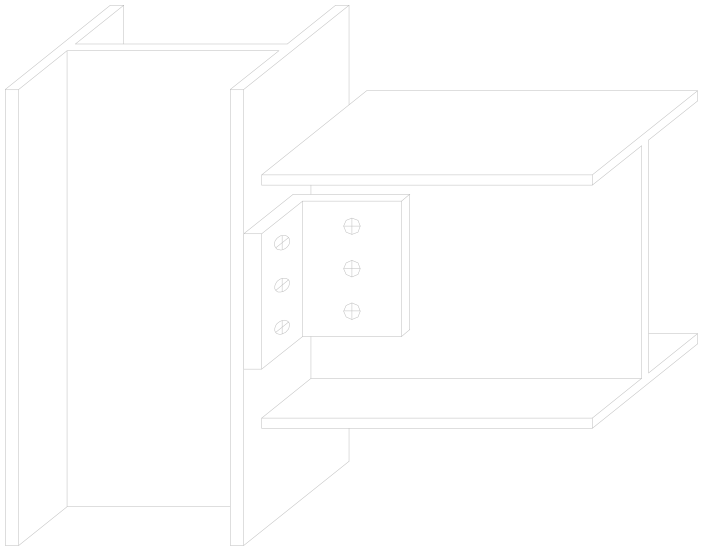
Université de Liège
Faculté des Sciences Appliquées
Département de Mécanique des matériaux et Structures

European recommendations for the design of simple joints in steel structures



First draft of a forthcoming Design Manual

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Document prepared under the supervision of ECCS TC10 by :
J.P. JASPART, S. RENKIN and M.L. GUILLAUME

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1 Preface

In some countries of the European Community, design rules for simple structural joints already exist. Unfortunately, these recommendations don't cover all the types of failure and give sometimes significantly different design rules for a typical failure mode.

In a first step, a comparative study [1] of design rules for pin connections has been achieved. In this work, reference is made to different normative documents or design recommendations :

- Eurocode 3 [2] and its revised Annex J [3];
- BS5950 [4] and BCSA-SCI recommendations [5, 6] ;
- NEN 6770 [7, 8] ;
- German "Ringbuch" [9] ;
- …

Each of these documents possesses its own application field which favours different failure modes. So, the comparison between them is rather difficult.

With the aim to establish a full design approach according to the general design principles stated in Eurocode 3, some design sheets for header plate and fin plate connections were prepared at the University of Liège and discussed at several meetings of the E.C.C.S. Technical Committee 10 « Connections ». The present report contains all these design rules. Explanations of these rules as well as their range of validity are available in [10].

In a few years, it is expected that the practical design recommendations presented in this booklet or in its eventual revised version will replace, in every country, the national normative documents or recommendations. In this way, it will simplify the free trade between the different European countries.

2 Scope and field of application

2.1 Types of structures

Simple structural joints are commonly met in steel framed buildings but they can be used also in other types of structures to connect each other steel elements (for example : bridge structures).

2.2 Types of connected elements

The shape of the structural connected elements which are considered in this report are :

- I or H beams ;
- I or H columns (with a possible extension to RHS and CHS).

2.3 Types of loading

The design methods are intended for joints subject to predominantly static or quasi-static loading. The influence of fatigue effects is disregarded.

The connection resistance is checked under shear and tying forces. The shear forces correspond to usual loading conditions of the structure during its life ; tying forces may develop when the frame is subjected to an explosion or when a supporting column collapses (Fig. 2.1).

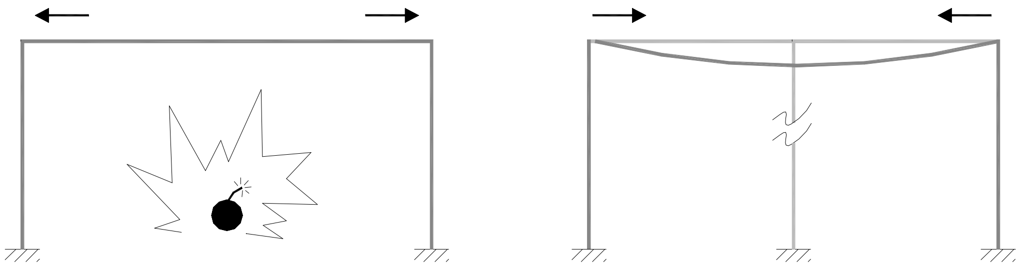


Figure 2.1 : Tying forces

2.4 Steel grades

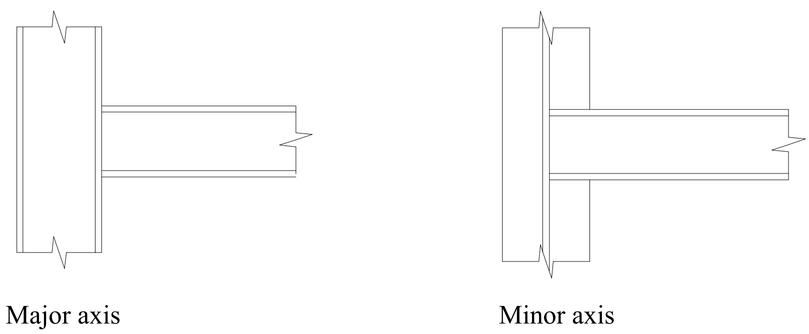
This draft applies to steel grades S 235, S 275, S 355, S 420 and S 460.

2.5 Possible joint configurations

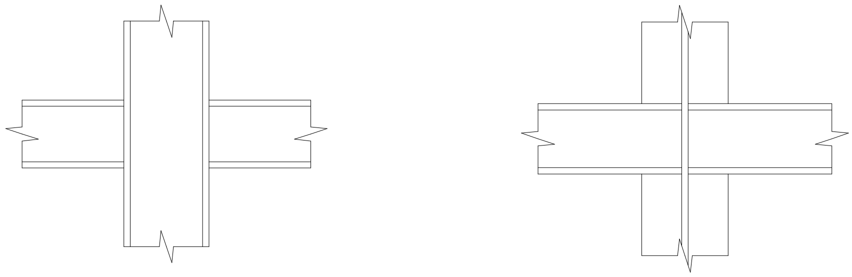
All the possible configurations of simple joints are as follows :

- Beam-to-column (Fig. 2.2) :

  a) Single-sided joint configuration



b) Double-sided joint configuration

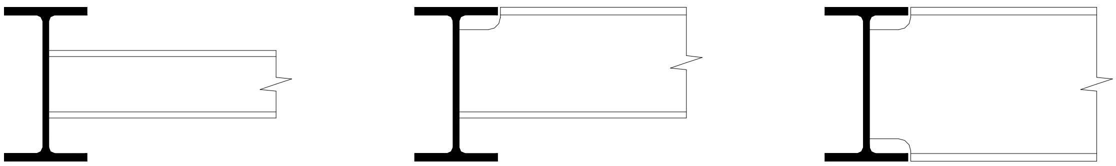


Major axis Minor axis

Figure 2.2 : Beam-to-column joint configurations

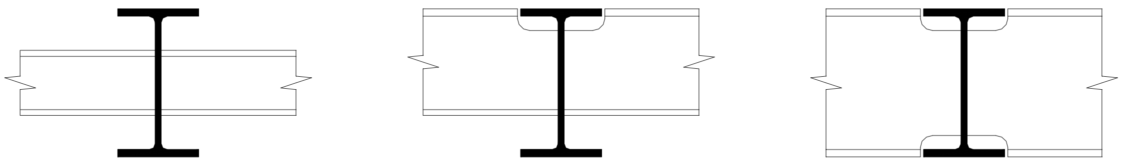
- Beam-to-beam (Fig. 2.3) :

a) Single-sided joint configuration



Uncoped supported beam web Single-coped supported beam web Double-coped supported beam web

b) Double-sided joint configuration



Uncoped supported beam web Single-coped supported beam web Double-coped supported beam web

Figure 2.3 : Beam-to-beam joint configurations

- Beam splice (Fig. 2.4 a and b) :

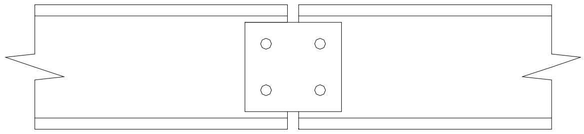


Figure 2.4 a : Beam splice joint

The possible localisations of this joint may be the following (Fig. 2.4 b) :

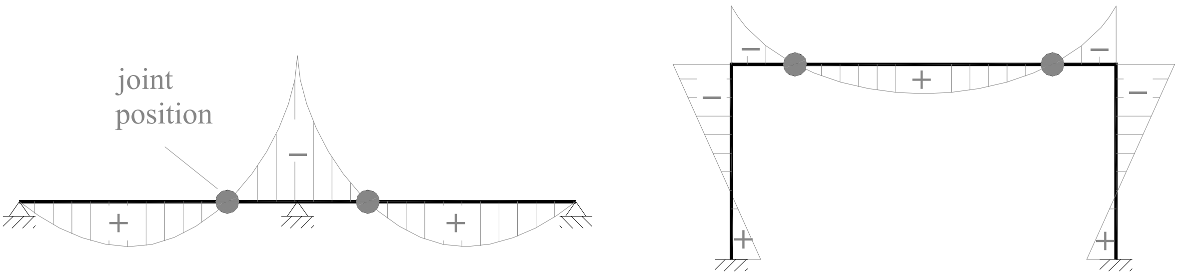


Figure 2.4 b : Diagrams of bending moments

- Column splice (Fig. 2.5) :

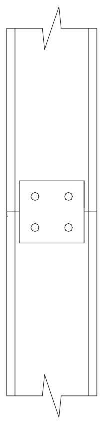


Figure 2.5 : Column splice joint

- Braced connection (Fig. 2.6) :

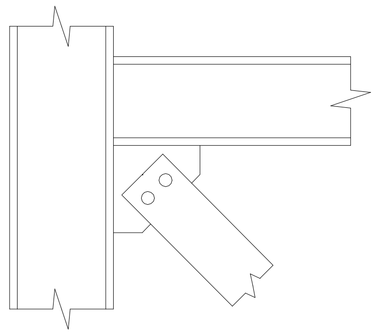


Figure 2.6 : Braced configuration

- Column base (Fig. 2.7) :

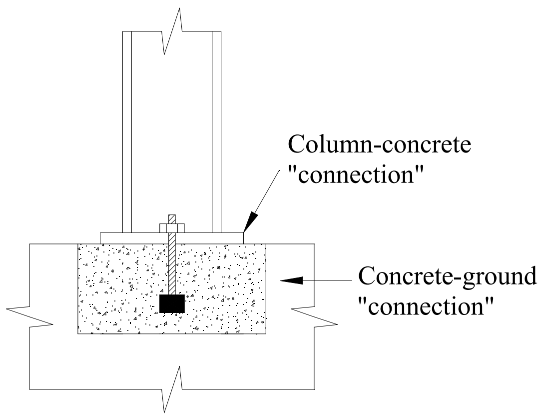


Figure 2.7 : Column base joint configuration

Amongst these joint configurations, only the two first ones will be explicitly covered : beam-to-column and beam-to-beam configurations. The others are expected to be covered in a revised edition of the present booklet.

2.6 Types of fasteners

2.6.1 Bolts

There exist two classes of bolts : normal bolts and high strength bolts. The second class can be used for preloaded bolts which are characterized by a slip-type resistance in shear.

In this document, only non-preloaded bolts are explicitly covered. Their design geometric and mechanical characteristics are respectively given in the tables 2.1 and 2.2. The extension of the

rules to preloaded bolts is not at all a difficulty and will be worked out when preparing the final draft of first edition of the present booklet.

d (mm)	**8**	**10**	**12**	**14**	**16**	**18**	**20**	**22**	**24**	**27**	**30**
A (mm²)	50	78	113	154	201	254	314	380	452	573	707
A_s (mm²)	36	58	84	115	157	192	245	303	353	459	561

with d = Nominal diameter of a bolt shank
A = Nominal area of a bolt
A_s = Resistant area of a bolt

Table 2.1 : Bolt Areas

Bolt grade	**4.6**	**5.6**	**6.8**	**8.8**	**10.9**
f_{yb} (N/mm²)	240	300	480	640	900
f_{ub} (N/mm²)	400	500	600	800	1000

Table 2.2 : Nominal values of yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts

2.6.2 Welds

The possible types of weld are fillet welds, fillet welds all round, butt welds, plug welds and flare groove welds. Only fillet welds are considered in the present draft.

2.7 Types of connections

The three connections types which are considered in these design recommendations are used to connect a beam to a column or a beam to a beam. These are the following :

- **Header plate connections :**

  The main components of the header plate steel connection are shown in Fig. 2.8. : a steel plate, a fillet weld on both sides of the supported beam web, and two single- or double-vertical row group. The plate is welded to the supported member and bolted to a supporting element such as a steel beam or column. Its height is not exceeding that of the beam web

and the plate is never extending beyond the beam web to the beam flanges. The end of the supported steel beam may be uncoped, single-coped or double-coped.

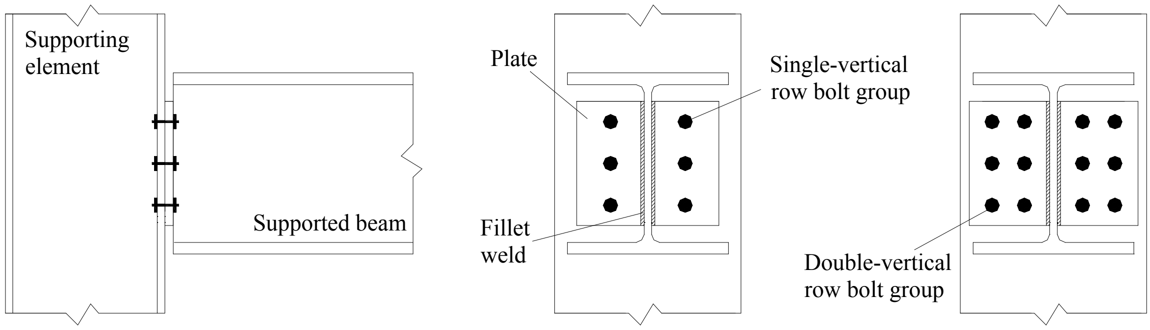


Figure 2.8 : Header plate connection

- **Fin plate connections :**

The main components of the fin plate steel connection are shown in Fig. 2.9. : a fin plate, a fillet weld on both sides of the plate, and a single- or double-vertical row group. The plate is welded to a supporting member such as a steel beam or column and bolted to the supported beam web. The end of the supported steel beam may be uncoped, single-coped or double-coped.

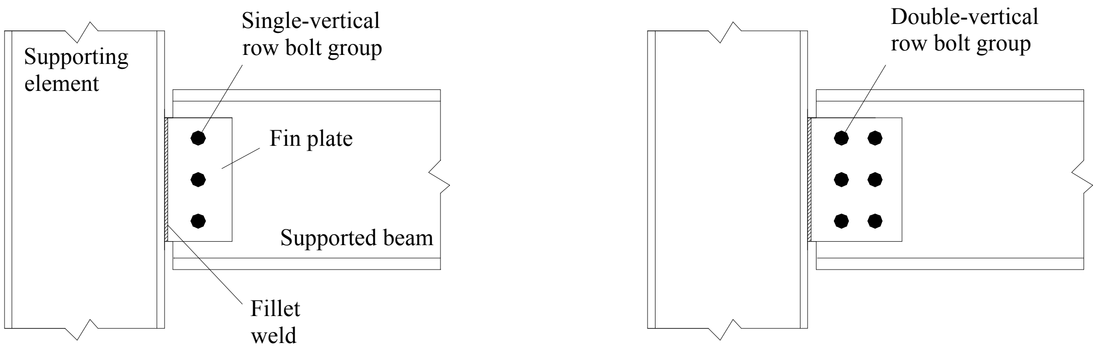


Figure 2.9 : Fin plate connection

- **Web cleat connections :**

The main components of the web cleat steel connection are shown in Fig. 2.10 : two web cleats and three single- or double-vertical row group (two on the supporting element and one on the supported member). The cleats are bolted to the supporting and supported members. The end of the supported steel beam may be uncoped, single-coped or double-coped.

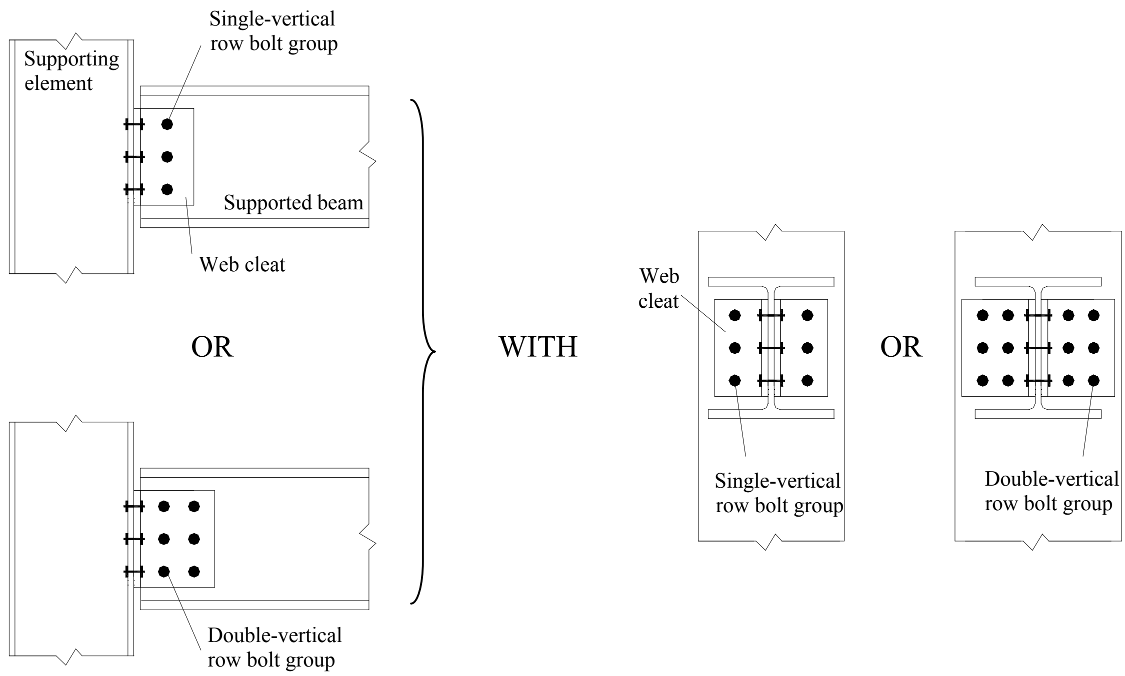


Figure 2.10 : Web cleat connection

Note :
Traditionally, other types of beam-to-column connections are considered as hinge. But, now, Eurocode 3 Part. 1.8 classes them as semi-rigid joints. Two examples are given in Fig. 2.11.

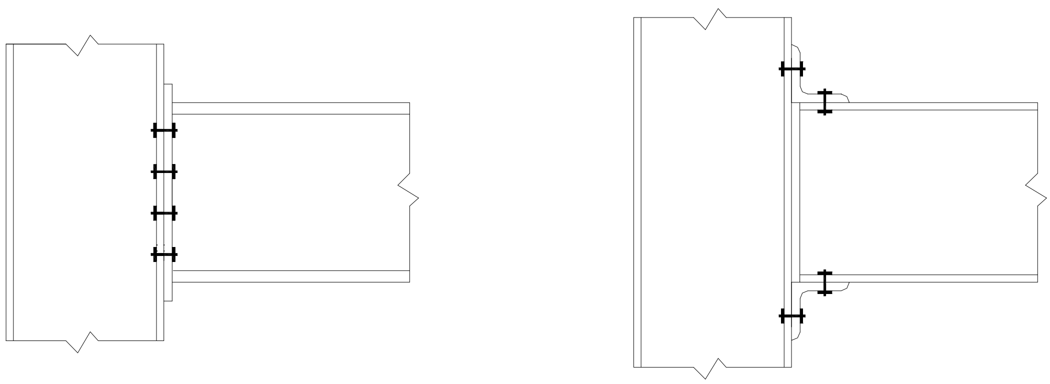


Figure 2.11 : Other simple connections

The present manual could be extended in the future to cover these connection types.

2.8 Reference code

The design rules are based on the resistance formula provided by Eurocode 3 Part. 1.8 as far as information is available. If not, the basic design principles prescribed by Eurocode 3 Annex J are anyway respected.

3 Joint modelling for frame analysis and design requirements

3.1 General

The effects of the actual response of the joints on the distribution of internal forces and moments within a structure, and on the overall deformations of the structure, should generally be taken into account ; but when these effects are sufficiently small, they may be neglected.

To identify whether the effects of joint behaviour on the analysis need be taken into account, a distinction should be made between the three following modellings :

- *simple*, in which the joint may be assumed not to transmit bending moments ;
- *continuous*, in which the behaviour of the joint may be assumed to have no effect on the analysis ;
- *semi-continuous*, in which the behaviour of the joint needs to be taken into account in the analysis.

The appropriate type of joint model depends on the classification of the joint and on the selected procedure for structural analysis and design.

3.2 EC 3 classification system

The joints can be classified according to the values of their main structural properties, i.e. rotational stiffness, strength in bending and rotational capacity (or ductility). It's very important that the structural properties of all the joints correspond to the assumptions made in the analysis of the structure and in the design of the members. In particular, as far as simple joints are concerned, the available rotation capacity of the joints should be sufficient to accept the rotations of the joints resulting from the analysis.

In the Eurocode 3 Part. 1.8, joints are classified by stiffness and by strength. Ductility aspects are also considered ; they will be more especially addressed in Section 4.

3.2.1 Classification by stiffness

This classification is only valid for the beam-to-column joint configurations. Through the comparison of actual rotational stiffness $S_{j,ini}$ with classification boundaries (Fig. 3.1), a joint may be considered as :

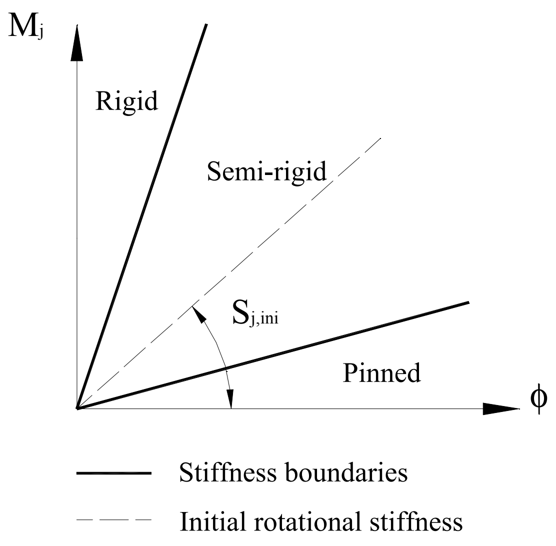


Figure 3.1 : Boundaries for stiffness classification of joints

- *Nominally pinned joints :*

  The joint shall be capable of transmitting the internal forces, without developing significant moments which might adversely affect the members of the structure. It shall be also capable of accepting the resulting rotations under the design loads.

  ⇒ Boundary : $S_{j,ini} \leq 0{,}5\ EI_b / L_b$

- *Rigid joints :*

  The joint behaviour is assumed not to have significant influence on the distribution of internal forces and moments in the structure, nor on its overall deformation.

  ⇒ Boundaries : $S_{j,ini} \geq k_b\ EI_b / L_b$

  where $k_b = 8$ for frames where the bracing system reduces the horizontal displacement by at least 80% ;
  $k_b = 25$ for other frames.

- *Semi-rigid joints :*

  The joint provides a predictable degree of interaction between members, based on the design moment-rotation characteristics of the joint. It should be able to transmit internal forces and moments.

⇒ Boundaries : A joint which doesn't meet the criteria for a rigid or a nominally pinned joint shall be classified as a semi-rigid joint.

Keys : E is the elastic modulus of the material whose the beam is formed ;
I_b is the second moment area of a beam ;
L_b is the span of a beam (distance between the axes of the supporting columns).

3.2.2 Classification by strength

Through the comparison of its actual design moment resistance $M_{j,Rd}$ with the design moment resistances of the members that it connects (Fig. 3.2), a joint may be classified as :

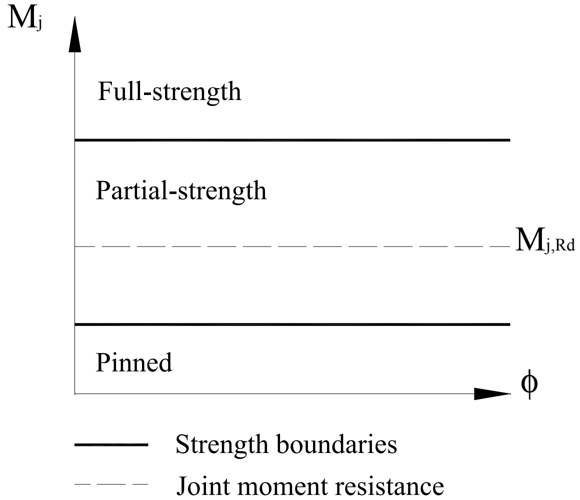


Figure 3.2 : Boundaries for strength classification of joints

- *Nominally pinned joints :*

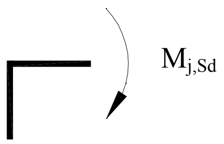
The joint shall be capable of transmitting the internal forces, without developing significant moments which might adversely affect the members of the structure. It shall be capable too of accepting the resulting rotations under the design loads.

⇒ Boundary : $M_{j,Rd} \leq 0{,}25\ M_{\text{full-strength}}$ (see Fig. 3.3)

- *Full-strength joints :*

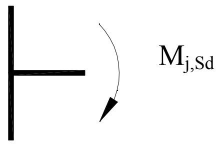
  The design resistance of a full strength joint shall be not less than that of the connected members.

  ⇒ Boundary : $M_{j,Rd} \geq M_{\text{full-strength}}$ (see Fig. 3.3)



Top column :

$M_{\text{full-strength}} = \min(M_{b,pl,Rd}, M_{c,pl,Rd})$



Within column height :

$M_{\text{full-strength}} = \min(M_{b,pl,Rd}, 2\,M_{c,pl,Rd})$

Keys : $M_{b,pl,Rd}$ is the plastic moment resistance of a beam ;
$M_{c,pl,Rd}$ is the plastic moment resistance of a column (possibly reduced by axial or shear forces in the column).

Figure 3.3 : Full-strength resistance

- *Partial-strength joints :*

  A joint which doesn't meet the criteria for full-strength or nominally pinned joints should be considered to have a partial-strength resistance.

3.3 EC 3 joint modelling

The joint modelling depends on the joint classification (see above) and the selected procedure of structural frame analysis and design. Eurocode 3 considers three simplified joint modellings (simple, continuous and semi-continuous) according as the effects of joint behaviour on the analysis can be neglected or no. The appropriate type of joint modelling should be determined from the Table 3.1.

METHOD OF GLOBAL ANALYSIS	CLASSIFICATION OF JOINT		
Elastic	Nominally pinned	Rigid	Semi-rigid
Rigid-Plastic	Nominally pinned	Full-strength	Partial-strength
Elastic-Plastic	Nominally pinned	Rigid and full-strength	Rigid and partial strength Semi-rigid and partial strength Semi-rigid and full-strength
TYPE OF JOINT MODEL	**Simple**	**Continuous**	**Semi-continuous**

Table 3.1 : Type of joint model

So, in the global analysis, the joint behaviour can be replaced by (Fig. 3.4) :

- a hinge, for the simple modelling ;
- a rotational spring, for the semi-continuous modelling [10] ;
- a infinitely rigid and resistant rotational spring, for the continuous modelling.

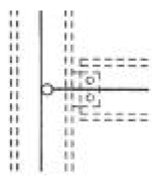
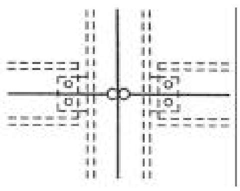
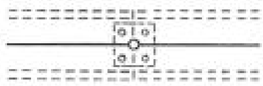
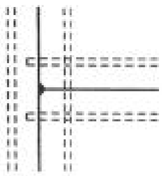
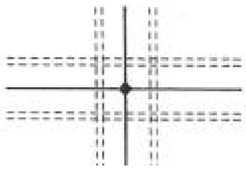
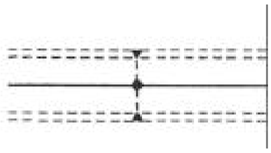
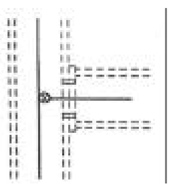
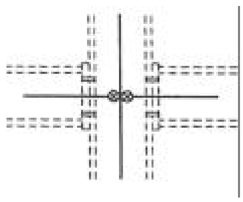
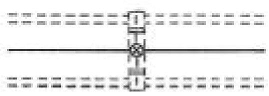
Type of joint model	Single-sided configuration	Double-sided configuration	Beam splice
Simple			
Continuous			
Semi-continuous			

Figure 3.4 : Local joint modelling

In the global structural analysis, the hinge or spring which models the joint is assumed to be located at the intersection of the axes of the connected elements.

3.4 Simple joint modelling

The design rules in this guide are given for joints which are assumed not to transmit bending moments. Thus, the joints should be modelled by a hinge. Unfortunately, a lot of joints which are traditionally considered as a hinge don't fulfilled the stiffness and/or strength limitations required by Eurocode 3 for nominally pinned joints. In front of that situation, what to do?

Two different attitudes may be adopted :

- According to the Eurocode 3 requirements, the joint is modelled by a rotational spring and is therefore considered as semi-rigid (what it is in reality). Its rotational stiffness, design bending resistance and shear resistance have to be evaluated and the actual properties of the joint have to be explicitly taken into consideration in the frame and joint design and analysis process. This approach is the more scientifically correct one but it requests more complex calculations in the global analysis and in the joint design.

- Despite its actual properties, the joint is considered as a hinge and the design rules presented in this present booklet for simple joints can be applied, but under some strict conditions in order to ensure the safe character of the approach. The global analysis and the joint design are more simple in this case as they are based on a common hinged approach.

If the second option is chosen, the joint is assumed not to transmit bending moments but it's not true in the reality. Bending moments develop actually into the joints which are designed only to resist to shear forces. This is potentially unsafe and is of course not basically acceptable at first sight.

But a careful examination of this problem allows anyway to conclude to the safe character of the "hinge assumption" if the two following requirements are fulfilled :

- the joint possesses a sufficient rotation capacity ;
- the joint possesses a sufficient ductility.

The first requirement relates to the rotational capacity that the joint should have, in order to "rotate" as a hinge, without developing too high internal bending moments.

The second requirement is there to ensure that the development of combined shear and bending forces into the joint is not leading to brittle failure modes (for instance, because of a rupture of a bolt or a weld). In other words, the design of the joint should be achieved to allow internal plastic deformation instead of brittle phenomena.

If this two requirements (sufficient rotation capacity and ductility) are fulfilled, it can be demonstrated that to consider an actually semi-rigid joint as a nominally pinned one, is safe for design purposes and, in particular, for the evaluation of :

- *the frame displacements* :
  the stiffness of the actual structure is always greater than the considered one, and all the actual displacements are therefore lower than the calculated ones ;

- *the plastic failure loading* :
  as the actual bending strength of the joint is higher than the considered one (equal to zero), the first order plastic resistance of the frame is higher than the one evaluated on the basis of a hinge behaviour ;

- *the critical loading of linear elastic instability* :
  the transversal stiffness of the actual structure is larger than the one of the structure with nominally pinned joints, and the rotational restraints at the end of the columns in the actual structure are higher than these calculated with a hinge assumption ; this ensures the safe character of the hinge assumption as far as global and local instability are concerned ;

- *the elastic-plastic phenomena of instability* :
  the actual stiffness of the structure is greater than the considered one but the actual solicitations are more important than these acting on the structure with nominally pinned joints ; nevertheless, experimental studies ([14] and [15]) show that the hinged approach is safety.

For further explanations, see [10].

In this guide, the design recommendations refer to the so-called "hinge model" and specific design requirements ensuring this safe character are presented for each of the considered connection types.

3.5 Summary of design requirements

In this booklet, the internal forces in the joint are determined by a structural analysis based on a simple "hinge" modelling joint. The hinge is assumed to be located at the intersection of the axes of the connected elements. As a result of this structural analysis, the maximum applied shear force and rotation in the joint, respectively V_{Sd} and $\phi_{required}$, are obtained.

From the geometrical properties of the joint and the mechanical properties of its constitutive materials, the available rotation capacity of the joint, $\phi_{available}$, can be estimated, as well as its design shear resistance, V_{Rd}. To ensure the validity of this approach, some ductility requirements have to be satisfied and the available rotation of the joint has to be higher than the required one. Finally, the joint will be considered as acceptable if the applied shear force is smaller than the design shear resistance.

Sometimes, the evaluation of the resistance to tying forces is requested for robustness purposes.

4 Practical ways to satisfy the ductility and rotation requirements

4.1 General principles

A simple joint is nothing else than an idealisation of the reality. Joints like those studied in the present document undergo a significant internal rotation but transfer anyway a certain bending moment. As explained above, to ensure the safety of the simple joint model, some requirements for sufficient ductility and rotation capacity are necessary.

These requirements can be written for each considered connection type, in the form of simple criteria based on the mechanical and geometrical characteristics of the different components forming the connection.

The rotation capacity requirements provide to the hinge a sufficient rotation without developing too significant bending moments which might adversely affect the members of the structure. These criteria are often expressed as geometrical limitations.

The ductility requirements avoid the occurrence of brittle failures, especially in bolts and welds, and buckling. Their derivation is more complex. In the "hinged" structural analysis, the joint is assumed to be only subjected to a shear force. In the reality, a bending moment and a shear force are acting simultaneously on the joint. In an "applied shear force – applied bending moment" graph (Fig. 4.1), the evolution of the actual and idealised loading types can be represented by two paths. The first is an horizontal one ($M_{Sd} = 0$) and the second an oblique one. The inclination of the actual loading path depends on the relative stiffness between the joint and the connected elements.

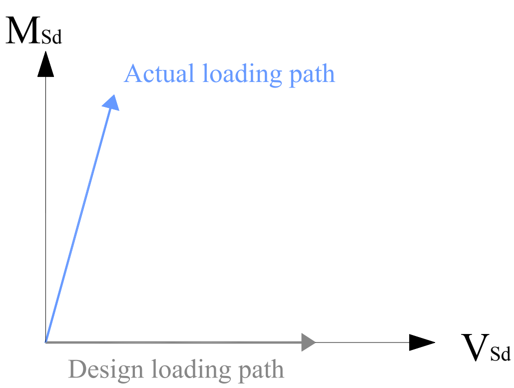


Figure 4.1 : Loading paths

Note : For fin plate connections, two different cross-sections inside the joint have to be considered separately. The first one is located at the external face of the supporting member ; while the second is at the level of the bolt group centre.
The actual loading situation is different in these two sections, so leading to two distinct M_{Sd} – V_{Sd} paths in the diagram shown on Figure 4.2.

But if a "hinge" model is considered, the first section is assumed to transfer only shear forces (M_{Sd} = 0) while the second one, for sake of equilibrium, transfers the same shear force V_{Sd} and a bending moment M_{Sd} equal to V_{Sd} . z.
z is defined as the distance between the external face of the supporting element and the bolt group centre.

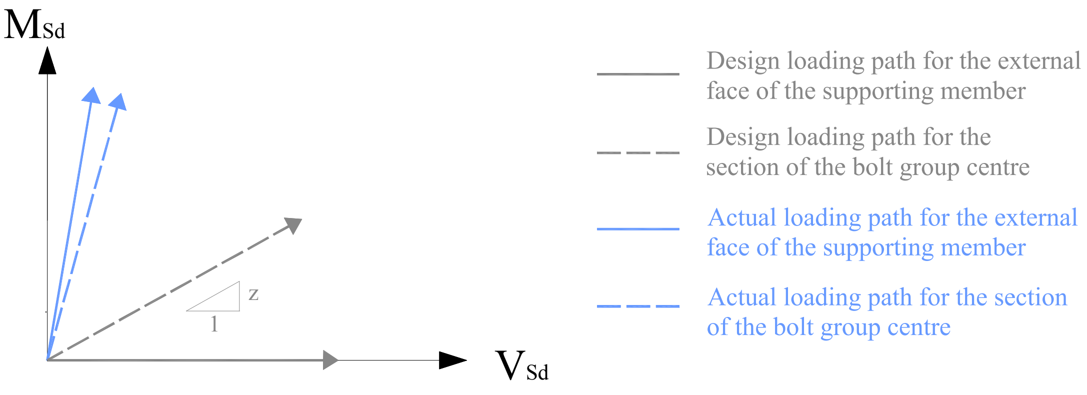


Figure 4.2 : Loading paths for a fin plate connection

The design shear resistance of each component of the joint can be represented in a "shear force – bending moment" graph. According as this resistance is influenced or not by the applied bending moment, its representation will be a curve or a vertical line. An example of three failure modes for a fin plate connection is given in Figure 4.3. The relative position between the different resistance curves or lines depends on the geometrical and mechanical characteristics of the joint components.

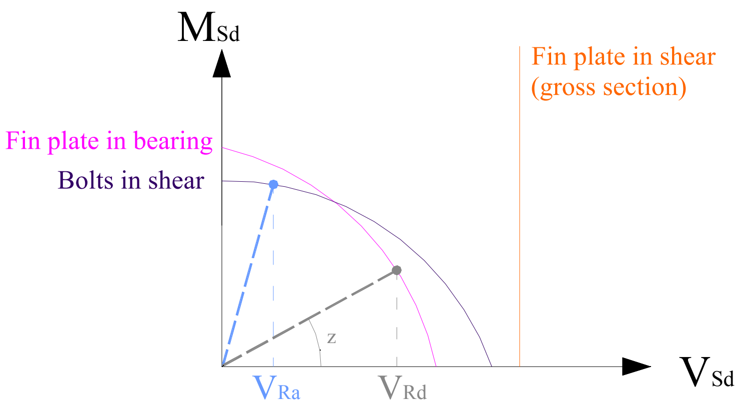
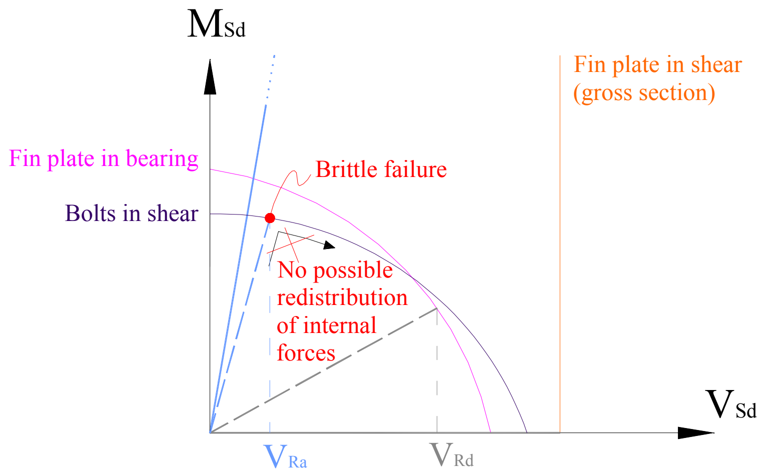


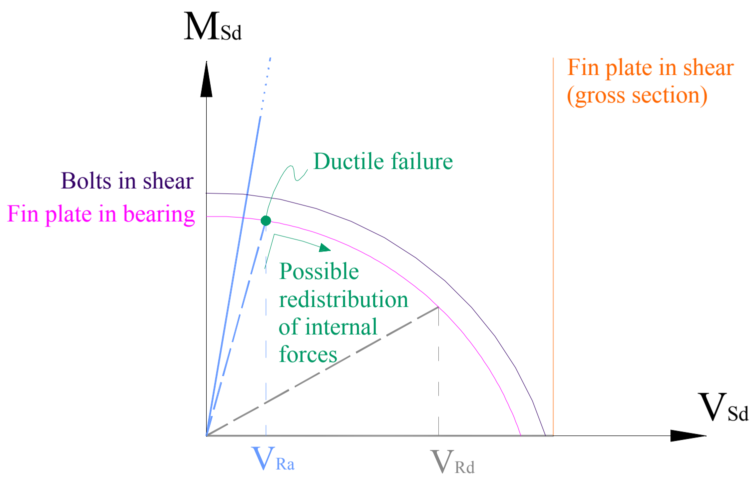
Figure 4.3 : Design resistances for some components of a fin plate connection and principle for the derivation of the shear resistance of the joint

In reality, the actual shear resistance, V_{Ra}, could be defined at the intersection between the actual loading path, in the appropriate cross-section, and the design resistance curves or lines of the weakest components (Fig. 4.3). If a similar principle is applied to the design loading path, a design shear resistance, V_{Rd}, is then obtained.

If the failure mode corresponding to the V_{Ra} value is brittle, the design shear resistance is seen as a full unconservative estimation of the joint resistance (Fig. 4.4 a). In fact, the only way to reach the design shear resistance V_{Rd}, is to rely on a plastic redistribution of internal forces inside the joint, as shown on Figure 4.4 b.



a) Premature brittle failure



b) Possible plastic redistribution of internal forces

Figure 4.4 : Determination of the shear resistance of the joint

As a conclusion, the ductility requirements will aim to ensure that the move from the actual to the design shear resistances may occur, as a result of a plastic redistribution of internal forces inside the joint.

In the next paragraphs, the design requirements to be fulfilled to allow sufficient rotation capacity and ductility are specified for all the connection types covered in the present booklet.

4.1.1 Header plate connection

4.1.1.1 Design requirements for sufficient rotation capacity

With the aim to permit a rotation without increasing too much the bending moment which develops into the joint, the contact between the lower beam flange and the supporting member has to be strictly avoided. So, it's imperative that the height of the plate is lower than that of the supported beam web (Fig. 4.5) :

$$h_p \leq d_b$$

where d_b is the clear depth of the supported beam web

If such a contact takes place, a compression force develops at the contact place; it is equilibrated by tension forces in the bolts and a significant bending moment develops (Fig. 4.5).

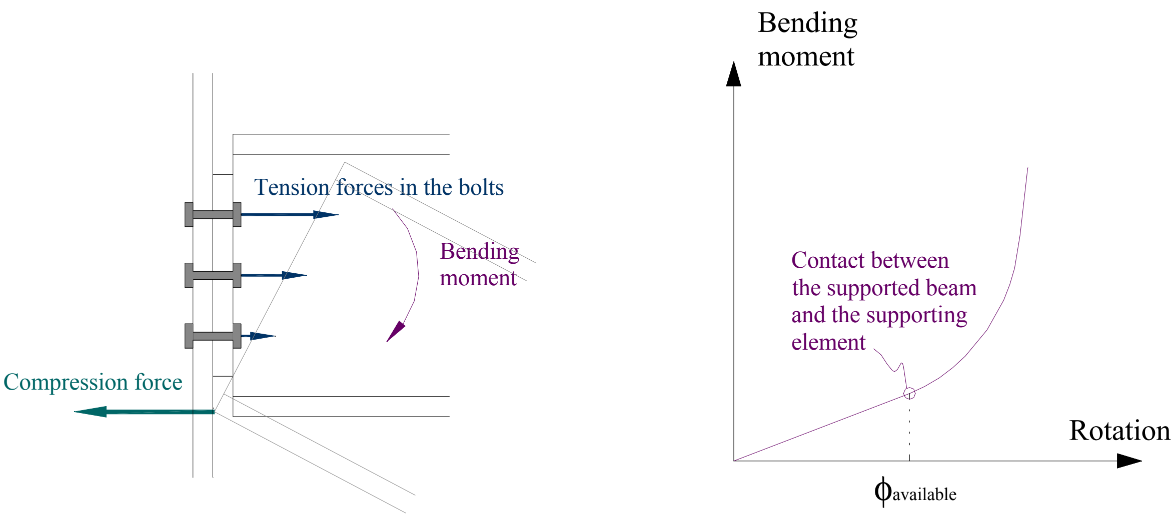


Figure 4.5 : Contact and evolution of the bending moment

The level of rotation at which the contact occurs is obviously dependent on the geometrical characteristics of the beam and of the header plate, but also on the actual deformations of the joint components.

In order to derive a simple criterion that the user could apply, before any calculation, to check whether the risk of contact may be disregarded, the following rough assumptions are made (see Fig. 4.6) :

- the supporting element remains undeformed ;
- the centre of rotation of the beam is located at the lower extremity of the header plate.

On the basis of such assumptions, a safe estimation (i.e. a lower bound) of the so-called "available rotation of the joint" $f_{available}$ may be easily derived :

$$\phi_{available} = \frac{t_p}{h_e}$$

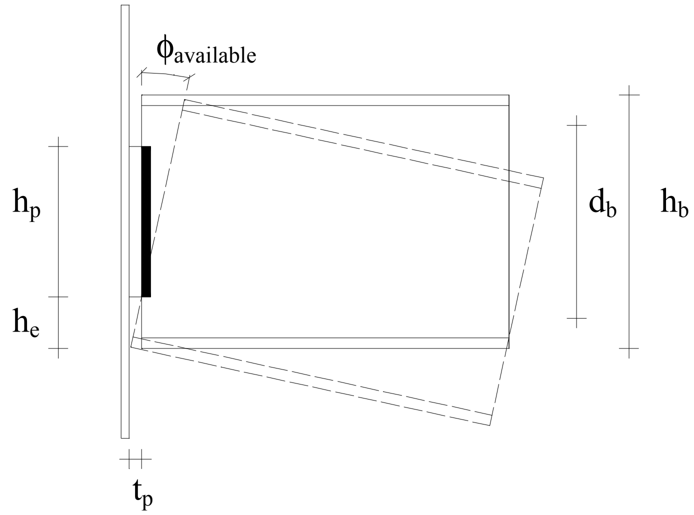


Figure 4.6 : Geometrical characteristics of the joint and illustration of the contact between the beam and the supporting element

This available rotation has to be greater than the "required rotation capacity" which varies according to the structural system and loading. So, simple criterion ensuring the sufficient joint rotation capacity may be written as :

$$\phi_{available} > \phi_{required}$$

For instance, the required rotation capacity, for a beam (length *L* and inertia *I*) simply supported at its extremities and subjected to an uniformly distributed load (factored load *g p* at ULS), writes:

$$\phi_{required} = \frac{\gamma \, p \, L^3}{24 \, EI}$$

By expressing that $\phi_{available} > \phi_{required}$, a simple criterion ensuring a sufficient joint rotation capacity may be derived. It writes:

$$\frac{t}{h_e} > \frac{\gamma \, p \, L^3}{24 \, EI}$$

Similar criteria may be derived for other load cases (Annexe 1).

4.1.1.2 Design requirements for sufficient joint ductility

As already said, bending moments develop in the joint and, as a result, the bolts and the welds are subjected to tension forces in addition to shear forces. The premature failure of these elements which exhibit a brittle failure and which are more heavily loaded in reality than in the calculation model, has therefore to be strictly avoided. Simple related criteria should therefore be proposed.

Criterion to avoid premature bolt failure because of tension forces

In Eurocode 3, a criterion based on the T-stub approach ensures that a yield lines mechanism develops in the plate before the strength of the bolts is exhausted (see [2]); its background is given in [12].

According to this criterion, at least one of the two following inequalities (1) and (2) has to satisfied :

(1) $$\frac{d}{t_p} \geq 2,8\sqrt{\frac{f_{yp}}{f_{ub}}}$$

(2) $$\frac{d}{t_{cf}} \geq 2,8\sqrt{\frac{f_{ycf}}{f_{ub}}}$$ for a supporting column flange

.... for a supporting column or beam web

Note :
A specific criterion has to be established. This criterion is expected to be satisfied by most of the supporting webs because of their slenderness.

where

- d is the nominal diameter of the bolt shank ;
- t_p is the thickness of the header plate ;
- t_{cf} is the thickness of the supporting column flange ;
- f_{yp} is the yield strength of the steel constituting the header plate ;
- f_{ycf} is the yield strength of the steel constituting the supporting column flange ;
- f_{ub} is the ultimate strength of the bolt.

Obviously, such a criterion does not ensure that the whole shear capacity of the bolt may be considered when evaluating the shear resistance of the joint. When this requirement is satisfied, it may be demonstrated :

- that the tension force in the bolts may amount $0,5\ B_{t.Rd}$, i.e. 50% of the design tension resistance $B_{t,Rd}$ of the bolts ;

- that, for such a tension force, the shear resistance only amounts 64% of the full shear resistance of the bolts according to the EC 3 resistance formula for bolts in shear and tension.

This looks at first side to be quite disappointing as the user tries to maximise the shear resistance of the joint. Obviously, it may be argued that only the bolt located in the upper half of the header plane are concerned by such a reduction, as the others are located in a compression zone, and are therefore not subjected to tension forces. Anyway, a reduction of the resistance of the joints when the "bolts in shear" is the governing failure mode is not welcome.

So finally a reduction is taken into consideration by multiplying the total resistance of the bolts in shear by a factor 0,8 (i.e. a reduction factor of 0,64 for half of the bolts located in the upper half of the header plate – 0,5.[1 + 0,64] ≈ 0,8).

Criterion to avoid premature weld failure because of tension forces

An easy way to avoid the brittle failure of the web is to design the latter so that the failure occurs by yielding in the beam web and not in the weld. A full-strength weld is therefore recommended.

According to clause 6.6.5.3 and Annex M in Eurocode 3, the following rule may be applied to estimate the weld resistance per unit length :

$$V_{Rd,Annex\ M} = 2\ a\ \frac{f_{ubw}}{\gamma_{M2}\ \beta_w\ \sqrt{3}} \qquad (\text{ 2 welds})$$

Standard and steel grade			Correlation factor β_w
EN 10025	**EN 10210**	**EN10219**	
S 235 S 235 W	S 235 H	S 235 H	0,8
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH	0,85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH	0,9
S 420 N/NL S 420 M/ML		S 420 MH/MLH	1,0
S 460 N/NL S 460 M/ML S 460Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH	1,0

Table 4.1 : Type of joint model

The weld may be considered as full-strength if its resistance per unit length is higher than the most important force per unit length which is acting in its vicinity on the beam web. This force may be estimated by :

$$V_{Rd,web} = t_{bw} \frac{f_{ybw}}{\gamma_{M0}}$$

Then the minimum value of the throat thickness to get full strength welds may be derived as follows:

$$a > 0{,}5\ t_{bw}\ \beta_w\ \sqrt{3}\ \frac{f_{ybw}}{f_{ubw}} \frac{\gamma_{M2}}{\gamma_{M0}}$$

But a less conservative approach may be followed by recognising, as in [13], that: “In Eurocode 3 (Version of April 1990), it is stated that the requirement for full strength will be satisfied if the design of the weld is not less than 80% of the design resistance of the weakest of connected parts”.

By applying this principle to the present situation, the precedent equation becomes :

$$a > 0{,}4\ t_{bw}\ \beta_w\ \sqrt{3}\ \frac{f_{ybw}}{f_{ubw}} \frac{\gamma_{M2}}{\gamma_{M0}}$$

and a significantly lower weld size is to be recommended (see Table 4.2).

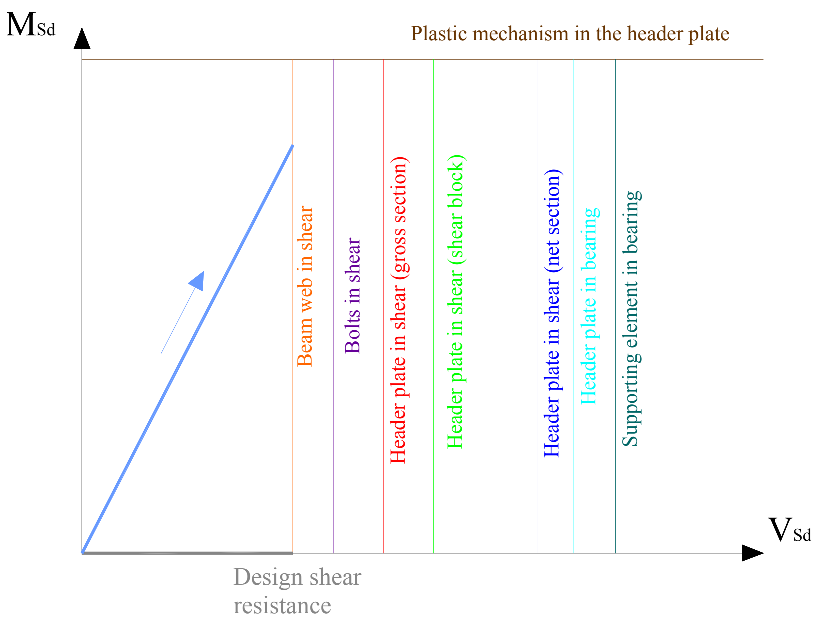
Steel grade (EN 10025)	a_{min}	
	(γ_{M2}=1,25; γ_{M0}=1,0)	(γ_{M2}=1,25; γ_{M0}=1,1)
S 235	0,453 t_{bw}	0,514 t_{bw}
S 275	0,471 t_{bw}	0,535 t_{bw}
S 355	0,543 t_{bw}	0,617 t_{bw}

Table 4.2 : Minimum weld size according EC3 Annex M

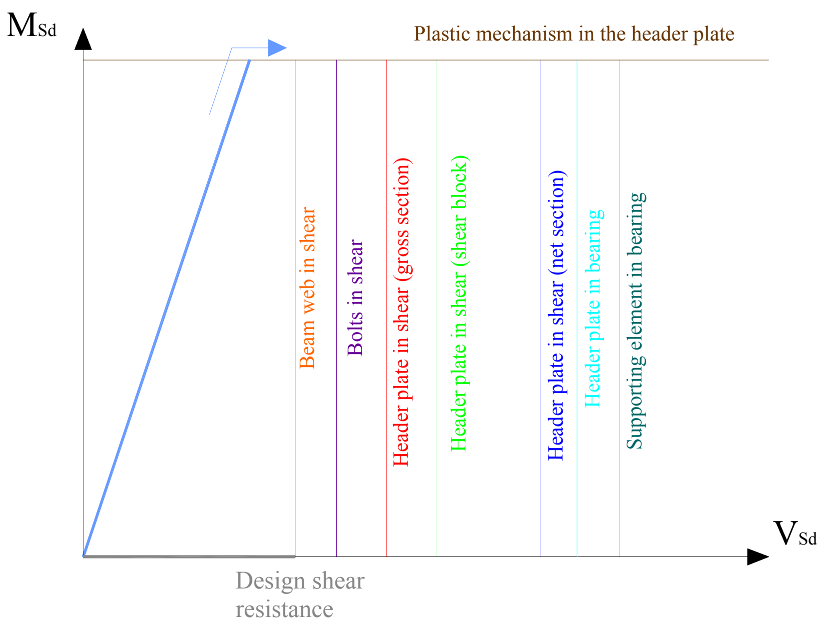
4.1.1.3 Conclusions

If the rotation capacity and ductility requirements are satisfied, the two following quite acceptable situations may occur (Fig. 4.7). For the first case (Fig. 4.7 a), the same failure mode is obtained by following the actual and design loading paths. For the second case (Fig. 4.7 b), the failure mode obtained with the actual loading path is enough ductile to permit a plastic redistribution of internal forces to take place until the design shear resistance is reached.

All these design requirements can be checked before any design calculation.



a) one single failure mode



b) Different failure modes

Actual loading path
Design loading path

Figure 4.7 : Possible failure modes for a header plate connection

4.1.2 Fin plate connection

4.1.2.1 Design requirements for sufficient rotation capacity

With the aim to permit a rotation without increasing too much the bending moment which develops into the joint, the contact between the lower beam flange and the supporting member has to be strictly avoided. So, it's imperative that the height of the fin plate is lower than that of the supported beam web (Fig. 4.8) :

$$h_p \leq d_b$$

where d_b is the clear depth of the supported beam web

If such a contact takes place, a compression force develops at the contact place ; it is equilibrated by tension forces in the welds and in the plate, and additional shear forces in the bolts.

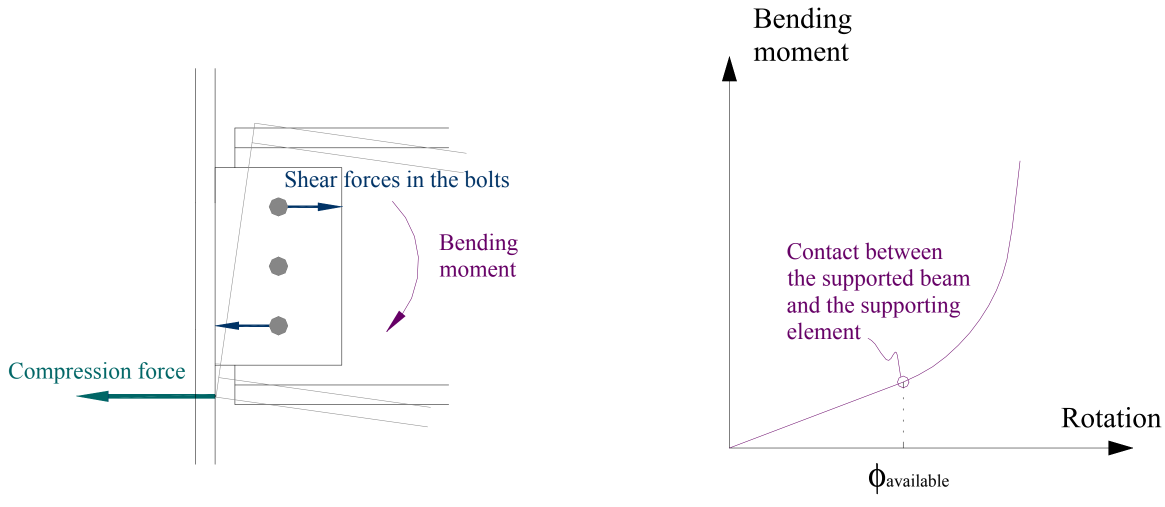


Figure 4.8 : Contact and evolution of the bending moment

The level of rotation at which the contact occurs is obviously dependent on the geometrical characteristics of the beam and of the fin plate, but also on the actual deformations of the joint components.

In order to derive a simple criterion that the user could apply, before any calculation, to check whether the risk of contact may be disregarded, the following rough assumptions are made (see Fig. 4.9) :

- the supporting element and the fin plate remain undeformed ;
- the centre of rotation of the beam is located at the gravity centre of the bolt group.

On the basis of such assumptions, a safe estimation (i.e. a lower bound) of the so-called "available rotation of the joint" $f_{available}$ may be easily derived :

- if $z > \sqrt{(z - g_h)^2 + \left(\frac{h_p}{2} + h_e\right)^2}$:

$$\phi_{available} = \text{"}\infty\text{"}$$

- else :

$$\phi_{available} = \arcsin\left(\frac{z}{\sqrt{(z-g_h)^2 + \left(\frac{h_p}{2} + h_e\right)^2}}\right) - \operatorname{arctg}\left(\frac{z - g_h}{\frac{h_p}{2} + h_e}\right)$$

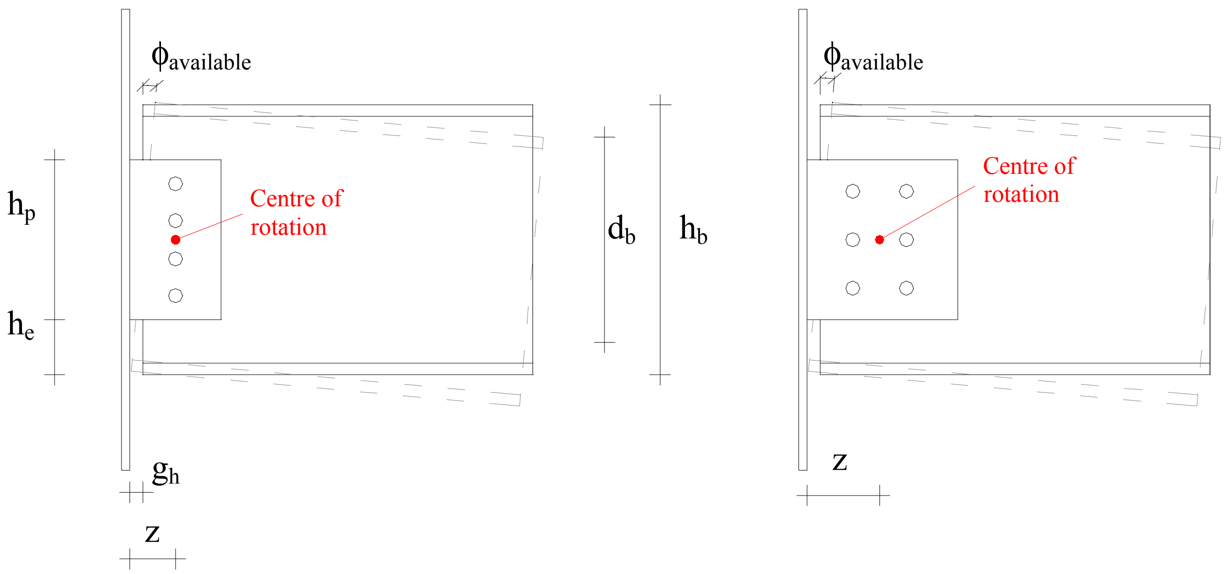


Figure 4.9 : Geometrical characteristics of the joint and illustration of the contact between the beam and the supporting element

This available rotation has to be greater than the "required rotation capacity" which varies according to the structural system and loading. So, a simple criterion ensuring the sufficient joint rotation capacity may be written as :

$$\phi_{available} > \phi_{required}$$

4.1.2.2 Design requirements for sufficient joint ductility

As explained previously, the design shear resistance of the joint may be reached, as a result of a plastic redistribution of internal forces between the different components of the joint. It requires that no local brittle failure mode or instability develop during this redistribution. The failure modes which could prevent for an eventual redistribution of internal forces are, for fin plate connections : the bolts and the welds in shear on account of their brittle nature, and the buckling of the fin plate which is not ductile in terms of plastic redistribution.

Criterion to avoid premature weld failure because of tension forces

A similar criterion as the one established for the header plate connection, may be written. Consequently, the requirement is :

$$a > 0{,}4\ t_p\ \beta_w\ \sqrt{3}\ \frac{f_{yp}}{f_{up}}\ \frac{\gamma_{M2}}{\gamma_{M0}}$$

The values of the correlation factor β_w are given in Table 4.1. This requirement can be checked before any design calculation.

Criterion to permit a plastic redistribution of internal forces between the "actual" and "design" resistance points

(1) First, the attainment of the design shear resistance should correspond to a ductile mode. The failures by bolts in shear or buckling of the fin plate are therefore excluded. A first criterion can be written :

$$\min(V_{Rd\,1}\ ;\ V_{Rd\,7}) > V_{Rd}$$

where $V_{Rd\,1}$ is the shear resistance of the bolts ;
$V_{Rd\,7}$ is the buckling resistance of the fin plate ;
V_{Rd} is the design shear resistance of the connection.

(2) Secondly, the "actual" resistance point has also to correspond to a ductile mode (so, no bolts in shear or buckling of the fin plate). According to the failure mode obtained by the design rules, different criteria can be written :

- *Failures by bolts in shear or buckling of the fin plate :*

  Excluded by the first criterion (1).

- *All the other failure modes :*

For one vertical bolt row, at least one of this two inequalities has to be satisfied :

$F_{b,hor,Rd} \leq \min (F_{v,Rd} \, ; V_{Rd\,7} \; \beta)$ for the beam web

$F_{b,hor,Rd} \leq \min (F_{v,Rd} \, ; V_{Rd\,7} \; \beta)$ for the fin plate

For two vertical bolt rows, at least one of this three inequalities has to be satisfied :

$$\max \left(\frac{1}{F_{v,Rd}^{\;2}} \left(\alpha^2 + \beta^2 \right) ; \; \frac{1}{V_{Rd\,7}^{\;2}} \right) \leq \left(\frac{\alpha}{F_{b,ver,Rd}} \right)^2 + \left(\frac{\beta}{F_{b,hor,Rd}} \right)^2$$ for the beam web

$$\max \left(\frac{1}{F_{v,Rd}^{\;2}} \left(\alpha^2 + \beta^2 \right) ; \; \frac{1}{V_{Rd\,7}^{\;2}} \right) \leq \left(\frac{\alpha}{F_{b,ver,Rd}} \right)^2 + \left(\frac{\beta}{F_{b,hor,Rd}} \right)^2$$ for the fin plate

$$V_{Rd\,6} \leq \min\left(\frac{2}{3\sqrt{\alpha^2 + \beta^2}} F_{v,Rd} \, ; \; \frac{2}{3} V_{Rd\,7} \right)$$

(3) Lastly, during the redistribution process, the "bolts in shear" failure mode should not be met. To avoid that, simple criteria can be written :

- *Failures by bolts in shear or buckling of the fin plate :*

Excluded by the first criterion (1).

- *Failures by fin plate or beam web in bearing :*

If the two first criteria (1) and (2) are fulfilled, no additional criterion is necessary.

- *All the other failure modes :*

$$V_{Rd\,1} > \min (V_{Rd\,2} \, ; \; V_{Rd\,8})$$

where $V_{Rd\,1}$ is the shear resistance of the bolts ;
$V_{Rd\,2}$ is the bearing resistance of the fin plate ;
$V_{Rd\,8}$ is the bearing resistance of the beam web.

The expressions of all the terms used in the above-mentioned requirements are given in the part "Design sheets for fin plate connections" of the present booklet.

The criteria (1), (2) and (3) can be only checked after the evaluation of the design shear resistance of the joint.

For further explanations about the derivation of these requirements, see [10].

4.1.3 Web cleat connection

4.1.3.1 General

The behaviour of a web cleat connection may be considered as the combination of the header and fin plates connections behaviours. The design rules and requirements for a safe approach may be simply deduced from those established for the two previous connection types.

4.1.3.2 Design requirements

They will be deduced from the previous requirements for header and fin plate connections in a revised version of this document.

5 Geometry of the three connection types

5.1 Symbols

5.1.1 General notations

- For the bolts :

n	Total number of bolts
A	Nominal area of a bolt
A_s	Resistant area of a bolt
d	Nominal diameter of a bolt shank
d_0	Diameter of a bolt hole
$f_{u,b}$	Ultimate strength of a bolt
$f_{y,b}$	Yield strength of a bolt

- For the welds :

a	Throat thickness of the welds
β_w	Correlation factor for the evaluation of the weld resistance

- For the supporting and supported elements :

t	Thickness of the supporting plate (t_{cf} and t_{cw} for respectively a column flange and web, t_{bw} for a beam web)
t_w	Thickness of the supported beam web
$A_{b,v}$	Gross shear area of the supported beam
$A_{b,v,net}$	Net shear area of the supported beam
f_u	Ultimate strength of a steel element (index *bw* for beam web, *cf* and *cw* for respectively column flange and web)
f_y	Yield strength of a steel element (index *bw* for beam web, *cf* and *cw* for respectively column flange and web)

- Safety coefficients :

γ_{M0}	Partial safety factor for steel sections ; it is equal to 1,0
γ_{M2}	Partial safety factor for net section at bolt holes, bolts, welds and plates in bearing ; it is equal to 1,25

- Loading :

V_{Sd}	Shear force applied to the joint

- Resistance :

V_{Rd}	Shear resistance of the joint
$F_{v.Rd}$	Design resistance in shear

5.1.2 Particular notations for header plate connections

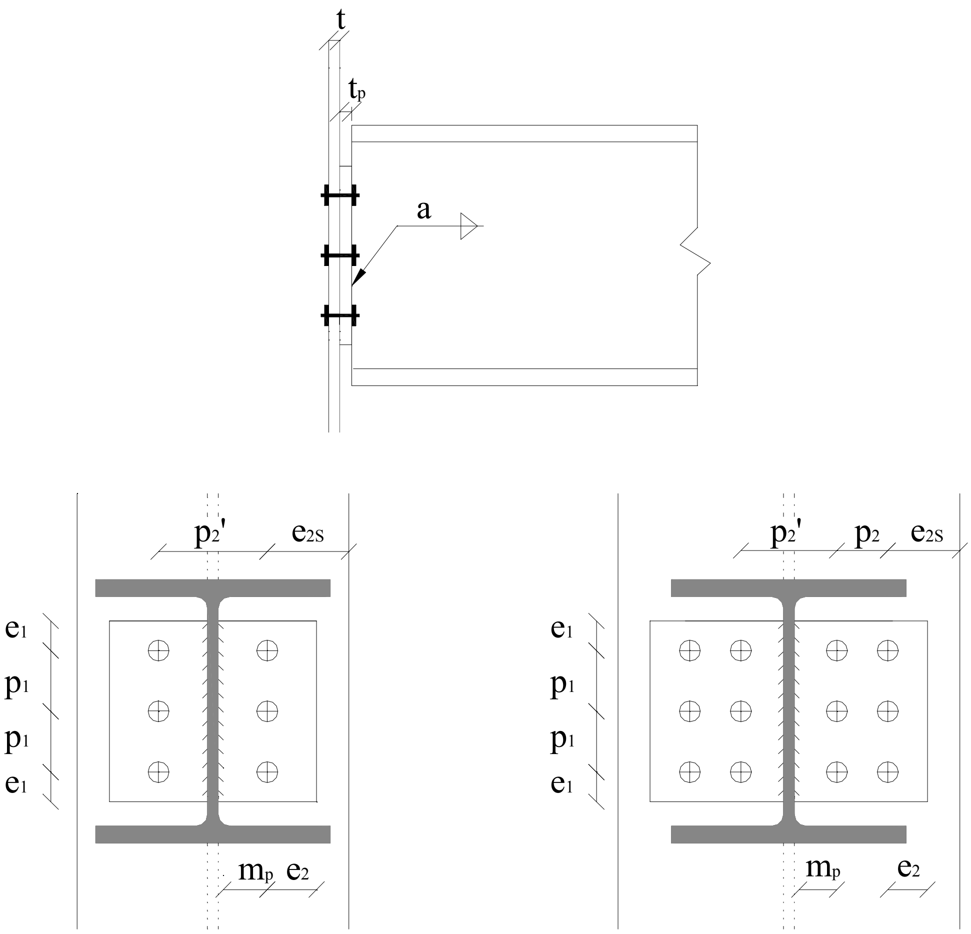


Figure 5.1 : Header plate notations

h_p	Height of the header plate
t_p	Thickness of the header plate
A_v	Gross shear area of the header plate
A_{vnet}	Net shear area of the header plate
f_{yp}	Yield strength of the header plate
n_1	Number of horizontal rows
n_2	Number of vertical rows
e_1	Longitudinal end distance
e_2	Transverse end distance
p_1	Longitudinal bolt pitch
p_2	Transversal bolt pitch
m_p	Distance between the bolt columns and the toe of the weld connecting the header plate to the beam web (definition according to prEN 1993 – Part. 1.8)

5.1.3 Particular notations for fin plate connections

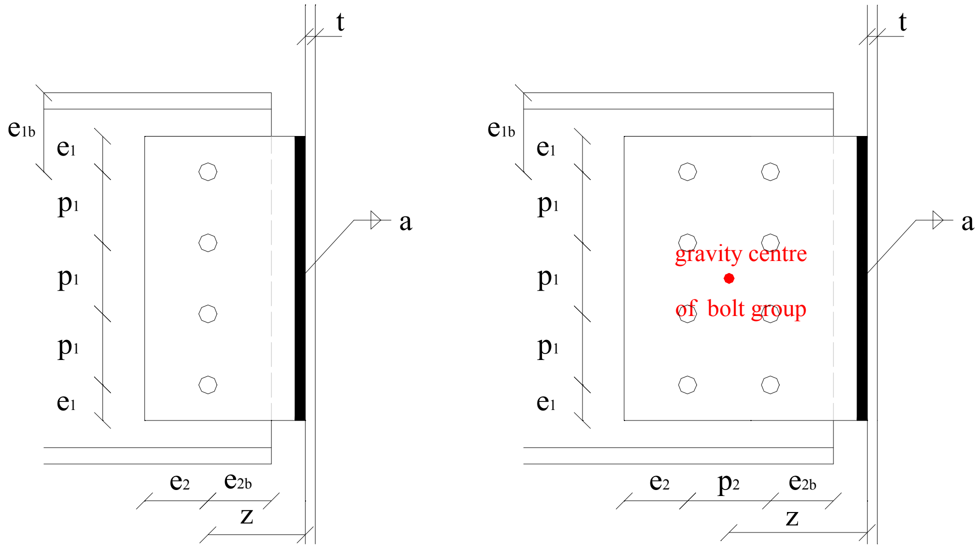


Figure 5.2 : Fin plate notations

h_p	Height of the fin plate
t_p	Thickness of the fin plate
A_v	Gross shear area of the fin plate
A_{vnet}	Net shear area of the fin plate
f_{yp}	Yield strength of the fin plate
n_1	Number of horizontal rows
n_2	Number of vertical rows
e_1	Longitudinal end distance (fin plate)
e_2	Transverse end distance (fin plate)
e_{1b}	Longitudinal end distance (beam web)
e_{2b}	Transverse end distance (beam web)
p_1	Longitudinal bolt pitch
p_2	Transverse bolt pitch
I	Moment of inertia of the bolt group

5.1.4 Particular notations for cleat web connections

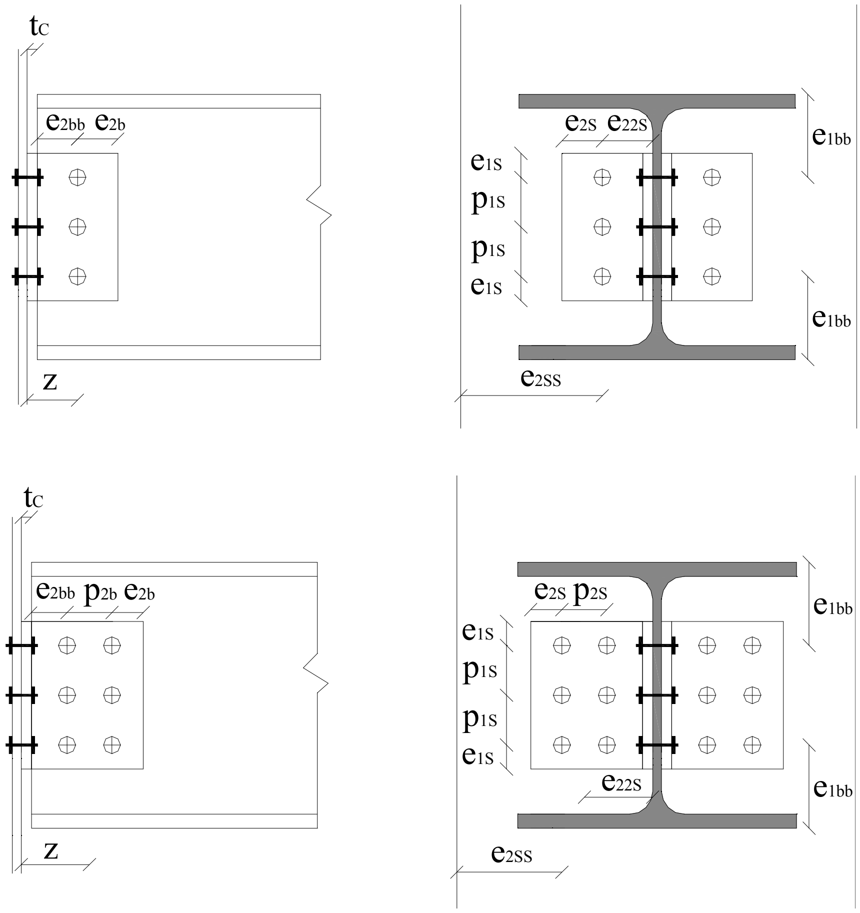


Figure 5.3 : Web cleat notations

h_c	Height of the cleat
t_c	Thickness of the cleat
A_v	Gross shear area of the cleat
A_{vnet}	Net shear area of the cleat

Supported beam side :

d_{sb}	Nominal diameter of a bolt shank
d_{0sb}	Diameter of a bolt hole
n_b	Total number of bolts
n_{1b}	Number of horizontal rows
n_{2b}	Number of vertical rows
e_{1b}	Longitudinal end distance (cleat)
e_{2b}	Transverse end distance (cleat)
p_{1b}	Longitudinal bolt pitch
p_{2b}	Transverse bolt pitch
e_{2bb}	Transverse end distance (beam web)

e_{1bb}	Longitudinal end distance (beam flange)
z	Lever arm
I	Moment of inertia of the bolt group

Supporting element side :

d_s	Nominal diameter of a bolt shank
d_{0s}	Diameter of a bolt hole
n_s	Total number of bolts
n_{1s}	Number of horizontal rows
n_{2s}	Number of vertical rows
e_{1s}	Longitudinal end distance (cleat)
e_{2s}	Transverse end distance (cleat)
p_{1s}	Longitudinal bolt pitch
p_{2s}	Transverse bolt pitch
e_{2ss}	Transverse end distance (supporting element)
e_{22s}	Longitudinal distance between the inner bolt column and the beam web

5.2 Geometrical requirements

The design rules may only be applied if the positioning of holes for bolts respects the minimum spacing, end and edge distances given in the following table (Eurocode 3 requirements).

Distances and spacings, see figure 5.4	Minimum	Maximum 1) 2) 3)		
		Structures made of steels according to EN 10025 except steels acc. to EN 10025-5		Structures made of steels according to EN 10025-5
		Steel exposed to the weather or other corrosive influences	Steel not exposed to the weather or other corrosive influences	Steel used unprotected
End distance e_1	1,2 d_0	$4t + 40$ mm		The larger of $8t$ or 125 mm
End distance e_2	1,2 d_0	$4t + 40$ mm		
Spacing p_1	2,2 d_0	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm
Spacing p_2	2,4 d_0	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm

1) Maximum values for spacings, edge and end distances are unlimited, except in the following cases :
   - for compression members in order to avoid local buckling and to prevent corrosion in exposed members and ;
   - for exposed tension members to prevent corrosion.
2) The local buckling resistance of the plate in compression between the fasteners should be calculated according to EN 1993-1-1 as column like buckling by using 0,6 p_i as buckling length. Local buckling between the fasteners need to be checked if p_1/t is smaller then 9 ε. The edge distance should not exceed the maximum to satisfy local buckling requirements for an outstand element in the compression members, see EN 1993-1-1. The end distance is not affected by this requirement.
3) t is the thickness of the thinner outer connected part.

Table 5.1 : Minimum spacing, end and edge distances

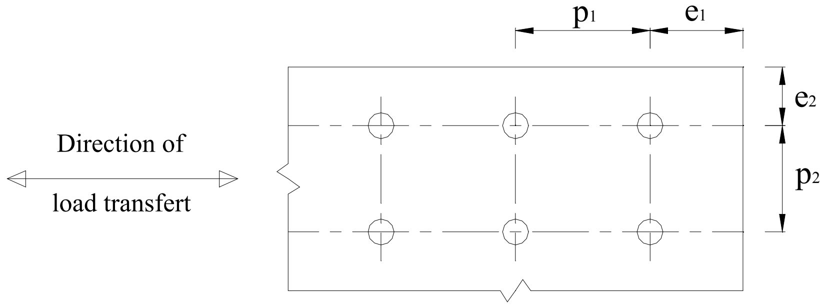


Figure 5.4 : Symbols for end and edge distances and spacing of fasteners

6 Design sheets

6.1 General

The forces applied to joints at the ultimate limit state shall be determined according to the principles in EN 1993-1-1. Linear-elastic analysis is used in the design of the joint.

The resistance of the joint is determined on the basis of the resistances of the individual fasteners, welds and other components of the joint.

6.2 Design sheet for connections with a header plate

6.2.1 Requirements to ensure the safety of the approach

To apply the design rules established in the section 6.2.2 of this document, all the following inequalities have to be satisfied.

(1) $h_p \leq d_b$

(2) $\frac{t_p}{h_e} > \phi_{required}$

(3) If the supporting element is a beam or column web :

$$\frac{d}{t_p} \geq 2,8 \sqrt{\frac{f_{yp}}{f_{ub}}} \quad \text{OR} \quad \text{criterion to establish}$$

If the supporting element is a column flange :

$$\frac{d}{t_p} \geq 2,8 \sqrt{\frac{f_{yp}}{f_{ub}}} \quad \text{OR} \quad \frac{d}{t_{cf}} \geq 2,8 \sqrt{\frac{f_{ycf}}{f_{ub}}}$$

(4) $a > 0,4\, t_{bw}\, \beta_w\, \sqrt{3}\, \frac{f_{ybw}}{f_{ubw}} \frac{\gamma_{M2}}{\gamma_{M0}}$

(β_w is given in Table 4.1)

Bolts in shear :

(1) The factor 0,8 takes the design shear resistance reduction due to the presence of actual tension forces into account. The explanation of this reduction is given in the part "Practical ways to satisfy the ductility and rotation requirements" (4.2.1.2).

(2) The shear force applied to the joint, is assumed to be equally distributed between the different bolts.

Header plate in bearing :

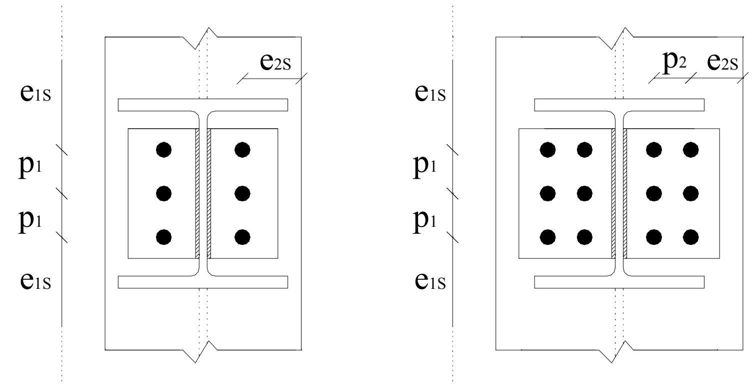
(1) The shear force applied to the joint, is assumed to be equally distributed between the different bolts.

6.2.2 Resistance to shear forces

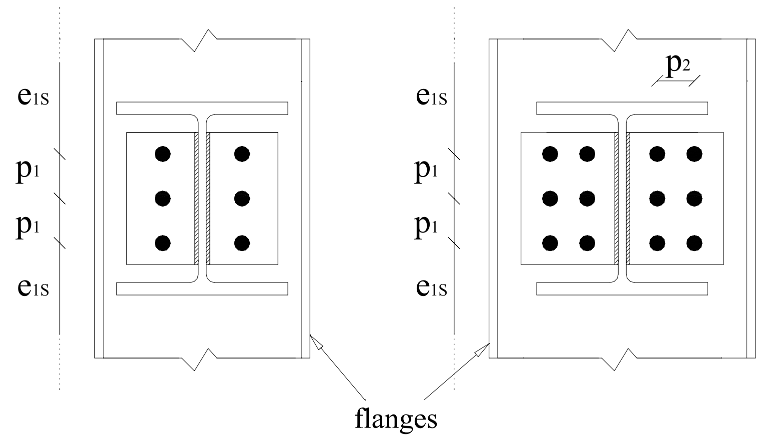
FAILURE MODE	VERIFICATION
Bolts in shear	$V_{Rd\,1} = 0{,}8\ n\ F_{v,Rd}$ $$F_{v,Rd} = \frac{\alpha_v\ f_{ub}\ A}{\gamma_{M2}}$$ • where the shear plane passes through the threaded portion of the bolt : $A = A_s$ (tensile stress area of the bolt) - for 4.6, 5.6 and 8.8 bolt grades : $\alpha_v = 0{,}6$ - for 4.8, 5.8, 6.8 and 10.9 bolt grades : $\alpha_v = 0{,}5$ • where the shear plane passes through the unthreaded portion of the bolt : A (gross cross area of the bolt) $\alpha_v = 0{,}6$ (according Table 3.4 in prEN 1993 Part. 1.8)
Header plate in bearing	$V_{Rd\,2} = n\ F_{b,Rd}$ $$F_{b,Rd} = \frac{k_1\ \alpha_b\ f_{up}\ d\ t_p}{\gamma_{M2}}$$ where $\alpha_b = \min\left(\frac{e_1}{3\,d_0}\ ;\ \frac{p_1}{3\,d_0} - \frac{1}{4}\ ;\ \frac{f_{ub}}{f_{up}}\ \text{ou}\ 1{,}0\right)$ $k_1 = \min\left(2{,}8\frac{e_2}{d_0} - 1{,}7\ ;\ 1{,}4\frac{p_2}{d_0} - 1{,}7\ ;\ 2{,}5\right)$ (see Table 3.4 in prEN 1993 Part. 1.8)

Supporting member in bearing :

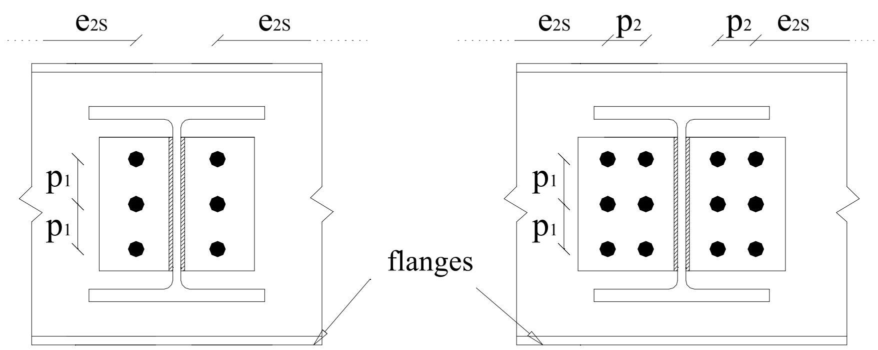
(1) When the supporting element is a column flange, $e_{1S} / 3d_0$ is greater than one.



(2) When the supporting element is a column web, $e_{1S} / 3d_0$ and $(2,8\ e_{2S} /d_0 - 1,7)$ are respectively greater than one and 2,5 (because of the presence of the column flanges).

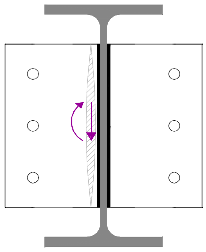


(3) When the supporting element is a beam web, $(2,8\ e_{2S} /d_0 - 1,7)$ and $e_{1S} / 3d_0$ are respectively greater than 2,5 and one (because of the presence of the beam flanges).



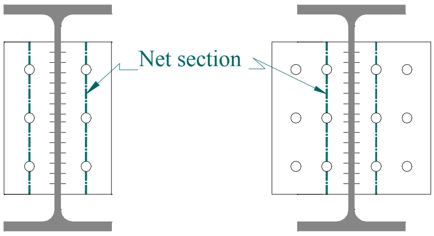
Supporting member in bearing	$V_{Rd\,3} = n\,F_{b,Rd}$ $F_{b,Rd} = \dfrac{k_1\ \alpha_b\ f_u\ d\ t}{\gamma_{M2}}$ • when the supporting element is a column flange : $t = t_{cf}$ $f_u = f_{ucf}$ $\alpha_b = \min\left(\dfrac{p_1}{3\,d_0} - \dfrac{1}{4}\ ;\ \dfrac{f_{ub}}{f_u}\ \text{ou}\ 1{,}0\right)$ $k_1 = \min\left(1{,}4\dfrac{p_2}{d_0} - 1{,}7\ ;\ 2{,}8\dfrac{e_{2s}}{d_0} - 1{,}7\ ;\ 2{,}5\right)$ • when the supporting element is a column web : $t = t_{cw}$ $f_u = f_{ucw}$ $\alpha_b = \min\left(\dfrac{p_1}{3\,d_0} - \dfrac{1}{4}\ ;\ \dfrac{f_{ub}}{f_u}\ \text{ou}\ 1{,}0\right)$ $k_1 = \min\left(1{,}4\dfrac{p_2}{d_0} - 1{,}7\ ;\ 2{,}5\right)$ • when the supporting element is a beam web : $t = t_{bw}$ $f_u = f_{ubw}$ $\alpha_b = \min\left(\dfrac{p_1}{3\,d_0} - \dfrac{1}{4}\ ;\ \dfrac{f_{ub}}{f_u}\ \text{ou}\ 1{,}0\right)$ $k_1 = \min\left(1{,}4\dfrac{p_2}{d_0} - 1{,}7\ ;\ 2{,}5\right)$ The formula as it is written here applies to major axis beam-to-column joints (connection to a column flange), to single-sided minor axis joints and to single-sided beam-to-beam joint configurations. In the other cases, the bearing forces result from both the left and right connected members, with the added problem that the number of connecting bolts may differ for the left and right connections. The calculation procedure may cover such cases without any particular difficulty. It could just bring some more complexity in the final presentation of the design sheet.

Header plate in shear : Gross section



(1) The factor 1,27 takes into account the reduction of the shear resistance, due to the presence of a bending moment. For further explanations, see [10].

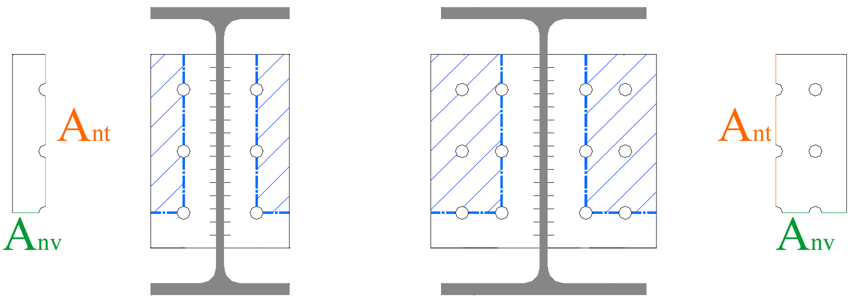
Header plate in shear : net section



Header plate in shear : shear block

(1) According as the fin plate is long or not, the effect of the eccentricity of the applied shear force relative to the bolt group centre has to take into account or not in the evaluation of the shear block resistance.

(2)



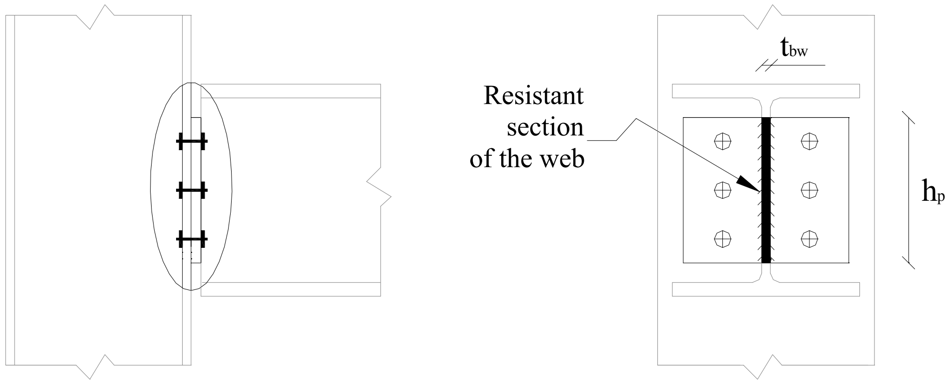
Header plate in shear : Gross section	$V_{Rd\,4} = \frac{2\,h_p\,t_p}{1{,}27}\,\frac{f_{yp}}{\sqrt{3}\,\gamma_{M_0}}$ (2 sections)
Header plate in shear : Net section	$V_{Rd\,5} = 2A_{v.net}\,\frac{f_{up}}{\sqrt{3}\,\gamma_{M2}}$ (2 sections) with $A_{v,net} = t_p\,(\,h_p - n_1\,d_0)$
Header plate in shear : Shear block	$V_{Rd\,6} = 2\,F_{eff,Rd}$ (2 sections) • if $h_p < 1{,}36\,p_{22}$ and $n_1 > 1$: $F_{eff,Rd} = F_{eff,2,Rd} = 0{,}5\,\frac{f_{up}A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}}\,f_{yp}\,\frac{A_{nv}}{\gamma_{M0}}$ • else : $F_{eff,Rd} = F_{eff,1,Rd} = \frac{f_{up}A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}}\,f_{yp}\,\frac{A_{nv}}{\gamma_{M0}}$ with $p_{22} = p_2'$ for $n_2 = 2$ $= p_2' + p_2$ for $n_2 = 4$ A_{nt} = net area subjected to tension - for one bolt vertical row ($n_2 = 2$) : $A_{nt} = t_p\,(\,e_2 - \frac{d_0}{2}\,)$ - for two bolt vertical rows ($n_2 = 4$) : $A_{nt} = t_p\,(\,p_2 + e_2 - 3\,\frac{d_0}{2}\,)$ A_{nv} = net area subjected to shear $= t_p\,(\,h_p - e_1 - (n_1 - 0{,}5)\,d_0\,)$ (see clause 3.10.2 in prEN 1993 Part. 1.8)

Header plate in bending :

(1) When the header plate is long, the effects of the bending moment in the central section become predominant and reduce its shear resistance ($V_{Rd\ 7} < V_{Rd\ 4}$). The evaluation of the bending effects is thus necessary.

Beam web in shear :

(1) The area of the beam web which transfers the applied shear force to the joint is equal to t_{bw} multiplied by h_p.



Header plate in bending	• if $h_p \geq 1{,}36\ p_{22}$: $V_{Rd\,7} = \infty$ • else : $V_{Rd\,7} = \dfrac{2\ W_{el}}{\dfrac{(p_{22} - t_w)}{2}} \dfrac{f_{yp}}{\gamma_{M_0}}$ with $p_{22} = p_2'$ for $n_2 = 2$ $= p_2' + p_2$ for $n_2 = 4$ $W_{el} = \dfrac{t_p\ h_p^2}{6}$
Beam web in shear	$V_{Rd\,8} = t_{bw}\ h_p \dfrac{f_{ybw}}{\gamma_{M_0}\ \sqrt{3}}$ (clause 5.4.6 in Eurocode 3)
Shear resistance of the joint	$V_{Rd} = \min_{i=1}^{8} V_{Rdi}$

NOTE :

The design shear resistance of the joint can only be considered if all the design requirements (section 6.2.1) are fulfilled.

NOTES :

(1) The evaluation of the tying resistance of the joint is made at the Ultimate Limit State.

(2) The determination of the applied tying force needs some further researches. It will be specified in a revised version of this document.

6.2.3 Resistance to tying forces

FAILURE MODE	VERIFICATION
Bolts in tension	$N_{u\,1} = n\, B_{t,u}$ with : $B_{t,u} = f_{ub}\; A_s$
Header plate in bending	$N_{u\,2} = \min\,(F_{hp,u,1}\;;F_{hp,u,2})$ $F_{hp,u,1} = \dfrac{(8\,n_p - 2\,e_w)\,l_{eff.p.t,1}\;m_{u.p}}{2\,m_p\;n_p - e_w\;(m_p + n_p)}$ $F_{hp,u,2} = \dfrac{2\,l_{eff.p.t,2}\;m_{u.p} + n\;B_{t.u}\;n_p}{m_p + n_p}$ where $n_p = \min\,(e_2\;;1{,}25\;m_p)$ $m_{u.p} = \dfrac{t_p^2\;f_{up}}{4}$ $l_{eff.p1} = l_{eff.p2} = h_p$ (usually safe value ; see EC3 – table with effective lengths for end plates, case “Bolt-row outside tension flange of beam” – for more precise values ; the effective lengths given in the table have however to be multiplied by a factor 2 before being introduced in the two above-written expressions)
Supporting member in bending	$N_{u\,3} =$ See prEN 1993 – Part. 1.8 for column flanges. See published reference documents for other supporting members (for instance [12])
Beam web in tension	$N_{u\,4} = t_w\;h_p\;f_{ubw}$
Welds	The full-strength character of the welds is ensured through recommendations for weld design given in the design sheet for shear resistance.
Tying resistance of the joint	$N_u = \min_{i=1}^{4} N_{u\,i}$

6.3 Design sheet for connections with fin plate

6.3.1 Requirements to ensure sufficient rotation capacity

The two following inequalities has to be fulfilled.

(1) $h_p \leq d_b$

(2) $\phi_{available} > \phi_{required}$

where :

- if $z > \sqrt{(z-g_h)^2 + \left(\frac{h_p}{2} + h_e\right)^2}$:

  $\phi_{available} = "\infty"$

- else :

$$\phi_{available} = \arcsin\left(\frac{z}{\sqrt{(z-g_h)^2 + \left(\frac{h_p}{2} + h_e\right)^2}}\right) - \text{arctg}\left(\frac{z-g_h}{\frac{h_p}{2} + h_e}\right)$$

6.3.2 Requirements to avoid premature weld failure

The following inequality has to be fulfilled.

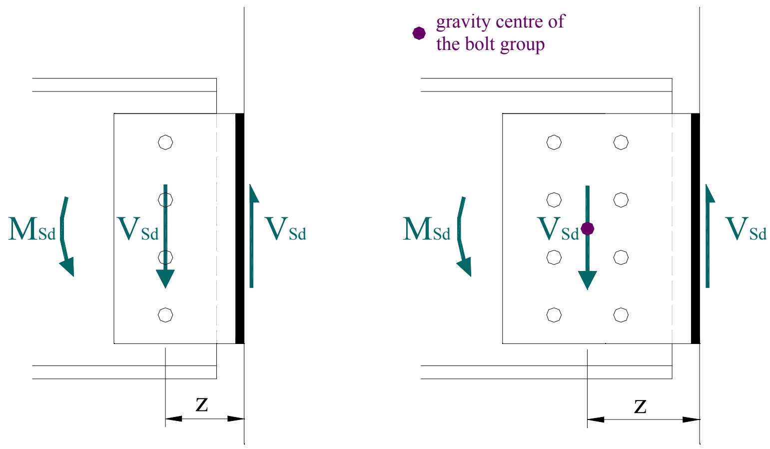
$$a > 0{,}4\ t_p\ \beta_w\ \sqrt{3}\ \frac{f_{yp}}{f_{up}}\ \frac{\gamma_{M2}}{\gamma_{M0}}$$

(β_w is given in Table 4.1)

Bolts in shear :

(1) The shear force acting on the bolts results from :

- the applied shear force V_{Sd} which is assumed to be equally distributed between the different bolts ;
- and the bending moment M_{Sd} which is assumed to be equal to the shear force multiplied by the distance z between the bolt group centre and the external face of the supporting element.



6.3.3 Resistance to shear forces

<table>
<tr><th>FAILURE MODE</th><th>VERIFICATION</th></tr>
<tr><td rowspan="2">Bolts in shear</td><td>for n₂ = 1 :

$$V_{Rd\,1} = \frac{n \quad F_{v,Rd}}{\sqrt{1 + \left(\frac{6\,z}{(n+1)\,p_1}\right)^2}}$$
for n₂ = 2 :

$$V_{Rd\,1} = \frac{F_{v,Rd}}{\sqrt{\left(\frac{z\,p_2}{2\,I} + \frac{1}{n}\right)^2 + \left(\frac{z\,p_1}{2\,I}\,(n_1 - 1)\right)^2}}$$
with :
$$I = \frac{n_1}{2}\;p_2^2 + \frac{1}{6}\;n_1\,(n_1^2 - 1)\;p_1^2$$</td></tr>
<tr><td>$$F_{v,Rd} = \frac{\alpha_v \quad f_{ub} \quad A}{\gamma_{M2}}$$
• where the shear plane passes through the threaded portion of the bolt :

$A = A_s$ (tensile stress area of the bolt)

- for 4.6, 5.6 and 8.8 bolt grades :
$\alpha_v = 0{,}6$

- for 4.8, 5.8, 6.8 and 10.9 bolt grades :
$\alpha_v = 0{,}5$

• where the shear plane passes through the unthreaded portion of the bolt :

A (gross cross area of the bolt)
$\alpha_v = 0{,}6$

according Table 3.4 in prEN 1993 Part. 1.8</td></tr>
</table>

Fin plate in bearing :

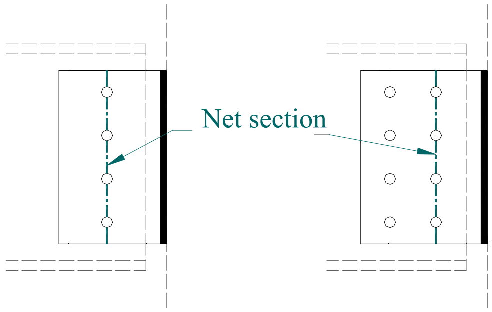
(1) The bearing force acting on the fin plate results from the applied shear force V_{Sd} and the bending moment M_{Sd}.

(2) The resultant bearing force which is acting on the fin plate can be split into two forces, a horizontal one and an other vertical one.

Fin plate in shear : Gross section

(1) The factor 1,27 takes into account the reduction of the shear resistance, due to the presence of a bending moment. For further explanations, see [10].

Fin plate in shear : Net section

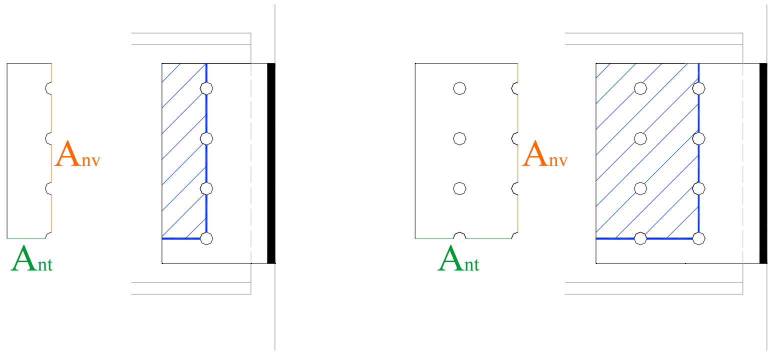


Fin plate in bearing	$V_{Rd\,2} = \dfrac{1}{\sqrt{\left(\dfrac{\frac{1}{n}+\alpha}{F_{b,ver,Rd}}\right)^2 + \left(\dfrac{\beta}{F_{b,hor,Rd}}\right)^2}}$ ***for $n_2 = 1$:*** - $\alpha = 0$; - $\beta = \dfrac{6\,z}{p_1\;n\,(n+1)}$. ***for $n_2 = 2$:*** - $\alpha = \dfrac{z}{I}\,\dfrac{p_2}{2}$; - $\beta = \dfrac{z}{I}\;\dfrac{n_1-1}{2}\;p_1$. with $I = \dfrac{n_1}{2}\,p_2^2 + \dfrac{1}{6}\,n_1\,(n_1^2-1)\,p_1^2$	
	$F_{b,ver,Rd} = \dfrac{k_1\;\alpha_b\;f_{up}\;d\;t_p}{\gamma_{M2}}$ where $\alpha_b = \min\left(\dfrac{e_1}{3\,d_0}\;;\;\dfrac{p_1}{3d_0}-\dfrac{1}{4}\;;\;\dfrac{f_{ub}}{f_{up}}\text{ ou }1{,}0\right)$ $k_1 = \min\left(2{,}8\,\dfrac{e_2}{d_0}-1{,}7\;;\;1{,}4\,\dfrac{p_2}{d_0}-1{,}7\;;\;2{,}5\right)$	$F_{b,hor,Rd} = \dfrac{k_1\;\alpha_b\;f_{up}\;d\;t_p}{\gamma_{M2}}$ where $\alpha_b = \min\left(\dfrac{e_2}{3d_0}\;;\;\dfrac{p_2}{3d_0}-\dfrac{1}{4}\;;\;\dfrac{f_{ub}}{f_{up}}\text{ ou }1{,}0\right)$ $k_1 = \min\left(2{,}8\,\dfrac{e_1}{d_0}-1{,}7\;;\;1{,}4\,\dfrac{p_1}{d_0}-1{,}7\;;\;2{,}5\right)$
	(see Table 3.4 in prEN 1993 Part. 1.8)	
Fin plate in shear : Gross section	$V_{Rd\,3} = \dfrac{h_p t_p}{1{,}27}\;\dfrac{f_{yp}}{\sqrt{3}\;\gamma_{M0}}$	
Fin plate in shear : Net section	$V_{Rd\,4} = A_{v,net}\;\dfrac{f_{up}}{\sqrt{3}\;\gamma_{M2}}$ with $A_{v,net} = t_p\,(\,h_p - n_1\,d_0)$	

Fin plate in shear : Shear block

(1) The applied shear force is acting with an eccentricity at the bolt group centre.

(2)



Fin plate in bending :

(1) When the fin plate is long, the effects of the bending moment become predominant and reduce its shear resistance ($V_{Rd\,6} < V_{Rd\,3}$). The evaluation of the bending effects is thus necessary.

Buckling of the fin plate :

(1) The buckling is due to the compression stresses which develop in the lower part of the fin plate under the action of the bending moment.

<table>
<tr>
<td>Fin plate in shear :
Shear block</td>
<td>$V_{Rd\,5} = F_{eff,2,Rd}$

$$F_{eff,2,Rd} = \frac{0{,}5\ f_{up}\ A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}}\ f_{yp}\ \frac{A_{nv}}{\gamma_{M0}}$$
with A_{nt} = net area subjected to tension
- for one bolt vertical row ($n_2 = 1$) :
$A_{nt} = t_p\ (e_2 - \frac{d_0}{2})$
- for two bolt vertical rows ($n_2 = 2$) :
$A_{nt} = t_p\ (p_2 + e_2 - 3\,\frac{d_0}{2})$

A_{vt} = net area subjected to shear
$= t_p\ (h_p - e_1 - (n_1 - 0{,}5)\ d_0)$

(see clause 3.10.2 in prEN 1993 Part. 1.8)</td>
</tr>
<tr>
<td>Fin plate in bending</td>
<td>• if $h_p \geq 2{,}73\ z$:
$V_{Rd\,6} = \infty$

• else :
$$V_{Rd\,6} = \frac{W_{el}}{z}\ \frac{f_{yp}}{\gamma_{M0}}$$
with $W_{el} = \frac{t_p\ h_p^2}{6}$</td>
</tr>
<tr>
<td>Buckling of the fin plate</td>
<td>$$V_{Rd\,7} = \frac{W_{el}}{z}\ \frac{\sigma}{\gamma_{M0}}$$
where $W_{el} = \frac{t_p\ h_p^2}{6}$
$$\sigma = 81 \left(\frac{t_p}{z} \right)^2 235$$</td>
</tr>
</table>

Beam web in bearing :

(1) The bearing force acting on the beam web results from the applied shear force V_{Sd} and the bending moment M_{Sd}.

(2) The resultant bearing force which is acting on the beam web can be split into two forces, a horizontal one and an other vertical one.

Beam web in shear : Gross section

(1) For the standard hot-rolled I and H sections, the shear area is equal to :

$$A_{bv} = A_b - 2\, b_b\, t_{bf} + (t_{bw} + 2\, r_b)\, t_{bf}$$

where
A_b area of section
b_b width of section
t_{bf} flange thickness
t_{bw} web thickness
r_b radius of root fillet

<table>
<tr>
<td rowspan="2">Beam web in bearing</td>
<td colspan="2">

$$V_{Rd\,8} = \frac{1}{\sqrt{\left(\frac{\frac{1}{n}+\alpha}{F_{b,ver,Rd}}\right)^2 + \left(\frac{\beta}{F_{b,hor,Rd}}\right)^2}}$$

for $n_2 = 1$:

- $\alpha = 0$;
- $\beta = \frac{6\,z}{p_1\;n\,(n+1)}$.

for $n_2 = 2$:

- $\alpha = \frac{z}{I}\;\frac{p_2}{2}$;
- $\beta = \frac{z}{I}\;\frac{n_1 - 1}{2}\;p_1$.

with $I = \frac{n_1}{2}\;p_2^2 + \frac{1}{6}\;n_1\,(n_1^2 - 1)\;p_1^2$

</td>
</tr>
<tr>
<td>

$$F_{b,ver,Rd} = \frac{k_1\;\alpha_b\;f_{ubw}\;d\;t_{bw}}{\gamma_{M2}}$$

where

$\alpha_b = \min\left(\frac{p_1}{3d_0} - \frac{1}{4}\;;\;\frac{f_{ub}}{f_{ubw}}\;\text{ou}\;1{,}0\right)$

$k_1 = \min\left(2{,}8\,\frac{e_{2b}}{d_0} - 1{,}7\;;\;1{,}4\,\frac{p_2}{d_0} - 1{,}7\;;\;2{,}5\right)$

</td>
<td>

$$F_{b,hor,Rd} = \frac{k_1\;\alpha_b\;f_{ubw}\;d\;t_{bw}}{\gamma_{M2}}$$

where

$\alpha_b = \min\left(\frac{e_{2b}}{3d_0}\;;\;\frac{p_2}{3d_0} - \frac{1}{4}\;;\;\frac{f_{ub}}{f_{ubw}}\;\text{ou}\;1{,}0\right)$

$k_1 = \min\left(1{,}4\,\frac{p_1}{d_0} - 1{,}7\;;\;2{,}5\right)$

</td>
</tr>
<tr>
<td>Beam web in shear :
Gross section</td>
<td colspan="2">

$$V_{Rd\,9} = A_{b,v}\;\frac{f_{ybw}}{\sqrt{3}\;\gamma_{M0}}$$

(clause 5.4.6 in Eurocode 3)

</td>
</tr>
<tr>
<td>Beam web in shear :
Net section</td>
<td colspan="2">

$$V_{Rd\,10} = A_{b,v,net}\;\frac{f_{ubw}}{\sqrt{3}\;\gamma_{M2}}$$

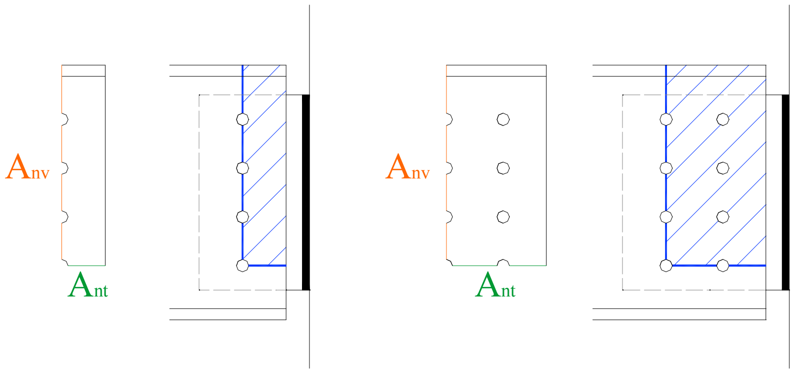
with $A_{b,v,net} = A_{b,v} - n_1\,d_0\,t_{bw}$

</td>
</tr>
</table>

Beam web in shear : Shear block

(1) The applied shear force is acting with an eccentricity at the bolt group centre.

(2)



Beam web in shear : Shear block	$V_{Rd\,11} = F_{eff,2,Rd}$ $$F_{eff,2,Rd} = \frac{0{,}5\ f_{ubw}\ A_{nt}}{\gamma_{M2}} + \frac{1}{\sqrt{3}}\ f_{ybw}\ \frac{A_{nv}}{\gamma_{M0}}$$ with A_{nt} = net area subjected to tension - for one bolt vertical row ($n_2 = 1$) : $A_{nt} = t_{bw}\ (e_{2b} - \frac{d_0}{2})$ - for two bolt vertical rows ($n_2 = 2$) : $A_{nt} = t_{bw}\ (p_2 + e_{2b} - 3\frac{d_0}{2})$ A_{nv} = net area subjected to shear $= t_{bw}\ (e_{1b} + (n_1 - 1)\ p_1 - (n_1 - 0{,}5)\ d_0)$ (see clause 3.10.2 in prEN 1993 Part. 1.8)
Shear resistance of the joint	$$V_{Rd} = \min_{i=1}^{11} V_{Rdi}$$

NOTE :

The design shear resistance of the joint can only be considered if all the design requirements (sections 6.3.1, 6.3.2 and 6.3.4) are fulfilled.

6.3.4 Requirements to permit a plastic redistribution of internal forces

All the following inequalities have to be satisfied.

(1) $V_{Rd} < \min(V_{Rd\,1} ; V_{Rd\,7})$

(2) **For $n_2 = 1$** :

$F_{b,hor,Rd} \leq \min (F_{v,Rd} ; V_{Rd\,7}\ \beta)$ for the beam web

OR

$F_{b,hor,Rd} \leq \min (F_{v,Rd} ; V_{Rd\,7}\ \beta)$ for the fin plate

For $n_2 = 2$:

$$\max \left(\frac{1}{F_{v,Rd}^{\ 2}} \left(\alpha^2 + \beta^2 \right) ; \ \frac{1}{V_{Rd\,7}^{\ 2}} \right) \leq \left(\frac{\alpha}{F_{b,ver,Rd}} \right)^2 + \left(\frac{\beta}{F_{b,hor,Rd}} \right)^2$$ for the beam web

OR

$$\max \left(\frac{1}{F_{v,Rd}^{\ 2}} \left(\alpha^2 + \beta^2 \right) ; \ \frac{1}{V_{Rd\,7}^{\ 2}} \right) \leq \left(\frac{\alpha}{F_{b,ver,Rd}} \right)^2 + \left(\frac{\beta}{F_{b,hor,Rd}} \right)^2$$ for the fin plate

OR

$$V_{Rd\,6} \leq \min\left(\frac{2}{3\sqrt{\alpha^2 + \beta^2}} F_{v,Rd} ; \ \frac{2}{3} V_{Rd\,7} \right)$$

(3) Moreover, if $\mathbf{V_{Rd} = V_{Rd\,3}, V_{Rd\,4}, V_{Rd\,5}, V_{Rd\,6}, V_{Rd\,9}, V_{Rd\,10}}$ **or** $\mathbf{V_{Rd\,11}}$, this following inequality has to be checked :

$V_{Rd\,1} > \min (V_{Rd\,2} ; V_{Rd\,8})$

NOTES :

(1) The evaluation of the tying resistance of the joint is made at the Ultimate Limit State.

(2) The determination of the applied tying force needs some further researches. It will be specified in a revised version of this document.

6.3.5 Resistance to tying forces

FAILURE MODE	VERIFICATION
Bolts in shear	$N_{u\,1} = n\,F_{v,u}$ with : $F_{v,u} = \alpha_v \; f_{ub} \; A$ • where the shear plane passes through the threaded portion of the bolt : $A = A_s$ (tensile stress area of the bolt) - for 4.6, 5.6 and 8.8 bolt grades : $\alpha_v = 0{,}6$ - for 4.8, 5.8, 6.8 and 10.9 bolt grades : $\alpha_v = 0{,}5$ • where the shear plane passes through the unthreaded portion of the bolt : A (gross cross area of the bolt) $\alpha_v = 0{,}6$
Fin plate in bearing	$N_{u\,2} = n\,F_{b,u,\,hor}$ with : $F_{b,u,hor} = k_1 \; \alpha_b \; f_{up} \; d \; t_p$ where $\alpha_b = \min\left(\frac{e_2}{3d_0} \; ; \; \frac{p_2}{3d_0} - \frac{1}{4} \; ; \; \frac{f_{ub}}{f_{up}} \; \text{ou } 1{,}0 \right)$ $k_1 = \min\left(2{,}8\frac{e_1}{d_0} - 1{,}7 \; ; 1{,}4\frac{p_1}{d_0} - 1{,}7 \; ; 2{,}5 \right)$
Fin plate in tension : Gross section	$N_{u\,3} = t_p \; h_p \; f_{up}$
Fin plate in tension : Net section	$N_{u\,4} = 0{,}9 \; A_{net,p} \; f_{up}$ with : $A_{net,p} = t_p \, h_p - d_0 \, n_1 \, t_p$

Beam web in bearing	$N_{u\,5} = n\, F_{b,u,hor}$ with : $F_{b,u,hor} = k_1\ \alpha_b\ f_{ubw}\ d\ t_{bw}$ where $\alpha_b = \min\left(\frac{e_{2b}}{3d_0}\ ;\ \frac{p_2}{3d_0} - \frac{1}{4}\ ;\ \frac{f_{ub}}{f_{ubw}}\ \text{ou } 1{,}0 \right)$ $k_1 = \min\left(1{,}4\frac{p_1}{d_0} - 1{,}7\ ;\ 2{,}5 \right)$
Beam web in tension : Gross section	$N_{u\,6} = t_{bw}\, h_{bw}\ f_{ubw}$
Beam web in tension : Net section	$N_{u\,7} = 0{,}9\ A_{net,bw}\ f_{ubw}$ with : $A_{net,bw} = t_{bw}\, h_{bw} - d_0\, n_1\, t_{bw}$
Supporting member in bending	$N_{u\,8} =$ See prEN 1993 – Part. 1.8 for column flanges. See published reference documents for other supporting members (for instance [12])
Welds	The full-strength character of the welds is ensured through recommendations for weld design given in the design sheet for shear resistance.
Tying resistance of the joint	$N_u = \min_{i=1}^{8} N_{u\,i}$

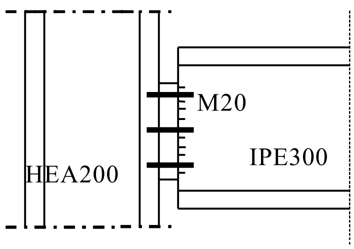
6.4 Design sheet for connections with web cleats

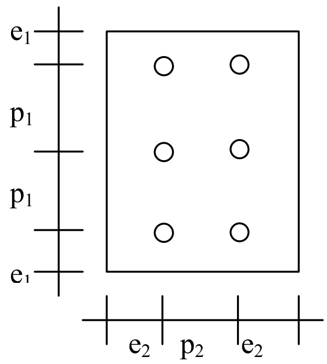
To complete in a revised version of this document.

7 Worked examples

7.1 Header plate connection

7.1.1 Geometrical and mechanical data





Main joint data

Configuration	Beam to column flange
Column	HEA 200 S 235
Beam	IPE 300 S 235
Type of connection	Header plate connection
Header plate	230 x 200 x 10, S 235

Detailed characteristics

Column HEA 200, S235

Depth	h	=	190.00	mm
Thickness of the web	t_{cw}	=	6.50	mm
Width	b_c	=	200.00	mm
Thickness of the flange	t_{cf}	=	10.00	mm
Root radius	r	=	18.00	mm
Area	A	=	53.83	cm^2
Inertia	I	=	3692.16	cm^4
Yield strength	f_{yc}	=	235.00	N/mm^2
Ultimate strength	f_{uc}	=	360.00	N/mm^2

Beam IPE 300, S235

Depth	h	=	300.00	mm
Thickness of the web	t_{bw}	=	7.10	mm
Width	b_b	=	150.00	mm
Thickness of the flange	t_{bf}	=	10.70	mm

Root radius	r	=	15.00	mm
Area	A	=	53.81	cm^2
Inertia	I	=	8356.11	cm^4
Yield strength	f_{yb}	=	235.00	N/mm²
Ultimate strength	f_{ub}	=	360.00	N/mm²

Header plate 230 x 200 x 10, S 235

Vertical gap	g_v	=	35.00	mm
Depth	h_p	=	230.00	mm
Width	b_p	=	200.00	mm
Thickness	t_p	=	10.00	mm

Direction of load transfer (1)

Number of bolts rows	n_1	=	3	
Edge to first bolt row distance	e_{11}	=	45.00	mm
Pitch between bolt row 1 and 2	$p_{1[1]}$	=	70.00	mm
Pitch between bolt row 2 and 3	$p_{1[2]}$	=	70.00	mm
last bolt row to edge distance	e_{1n}	=	45.00	mm

Direction perpendicular to Load transfer (2)

Number of bolts rows	n_2	=	2	
Edge to first bolt row distance	e_{21}	=	50.00	mm
Pitch between bolt row 1 and 2	p_2'	=	100.00	mm
last bolt row to edge distance	e_{2n}	=	50.00	mm
last bolt row to edge distance (column flange)	e_{2s}	=	50.00	mm
Yield strength	f_{yp}	=	235.00	N/mm²
Ultimate strength	f_{up}	=	360.00	N/mm²

Bolts M20, 8.8

Resistant area	A_s	=	245.00	mm²
Diameter of the shank	d	=	20.00	mm
Diameter of the holes	d_0	=	22.00	mm
Yield strength	f_{yb}	=	640.00	N/mm²
Ultimate strength	f_{ub}	=	800.00	N/mm²

Welds

Throat thickness of the weld	a_w	=	4.00	mm
Length of the weld	l_w	=	230.00	mm

Safety factors

γ_{M0} = 1.00
γ_{M2} = 1.25

Applied shear force

V_{Sd} = 200 kN

7.1.2 Ductility and rotation requirements

Rotation requirements

(1) $h_p \leq d_b$

h_p = 230.00 mm
d_b = $h - 2\ t_{bf} - 2\ r$
= 300.00 – 2 10.70 – 2 15.00 = 248.60 mm
→ ok

(2) $\phi_{available} > \phi_{required}$ we suppose that this requirement is fulfilled.

Ductility requirements

(1) $$\frac{d}{t_p} \geq 2{,}8\sqrt{\frac{f_{yp}}{f_{ub}}}$$

d / t_p = 2.00
f_{yp} / f_{ub} = 0.29
→ 2.00 ≥ 1.52 ok

(2) $$a \geq 0.4\ t_{bw}\ \beta_w\ \sqrt{3}\ \frac{f_{ybw}}{f_{ubw}}\ \frac{\gamma_{M2}}{\gamma_{M0}} = 3.21\ \text{mm}$$

t_{bw} = 7.1 mm
f_{ybw} = 235.00 N/mm²
f_{ubw} = 360.00 N/mm²
β_w = 0.80

a = 4.00 mm → ok

7.1.3 Joint shear resistance

Bolts in shear

$V_{Rd\,1} = 0{,}8\ n\ F_{v,Rd} = 451.58$ kN

$n = 6$

$F_{v,Rd} = \alpha_v\ A\ f_{ub} / \gamma_{M2} = 94.08$ kN

$\alpha_v = 0.6$

$A = A_S = 245.00$ mm²

$f_{ub} = 800.00$ N/mm²

Header plate in bearing

$V_{Rd\,2} = n\ F_{b,Rd} = 589.09$ kN

$n = 6$

$F_{b,Rd} = k_1\ \alpha_b\ d\ t_p\ f_{up} / \gamma_{M2} = 98.18$ kN

$\alpha_b = \min(\alpha_1 , \alpha_2 , \alpha_3 , 1) = 0.68$

$\alpha_1 = e_1 / 3d_0 = 0.68$

$\alpha_2 = p_1 / 3d_0 - 1/4 = 0.81$

$\alpha_3 = f_{ub} / f_{up} = 2.22$

$k_1 = \min(2.8\ e_2 / d_0 – 1.7\ ; 2.5)$
$= \min(4.66\ ; 2.5) = 2.5$

$d = 20.00$ mm

$t_p = 10.00$ mm

$f_{ub} = 800.00$ N/mm²

$f_{up} = 360.00$ N/mm²

Column flange in bearing

$V_{Rd\,3} = n\ F_{b,Rd} = 700.36$ kN

$n = 6$

$F_{b,Rd} = k_1\ \alpha_b\ d\ t_{cf}\ f_{ucf} / \gamma_{M2} = 116.73$ kN

$\alpha = \min(\alpha_1 , \alpha_2 , 1) = 0.81$

$\alpha_1 = p_1 / 3d_0 - 1/4 = 0.81$

$\alpha_2 = f_{ub} / f_{ucf} = 2.22$

$k_1 = \min(2.8\ e_{2s} / d_0 - 1.7\ ;\ 2.5)$
$= \min(4.66\ ;\ 2.5) = 2.5$

$d = 20.00$ mm
$t_{cf} = 10.00$ mm
$f_{ub} = 800.00$ N/mm²
$f_{ucf} = 360.00$ N/mm²

Gross section of the header plate in shear

$V_{Rd\ 4} = 2\ F_{v,Rd} = 491.44$ kN

$F_{v,Rd} = A_v\ f_{yp} / (1{,}27\ \sqrt{3}\ \gamma_{M0}) = 245.72$ kN
$A_v = h_p\ t_p = 23.00$ cm²
$f_{yp} = 235.00$ N/mm²

Net section of the header plate in shear

$V_{Rd\ 5} = 2\ F_{v,Rd} = 545.39$ kN

$F_{v,Rd} = A_{v,net}\ f_{up} / (\sqrt{3}\ \gamma_{M2}) = 272.69$ kN
$A_{v,net} = (h_p - n_1\ d_0)\ t_p = 16.40$ cm²
$h_p = 230.00$ mm
$n_1 = 6$
$d_0 = 22.00$ mm
$t_p = 10.00$ mm
$f_{up} = 360.00$ N/mm²

Shear block of the header plate

$V_{Rd\ 6} = 2\ F_{eff,Rd} = 577.40$ kN

$1{,}36\ p_2' = 136.00$ mm $\rightarrow h_p > 1{,}36\ p_2'$
$n_1 = 3$ $\rightarrow n_1 > 1$

$F_{eff,Rd} = F_{eff,1,Rd} = f_{up}\ A_{nt} / \gamma_{M2} + f_{yp}\ A_{nv} / (\sqrt{3}\ \gamma_{M0}) = 288.70$ kN
$A_{nt} = t_p\ (e_2 - d_0/2) = 390.00$ mm²
$t_p = 10.00$ mm
$e_2 = 50.00$ mm
$d_0 = 22.00$ mm
$A_{nv} = t_p\ (h_p - e_1 - (n_1 - 0.5)\ d_0) = 1300.00$ mm²

n_1 = 3
h_p = 230.00 mm
e_1 = 45.00 mm

f_{yp} = 235.00 N/mm²
f_{up} = 360.00 N/mm²

Header plate in bending

$V_{Rd\ 7} = \infty$

h_p = 230.00 mm
1,36 p_2' = 136.4 mm → $h_p > 1{,}36\ p_2'$

Beam web in shear

$V_{Rd\ 8} = F_{v,Rd}$ = 221.56 kN

$F_{v.Rd} = A_v\ f_{ybw} / (\sqrt{3}\ \gamma_{M0})$ = 221.56 kN
$A_v = h_p\ t_{bw}$ = 16.33 cm²
f_{ybw} = 235.00 N/mm²

Joint shear resistance

Shear resistance of the joint	V_{Rd}	= 221.56 kN
Failure Mode:	Beam web in shear	

7.1.4 Design check

Applied shear force:	V_{Sd}	= 200 kN		
Shear resistance:	V_{Rd}	= 221.56 kN	⇒	Design O.K.

7.1.5 Joint tying resistance

Bolts in tension

$N_{u\,1} = n\,B_{t,u} = 1176.00$ kN

$n = 6$

$B_{t,u} = f_{ub}\,A_s = 196.00$ kN

$A_s = 245.00$ mm²

$F_{ub} = 800.00$ N/mm²

Header plate in bending

$N_{u\,2} = \min(F_{hp,u,1}\,;\,F_{hp,u,2}) = 684.69$ kN

$$F_{hp,u,1} = \frac{(8\,n_p - 2\,e_w)\,l_{eff.p.t,1}\,m_{u.p}}{2\,m_p\,n_p - e_w\,(m_p + n_p)} = 852.83 \text{ kN}$$

$$F_{hp,u,2} = \frac{2\,l_{eff.p.t,2}\,m_{u.p} + n\,B_{t.u}\,n_p}{m_p + n_p} = 684.69 \text{ kN}$$

$n = 6$

$m_p = (p_2' - t_w - 2 \times 0{,}8\,a\,2^{-0{,}5}) / 2 = 41.925$ mm

$n_p = \min(e_2\,;\,1{,}25\,m_p) = \min(50\,;\,52.4) = 50.00$ mm

$m_{u.p} = \dfrac{t_p^2\,f_{up}}{4} = 9000.00$ N mm/mm

$l_{eff.p1} = l_{eff.p2} = h_p = 230.00$ mm

$e_w = 37.00$ mm

Supporting member in bending (column flange)

$N_{u\,2} = \min(F_{cf,u,1}\,;\,F_{cf,u,2}) = \ldots$

$$F_{cf,u,1} = \frac{(8\,n_{cf} - 2\,e_w)\,l_{eff.cf.t,1}\,m_{u.cf}}{2\,m_{cf}\,n_{cf} - e_w\,(m_{cf} + n_{cf})} = \ldots$$

$$F_{cf,u,2} = \frac{2\,l_{eff.cf.t,2}\,m_{u.cf} + n\,B_{t.u}\,n_{cf}}{m_{cf} + n_{cf}} = \ldots$$

$n = 6$

$m_{cf} = (p_2' - t_{cw} - 2 \times 0{,}8\,r_c) / 2 = 32.35$ mm

$n_{cf} = \min(e_{2s}\,;\,1{,}25\,m_p) = \min(50\,;\,40.438) = 40.438$ mm

$m_{u.cf} = \dfrac{t_{cf}^2\,f_{ucf}}{4} = 9000.00$ N mm/mm

$l_{eff.cf1} = \ldots$

$l_{eff.cf2}$ = …
e_w = 37.00 mm

Comment : More resistance than for the header plate (higher l_{eff} values and smaller values of m and n).

Beam web in tension

$$N_{u\,4} = t_w\ h_p\ f_{ubw} = 587.88 \text{ kN}$$

t_w = 7.10 mm
h_p = 230.00 mm
f_{ubw} = 360.00 N/mm²

Welds

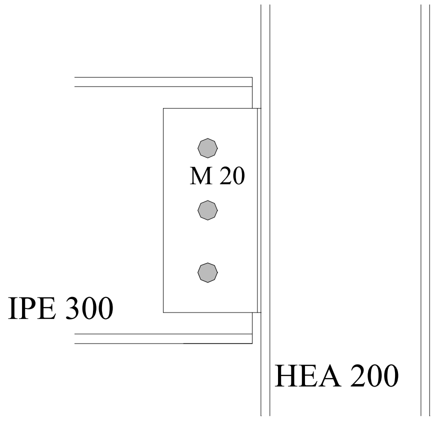
Conditions for full-strength behaviour of the welds are fulfilled

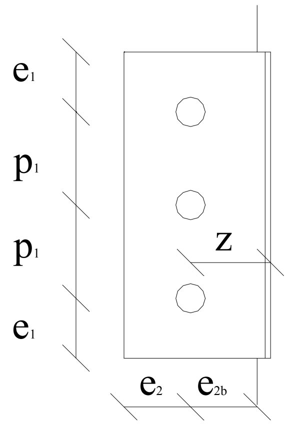
Joint tying resistance

Tying resistance of the joint	N_u = 587.88 kN
Failure mode :	Beam web in tension

7.2 Fin plate connection

7.2.1 Geometrical and mechanical data





Main joint data

Configuration	Beam to column flange
Column	HEA 200 S 235
Beam	IPE 300 S 235
Type of connection	Fin plate connection
Fin plate	230 x 110 x 10, S 235

Detailed characteristics

Column HEA 200, S235

Depth	h	=	190.00	mm
Thickness of the web	t_{cw}	=	6.50	mm
Width	b_f	=	200.00	mm
Thickness of the flange	t_{cf}	=	10.00	mm
Root radius	r	=	18.00	mm
Area	A	=	53.83	cm^2
Inertia	I	=	3692.16	cm^4
Yield strength	f_{yc}	=	235.00	N/mm^2
Ultimate strength	f_{uc}	=	360.00	N/mm^2

Beam IPE 300, S235

Depth	h	=	300.00	mm
Thickness of the web	t_{bw}	=	7.10	mm
Width	b_f	=	150.00	mm

Thickness of the flange	t_{bf}	=	10.70	mm
Root radius	r	=	15.00	mm
Area	A	=	53.81	cm²
Inertia	I	=	8356.11	cm^4
Yield strength	f_{yb}	=	235.00	N/mm²
Ultimate strength	f_{ub}	=	360.00	N/mm²

Fin plate 230 x 110 x 10, S 235

Vertical gap	g_v	=	35.00	mm
Horizontal gap (end beam to column flange)	g_h	=	10.00	mm
Depth	h_p	=	230.00	mm
Width	b_p	=	110.00	mm
Thickness	t_p	=	10.00	mm

Direction of load transfer (1)

Number of bolts rows	n_1	=	3	
Edge to first bolt row distance	e_{11}	=	45.00	mm
Beam edge to first bolt row distance	e_{1b}	=	80.00	mm
Pitch between bolt row 1 and 2	$p_{1[1]}$	=	70.00	mm
Pitch between bolt row 2 and 3	$p_{1[2]}$	=	70.00	mm
last bolt row to edge distance	e_{1n}	=	45.00	mm

Direction perpendicular to Load transfer (2)

Number of bolts rows	n_2	=	1	
Edge to first bolt row distance	e_{21}	=	50.00	mm
last bolt row to beam edge distance	e_{2b}	=	50.00	mm
Lever arm	z	=	60.00	mm
Yield strength	f_{yp}	=	235.00	N/mm²
Ultimate strength	f_{up}	=	360.00	N/mm²

Bolts M20, 8.8

Resistant area	A_s	=	245.00	mm²
Diameter of the shank	d	=	20.00	mm
Diameter of the holes	d_0	=	22.00	mm
Yield strength	f_{yb}	=	640.00	N/mm²
Ultimate strength	f_{ub}	=	800.00	N/mm²

Welds

Throat thickness of the weld	a_w	=	5.00	mm
Length of the weld	l_w	=	230.00	mm

Safety factors

γ_{M0} = 1.00

γ_{M2} = 1.25

Applied shear force

V_{Sd} = 100 kN

7.2.2 Requirements to ensure sufficient rotation capacity

(1) $h_p \leq d_b$

h_p = 230.00 mm

d_b = $h - 2\ t_{bf} - 2\ r$

= 300.00 – 2 10.70 – 2 15.00 = 248.60 mm

→ ok

(2) $\phi_{available} > \phi_{required}$ we suppose that this requirement is fulfilled.

7.2.3 Requirements to avoid premature weld failure

$$a > 0{,}4\ t_p\ \beta_w\ \sqrt{3}\ \frac{f_{yp}}{f_{up}}\ \frac{\gamma_{M2}}{\gamma_{M0}}\ =\ 4.52\text{ mm}$$

t_p = 10.00 mm

f_{yp} = 235.00 N/mm²

f_{up} = 360.00 N/mm²

β_w = 0.80

a = 5.00 mm → ok

7.2.4 Joint shear resistance

Bolts in shear

$$V_{Rd\,1} = \frac{n \quad F_{v,Rd}}{\sqrt{1 \quad + \quad \left(\dfrac{6\,z}{(n+1)\,p_1}\right)^2}} = 173.28 \text{ kN}$$

$n \quad = 3$

$z \quad = 60.00 \text{ mm}$

$F_{v,Rd} = \alpha_v\, A \;\; f_{ub} / \gamma_{M2} = 94.08 \text{ kN}$

$\alpha_v \quad = 0.6$

$A \quad = A_s = 245.00 \text{ mm}^2$

$f_{ub} \quad = 800.00 \text{ N/mm}^2$

Fin plate in bearing

$$V_{Rd\,2} = \frac{1}{\sqrt{\left(\dfrac{\frac{1}{n}+\alpha}{F_{b,ver,Rd}}\right)^2 + \left(\dfrac{\beta}{F_{b,hor,Rd}}\right)^2}} = 192.59 \text{ kN}$$

$n \quad = 3$

$\alpha \quad = 0$

$1 / n \quad = 1 / 3$

$$\beta = \frac{6\,z}{p_1 \;\; n\,(n+1)} = 0.43$$

$F_{b,Rd,ver} = \; k_1\, \alpha_b\, d \;\; t_p \;\; f_{up} / \gamma_{M2} = 98.18 \text{ kN}$

$\alpha_b \quad = \min(\alpha_1 , \alpha_2 , \alpha_3 , 1) = 0.68$

$\alpha_1 \quad = e_1 / 3d_0 = 0.68$

$\alpha_2 \quad = p_1 / 3d_0 - 1/4 = 0.81$

$\alpha_3 \quad = f_{ub} / f_{up} = 2.22$

$k_1 \quad = \min(2.8\, e_2 / d_0 - 1.7 \,;\, 2.5)$

$\quad = \min(4.66 \,;\, 2.5) = 2.5$

$F_{b,Rd,hor} = \; k_1\, \alpha_b\, d \;\; t_p \;\; f_{up} / \gamma_{M2} = 109.09 \text{ kN}$

$\alpha_b \quad = \min(\alpha_1 , \alpha_2 \;, 1) = 0.75$

$\alpha_1 \quad = e_2 / 3d_0 = 0.75$

$\alpha_2 = f_{ub} / f_{up} = 2.22$

$k_1 = \min (2.8\, e_1 / d_0 - 1.7 \,;\, 1.4\, p_1 / d_0 - 1.7 \,;\, 2.5)$
$\quad = \min (4.03 \,;\, 2.75 \,;\, 2.5) = 2.5$

$d = 20.00$ mm
$t_p = 10.00$ mm
$f_{ub} = 800.00$ N/mm²
$f_{up} = 360.00$ N/mm²

Gross section of the fin plate in shear

$$V_{Rd\,3} = A_v\, f_{yp} / (1.27\, \sqrt{3}\, \gamma_{M0}) = 245.72 \text{ kN}$$

$A_v = h_p\, t_p = 23.00$ cm²
$f_{yp} = 235.00$ N/mm²

Net section of the fin plate in shear

$$V_{Rd\,4} = A_{v,net}\, f_{up} / (\sqrt{3}\, \gamma_{M2}) = 272.69 \text{ kN}$$

$A_{v,net} = (h_p - n_1\, d_0)\, t_p = 16.40$ cm²
$h_p = 230.00$ mm
$n_1 = 3$
$d_0 = 22.00$ mm
$t_p = 10.00$ mm
$f_{up} = 360.00$ N/mm²

Shear block of the fin plate

$$V_{Rd\,5} = F_{eff,2,Rd} = 232.54 \text{ kN}$$

$$F_{eff,2,Rd} = 0.5\, f_{up}\, A_{nt} / \gamma_{M2} + f_{yp}\, A_{nv} / (\sqrt{3}\, \gamma_{M0}) = 232.54 \text{ kN}$$

$A_{nt} = t_p\, (e_2 - d_0/2) = 390.00$ mm²
$t_p = 10.00$ mm
$e_2 = 50.00$ mm
$d_0 = 22.00$ mm
$A_{nv} = t_p\, (h_p - e_1 - (n_1 - 0.5)\, d_0) = 1300.00$ mm²
$n_1 = 3$
$h_p = 230.00$ mm
$e_1 = 45.00$ mm

f_{yp} = 235.00 N/mm²
f_{up} = 360.00 N/mm²

Fin plate in bending

$h_p = 230\text{ mm} \geq 2{,}73\ z = 163{,}8\text{ mm}$

$V_{Rd\,6} = \infty$

Buckling of the fin plate

$$V_{Rd\,7} = \frac{W_{el}}{z}\ \frac{\sigma}{\gamma_{M0}} = 776.97\text{ kN}$$

$$W_{el} = \frac{t_p\ h_p^2}{6} = 88\ 166.67\text{ mm}^3$$

$$\sigma = 81\ \left(\frac{t_p}{z}\right)^2\ 235 = 528.75\text{ N/mm}^2$$

Beam web in bearing

$$V_{Rd\,8} = \frac{1}{\sqrt{\left(\dfrac{\frac{1}{n}+\alpha}{F_{b,ver,Rd}}\right)^2 + \left(\dfrac{\beta}{F_{b,hor,Rd}}\right)^2}} = 146.19\text{ kN}$$

n = 3
α = 0
$1 / n$ = 1 / 3

$$\beta = \frac{6\ z}{p_1\ n\ (n+1)} = 0.43$$

$F_{b,Rd,ver} = k_1\ \alpha_b\ d\ t_{bw}\ f_{ubw} / \gamma_{M2} = 82.88\text{ kN}$
α_b = min (α_1 , α_2 , 1) = 0.81
α_1 = $p_1 / 3d_0 - 1/4$ = 0.81
α_3 = f_{ub} / f_{ubw} = 2.22

k_1 = min (2.8 e_{2b} / d_0 – 1.7 ; 2.5)
= min (4.66 ; 2.5) = 2.5

$F_{b,Rd,hor} = k_1 \alpha_b d t_{bw} f_{ubw} / \gamma_{M2} = 77.45$ kN

$\alpha_b = \min(\alpha_1, \alpha_2, 1) = 0.75$

$\alpha_1 = e_{2b} / 3d_0 = 0.75$

$\alpha_2 = f_{ub} / f_{ubw} = 2.22$

$k_1 = \min(1.4 p_1 / d_0 - 1.7 ; 2.5)$

$= \min(2.75 ; 2.5) = 2.5$

$d = 20.00$ mm

$t_{bw} = 7.10$ mm

$f_{ub} = 800.00$ N/mm²

$f_{ubw} = 360.00$ N/mm²

Gross section of the beam web in shear

$V_{Rd\,9} = A_{b,v} f_{ybw} / (\sqrt{3}\,\gamma_{M0}) = 348.42$ kN

$A_{b,v} = 25.68$ cm²

$f_{ybw} = 235.00$ N/mm²

Net section of the beam web in shear

$V_{Rd10} = A_{v,net} f_{ubw} / (\sqrt{3}\,\gamma_{M2}) = 349.11$ kN

$A_{b,v,net} = A_{b,v} - n_1 d_0 t_{bw} = 21.00$ cm²

$A_{b,v} = 25.68$ cm²

$n_1 = 3$

$d_0 = 22.00$ mm

$t_{bw} = 7.10$ mm

$f_{ubw} = 360.00$ N/mm²

Shear block of the beam web

$V_{Rd11} = F_{eff,2,Rd} = 198.82$ kN

$F_{eff,2,Rd} = 0.5 f_{ubw} A_{nt} / \gamma_{M2} + f_{ybw} A_{nv} / (\sqrt{3}\,\gamma_{M0}) = 198.82$ kN

$A_{nt} = t_{bw} (e_{2b} - d_0/2) = 276.9$ mm²

$t_{bw} = 7.10$ mm

$e_{2b} = 50.00$ mm

$d_0 = 22.00$ mm

$A_{nv} = t_{bw} (e_{1b} + (n_1 - 1) p_1 - (n_1 - 0{,}5) d_0) = 1171.50$ mm²

n_1 = 3
p_1 = 70.00 mm
e_{1b} = 45.00 + 35.00 = 80.00 mm
f_{ybw} = 235.00 N/mm²
f_{ubw} = 360.00 N/mm²

Joint shear resistance

Shear resistance of the joint V_{Rd} = 146.18 kN
Failure Mode: Beam web in bearing

7.2.5 Requirements to ensure the safety of the shear design rules

(1) $V_{Rd} < \min(V_{Rd\,1} ; V_{Rd\,7})$

V_{Rd} = 146.18 kN
$\min(V_{Rd\,1} ; V_{Rd\,7})$ = 178.28 kN
$V_{Rd\,1}$ = 178.28 kN
$V_{Rd\,7}$ = 776.97 kN
→ ok.

(2) $\mathbf{n_2 = 1}$:

$F_{b,hor,Rd} \leq \min(F_{v,Rd} ; V_{Rd\,7}\ \beta)$

$V_{Rd\,7}$ = 776.97 kN
$F_{v,Rd}$ = 94.08 kN

for the beam web :
$F_{b,hor,Rd}$ = 77.45 kN
β = 0.43
$\min(F_{v,Rd} ; V_{Rd\,7}\ \beta)$ = min (94.08 ; 334.09) = 94.08 kN
→ ok.

One of the two inequalities is satisfied. → ok.

(3) $V_{Rd} = V_{Rd\,8}$ → ok.

7.2.6 Design check

Applied shear force:	V_{Sd}	= 100 kN		
Shear resistance:	V_{Rd}	= 146.18 kN	$\Rightarrow$	Design O.K.

7.2.7 Joint tying resistance

Bolts in shear

$N_{u\,1} = n\,F_{v,u} = 352.80$ kN

$n = 3$

$F_{v,u} = \alpha_v \; f_{ub} \; A = 117.60$ kN

$A = A_s = 245.00$ mm²

$\alpha_v = 0{,}6$

Fin plate in bearing

$N_{u\,2} = n\,F_{b,u,\,hor} = 409.09$ kN

$n = 3$

$F_{b,u,hor} = k_1 \; \alpha_b \; f_{up} \; d \; t_p = 136.36$ kN

$\alpha_b = \min(\alpha_1 , \alpha_2 , 1) = 0.75$

$\alpha_1 = e_2 / 3d_0 = 0.75$

$\alpha_2 = f_{ub} / f_{up} = 2.22$

$k_1 = \min(2.8\,e_1 / d_0 - 1.7 \,;\, 1.4\,p_1 / d_0 - 1.7 \,;\, 2.5)$

$= \min(4.03 \,;\, 2.75 \,;\, 2.5) = 2.5$

$d = 20.00$ mm

$t_p = 10.00$ mm

$f_{ub} = 800.00$ N/mm²

$f_{up} = 360.00$ N/mm²

Fin plate in tension : gross section

$N_{u\,3} = t_p\,h_p\,f_{up} = 828.00$ kN

Fin plate in tension : net section

$N_{u\,4} = 0{,}9\,A_{net,p}\,f_{up} = 531.36$ kN

$A_{net,p} = t_p\,h_p - d_0\,n_1\,t_p = 1640.00$ mm²

$n_1 = 3$

$h_p = 230.00$ mm

$t_p = 10.00$ mm

$d_0 = 22.00$ mm

Beam web in bearing

$N_{u\,5} = n\, F_{b,u,\,hor} = 290.45$ kN

$n = 3$

$F_{b,u,hor} = k_1\ \alpha_b\ f_{ubw}\ d\ t_{bw} = 96.82$ kN

$\alpha_b = \min(\alpha_1 , \alpha_2 , 1) = 0.75$

$\alpha_1 = e_{2b} / 3d_0 = 0.75$

$\alpha_2 = f_{ub} / f_{ubw} = 2.22$

$k_1 = \min(1.4\ p_1 / d_0 - 1.7 ; 2.5)$
$= \min(2.75 ; 2.5) = 2.5$

$d = 20.00$ mm
$t_{bw} = 7.10$ mm
$f_{ub} = 800.00$ N/mm²
$f_{ubw} = 360.00$ N/mm²

Beam web in tension : gross section

$N_{u\,6} = t_{bw}\ h_{bw}\ f_{ubw} = 587.88$ kN

Beam web in tension : net section

$N_{u\,7} = 0{,}9\ A_{net,bw}\ f_{ubw} = 377.27$ kN

$A_{net,bw} = t_{bw}\ h_{bw} - d_0\ n_1\ t_{bw} = 1164.40$ mm²
$t_{bw} = 7.10$ mm
$h_{bw} = 230.00$ mm
$n_1 = 3$
$d_0 = 22.00$ mm

Supporting member in bending

….

Welds

Conditions for full-strength behaviour of the welds are fulfilled

Joint tying resistance

Tying resistance of the joint	N_u = 290.45 kN
Failure mode :	Beam web in bearing

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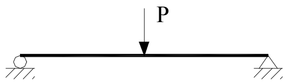
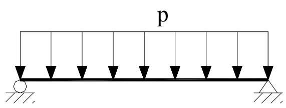
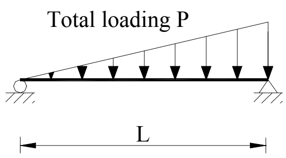
9 Annexe 1 : Practical values for $f_{required}$

System of loading	M_{max}	$\phi_{required}$
	M	$\phi_A = \frac{\gamma M L}{6 E I}$ $\phi_B = -\frac{\gamma M L}{3 E I}$
	$\frac{P L}{4}$	$\pm \frac{\gamma P L^2}{16 E I}$
	$\frac{p L^2}{8}$	$\pm \frac{\gamma p L^3}{24 E I}$
	$\frac{2 P L}{9 \sqrt{3}}$	$\phi_A = \frac{7 \gamma P L^2}{180 E I}$ $\phi_B = -\frac{8 \gamma P L^2}{180 E I}$

where E is the elastic modulus of the material whose the beam is formed ;
I is the second moment area of a beam ;
L is the span of a beam (centre-to-centre of columns) ;
γ is the loading factor at ULS.