall be selected from Table 3.1 EN 1993-1-10 depending on the consequences of lamellar tearing.

Depending on the Quality Class selected from Table 3.1, either: through thickness properties for the steel material shall be specified from EN 10164, or post-fabrication inspection shall be used to identify whether lamellar tearing has occurred.

Guidance on the avoidance of lamellar tearing during welding is given in EN 1011-2.

National choice is allowed through clauses listed in the Foreword to EN 1993 1-10.

Designers and fabricators must work closely together to ensure the accuracy and clarity of the drawings. On occasions, the designer fails to recognize situations where what has been drawn cannot actually be executed, for example due to a lack of adequate space for welding. Often several meetings are required before both parties are satisfied that the graphic representation of the design is correct and can be fabricated.

The fabrication of the elements, including assembly, transportation and erection on site, must all be carefully managed in order to ensure the final quality of the structure is appropriate.

Structures must be executed in accordance with the relevant execution standards, namely EN1090-2 in Europe and AISC 303-10 in the USA, which set minimum quality requirements. Based on experience some fabricators may choose to exceed these requirements, and thus avoid known issues that often arise during erection on site.

3. CHARACTERIZATION OF THE PREQUALIFIED JOINTS

3.1 General performance objectives

Strength criterion: According to EC8, the seismic design of steel structures is based on the concept of dissipative structures, where specific zones of the structures should be able to develop plastic deformation in order to dissipate the seismic energy. On the contrary, the non-dissipative parts should behave elastically under seismic action in order to avoid brittle collapse. The hierarchy of the resistances is the fundamental principle allowing this performance by detailing non-dissipative zones to resist the full plastic strength of the related dissipative members. The design criteria used within Equaljoints project aim at harmonizing the hierarchy requirements among the strengths of macro-components (e.g. the web panel, the connection, the beam and the column), and their sub-components (e.g. end-plate, bolts, welds, etc.), as well.
According to design procedure developed within the project, the joint is considered as made of three macro-components (i.e. the column web panel, the connection zone, and the beam zone, see Figure 3.1); each macro-component is individually designed according to specific assumptions and then simply capacity design criteria are applied, in order to obtain different design objectives.

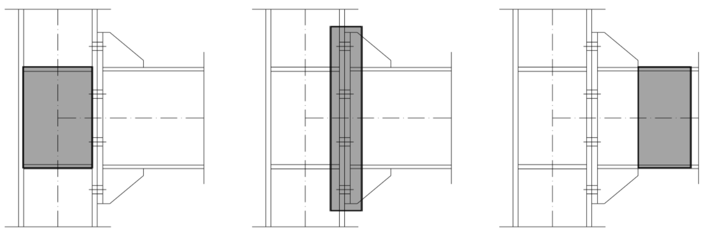


Figure 3.1: Plastic regions for the examined performance design objectives:
a) web panel, b) connection and c) beam

The following design objectives can be adopted for the connection:
- Full strength connection is designed to be stronger than other macro-components, such that yielding occur in other parts of the joint (beam or column web panel).
- Equal strength connection is designed to have a strength close to the one of the beam or column web panel, or both. Theoretically yielding should occur in all, or two of the three macro-components.
- Partial strength connection is designed to develop plastic deformations with its components.

For the column web panel an addition design objective can be introduced:
- Full strength column web panel is designed to be stronger than other macro-components, such that yielding occur in other parts of the joint (beam or connection).

- Equal strength column web panel is designed to have a strength close to the one of the beam or the connection, or both.
- Partial strength column web panel is designed to develop plastic deformations exclusively with itself.

It should be also noted that both EC3 and EC8 do not consider the case of equal strength joint, which is proposed within the project as an intermediate performance level. According to the current Eurocode classification, an equal strength joint falls on the category of partial strength.
The primary source of plastic deformations in the seismic design situation is the beam end. Depending on the location of the plastic hinge, amount of strain hardening and the expected yield strength at the plastic hinge, the design moment at the column face may be obtained as:

$$M_{con,Ed} = \alpha \cdot \left(M_{B,Rd} + V_{B,Ed} \cdot s_h\right) \tag{3.1}$$

Where $M_{con,Ed}$ is the design bending moment at the column face; α depends on the design performance level. It is equal to $\gamma_{sh} \cdot \gamma_{ov}$ for the full strength joints (being γ_{ov} the overstrength factor due to the material randomness, and γ_{sh} the strain hardening factor corresponding to the ratio between the ultimate and the plastic moment of the beam), while equal to 1 for equal strength joints and smaller than 1 for partial strength joints. In order to avoid too severe damage concentration in the connection zone, the strength ratio for partial strength joints is assumed equal to 0.6 or 0.8. $M_{B,Rd}$ is the plastic flexural strength of the connected beam; s_h is the distance between the column face and the plastic hinge occurring in the connected beam (namely it is the distance between the column face and at the tip of the stiffener for haunched and stiffened endplate joints), it is equal to zero for unstiffened endplate joints); $V_{B,Ed}$ is the shear force corresponding to the occurring of the plastic hinge in the connected beam; it is given by:

$$V_{B,Ed} = V_{B,Ed,M} + V_{B,Ed,G} \tag{3.2}$$

where $V_{B,Ed,M}$ is the shear force due to the formation of plastic hinges at both beam ends, spaced by the length L_h and calculated as:

$$V_{B,Ed,M} = \frac{2 \cdot M_{B,Rd}}{L_h} \tag{3.3}$$

$V_{B,Ed,G}$ is the contribution due to the gravity loads; it should be noted that this amount does not account for the distance between the column face and plastic hinge e L_h is the approximate distance between plastic hinges.

Concerning both the overstrength factors, further considerations are necessary: γ_{ov} is assumed equal to 1.25, as recommended by EC8. The strain hardening factor γ_{sh} is assumed differently by EN1993-1-8 and EN1998-1. In particular, EN1993-1-8 recommends to consider an overstrength ratio equal to 1.2 for full strength joints, while EN1998-1 contradictorily assumes a value equal to 1.1. Several empirical equations are available in literature to estimate the flexural overstrength γ_{sh} developed by steel beams. Based on the main findings obtained by Mazzolani and Piluso (1992), D'Aniello *et al* (2012), Güneyisi *et al* (2013, 2014) it can be argued that γ_{sh} factor ranges within 1.1-1.2 for European profiles commonly used for beams (e.g. IPE), thus larger than the value recommended by EC8, but in line with AISC358-10 that assumes the following overstrength factor:

$$\gamma_{sh,AISC} = \frac{f_y + f_u}{2 \cdot f_y} \leq 1.20 \tag{3.4}$$

Therefore, in the current procedure γ_{sh} is conservatively assumed equal to 1.20, based also on the characteristic yield and ultimate strength of European mild carbon steel grades.

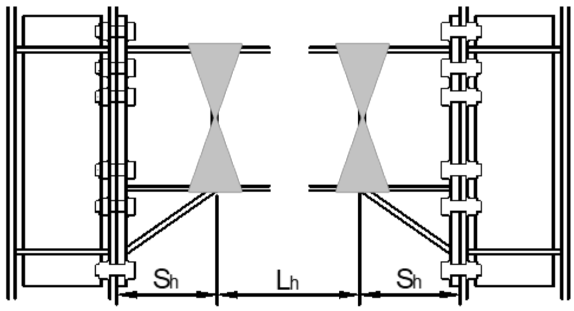


Figure 3.2: Position of plastic hinges in haunched and extended stiffened endplate joints

The shear force in the column web panel may be determines as:

$$V_{wp,Ed} = \alpha \cdot \left(M_{B,Rd} + V_{B,Ed} \cdot s_h \right) / z - V_{c,Ed} \tag{3.5}$$

where

$V_{wp,Ed}$ is the design shear force in the column web panel;

$V_{c,Ed}$ is the shear force in the column;

z is the internal lever arm;

α depends on the design performance level, and may be different from the one used to design the connection.

Depending on the required design objectives of the joint, the following inequalities shall be checked:

$$M_{con,Rd} \geq M_{con,Ed} \tag{3.6}$$

$$V_{wp,Rd} \geq V_{wp,Ed} \tag{3.7}$$

where:

$M_{con,Rd}$ is the flexural strength of the connection.

$V_{wp,Rd}$ is the shear resistance of the column web panel.

Ductility criterion: The joint ductility depends on the type of failure mode and the corresponding plastic deformation capacity of the activated component. Figure 7.10 concisely depicts the dependency of failure mode on geometric properties and end-plate to bolt strength ratio (Jaspart, 1997). In abscissa it is reported the ratio β between the flexural strength ($M_{pl,Rd}$) of the plates or column flanges, and the axial strength of the bolts ($F_{t,Rd}$), while the vertical axis reports the ratio η between the T-stub strength (F) over $F_{t,Rd}$. The strength for mode 1 in case of non-circular pattern depends on the ratio $v = n/m$, where m is the distance between the bolt axis and the flange-to-web expected location of the plastic hinge, and n is the minimum of the distance between the edge of the flange and the bolts axis or 1.25m. In line with Figure 7.10, two possible ductility criteria can be adopted to avoid mode 3, namely:

Level-1: $\beta \leq 1$ this condition imposes either a failure mode I or failure mode II (but very close to mode I), which provide very high ductility.

Level-2: $\beta < 2$ and $\eta \leq 0.95$, this condition imposes a failure mode II with limited ductility, but avoiding brittle failure.

The level of ductility to be guaranteed obviously depends on the design performance objectives: it is crucial providing the larger ductility for equal and partial strength, less for full strength joints.

According to the EN1993-1-8, the joint rotation capacity should be checked if M_{jRd} is less than 1.2 $M_{B,pl,Rd}$ and two alternative ways can be pursued: 1) performing experimental tests; 2) controlling the thickness t of either end-plate or column flange, provided that the joint design moment resistance is governed by those components, which should satisfy the following inequality:

$$t \leq 0.36d\sqrt{\frac{f_{ub}}{f_y}} \tag{3.8}$$

where d is the nominal bolt diameter, f_y is the yield strength of the relevant basic component and f_{ub} is the bolt ultimate strength.

Eq. (3.8) would theoretically comply with the ductility Level-1 depicted in Figure 3.3, assuming that the resistance of each individual bolt ($F_{t,Rd}$) is greater than the resistance ($F_{p,Rd}$) of the connected plates (end-plate or column flange). In particular, the design resistance of a bolt in tension ($F_{t,Rd}$) is given as follows:

$$F_{t,Rd} = \frac{0.9 A_s f_{ub}}{\gamma_{M2}} \tag{3.9}$$

where A_s is the tensile stress area of the bolt and γ_{M2} is the relevant partial safety factor (i.e. Eurocode recommended value is equal to 1.25).

In addition, Eq. (3.8) uses the design resistance ($F_{p,Rd}$) corresponding to a circular mechanism, which can be assumed as follows:

$$F_{p,Rd} = \frac{\pi t^2 f_y}{\gamma_{M0}} \tag{3.10}$$

where t is the plate thickness and γ_{M0} is the relevant partial safety factor (recommended equal to 1).

It should be noted that Eq. (3.9 and 3.10) assume perfectly plastic behaviour of steel plates. However, in light of the considerations previously discussed, the ductility Level-1 for seismic resistant Partial strength joints should be expressed accounting for both the random variability of plate material and its relevant strain hardening, so that the following inequality can be used:

$$F_{t,Rd} \geq \gamma \cdot F_{p,Rd} = \gamma_{ov} \cdot \gamma_{sh} \cdot F_{p,Rd} \tag{3.11}$$

The overstrength factor γ in Eq. (3.11) can be taken equal to 1.5, since the Eurocode recommended value for γ_{ov} is equal to 1.25, the value for γ_{sh} is equal to 1.2 for European mild carbon steel, and the recommended partial safety factor γ_{M0}

is equal to 1.0. Thus, rearranging the inequality (3.11) with Eq. (3.8), the ductility condition accounting for capacity design criteria can be expressed as following:

$$t \le \frac{0.42 \cdot d}{\sqrt{\gamma_{ov} \cdot \gamma_{sh}}} \cdot \sqrt{\frac{\gamma_{M0} \cdot f_{ub}}{\gamma_{M2} \cdot f_y}} \cong 0.30 \cdot d \cdot \sqrt{\frac{f_{ub}}{f_y}} \qquad (3.12)$$

Regarding full and equal strength joints, even though either no or poor ductility should be exploited respectively, a local hierarchy criterion is advisable in order to avoid undesirable failure mode in the brittle components due to material variability. Hence, in line with ductility Level-2, the strength of bolts should satisfy the following inequality:

$$F_{t,Rd} \ge \gamma_{ov} \cdot F_{p,Rd} \qquad (3.13)$$

Eq. (3.13), can be rearranged and after some algebraic manipulations it provides a similar criterion given by Eq. (3.8).
It is important to highlight that all criteria previously described require that failure of welds has to be unquestionably avoided, because of their brittle collapse mechanism.

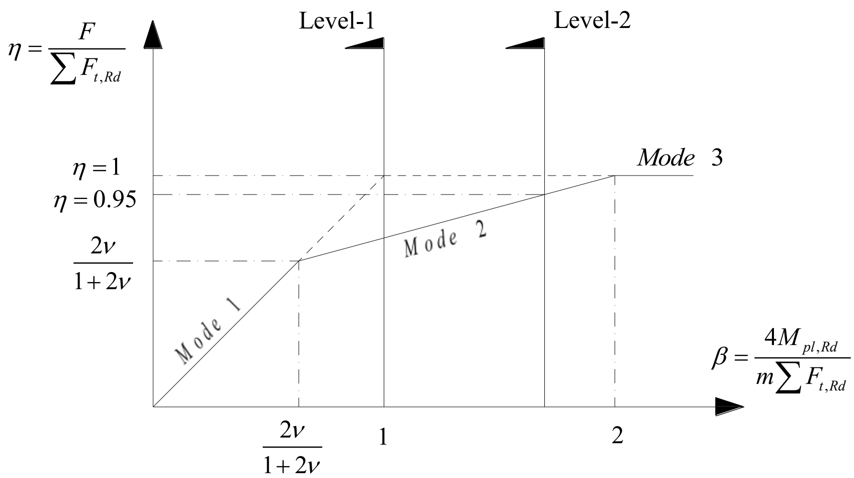


Figure 3.3: Ductility criterion: T-Stub resistance and corresponding failure mechanism

3.2 Further design assumptions

3.2.1 Active bolt row in tension

Differently from the component method implemented in EN1993-1-8, where all bolt rows in tension are rigorously obtained by imposing the equilibrium with compression internal resultant, the number of active bolt-rows in tension is assumed *a-priori* as shown, since the contribution of bolt rows below the central axis of the connection is reasonably negligible under pure bending condition.

3.2.2 Centre of compression and lever arm

For end-plate joints EN 1993-1-8 specifies that the compression centre is located in the middle of thickness of beam flange, or at the tip of the haunch in case of haunched joints. Experimental and numerical results showed that the location of compression centre depends on both the joint type and the rotation demand due to the formation of plastic modes with different engagement of each joint component. According to the proposed design procedure and based on both experimental and numerical results from literature (Lee, 2002; Lee *et al*, 2005; Abidelah *et al*, 2012) and achieved within the project condition (Maris *et al*, 2015, Stratan *et al*, 2016, D'Aniello *et al*, 2017; Tartaglia and D'Aniello, 2017, Tartaglia *et al*, 2018), the location of compression centre is assumed as follows: (i) in the middle of thickness of beam flange for unstiffened endplate joints (see Figure 3.4a); (ii) at the centroid of the section made by the beam flange and the rib stiffeners, for the stiffened endplate joints (see Figure 3.4b); (iii) at 0.5 the haunch height h_h, in case of haunched joints (see Figure 3.4c).

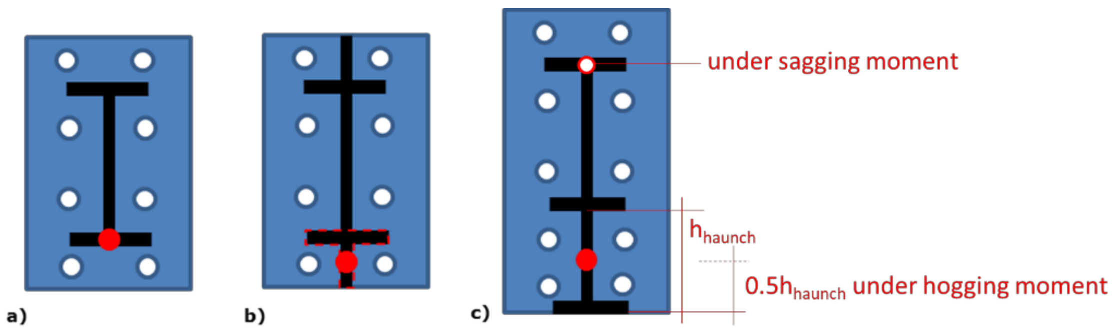


Figure 3.4: Location of compression centre for different joint types (a) unstiffened endplate (b) stiffened endplate (c) haunched connections.

3.3 Haunched beam-to-column joints

3.3.1 Description of the joint configuration

Haunched extended end-plate beam-to-column connections are intended to provide a full-strength and rigid connection, with strong or balanced column web panel. The configuration of haunched extended end-plate beam-to-column joints is described in Figure 3.5. The connection uses an extended end-plate with high-strength bolts, and is reinforced using a haunch below the bottom flange of the beam. Transverse column and beam stiffeners are mandatory. Supplementary web plates are optional, and can be used to enhance the stiffness and strength of the column web panel.

Haunch angle is measured between the bottom flange of the beam and the flange of the haunch, and can range from 30° to 45°.Types of welds for which the haunched beam-to-column joints were prequalified are shown in Figure 3.6. All welds are designed to allow transfer of forces corresponding to the resistance of the welded parts. This is accomplished by using two fillet welds (both sides of the plate) with a minimum throat of 0.55 times the thickness of the plate. Critical welds (top beam flange, haunch flange, supplementary web plate to column flange) are full-penetration groove welds. Top beam flange and haunch flange groove welds are further reinforced with additional fillet welds.

3.3.2 List of systems for which connection is prequalified

Haunched extended end-plate beam-to-column connections described in this document are prequalified for the following structural systems:

- Moment Resisting Frames (MRFs);
- Dual Concentrically Braced Frames (i.e. MRF+ CBFs);
- Dual Eccentrically Braced Frames (i.e. MRF+ EBFs).

In addition, these joints should be used only in frames with perpendicular beam-column axis and regular span layout of the seismic resisting system, namely no sloped beams.

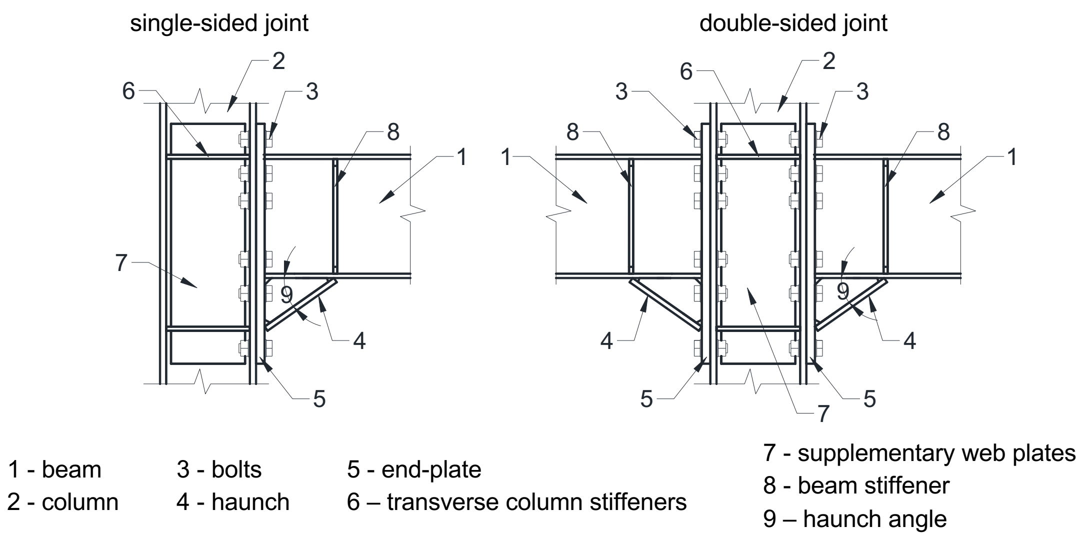


Figure 3.5: Description of haunched extended end-plate joints

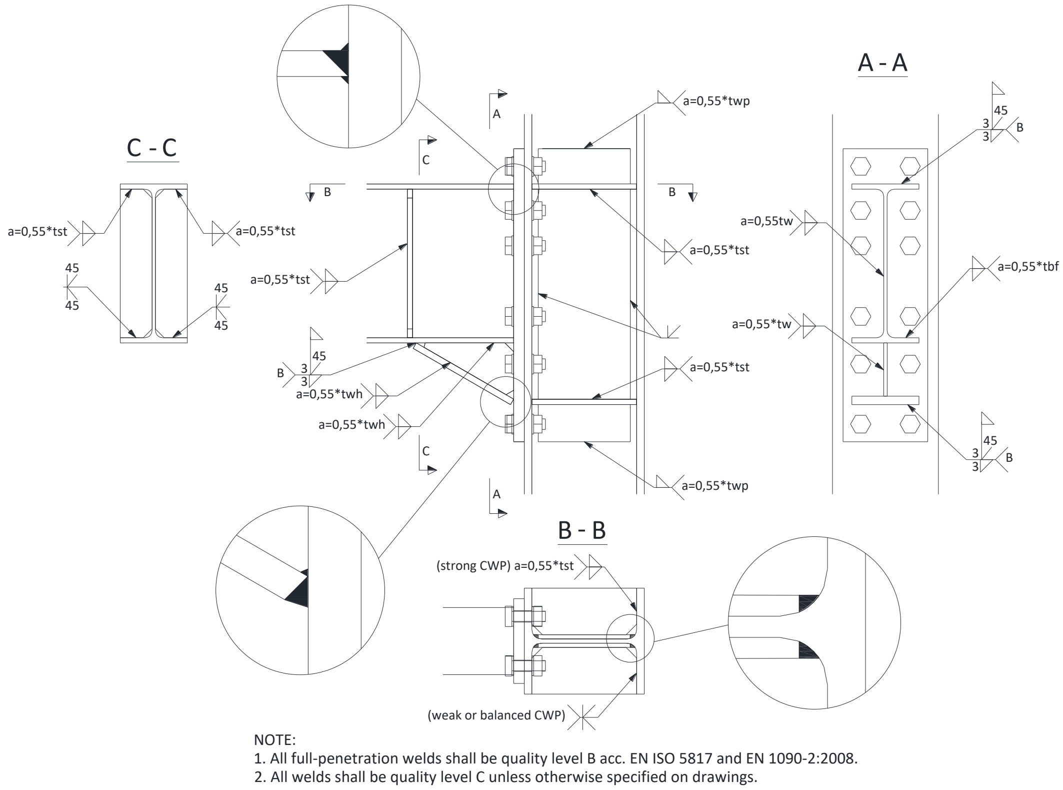


Figure 3.6: Weld details for haunched extended end-plate joints

3.3.3 List of limit values for prequalified data

Table 3.1: **List of limit values for prequalified data**

Elements	**Application range**
Beam	Hot-rolled wide-flange beams ranging from IPE330 up to IPE600. Cross-section shall be class 1 according to EN 1993-1-1. Built-up beams with similar cross-section shape may be used, provided welds between the web and flanges are full-penetration groove welds reinforced with fillet welds.
Depth	330 to 600 mm
Clear span-to-depth ratio (between the assumed location of plastic hinges)	Minimum 7
Flange thickness	Minimum: 11 mm Maximum: 21 mm* (10% extrapolation with respect to the maximum tested)
Material	S235 to S355
Column	Hot-rolled wide-flange columns ranging from HEB260/HEM260 up to HEB550/HEM550. Cross-section shall be class 1 according to EN 1993-1-1. Built-up columns with similar cross-section shape may be used, provided welds between the web and flanges are full-penetration groove welds reinforced with fillet welds.

Table 3.1: **List of limit values for prequalified data (cont.)**

Elements	Application range
Depth	260 to 550 mm
Flange thickness	Minimum: 17.5 mm Maximum: 40 mm
Material	From S235 to S355
Beam/column depth	0.60-2.00
End-plate	20-40
Thickness	Minimum: 20 mm Maximum: 40 mm
Width	Minimum: beam flange width + 30 mm Maximum: column flange width
Material	From S235 to S355
Transverse column and beam stiffeners	According to requirements of EN 1993-1-8 and EN 1998-1.
Material	From S235 to S355
Supplementary web plates	According to requirements of EN 1993-1-8 and EN 1998-1. It is allowed to consider the full area of the supplementary web plates in computing the additional shear strength of column web panel.
Height	At least equal to the height of the end plate.
Material	From S235 to S355
Bolts	High strength structural bolting assemblies for preloading, according to EN 14399-3 (system HR) and EN 14399-4 (system HV). Bolts shall be fully preloaded according to EN 1090-2.
Size	M24 to M36
Grade	8.8 or 10.9
Holes	According to EN 1993-1-8
Haunch	
Angle	Haunch angle measured between the bottom flange of the beam and the flange of the haunch can range from 30° to 45°.
Welds	According to in Figure 3.6
End-plate to top beam flange and haunch flange	Reinforced full penetration groove welds
Continuity plates to column flanges	Full penetration groove welds
Supplementary web plates to column flanges	Full penetration groove welds
Other welds	Fillet welds both sides with a throat thickness greater than 0.55 time of the thickness the connected plates.

Note. Prequalification tests were performed on beams ranging from IPE360 to IPE600. The lower limit is extended to IPE330 as it represents less than 10% variation of the beam height, and smaller beam sizes were shown to be characterised by larger ductility in requalifying tests.

3.3.4 Design procedure

3.3.4.1 Design concept

Haunched extended end-plate beam-to-column joints are intended to provide a full-strength and rigid connection, with strong or balanced column web panel. The design procedure is based on the component method implemented in EN 1993-1-8, with some adjustments as outlined below, and accounts for requirements in EN 1998-1.

The joint is considered to consist of a connection, column web panel and the connected member (beam). The connection is designed for the bending moment and shear force at the column face corresponding to the formation of the plastic hinges in the beam (near to the haunch), accounting for material overstrength and strain hardening.

Numerical simulations performed in the EQUALJOINTS project showed that for hogging bending moment the centre of compression is located at a distance Δ_C above the haunch flange. Based on the results available so far, it may be assumed that the centre of compression is shifted up by 50% of the haunch depth ($\Delta_C = 0.5\ h_h$, see Figure 3.7a). For sagging moment, the usual assumption of centre of compression located at the middle of the compression flange is adopted (Figure 3.7b). On the other hand, bolt rows located close to the compression centre develop negligible tension forces, due to flexibility of the end plate and limited ductility of the bolt rows at the tension flange.

Consequently, it was assumed that only the bolt rows which are above the mid-depth of the beam cross-section (without haunch) are active under hogging moment. Under sagging moment, only bolt rows located beyond mid-depth of the beam cross-section including haunch were assumed active.

Column web panel can be designed to be balanced with the beam, sharing with the latter plastic deformation demands, or to be stronger than the beam.

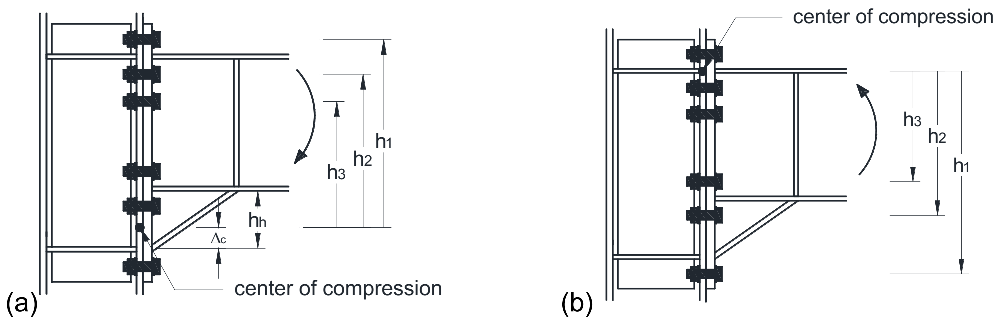


Figure 3.7: Centre of compression and active bolts rows for hogging (a) and sagging (b) moments

3.3.4.2 Global procedure

Step 1: Initial choice of the connection geometries and materials

- Bolt grade, bolt size and number of bolt rows
- Thickness and dimension of the end-plate
- Thickness and dimensions of haunch

- Thickness and dimensions of transverse stiffeners
- Thickness and dimensions of the supplementary web plates (if necessary)
- The weld specification

Step 2: Component characterisation

- Component resistances (joint under bending)
- Component stiffness (joint under bending)
- Component resistances (joint under shear)

Step 3: Assembly procedures

- Connection resistance in bending
- Connection resistance in shear
- Column web panel resistance
- Joint stiffness in bending

Step 4: Connection classification and check

3.3.4.3 Initial choice of the connection

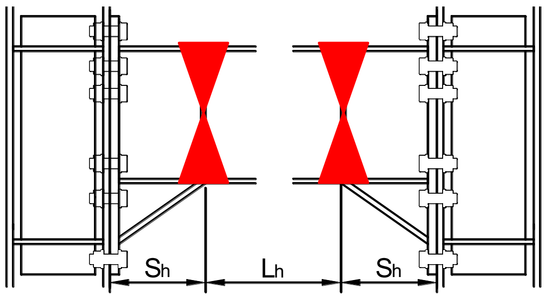
The recommendations given in the following table can be used for initiating the connection geometries and materials.

<table>
<tr><td rowspan="2">Connection elements</td><td colspan="3">Beam sizes</td></tr>
<tr><td>Small (≈IPE360)</td><td>Middle (≈ IPE450)</td><td>High (≈ IPE600)</td></tr>
<tr><td>Bolt grade</td><td colspan="3">10.9</td></tr>
<tr><td>Bolt size</td><td>M27</td><td>M30</td><td>M36</td></tr>
<tr><td>Number of bolt rows</td><td>6</td><td>6</td><td>6</td></tr>
<tr><td>End-plate</td><td colspan="3">Thickness: $t_{ep}=d_b$.
Dimensions: The width should be larger than the beam flange width (by at least 30 mm in order to accommodate the weld) and smaller than the column flange. The extended part should be enough to position one bolt row, respecting the rules given in EN 1993-1-8 (§3.5).</td></tr>
<tr><td>Haunch</td><td colspan="3">Haunch flange width equal to beam flange width.
Haunch flange thickness should be larger than γ_{ov} times the beam flange thickness.
Haunch web thickness should be equal or larger than the beam web thickness.
Haunch depth:
• $h_h = 0.4 \times h_b$ for haunch angle of $30° \leq \alpha < 40°$;
• $h_h = 0.5 \times h_b$ for haunch angle of $40° \leq \alpha \leq 45°$.</td></tr>
<tr><td>Supplementary web plates</td><td colspan="3">The thickness and the dimensions of the supplementary web plates should respect the rules given in EN 1993-1-8 (§ 6.2.6.1), otherwise plug welds should be used to guarantee the stability strength of the supplementary plates.</td></tr>
<tr><td>Transverse stiffeners</td><td colspan="3" rowspan="2">Table 3.1</td></tr>
<tr><td>Weld details</td></tr>
<tr><td colspan="4">Note: t_{ep} is the thickness of the end-plate and d_b is the nominal diameter of the bolt.</td></tr>
</table>

3.3.4.4 Assembly procedure and resistance checks

Classification type	Criterion	References
Connection resistance in bending	Full strength connection: $M_{con,Rd} \geq M_{con,Ed} = \alpha \cdot (M_{b,Rd} + V_{b,Ed} \cdot s_h)$ $\alpha = \gamma_{sh} \cdot \gamma_{ov}$	Equaljoints
Connection resistance in shear	$V_{con,Rd} \geq V_{b,Ed}$	Equaljoints
Resistance of column web panel in shear	Strong web panel: $V_{wp,Rd} \geq V_{wp,Ed}$ with $V_{wp,Ed} = \alpha \cdot (M_{b,Rd} + V_{b,Ed} \cdot s_h) / z - V_{c,Ed}$	Equaljoints
Rigidity classification	Classification / braced frames / Unbraced frames Semi-rigid joints / $0.5 \leq k_b < 8$ / $0.5 \leq k_b < 25$ Rigid joints / $k_b \geq 8$ / $k_b \geq 25$ $k_b = S_{j,ini} / (EI_b / L_b)$	EC3-1-8 5.2.2

Determine the design bending moment at the column face and the corresponding shear force.



The design bending moment at the column face, corresponding to yielded and fully strain hardened plastic hinge at the end of the haunch is:

$$M_{con,Ed} = M_{b,Rd} + V_{b,Ed} \cdot s_h$$

The design shear force in the connection $V_{con,Ed}$ is determined based on the assumption that fully yielded and strain hardened plastic hinges form at both ends of the beam:

$$V_{con,Ed} \cong V_{b,Ed} = V_{Ed,M} + V_{Ed,G}$$

where:

$M^{*}_{pl,Rd} = \gamma_{sh} \cdot \gamma_{ov} \cdot W_{pl,beam} \cdot f_{y,beam}$ is the expected plastic moment at the plastic hinge location;

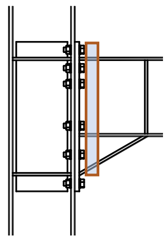
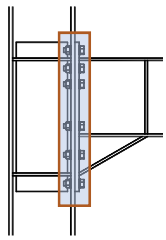
$W_{pl,beam}$ is the plastic modulus of the beam;

$f_{y,beam}$ is the specified minimum yield stress of the yielding element;

γ_{sh} is the strain-hardening factor to account for the peak connection strength;

γ_{ov}- is the material overstrength factor;

$V_{Ed,M}$ is the shear force due to the plastic hinges;

$V_{Ed,G}$ is the shear force due to gravity loads in the seismic design situation; s_h is the distance from the face of the column to the plastic hinge; L_h is the distance between plastic hinges. Note: Experimental tests show that plastic hinge forms at some distance away from the haunch end. However, as a simplification, it may be assumed that the plastic hinge is located at the haunch end. More exact position may be used if needed.	
<u>Check the beam end including the haunch</u> The beam end including the haunch is checked according to EN 1993-1-1 for the expected design bending moment at the column face: $$\frac{M_{con,Ed}}{M_{bh,Rd}} \leq 1.0$$ where: $M_{bh,Rd}$ is the plastic moment resistance of the double T section composed of beam top flange, haunch flange and beam-haunch web, neglecting the bottom beam flange, see subclause 6.2.6.7 of EN 1993-1-8 ; $M_{con,Ed}$ is the expected maximum moment at the column face. In order to account for possible material overstrength in beam with respect to the one in haunch, the haunch flange thickness is then increased by γ_{ov}.	
<u>Check the bending resistance of the end-plate connection.</u> Check the resistance of the connection in bending, under both hogging and sagging moments: $$\frac{M_{con,Ed}}{M_{con,Rd}} \leq 1.0$$ where $M_{con,Rd}$ is the bending resistance of the connection. The following components are used to obtain the moment resistance of the connections: • Column flange in bending; • End-plate in bending; • Beam web in tension; • Column web in tension; • Column web in compression. $M_{con,Rd}$ is determined according to EN 1993-1-8, with the following modifications: • under hogging moment only the bolt rows are above the mid-depth of the beam cross-section (without haunch) are assumed active. • under sagging moment, only bolt rows located beyond mid-depth of the beam cross-section including haunch are assumed active. • for hogging bending moment the center of compression is shifted up by 50% of the haunch depth ($\Delta_C = 0.5\ h_h$, see Figure 3.7a); • the following components are not taken into account: column web panel in shear, beam flange and web (and haunch) in compression.	

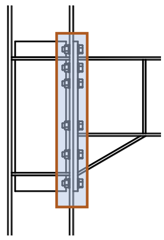
<u>Check shear resistance of the connection</u>

$$\frac{V_{b,Ed}}{V_{con,Rd}} \leq 1.0$$

where $V_{con,Rd}$ is shear resistance of the connection.

The following components are used to obtain the shear resistance of the connections:

- Beam web in shear;
- Bolts in bearing on column flange;
- Bolts in bearing on end-plate;
- Bolts in shear. Only the bolts not accounted for bending resistance of the connection should be accounted for.



<u>Check column web panel</u>

The design shear force in the column web panel is determined based on bending moments and shear forces acting on the web panel.

$$V_{wp,Ed} = \alpha \cdot \left(M_{b,Rd} + V_{b,Ed} \cdot s_h\right) / z - V_{c,Ed}$$

where

$V_{wp,Ed}$ is the design shear force in the column web panel;

$V_{c,Ed}$ is the shear force in the column;

z is the internal lever arm.

For a *strong column web panel*, the design shear force should be obtained accounting for the development of fully yielded and strain hardened plastic hinges in the beam:

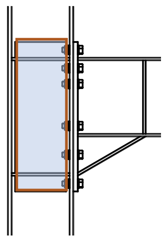
$$\alpha = \gamma_{sh} \cdot \gamma_{ov}$$

The resistance of the column web panel checked with the following relation:

$$\frac{V_{wp,Ed}}{V_{wp,Rd}} \leq 1.0$$

$V_{wp,Rd}$ is determined according to EN 1993-1-8. The following limitations apply:

- It is allowed to consider the full area of the supplementary web plates in computing the additional shear strength of column web panel.
- The additional shear resistance $V_{wp,add,Rd}$ due to column flanges and transverse stiffeners may be disregarded.



3.3.4.5 <u>Component characterization</u>

Component	Detailed rules	References
Column web panel in shear	Rules in EN 1993-1-8, 6.2.6.1 apply, with the following observations: • It is allowed to consider the full area of the supplementary web plates in computing the additional shear strength of column web panel. • The additional shear resistance $V_{wp,add,Rd}$ due to column flanges and transverse stiffeners may be disregarded.	EN 1993-1-8 6.2.6.1 6.3.2
Column flange in bending	Rules in EN 1993-1-8 apply.	EN 1993-1-8 6.2.6.4 6.3.2

Component	Detailed rules	References
End-plate in bending	Rules in EN 1993-1-8 apply.	EN 1993-1-8 6.2.6.5 6.3.2
Column web in compression	Rules in EN 1993-1-8 apply.	EN 1993-1-8 6.2.6.2 6.3.2
Beam web in tension	Rules in EN 1993-1-8 apply.	EN 1993-1-8 6.2.6.8 6.3.2
Column web in tension	Rules in EN 1993-1-8 apply.	EN 1993-1-8 6.2.6.3 6.3.2
Beam web in shear	Rules in EN 1993-1-1 apply.	EN 1993-1-1 6.2.6
Bolts in bearing on column flange	Rules in EN 1993-1-8 apply.	EN 1993-1-8 3.6.1
Bolts in bearing on end-plate	Rules in EN 1993-1-8 apply.	EN 1993-1-8 3.6.1
Bolts in shear	Rules in EN 1993-1-8 apply.	EN 1993-1-8 3.6.1

3.3.4.6 Stiffness classification

Haunched extended end-plate beam-to-column joints may be considered to be rigid, provided:

- column web panel resistance is obtained using equation (6.7) in EN 1993-1-8, neglecting the additional shear resistance $V_{wp,add,Rd}$ due to column flanges and transverse stiffeners;
- centre-line model is used for the global structural analysis;
- bolts are category E (fully preloaded) according to EN 1993-1-8.

Rules in EN 1993-1-8 may be used to quantify connection and column web panel stiffness. Advanced modelling of connection and column web panel may be used in global structural analysis if needed.

3.3.4.7 Ductility classification

Haunched extended end-plate beam-to-column joints designed according to the provisions above are deemed to be qualified for application in DCH and DCM structural systems (moment resisting frames, dual concentrically braced frames and dual eccentrically braced frames).

This is based on the fact that all tested connections satisfied the following requirements (ANSI/AISC 341-16):

- The connection was capable of accommodating a story drift angle of at least 0.04 rad

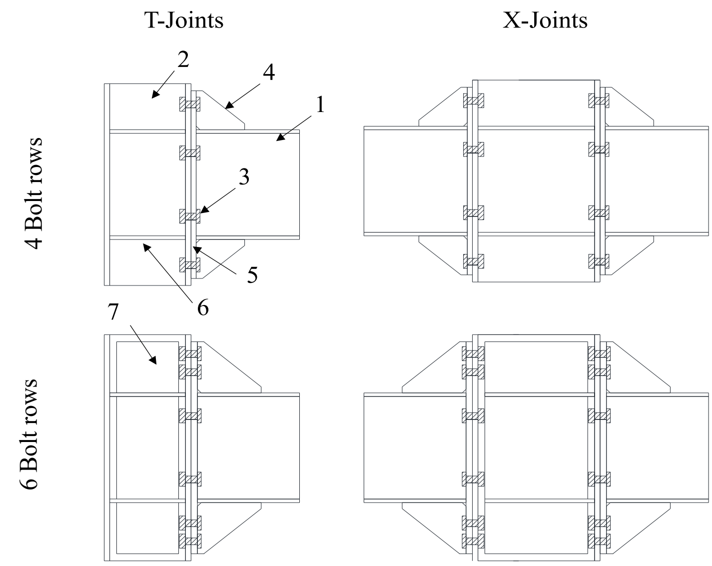
- The measured flexural resistance of the connection, determined at the column face, was equal to least 0.80Mp of the connected beam at a story drift angle of 0.04 rad.

The user is cautioned though that storey drifts corresponding to 20% drop of the maximum moment were less than 0.04 rad (but larger than 0.03 rad) for haunches.

3.4 Stiffened extended end-plate beam-to-column joints

3.4.1 Description of the joint configuration

The joint configuration is described in Figure. 3.8, it concerns the unstiffened extended end-plate joints. Depending on the beam depth and the design criteria, 4 or 6 bolt rows can be adopted. The use of the additional plates is an option to reinforce the column web it is necessary, while the use of the continuity plates (transverse column stiffeners) is recommended for all cases.
Weld types prescribed in accordance with the design criteria are listed in Table 3.2.



1: Beam 3: Bolts 5: End-plate 7: Supplementary
2: Column 4: Ribs 6: Continuity plates

Figure 3.8: Description of stiffened extended end-plate joints

Table 3.2: **Weld types in accordance with the design criteria**

Welded Elements	Joint strength		
	Full	Equal	Partial
Beam flange to End-plate (bf-ep)	FPW	FPW	FPW
Beam web to End-plate (bw-ep)	FPW	FPW	FW
Continuity plates to column (cp-c)	FW	FW	FPW
Rib to End-plate (r-ep)	FPW	FPW	FPW
Rib to Beam flange (r-bf)	FPW	FPW	FPW
Supp. web plates to Column (Swp-c)	FPW+PW	FPW+PW	FPW+PW

3.4.2 List of systems for which connection is prequalified

Extended stiffened end-plate bolted joints prequalified in this document can be used for the following structural systems:

- Moment Resisting Frames (MRFs);
- Dual Concentrically Braced Frames (i.e. MRF+ CBFs);
- Dual Eccentrically Braced Frames (i.e. MRF+ EBFs);

In addition, these joints should be used only in frames with perpendicular beam-column axis and regular span layout of the seismic resisting system, namely no sloped beam.

3.4.3 List of limit values for prequalified data

Table 3.3: **List of limit values for prequalified data**

Elements	**Application range**
Beam	
Depth	Maximum=600mm
Span-to-depth ratio	Maximum=23, Minimum=10
Flange thickness	Maximum=19mm
Material	From S235 to S355
Column	
Depth	Maximum=550mm
Flange thickness	Maximum=29mm
Material	From S235 to S355
Beam/column depth	0.65-2.15
End-plate	18-30mm
Thickness	Table 3.4
Material	From S235 to S355
Continuity plates	
Thickness	Equal or larger than the thickness of the connected beam flange
Material	From S235 to S355
Additional plates	
Thickness	Table 3.4
Material	From S235 to S355
Bolts	HV or HR
Size	Table 3.4
Grade	10.9
Number of bolt rows	Table 3.4
Washer	According to EN 14399-4
Holes	According to EN1993:1-8
Welds	
End-plate to beam flanges	Reinforced groove full penetration (Figure 3.9)
Continuity plates to column flanges	Groove full penetration (Figure 3.9)
Additional plates to column flanges	Groove full penetration (Figure 3.9)
Other welds	Fillet welds: throat thickness greater than 0.55 time of the thickness the connected plates.

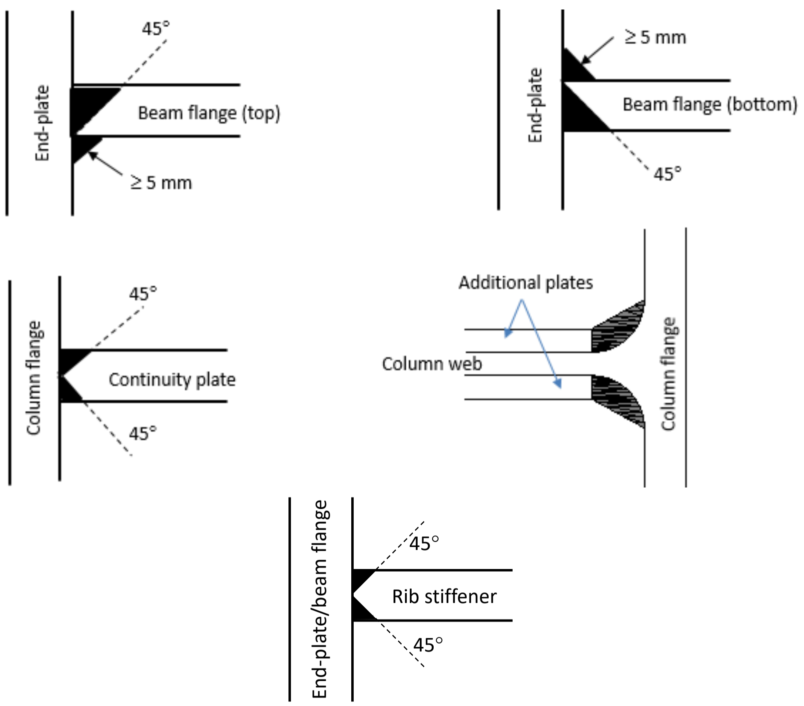


Figure 3.9: Details of the groove full penetration welds

3.4.4 Design procedure

Further to the choices made in terms of joint geometries and materials, the three main design steps of the component method are successively address:

- Component characterization
- Assembly procedure
- Joint classification and design check

3.4.4.1 <u>Global Procedure</u>

<u>Step 1</u>: Initial choice of the connection geometries and materials

- Bolt grade, bolt size and number of bolt rows
- Thickness and dimension of the end-plate
- Thickness and dimensions of the continuity plates
- Thickness and dimensions of the additional plates (if the case)
- The weld specification

<u>Step 2</u>: Component characterization

- Component resistances (joint under bending)
- Component rigidities (joint under bending)
- Component resistances (joint under shear)

<u>Step 3</u>: Assembly procedures

- Joint resistance in bending
- Joint rigidity in bending

- Connection resistance in shear
- Ductility degree of the connection

Step 4: Joint classification and design check

- Resistance in bending
- Rigidity in bending
- Resistance in shear
- Ductility
- Check

3.4.4.2 Initial choice of the connection

The recommendations given in Table 3.4 can be used for initiating the connection geometries and materials.

Table 3.4: **Initial choice of connection geometries and materials**

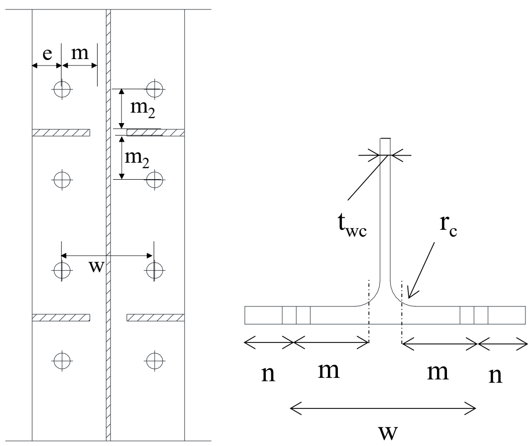
Connection elements	Beam sizes		
	Small (≈IPE360)	Middle (≈ IPE450)	High (≈ IPE600)
Bolt grade	10.9		
Bolt size	M27	M30	M36
Number of bolt rows	4/6	4/6	6
End-plate	*Thickness*: $t_{ep} = \left(2/3 \div 5/6\right) d_b$ for full joints it can be slightly larger than the column flanges; $t_{ep} = \left(2/3 \div 5/6\right) d_b$ for equal joints; but should be less than the thickness of the column flanges. *Dimensions*: The width should be equal to or smaller than the column flange one. The extended part should be enough to position one or two bolt rows, respecting the rules given in EC3-1-8 (§3.5).		
Additional plates	The thickness and the dimensions of the additional plates should be respected the rules given in EC3-1.8 (§ 6.2.6.1), otherwise plug welds should be used to guarantee the stability strength of the supplementary plates.		
Continuity plates	Table 3.3		
Weld details			
Note: t_{ep} is the thickness of the end-plate and d_b is the nominal diameter of the bolt.			

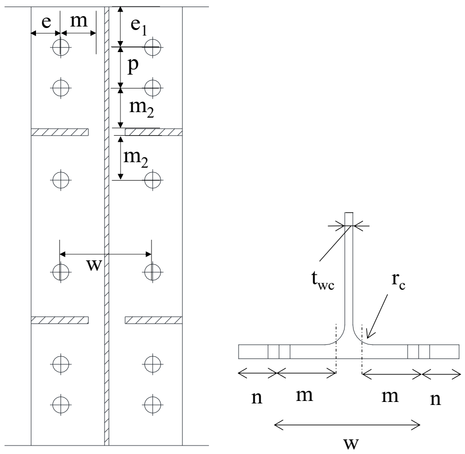
3.4.4.3 Assembly procedure and resistance checks

Classification type	Criterion	References
Resistance in bending	$M_{con,Rd} \approx M_{Ed}$: equal connection $M_{con,Rd} > M_{Ed}$: full strength connection $V_{wp,Rd,} > \min\left[F_{con,Rd}, F_{fbc,Rd}\right]$: strong web panel With: $F_{con,Rd} = \sum F_{Rd,ri}$ (I = 1 to 5 for joints with 6 bolt rows and i = 1 to 3 for joints with 4 bolt rows), is the transversal shear force in the connection due to the bolt rows in tension. $F_{fbc,Rd}$ is the resistance of the beam flanges and web in compression.	Equaljoints
Rigidity classification	Classification / braced frames / Unbraced frames Semi-rigid joints: $0.5 \leq k_b < 8$ / $0.5 \leq k_b < 25$ Rigid joints: $k_b \geq 8$ / $k_b \geq 25$ $k_b = S_{j,ini} / \left(EI_b / L_b\right)$	EC3-1-8 5.2.2
Resistance in shear	$V_{con,Rd} \approx V_{b,Rd}$: equal resistance in shear $V_{con,Rd} > V_{b,Rd}$: full resistance in shear	
Ductility classification	$\beta_{max} \leq 1.0$: ductility degree 1 $\beta_{max} > 1.0$ and $\eta_{max} \leq 0.95$: ductility degree 2 With: $\beta_{max} > \max\left[\beta_{r1}, \beta_{r2}\right]; \eta_{max} > \max\left[\eta_{r1}, \eta_{r2}\right]$	Equaljoints

3.4.4.4 Component characterization

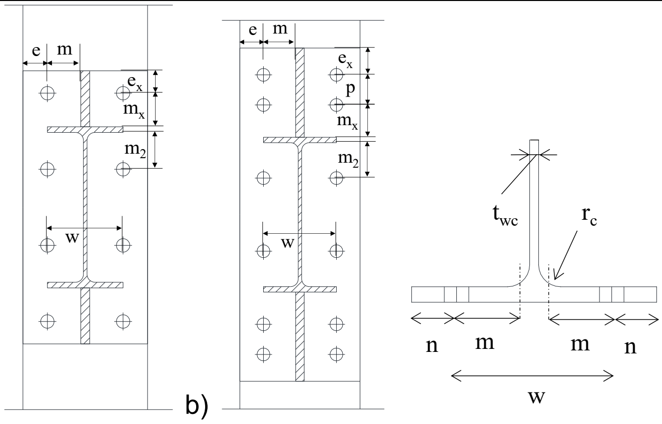
Component resistance (under bending)

Component	Detailed rules	References
Column flange in bending	Cases of 4 bolt rows 	EC3-1-8 6.2.6.4

Component	Detailed rules	References
Column flange in bending	Cases of 6 bolt rows  For each bolt row or for a group of bolt rows, the resistance is obtained using the following formula: $F_{cfb,Rd} = \min\left[F_{T,1,Rd};F_{T,2,Rd};F_{T,3,Rd}\right]$* or $F_{cfb,Rd} = \min\left[F_{T,1-2,Rd};F_{T,3,Rd}\right]$** with: • $F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m}$ • $F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$ • $F_{T,3,Rd} = \sum \frac{0.9f_{ub}A_s}{\gamma_{M2}}$ • $F_{T,1-2,Rd} = \frac{2M_{pl,1,Rd}}{m}$ In which: $M_{pl,1,Rd} = 0,25\Sigma\ell_{eff,1} t_{fc}^2 f_{y,fc} / \gamma_{M0}$ $M_{pl,2Rd} = 0.25\Sigma\ell_{eff,2} t_{fc}^2 f_{y,fc} / \gamma_{M0}$ $m = \left(w/2 - t_{wc}/2 - 0.8r_c\right)$ n = min [e,1.25m], with circular patterns $n = \infty$ can be used. $e_w = d_w / 4$ d_W is the diameter of the washer, or the width across points of the bolt head or nut. *If prying forces develop **If prying forces do not develop NB EC1993-1-8 allows considering prying forces in any case for bolted connection. In light of the results described within EQUALJOINTS project, this statement is not conservative and the prying force activation should be verified case by case.	EC3-1-8 6.2.6.4

<table>
<tr><th>Component</th><th>Detailed rules</th><th>References</th></tr>
<tr><td></td><td>
Effective lengths
❖ Connection 4 bolt rows
<u>Bolt row 1</u>:
$l_{eff,1} = \min\left[2\pi m, \alpha m\right]$
$l_{eff,2} = \alpha m$
<u>Bolt row 2</u>:
$l_{eff,1} = \min\left[2\pi m, \alpha m\right]$
$l_{eff,2} = \alpha m$
α is given by figure 6.11 in EC3-1-8, depending on:
$\lambda_1 = \frac{m}{m+e}$; $\lambda_2 = \frac{m_2}{m+e}$
N.B. Between the first and the second bolt row line, no group can be activated since the continuity plate are introduced.
❖ Connection 6 bolt rows
<u>Bolt row 1</u>:
$l_{eff,1} = \min\left[2\pi m; 4m + 1.25e\right]$
$l_{eff,2} = 4m + 1.25e$
First row of the group 1+2
$l_{eff,1} = \min\left[2p; p\right]$
$l_{eff,nc} = p$
<u>Bolt row 2</u>:
$l_{eff,1} = \min\left[2\pi m, \alpha m\right]$
$l_{eff,2} = \alpha m$
Second row of the group 1+2
$l_{eff,1} = \min\left[\pi m + p; 0.5p + \alpha m - \left(2m + 0.625e\right)\right]$
$l_{eff,1} = 0.5p + \alpha m - \left(2m + 0.625e\right)$
<u>Bolt row 3</u>:
$l_{eff,1} = \min\left[2\pi m, \alpha m\right]$
$l_{eff,2} = \alpha m$
α is given by figure 6.11 in EC3-1-8, depending on:
$\lambda_1 = \frac{m}{m+e}$
$\lambda_2 = \frac{m_2}{m+e}$
N.B. Between the first and the second bolt row line the group effect can influence the line resistance; conversely, no group effect with the third bolt row line can be activated since the continuity plate are introduced.
</td><td>Table 6.5
(EC3-1-8)</td></tr>
</table>

<table>
<tr><th>Component</th><th>Detail</th><th>References</th></tr>
<tr><td>End-plate in bending</td><td>



a) b)

Cases of 4 bolt rows (a) and 6 bolt rows (b)

For each bolt row or for a group of bolt rows, the resistance is obtained using the following formula:

$F_{cfb,Rd} = \min\left[F_{T,1,Rd};F_{T,2,Rd};F_{T,3,Rd}\right]$* or

$F_{cfb,Rd} = \min\left[F_{T,1-2,Rd};F_{T,3,Rd}\right]$**

with:

- $F_{T,1,Rd} = \dfrac{4M_{pl,1,Rd}}{m}$
- $F_{T,2,Rd} = \dfrac{2M_{pl,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$
- $F_{T,3,Rd} = \sum \dfrac{0.9 f_{ub} A_s}{\gamma_{M2}}$
- $F_{T,1,Rd} = \dfrac{2M_{pl,1,Rd}}{m}$

where:

$M_{pl,1,Rd} = 0.25\Sigma\ell_{eff,1} t_{fc}^{\ 2} f_{y,fc} / \gamma_{M0}$

$M_{pl,2Rd} = 0,25\Sigma\ell_{eff,2} t_{fc}^{\ 2} f_{y,fc} / \gamma_{M0}$

$m = \left(w/2 - t_{wc}/2 - 0.8 r_c\right)$

$n = \min\left[e, 1.25m\right]$, with circular patterns n=∞ can be used.

$e_w = d_w/4$

d_W is the diameter of the washer, or the width across points of the bolt head or nut.

*If the prying force will be developed

** If the prying force will not be developed

</td><td>EC3-1-8
6.2.6.5</td></tr>
</table>

Component	Detailed rules	References
	Effective lengths ❖ Connection 4 bolt rows <u>Bolt row 1</u>: $l_{eff,1} = \min\begin{cases} 2\pi m \\ \pi m + 2e_x \\ \alpha m - (2m + 0.625e) + e_x \\ 2m + 0.625e + e_x \\ 4m + 1.25e \end{cases}$ $l_{eff,2} = \min\begin{cases} \alpha m - (2m + 0.625e) + e_x \\ 2m + 0.625e + e_x \\ 4m + 1.25e \end{cases}$ <u>Bolt row 2</u>: $l_{eff,1} = \min[2\pi m, \alpha m]$ $l_{eff,2} = \alpha m$ α is given by figure 6.11 in EC1993-1-8, depending on: $\lambda_1 = \frac{m}{m+e}$ $\lambda_2 = \frac{m_2}{m+e}$ ❖ Connection 6 bolt rows <u>Bolt row 1</u>: $l_{eff,1} = \min\begin{cases} 2\pi m \\ \pi m + 2e_x \\ 4m + 1.25e \\ 2m + 0.625e + e_x \end{cases}$ $l_{eff,2} = \min\begin{cases} 4m + 1.25e \\ 2m + 0.625e + e_x \end{cases}$ *First row of the group 1+2* $l_{eff,1} = \min\begin{cases} \pi m + p \\ 2e_x + p \\ 2m + 0.625e + 0.5p \\ e_x + 0.5p \end{cases}$ $l_{eff,2} = \min\begin{cases} 2m + 0.625e + 0.5p \\ e_x + 0.5p \end{cases}$ <u>Bolt row 2</u>: $l_{eff,1} = \min[2\pi m, \alpha m]$ $l_{eff,2} = \alpha m$ *Second row of the group 1+2* $l_{eff,1} = \min[\pi m + p; 0.5p + \alpha m - (2m + 0.625e)]$ $l_{eff,2} = 0.5p + \alpha m - (2m + 0.625e)$	

Component	Detailed rules	References
	<u>*Bolt row 3:*</u> $l_{eff,1} = \min\left[2\pi m, \alpha m\right]$ $l_{eff,2} = \alpha m$ α is given by figure 6.11 in EC3-1-8.	

Component	Detailed rules	References
Beam flanges and web in compression	$F_{fbc,Rd} = M_{c,Rd} f_{y,b} / \left(h + \xi b - 0.5 t_{fb}\right)$ • h is the depth of the connected beam; • $M_{c,Rd}$ is the design moment resistance of the beam plus the rib cross-section, reduced if necessary to allow for shear, see EN 1993-1-1. • t_{fb} is the flange thickness of the connected beam. • ξ_b is the position of the compression center; • b is the rib height.	EC3-1-8 6.2.6.7
Column web and continuity plates in compression	The resistance of the column web and continuity plates may be computed by: $F_{wcc,Rd} = \frac{\omega k_{wc} b_{eff,c,cf} t_{wc} f_{y,wc}}{\gamma_{M0}} + \frac{A_{cp} f_{y,cp}}{\gamma_{M0}}$ where: $b_{eff,c,cf} = t_{fb} + \sqrt{2}\left(a_{w1} + a_{w2}\right) + 5\left(t_{fc} + r_c\right) + 2t_{ep}$ A_{cp} is the area of the continuity plates (both two sides); The reduction factor k_{wc} taking into account the axial stress in the column web, given in 6.2.6.2(2) of EC3-1-8. The reduction factor ω is given by Table 6.3 in EC3-1-8; NB: when the continuity plates are used, the reduction due to buckling of the column web under transverse compression can be neglected	EC3-1-8 6.2.6.2
Beam web in tension	$F_{wbt,Rd} = b_{eff,t,wb} t_{wb} f_{y,wb} / \gamma_{M0}$ The effective width $b_{eff,t,wb}$ of the beam web in tension should be taken as equal to the effective length of the equivalent T-stub representing the end-plate in bending for an individual bolt-row or a bolt-group.	EC3-1-8 6.2.6.8
Column web in tension	$F_{wct,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$ The effective width $b_{eff,t,wc}$ of the column flange in tension should be taken as equal to the effective length of the equivalent T-stub representing the column flange in bending for an individual bolt-row or a bolt-group. The reduction factor ω is given by Table 6.3 in EC3-1-8.	EC3-1-8 6.2.6.3
Bolts in tension	The resistance of a bolt row (two bolts) in tension is given by: $F_{bt,Rd} = 2\ \frac{0.9 f_{ub}\ A_s}{\gamma_{M2}}$ where: • f_{ub} is the ultimate tensile strength of the bolt; • A_s is the tensile stress area of the bolt.	EC3-1-8 3.6.1

3.4.4.5 Component rigidities (joint under bending)

Component	Detailed rules	References
Column web panel in shear	For the stiffened joint, k_1 contribution is equal to infinite, while if the joint is un-stiffened: $$k_1 = \frac{0.38 \cdot A_{VC}}{\beta z}$$ where: β is the transformation parameter defined in EN1993-1-8 pr. 5.3(7), and z is the level arm.	EC3-1-8 6.3.2
Column flange in bending	For simple bolt row in tension: $$k_4 = \frac{0.9 \cdot l_{eff} \cdot t_{fc}^2}{m^3}$$ The effective width b_{eff} is the smallest effective lengths of the bolt row (individual or as part of a group bolt rows).	EC3-1-8 6.3.2
End-plate in bending	For simple bolt row in tension: $$k_5 = \frac{0.9 \cdot l_{eff} \cdot t_p^2}{m^3}$$ The effective width b_{eff} is the smallest effective lengths of the bolt row (individual or as part of a group bolt rows).	EC3-1-8 6.3.2
Column web in tension	For stiffened welded connection, k_3 contribution is equal to infinite, while if the joint is un-stiffened: $$k_3 = \frac{0.7 \cdot b_{eff,t,wc} \cdot t_{wc}}{d_c}$$ The effective width b_{eff} is the smallest effective lengths of the bolt row (individual or as part of a group bolt rows) of the column flange in bending component.	EC3-1-8 6.3.2
Bolts in tension	For simple bolt row in tension: $$k_{10} = \frac{1.6 \cdot A_s}{L_b}$$	EC3-1-8 6.3.2
Rib on the compression side	$$k_{RIB} = \frac{A_{eq}}{L_{Strut}} \cdot \cos(\alpha)$$ where (as defined by Lee): $$A_e = \frac{\eta\left(ab - c^2\right)}{\sqrt{\left(a-c\right)^2 + \left(b-c\right)^2}}$$ $$L_e = (0.6)\sqrt{\left(a^2 + b^2\right)}$$ α is the rib strut inclination.	Equaljoints

3.4.4.6 Component resistance (joint under shear)

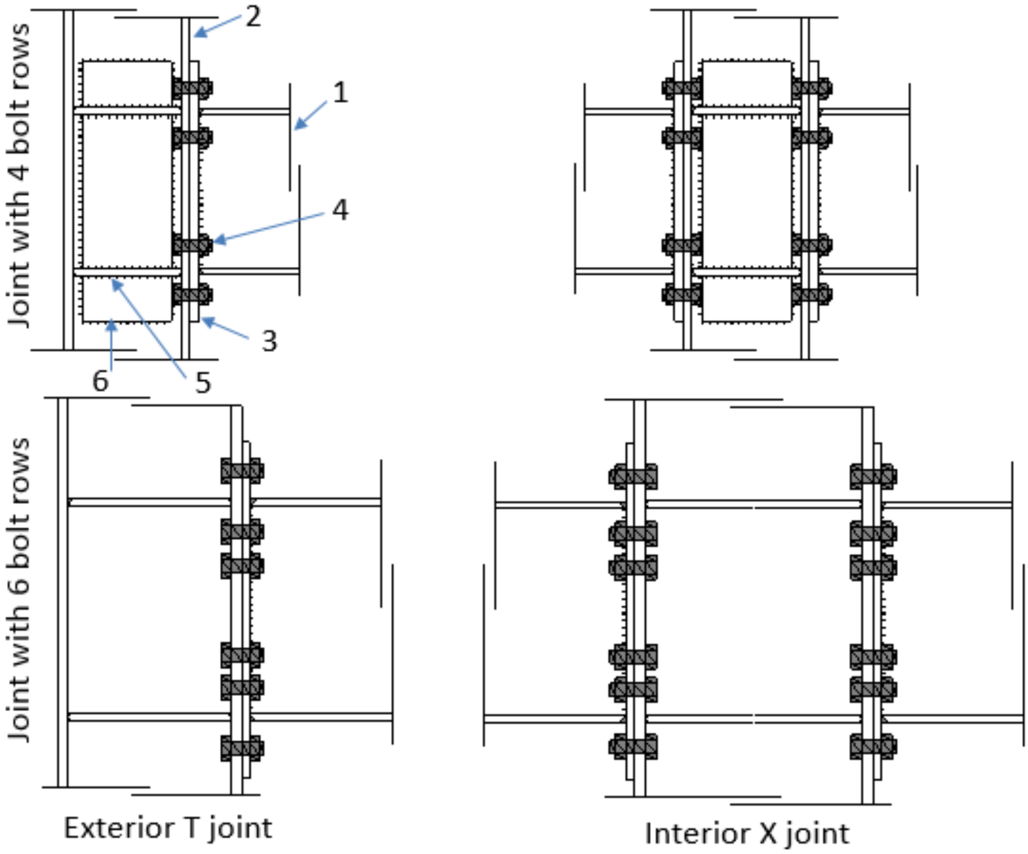
Component	Detailed rules	References
Beam web in shear	$V_{b,RD} = \chi_w A_{vb} f_{y,b} / \sqrt{3}\gamma_{M1}$ where: $A_{vb} = A_b - 2b_b t_{fb} + (t_{wb} + 2r_b) t_{fb}$ $\chi_w = 0.83/\overline{\lambda}_w$ if $\overline{\lambda}_w \geq 0.83$; $\chi_w = 1.0$ if $\overline{\lambda}_w < 0.83$ with $\overline{\lambda}_w = 0.3467 (h_{wb}/t_{wb}) \sqrt{f_{y,b}/E}$	EC3-1-5 5.3
Column flange in bearing	For simple bolt row (two bolts) in shear: $F_{b,Rd} = 2\dfrac{k_1 \alpha_b f_u d t_{fc}}{\gamma_{M2}}$ where: for edge bolts: $k_1 = \min\left[2.8\dfrac{e}{d_0} - 1.7,\quad 2.5\right]$ for inner bolts: $k_1 = \min\left[1.4\dfrac{p_2}{d_0} - 1.7,\quad 2.5\right]$ α_b depending on the sear load direction and bolt row position: Connection with 4 bolt rows *Shear load going down* Bolt rows 1, 3 and 4: $\alpha_b = 1.0$ Bolt rows 2: $\alpha_b = \min\left[1.0,\quad p/3d_0 - 0.25\right]$ *Shear load going up* Bolt rows 1, 2 and 4: $\alpha_b = 1.0$ Bolt rows 3: $\alpha_b = \min\left[1.0,\quad p/3d_0 - 0.25\right]$ Connection with 6 bolt rows *Shear load going down* Bolt rows 1, 3 and 5: $\alpha_b = \min\left[1.0,\quad p/3d_0 - 0.25\right]$ Bolt rows 2, 4 and 6: $\alpha_b = 1.0$ *Shear load going up* Bolt rows 1, 3 and 5: $\alpha_b = 1.0$ Bolt rows 2, 4 and 6: $\alpha_b = \min\left[1.0,\quad p/3d_0 - 0.25\right]$	EC3-1-8 3.6.1
End-plate in bearing	For simple bolt row (two bolts) in shear: $F_{b,Rd} = 2\dfrac{k_1 \alpha_b f_u d t_{fc}}{\gamma_{M2}}$ for edge bolts: $k_1 = \min\left[2.8\dfrac{e}{d_0} - 1.7,\quad 2.5\right]$	EC3-1-8 3.6.1

<table>
<tr><th>Component</th><th colspan="2">Detailed rules</th><th>References</th></tr>
<tr><td rowspan="6"></td><td colspan="2">for inner bolts: $k_1 = \min\left[1.4\frac{p_2}{d_0} - 1.7, \quad 2.5\right]$
α_b depending on the sear load direction and bolt row position:</td><td rowspan="6"></td></tr>
<tr><td colspan="2">Connection with 4 bolt rows</td></tr>
<tr><td>Shear load going down
Bolt rows 1:
$\alpha_b = \min\left[1.0, \quad e_x/3d_0\right]$
Bolt rows 2 and 4:
$\alpha_b = 1.0$
Bolt rows 3:
$\alpha_b = \min\left[1.0, \quad p/3d_0 - 0.25\right]$</td><td>Shear load going up
Bolt rows 1 and 3:
$\alpha_b = 1.0$
Bolt rows 2:
$\alpha_b = \min\left[1.0, \quad p/3d_0 - 0.25\right]$
Bolt rows 4:
$\alpha_b = \min\left[1.0, \quad e_x/3d_0\right]$</td></tr>
<tr><td colspan="2">Connection with 6 bolt rows</td></tr>
<tr><td>Shear load going down
Bolt rows 1:
$\alpha_b = \min\left[1.0, \quad e_x/3d_0\right]$
Bolt rows 2, 4 and 6:
$\alpha_b = \min\left[1.0, \quad p/3d_0 - 0.25\right]$
Bolt rows 3 and 5:
$\alpha_b = 1.0$</td><td>Shear load going up
Bolt rows 1, 3 and 5:
$\alpha_b = \min\left[1.0, \quad p/3d_0 - 0.25\right]$
Bolt rows 2 and 4:
$\alpha_b = 1.0$
Bolt rows 6:
$\alpha_b = \min\left[1.0, \quad e_x/3d_0\right]$</td></tr>
<tr><td colspan="2"></td></tr>
<tr><td>Bolts in shear</td><td colspan="2">For simple bolt row (two bolts) in shear:
$F_{b,Rd} = 2\frac{\alpha_v f_{ub} A_s}{\gamma_{M2}}$
α_v =0.5 for 10.9 bolts.</td><td>EC3-1-8
3.6.1</td></tr>
</table>

3.5 Unstiffened extended end-plate beam-to-column joints

3.5.1 Description of the joint configuration

The joint configuration is described in Figure 3.10, it corresponds to unstiffened extended end-plate joints. Depending on the beam depth, 4 or 6 bolt rows can be adopted. The use of the additional plates is an option to reinforce the column web if it is necessary, while the use of the continuity plates (transverse column stiffeners) is recommended for all cases.



1: beam
2: column
3: end-plate
4: bolts
5: continuity plates
6: additional plates

Figure 3.10 Description of unstiffened extended end-plate joints

3.5.2 List of systems for which connection is prequalified

Extended unstiffened end-plate bolted joints prequalified in this document can be used for the following structural systems:

- Dual Concentrically Braced Frames (i.e. MRF+ CBFs);
- Dual Eccentrically Braced Frames (i.e. MRF+ EBFs);

In addition, these joints should be used only in frames with perpendicular beam-column axis and regular span layout of the seismic resisting system, namely no sloped beam.

3.5.3 List of limit values for prequalified data

Table 3.5 summarises the limit values for the prequalified data. This table will be completed until the end of the project.

Table 3.5: **Limit values for prequalified data**

Elements	Parameters	Application range
Beam	Depth	Maximum = 600mm
	Span-to-depth ration	Maximum = 23, Minimum = 10
	Flange thickness	Maximum = 19mm
	Material	From S235 to S355
Column	Depth	Maximum = 550mm
	Flange thickness	Maximum = 31mm
	Material	From S235 to S355
Beam/column depth		
End-plate	Thickness	18-25mm
	Material	From S235 to S355
Continuity plates	Thickness	Equal or larger than the thickness of the connected beam flange
	Material	From S235 to S355
Additional plates	Thickness	Table 3.6
	Material	From S235 to S355
Bolts		HV or HR
	Size	Table 3.6
	Grade	10.9
	Number of bolt rows	Table 3.6
	Washer	
	Holes	
Welds	End-plate to beam flanges	Reinforced groove full penetration (Figure 3.9)
	Continuity plates to column flanges	Groove full penetration (Figure 3.9)
	Additional plates to column flanges	Groove full penetration ((Figure 3.9)
	Other welds	Fillet welds: throat thickness is greater than 0.55 times the thickness of the connected plates.

3.5.4 Design procedure

Further to the choices made in terms of joint geometries and materials, the three main design steps of the component method are successively address:

- Component characterization
- Assembly procedure
- Joint classification and design check

3.5.4.1 Global procedure

Step 1: Initial choice of the joint geometries and materials

- Bolt grade, bolt size and number of bolt rows
- Thickness and dimension of the end-plate

- Thickness and dimensions of the continuity plates
- Thickness and dimensions of the additional plates (if the case)
- The weld specification

Step 2: Component characterization

- Component resistances (joint under bending)
- Component rigidities (joint under bending)
- Component resistances (joint under shear)

Step 3: Assembly procedures

- Connection resistance in bending
- Joint rigidity in bending
- Connection resistance in shear
- Ductility degree of the connection

Step 4: Joint classification and design check

- Resistance in bending
- Rigidity in bending
- Resistance in shear
- Ductility
- Check

3.5.4.2 Initial choice of the joint geometries and materials

The recommendations given in Table 3.6 can be used as a good guess for the definition of the connection geometries and materials.

Table 3.6: **Initial choice of joint geometries and materials**

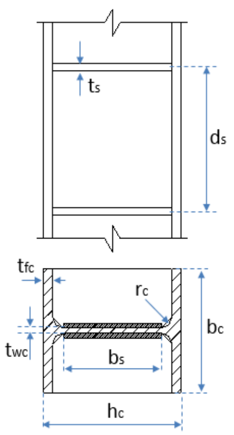
Connection elements	Beam sizes		
	Small (≈IPE360)	Middle (≈ IPE450)	High (≈ IPE600)
Bolt grade	10.9		
Bolt size	M27	M30	M36
Number of bolt rows	4	4	6
End-plate	*Thickness*: t_{ep}=(1/2÷2/3)d_b for partial joints; t_{ep}=(2/3÷5/6)d_b for equal joints; but should be less than the thickness of the column flanges. *Dimensions*: The width should be equal to the column flange one. The extended part should be enough to position one bolt row, respecting the rules given in EC3-1-8 (§3.5).		
Additional plates	With HEB columns and IPE beams, the additional plates are only to be considered when the strong web panel is required. The thickness and the dimensions of the additional plates should be respected, so by following the rules given in EC3-1.8 (§ 6.2.6.1).		
Continuity plates	Table 3.5		
Weld details			
Note: t_{ep} is the thickness of the end-plate and d_b is the nominal diameter of the bolt.			

3.5.4.3 Assembly procedure and design checks

Classification type	Criterion	References
Resistance in bending	$M_{con,Rd} < M_{con,Ed}$: partial connection $M_{con,Rd} \approx M_{con,Ed}$: equal connection $M_{con,Rd} > M_{con,Ed}$: full strength connection $V_{wp,Rd} <$ min $[F_{con,Rd}, F_{fbc,Rd}]$: weak web panel $V_{wp,Rd} \approx$ min $[F_{con,Rd}, F_{fbc,Rd}]$: balance web panel $V_{wp,Rd} >$ min $[F_{con,Rd}, F_{fbc,Rd}]$: strong web panel with: $F_{con,Rd} = \Sigma\, F_{Rd,ri}$ (I = 1 to 5 for joints with 6 bolt rows and i = 1 to 3 for joints with 4 bolt rows), is the transversal shear force in the connection due to the bolt rows in tension. $F_{fbc,Rd}$ is the resistance of the beam flanges and web in compression.	Equaljoints
Rigidity classification	Classification / Braced frames / Unbraced frames Semi-rigid jonts / $0.5 \le k_b < 8$ / $0.5 \le k_b < 25$ Rigid joints / $k_b \ge 8$ / $k_b \ge 25$ $k_b = S_j / \left(E I_b / L_b\right)$	EC3-1-8 5.2.2
Resistance in shear	$V_{con,Rd} < V_{b,Rd}$: partial resistance in shear $V_{con,Rd} \approx V_{b,Rd}$: equal resistance in shear $V_{con,Rd} > V_{b,Rd}$: full resistance in shear	
Ductility classification	$\beta_{max} \le 1.0$: ductility degree 1 $\beta_{max} > 1.0$ and $\eta_{max} \le 0.95$: ductility degree 2 With: $\beta_{max} >$ max $[\beta_{r1}, \beta_{r2}]$; $\eta_{max} >$ max $[\eta_{r1}, \eta_{r2}]$	Equaljoints

3.5.4.4 Component characterization

Component resistances (joint under bending)

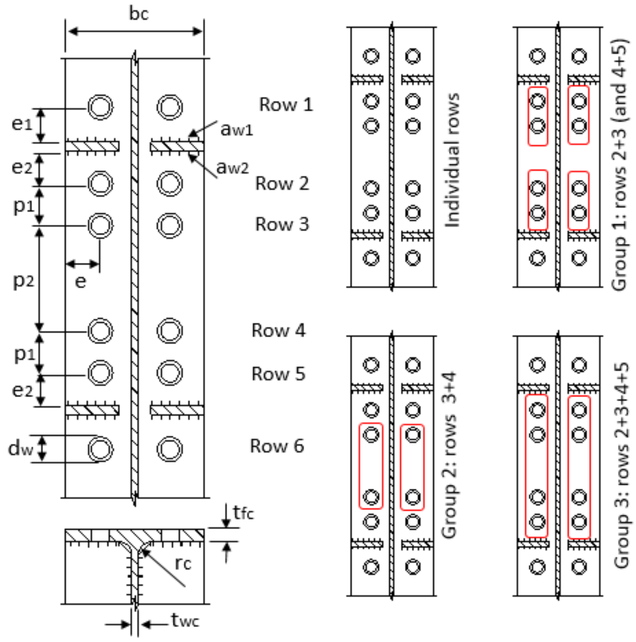
Component	Detailed rules	References
Column web panel in shear	$V_{wp,Rd} = \frac{0.9 A_{vc} f_{y,wc}}{\sqrt{3}\gamma_{M0}} + \frac{4\left(0.25 t_{fc}^2 f_{y,fc}\right)\left(b_c - t_{wc} - 2r_c\right)}{d_s}$  • Column web panel in shear with transverse web stiffeners and no additional plate: $A_{vc} = A_c - 2b_c t_{fc} + \left(t_{wc} + 2r_c\right) t_{fc}$ • Column web panel in shear with transverse web stiffeners and additional plate: $A_{vc} = A_c - 2b_c t_{fc} + \left(t_{wc} + 2r_c\right) t_{fc} + t_{wc} b_s$	EC3-1-8 6.2.6.1

<table>
<tr><th>Component</th><th>Detailed rules</th><th>References</th></tr>
<tr><td>Beam cross-section in bending</td><td>

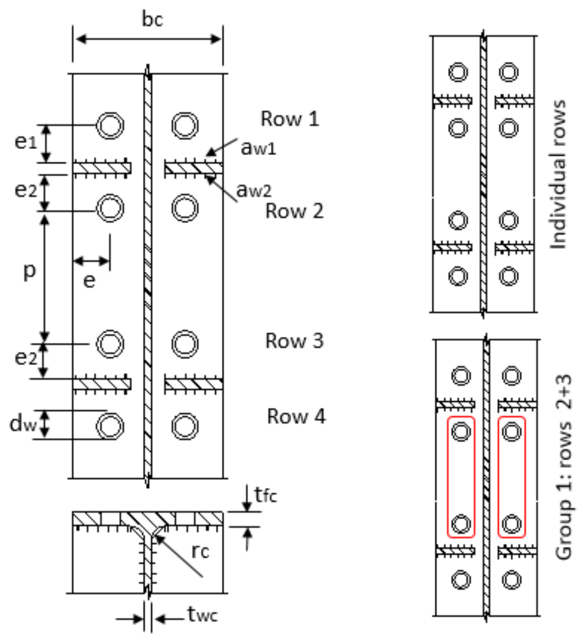
$M_{b,Rd} = W_{b,p} f_{y,b}$

- $W_{b,p}$ is the plastic modulus in bending of the beam section.
- $f_{y,b}$ is the yield strength of the beam material.

</td><td></td></tr>
<tr><td>Column flange in bending</td><td>



Cases of 6 bolt rows



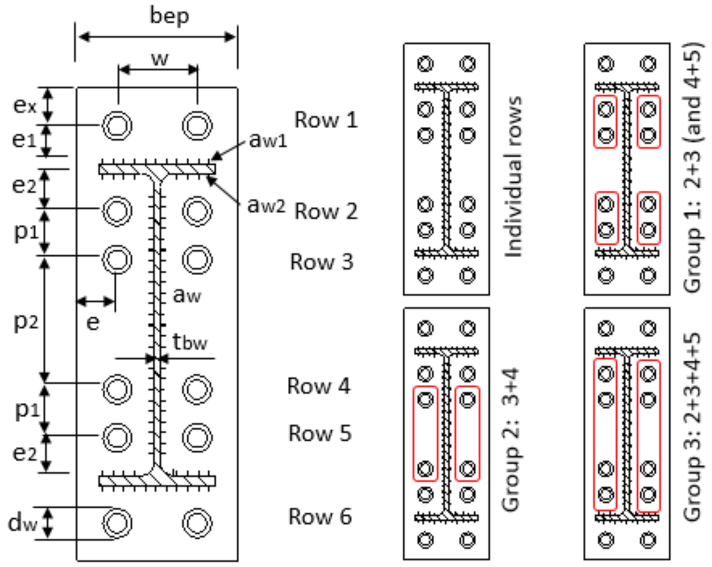
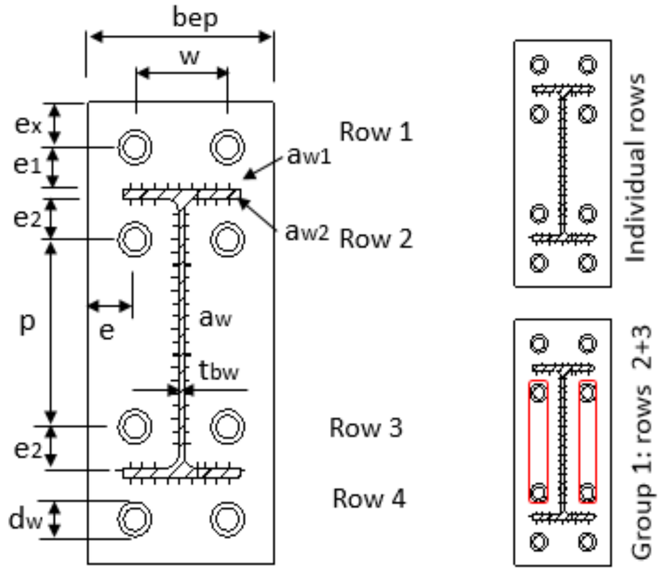
Cases of 4 bolt rows

For each bolt row or for a group of bolt rows, the resistance is obtained using the following formula:

</td><td>EC3-1-8
6.2.6.4</td></tr>
</table>

Component	Detailed rules	References
	$F_{cfb,Rd}=\min\left[F_{T,1,Rd};F_{T,2,Rd}\right]$ with • $F_{T,1,Rd}=\dfrac{\left(8n-2e_w\right)M_{pl,1,Rd}}{2mn-e_w\left(m+n\right)}$ • $F_{T,2,Rd}=\dfrac{2M_{pl,2,Rd}+n\Sigma F_{t,Rd}}{m+n}$ In which: $M_{pl,1,Rd}=0.25\Sigma\ell_{eff,1}t_{fc}{}^2f_{y,fc}/\gamma_{M0}$ $M_{pl,2Rd}=0.25\Sigma\ell_{eff,2}t_{fc}{}^2f_{y,fc}/\gamma_{M0}$ $m=0.5\left(b_c-2e-t_{wc}-1.6r_c\right)$ $n=\min\left[e,1.25m\right]$, with circular patterns $n=\infty$ can be used. $e_w=0.25d_w$ (with d_w is the diameter of the washer) ***Effective lengths*** ❖ Connection with 6 bolt rows <u>*Bolt row 1*</u>: $l_{eff,1}=\min\left[2\pi m,\alpha m\right]$ $l_{eff,2}=\alpha m$ <u>*Bolt row 2 (or row 5)*</u>: *Individual row*: $l_{eff,1}=\min\left[2\pi m,\alpha m\right]$ $l_{eff,2}=\alpha m$ *First row of the group 1 or group 3* $l_{eff,1}=\min\left[\pi m+p_1,\ 0.5p_1+\alpha m-\left(2m+0.625e\right)\right]$ $l_{eff,2}=0.5p_1+\alpha m-\left(2m+0.625e\right)$ <u>*Bolt row 3 (or row 4)*</u>: *Individual*: $l_{eff,1}=\min\left[2\pi m,4m+1.25e\right]$ $l_{eff,2}=4m+1.25e$ *Last row of the group 1:* $l_{eff,1}=\min\left[\pi m+p_1,\ 2m+0.625e+0.5p_1\right]$ $l_{eff,2}=2m+0.625e+0.5p_1$ *One row of the group 2:* $l_{eff,1}=\min\left[\pi m+p_2,0.5p_2+0.5\alpha m\right]$ $l_{eff,2}=0.5p_2+0.5\alpha m$ *Intermediate row of a group 3:* $l_{eff,1}=p_1+p_2$	

Component	Detailed rules	References
	$l_{eff,2} = 0.5(p_1 + p_2)$ α is given by figure 6.11 in EC3-1-8, depending on: $\lambda_1 = \frac{m}{m+e}$; $\lambda_2 = \frac{m_2}{m+e}$ where: $m_2 = e_1 - 0.8a_{w1}\sqrt{2}$ for bolt row 1 $m_2 = e_2 - 0.8a_{w2}\sqrt{2}$ for bolt row 2 or 5 ❖ Connection with 4 bolt rows <u>*Bolt row 1*</u>: $l_{eff,1} = \min\left[2\pi m, \alpha m\right]$ $l_{eff,2} = \alpha m$ <u>*Bolt row 2*</u>: *Individual*: $l_{eff,1} = \min\left[2\pi m, \alpha m\right]$ $l_{eff,2} = \alpha m$ *One row of the group 2+3* $l_{eff,1} = \min\left[\pi m + p, 0.5p + 0.5\alpha m\right]$ $l_{eff,2} = 0.5p + 0.5\alpha m$ <u>*Bolt row 3: the similar with the bolt row 2*</u> α is given by the figure 6.11 in EN-1993-1-8, depending on: $\lambda_1 = \frac{m}{m+e}$; $\lambda_2 = \frac{m_2}{m+e}$ where: $m_2 = e_1 - 0.8a_{w1}\sqrt{2}$ for bolt row 1 $m_2 = e_2 - 0.8a_{w2}\sqrt{2}$ for bolt row 2 or 3	

Component	Detailed rules	References
End-plate in bending	 Cases of 6 bolt rows  Cases of 4 bolt rows For each bolt row or for a group of bolt rows, the resistance is obtained using the following formula: $F_{pb,Rd} = \min\left[F_{T,1,Rd}; F_{T,2,Rd}\right]$ with • $F_{T,1,Rd} = \dfrac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)}$ • $F_{T,2,Rd} = \dfrac{2M_{pl,2,Rd} + n\Sigma F_{t,Rd}}{m+n}$ In which: $M_{pl,1,Rd} = 0.25\Sigma\ell_{eff,1} t_{ep}^{2} f_{y,ep} / \gamma_{M0}$ $M_{pl,2Rd} = 0.25\Sigma\ell_{eff,2} t_{ep}^{2} f_{y,ep} / \gamma_{M0}$	EC3-1-8 6.2.6.5

<table>
<tr><th>Component</th><th>Detailed rules</th><th>References</th></tr>
<tr><td></td><td>
$\begin{cases} m = 0.5\left(b_{ep} - 2e - t_{bw} - 1.6a_w\sqrt{2}\right) \\ n = \min\left[e, 1.25m\right] \end{cases}$ for bolt rows inside the beam flanges
$\begin{cases} m = e_1 - 0.8a_{w1}\sqrt{2} \\ n = \min\left[e_x, 1.25m\right] \end{cases}$ for bolt rows outside the beam flanges
(with circular patterns, n=∞ can be used).
$e_w = 0.25d_w$
Effective lengths
❖ Connection with 6 bolt rows
<u>Bolt row 1</u>:
$l_{eff,1} = \min\begin{cases} 2\pi m,\ \pi m + w,\ \pi m + 2e \\ 4m + 1.25e_x,\ e + 2m + 0.625e_x,\ 0.5b_{ep},\ 0.5w + 2m + 0.625e_x \end{cases}$
$l_{eff,2} = \min\left[4m + 1.25e_x,\ e + 2m + 0.625e_x,\ 0.5b_{ep},\ 0.5w + 2m + 0.625e_x\right]$

<u>Bolt row 2 (or row 5)</u>:
Individual row:
$l_{eff,1} = \min\left[2\pi m, \alpha m\right]$
$l_{eff,2} = \alpha m$
First row of the group 1(rows 2+3 or 4+5)
$l_{eff,1} = \min\left[\pi m + p_1, 0.5p_1 + \alpha m - \left(2m + 0.625e\right)\right]$
$l_{eff,2} = 0.5p_1 + \alpha m - \left(2m + 0.625e\right)$
<u>Bolt row 3 (or row 4)</u>:
Individual row:
$l_{eff,1} = \min\left[2\pi m, 4m + 1.25e\right]$
$l_{eff,2} = 4m + 1.25e$
Last row of the group 1 (rows 2+3 or 4+5):
$l_{eff,1} = \min\left[\pi m + p_1,\ 2m + 0.625e + 0.5p_1\right]$
$l_{eff,2} = 2m + 0.625e + 0.5p_1$
First row (or last row) of the group 2 (rows 3+4):
$l_{eff,1} = \min\left[\pi m + p_2,\ 2m + 0.625e + 0.5p_2\right]$
$l_{eff,2} = 2m + 0.625e + 0.5p_2$
Intermediate row of a group 3 (rows 2+3+4+5):
$l_{eff,1} = p_1 + p_2$
$l_{eff,2} = 0.5\left(p_1 + p_2\right)$
α is given by the figure 6.11 in EN-1993-1-8, depending on:
</td><td></td></tr>
</table>

Component	Detailed rules	References
	$\lambda_1 = \frac{m}{m+e}$; $\lambda_2 = \frac{m_2}{m+e}$ where: $m_2 = e_1 - 0.8a_{w1}\sqrt{2}$ for bolt row 1 $m_2 = e_2 - 0.8a_{w2}\sqrt{2}$ for bolt row 2 or 5 ❖ Connection with 4 bolt rows *Bolt row 1*: $l_{eff,1} = \min\begin{cases} 2\pi m,\ \pi m + w,\ \pi m + 2e \\ 4m + 1.25e_x,\ e + 2m + 0.625e_x,\ 0.5b_{ep},\ 0.5w + 2m + 0.625e_x \end{cases}$ $l_{eff,2} = \min\left[4m + 1.25e_x,\ e + 2m + 0.625e_x,\ 0.5b_{ep},\ 0.5w + 2m + 0.625e_x\right]$ *Bolt row 2:* *Individual row*: $l_{eff,1} = \min\left[2\pi m, \alpha m\right]$ $l_{eff,2} = \alpha m$ *One row of the group 2+3* $l_{eff,1} = \min\left[\pi m + p, 0.5p + 0.5\alpha m\right]$ $l_{eff,2} = 0.5p + 0.5\alpha m$ *Bolt row 3: similar to the bolt row 2:* α is given by the figure 6.11 in EC3-1-8, depending on: $\lambda_1 = \frac{m}{m+e}$; $\lambda_2 = \frac{m_2}{m+e}$ $m_2 = e_1 - 0.8a_{w1}\sqrt{2}$ for bolt row 1 $m_2 = e_2 - 0.8a_{w2}\sqrt{2}$ for bolt row 2 or 3	
Beam flanges and web in compression	$F_{fbc,Rd} = M_{c,Rd} / (h - t_{fb})$ where: • h is the depth of the connected beam; • $M_{c,Rd}$ is the design moment resistance of the beam cross-section, reduced if necessary to allow for shear, see EN 1993-1-1. • t_{fb} is the flange thickness of the connected beam.	EC3-1-8 6.2.6.7
Column web and continuity plates in compression	The resistance of the column web and continuity plates may be computed by: $F_{wcc,Rd} = \frac{\omega k_{wc} b_{eff,cf} t_{wc} f_{y,wc}}{\gamma_{M0}} + \frac{A_{cp} f_{y,cp}}{\gamma_{M0}}$ where:	EC3-1-8 6.2.6.2

Component	Detailed rules	References
	$b_{eff,c,cf} = t_{fb} + \sqrt{2}\left(a_{w1} + a_{w2}\right) + 5\left(t_{fc} + r_c\right) + 2t_{ep}$ A_{cp} is the area of the continuity plates (both two sides); The reduction factor k_{wc} taking into account the axial stress in the column web, given in 6.2.6.2(2) of EC3-1-8; The reduction factor ω is given by Table 6.3 in EC3-1-8; *Note*: the reduction due to buckling of the column web and the continuity plates under transverse compression are neglected. The geometries (the slender) of the continuity plates to satisfy this condition will be shown in Table 4.3.1.	
Beam web in tension	$F_{wbt,Rd} = b_{eff,wb}\, t_{wb}\, f_{y,wb} / \gamma_{M0}$ The effective width $b_{eff,t,wb}$ of the beam web in tension should be taken as equal to the effective length of the equivalent T-stub representing the end-plate in bending for an individual bolt-row or a bolt-group.	EC3-1-8 6.2.6.8
Column web in tension	$F_{wct,Rd} = \dfrac{\omega b_{eff,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$ The effective width $b_{eff,t,wc}$ of the column flange in tension should be taken as equal to the effective length of the equivalent T-stub representing the column flange in bending for an individual bolt-row or a bolt-group. The reduction factor ω is given by Table 6.3 in EC3-1-8.	EC3-1-8 6.2.6.3
Bolts in tension	The resistance of a bolt row (two bolts) in tension is given by: $F_{bt,Rd} = 2\dfrac{0.9\, f_{ub}\, A_s}{\gamma_{M2}}$ where: • f_{ub} is the ultimate tensile strength of the bolt; • A_s is the tensile stress area of the bolt.	EC3-1-8 3.6.1

3.5.4.5 Component rigidities (joint under bending)

Component	Detailed rules	References
Column web panel in shear	$k_1 = \dfrac{0.38 A_{vc}}{\beta z}$ The transformation parameter β is given in Table 5.4 of EC3-1-8. The lever arm, z, of the connection is given in EC-1-8, 6.3.3.1.	EC3-1-8 6.3.2
Column flange in bending	For simple bolt row in tension: $k_4 = \dfrac{0.9 b_{eff,cf} t_{fc}^3}{m^3}$ The effective width b_{eff} is the smallest effective lengths of the bolt row (individual or as part of a group bolt rows).	EC3-1-8 6.3.2
End-plate in bending	For simple bolt row in tension: $k_5 = \dfrac{0.9 b_{eff,ep} t_{ep}^3}{m^3}$ The effective width b_{eff} is the smallest effective lengths of the bolt row (individual or as part of a group bolt rows).	EC3-1-8 6.3.2

Column web in tension	For simple bolt row in tension: $k_3 = \dfrac{0.7 b_{eff,wc} t_{wc}}{d_c}$ The effective width b_{eff} is the smallest effective lengths of the bolt row (individual or as part of a group bolt rows) of the column flange in bending component.	EC3-1-8 6.3.2
Bolts in tension	For simple bolt row in tension: $k_{10} = 1.6 A_s / L_b$	EC3-1-8 6.3.2

3.5.4.6 Component resistance (joint under shear)

Component	Detailed rules	References
Beam web in shear	$V_{b,RD} = \chi_w A_{vb} f_{y,b} / \sqrt{3}\gamma_{M1}$ where: $A_{vb} = A_b - 2b_b t_{fb} + (t_{wb} + 2r_b) t_{fb}$ $\chi_w = 0.83/\bar{\lambda}_w$ if $\bar{\lambda}_w \geq 0.83$; $\chi_w = 1.0$ if $\bar{\lambda}_w < 0.83$ with $\bar{\lambda}_w = 0.3467 (h_{wb}/t_{wb}) \sqrt{f_{y,b}/E}$	EC3-1-5 5.3
Column flange in bearing	For simple bolt row (two bolts) in shear: $F_{b,Rd} = 2\dfrac{k_1 \alpha_b f_u d t_{fc}}{\gamma_{M2}}$ where: $k_1 = \min\left[2.8\dfrac{e}{d_0} - 1.7,\ 2.5\right]$ α_b depending on the sear load direction and bolt row position *Shear load going down* Bolt rows 1, 5 and 6 (or (*) rows 1, 3 and 4): $\alpha_b = 1.0$ Bolt rows 2 and 4 (or(*): row 2): $\alpha_b = \min[1.0,\ p_1/3d_0 - 0.25]$ Bolt row 3: $\alpha_b = \min[1.0,\ p_2/3d_0 - 0.25]$ *Shear load going up* Bolt rows 1, 2 and 6 (or(*) rows 1, 2 and 4) $\alpha_b = 1.0$ Bolt rows 3 and 5 (or(*) row 3) $\alpha_b = \min[1.0,\ p_1/3d_0 - 0.25]$ Bolt row 4: $\alpha_b = \min[1.0,\ p_2 / 3d_0 - 0.25]$ (*): used for joint with 4 bolt rows (p_1 should be replaced by p)	EC3-1-8 3.6.1
End-plate in bearing	For simple bolt row (two bolts) in shear: $F_{b,Rd} = 2\dfrac{k_1 \alpha_b f_u d t_{fc}}{\gamma_{M2}}$	EC3-1-8 3.6.1

<table>
<tr><th>Component</th><th colspan="2">Detailed rules</th><th>References</th></tr>
<tr><td rowspan="2">End-plate in bearing</td><td colspan="2">$k_1 = \min\left[2.8\frac{e}{d_0} - 1.7,\ 2.5\right]$</td><td rowspan="2">EC3-1-8 3.6.1</td></tr>
<tr><td>Shear load going down:
Bolt rows 2 and 6 (or(*) rows 2 and 4):
$\alpha_b = 1.0$
Bolt row 1 (or(*) row 1):
$\alpha_b = \min\left[1.0,\ e_x/3d_0\right]$
Bolt rows 3 and 5 (or(*) row 3):
$\alpha_b = \min\left[1.0,\ p_1/3d_0 - 0.25\right]$
Bolt row 4:
$\alpha_b = \min\left[1.0,\ p_2/3d_0 - 0.25\right]$

(*): used for joint with 4 bolt rows (p_1 should be replaced by p)</td><td>Shear load going up:
Bolt rows 1 and 5 (or(*) rows 1 and 3):
$\alpha_b = 1.0$
Bolt row 6 (or(*) row 4):
$\alpha_b = \min\left[1.0,\ e_x/3d_0\right]$
Bolt rows 2 and 4 (or(*): row 2)
$\alpha_b = \min\left[1.0,\ p_1/3d_0 - 0.25\right]$
Bolt row 3:
$\alpha_b = \min\left[1.0,\ p_2/3d_0 - 0.25\right]$</td></tr>
<tr><td>Bolts in shear</td><td colspan="2">For simple bolt row (two bolts) in shear:
$F_{b,Rd} = 2\frac{\alpha_v f_{ub} A_s}{\gamma_{M2}}$
α_v =0.6 for 8.8 bolts and α_v =0.5 for 10.9 bolts.</td><td>EC3-1-8 3.6.1</td></tr>
</table>

3.6 Dog-bone joints

The dog-bone or RBS (reduced beam section) joints were considered in the EqualJoints project as part of examining the use of European steel for large beam-column assemblies incorporating this type of dissipative connection in US construction. Accordingly, they represent a special case which is not directly related to the other connection configurations discussed above (i.e. haunched, extended stiffened and extended unstiffened. The design in this case therefore mainly follows AISC provisions (see typical configuration in Figure 6.1), although some aspects such as those related to the design of the web panel can be replaced directly by recommendations made above for full strength joints in other previous configurations for design in European practice.

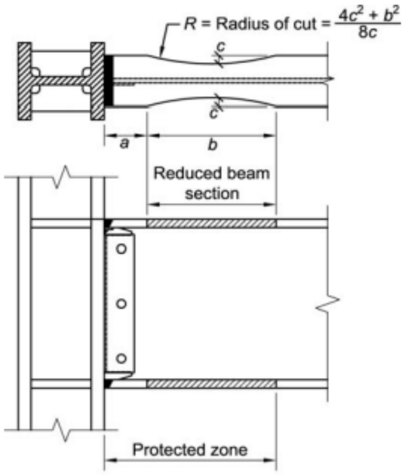


Figure 3.11: Configuration and dimensions of Reduced Beam Section Connections (ANSI/AISC 358)

On this basis, the design needs to follow the requirements of AISC 341 (Seismic Provisions for Structural Steel Buildings), AISC 358-16 (Prequalified Connections for Seismic Applications) and AISC 360 (Specification for Structural Steel Buildings).
Based on the above, the design follows the following procedure:

1. Check beam local buckling for seismic compactness
   $b_{bf}/(2t_{fb}) \quad < \quad \lambda_{ps} \quad = \quad 0.3\sqrt{(E/f_y)}$

2. Check column local buckling for seismic compactness
   $b_{cf}/(2t_{fc}) \quad < \quad \lambda_{ps} \quad = \quad 0.3\sqrt{(E/f_y)}$

3. Check Beam Limitations of AISC 358 Sect 5.3.1
   Noting however that based on the tests carried out in the EqualJoints project, the beam sizes can be extended from depth of W36 to W44 which demonstrated adequate behaviour under the prequalification requirements.

4. Check Column Limitations of AISC 358 Sect 5.3.2
   Noting however that based on the tests carried out in the EqualJoints project, the column sizes can be extended from depth of W36 to W40 which demonstrated adequate behaviour under the prequalification requirements.

5. Determine plastic section modulus at the centre of the reduced beam section (AISC 358 Sect 5.8, Step 2)
   $Z_{RBS} \quad = \quad Z_x - 2ct_{fb}(h_b - t_{fb})$

Where:

Z_{RBS} is the plastic section modulus at the centre of the reduced beam section

$Z_{pl.x}$ is the plastic section modulus about the x-axis for full beam cross section

t_{fb} is the thickness of the beam flange

h_b is the beam depth

c is the depth of cut at center of the reduced beam section

6. Determine probable maximum moment at the reduced beam section (AISC 358 Sect 5.8 Step 3)

$$M_{pr} = M_{RBS} = C_{pr} R_y f_y Z_e$$

Where:

C_{pr} is a factor to account for the peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions, calculated as follows:

$$C_{pr} = \frac{f_y + f_u}{2f_y} \leq 1.2$$

R_y Ratio of the expected yield stress to the specified minimum yield stress, f_y

7. Compute the shear force at the centre of the RBS (AISC 358 Sect 5.8 Step 4)

$$V_p = V_{RBS} = 2M_{pr} / L_h + V_g$$

8. Compute the corresponding shear in the column

$$V_c = N_b V_e L_b / (N_c h_c)$$

9. Compute the probable maximum moment at the face of the column (AISC 358 Sect 5.8 Step 5)

$$M_f = M_{pr} + V_{RBS} S_h + M_g$$

Where:

$$M_g = 1/2 W_{ub} S_h^2$$

10. Compute the expected plastic moment of the beam (AISC 358 Sect 5.8 Step 6)

$$M_{pe} = R_y f_y Z_{bx}$$

11. Check the flexural strength does not exceed $\phi_d M_{pe}$ (AISC 358 Sect 5.8 Step 7)

$$M_f < \phi_d M_{pe}$$

12. Calculate and check the concentrated force in the column

$$P_b \leq \phi f_y w_{tw} \left(5k + l_b\right)$$

$$\leq \phi 0.8 t_w^2 \left[1 + 3\left(l_b/d\right)\left(t_w/t_f\right)^{1.5}\right]\left(E f_{yw} t_f / t_w\right)^{1/2}$$

$$\leq \phi 6.25 f_{yf} t_f^2$$

Where:

$$P_b = M_f b_{fb} t_{fb} / Z_x$$

13. Check column-beam moment ratio (AISC 341 Sect. 9.6)

$$\Sigma M^*_{pc} / \Sigma M^*_{pb} > 1.0$$

Where:

ΣM^*_{pc} is the sum of moments in the column above and below the joint at the intersection of the beam and column centreline

$$= \Sigma\left[Z_c\left(f_{yc} - P_{uc}/A_g\right) + V_c d_b/2\right)\right]$$

ΣM^*_{pb} is the sum of moments in the beams at the intersection of the beam column centreline

$$= N_b M_{RBS} + \Sigma M_v$$

ΣM_v is the additional moment due to the shear amplification from the location of the plastic hinge to the column centreline

$$= \left(V_{RBS} + V'_{RBS}\right)\left(a + b/2 + d_c/2\right)$$

14. Check column panel zone shear strength (AISC341 Sect 9.3)

$$0.75 P_c > P_r$$

$$\varphi_v R_n > \Sigma M_f / \left(d_b - t_{fb}\right) - V_c$$

15. Calculate required thickness of doubler plate

$$R_u \leq \varphi R_{ncol} + \varphi R_{ndp}$$

$$t_{dp} \geq \left(R_u - \varphi R_{ncol}\right) / \left(0.6 f_y d_c\right)$$

16. Check required thickness of column web and doubler plate if provided

$$t \geq \left(d_z + w_z\right) / 90$$

17. Check if continuity plates are required (AISC 358 Step 10)

$$t_{fc} \geq 0.4\left[1.8 b_b f_t b_f \left(F_{yb} R_{yb}\right) / \left(F_{yc} R_{yc}\right)\right] 0.5$$

$$t_{fc} \geq b_{fb} / 6 \text{ or } 12$$

18. Calculate required thickness of continuity plate

Check 1: $t_s \geq 0.5 t_{bf}$

Check 2: $P_b \leq \varphi R_{ncol} + \varphi R_{ncp}$

$$t_s \geq \left(P_b - \varphi R_{ncol}\right) / \left(0.9 f_y b_{bf}\right)$$

As noted above, the design follows the requirements of AISC 341 (Seismic Provisions for Structural Steel Buildings), AISC 358-16 (Prequalified Connections for Seismic Applications) and AISC 360 (Specification for Structural Steel Buildings). This is in consistency with the purpose of the tests carried out within the project which did not examine European design or European sections, but instead focused on validating the use of large European steel designed according to US provisions and adopted in US construction practice.

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