The Guidelines For The Design Of Steel-Concrete Composite Joints In Seismic Areas

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ABSTRACT

The behaviour of steel and steel-concrete composite joints has been thoroughly studied in the last decades, with focus on their influence on the global response of framed structures. Nevertheless, especially as far as composite steel-concrete structures are concerned, there are no well-established design guidelines to optimize choices and adequately design composite joints. For this purpose, the Guidelines for the seismic design of steel-concrete composite framed structures and the non-linear analysis and the Guidelines for the seismic design of steel-concrete composite joints were developed within the RELUIS 2014-16 project, financed by the 'Dipartimento di Protezione Civile' (Department of Civil Protection). This paper highlights the main issues that need to be addressed with regards to the seismic design of the beam to column joints and frames belonging to steel-concrete structures, discussing rules reported within codes, as well as modelling approaches. The provisions that are referred to this paper are the 'Norme Tecniche per le Costruzioni' (NTC2008) and the Eurocode 4 [EN 1994-1-1:2004] as far as the general rules to be applied to the design of a composite structure are concerned, and the Eurocode 8 [EN 1998-1:2004] as regards the specific rules to be applied.

KEYWORDS

Composite joints, Steel-concrete frames, slab mechanisms, slab-column interactions.

1 INTRODUCTION

This paper focuses on the main issues affecting the modelling and design of steel-concrete composite structures in seismic areas, with focus on the evaluation of joints behavior. Indications for the design of seismic-resistant composite frames and structural joints have recently been collected in specific Guidelines developed under the ReLUIS Project 2014-16.
The regulations referred to this note are the Italian building code (NTC2008) and the Eurocode 4 [EN 1994-1-1: 2004] as regards the general rules to be applied in the design of a composite structure and the Eurocode 8 [EN 1998-1: 2004] for the specific rules to be applied in seismic areas. Specifically, the different resistant mechanisms that might occur in the concrete slab at the beam-to-column intersection are discussed. Moreover, a design procedure of the slab able to assure a high ductility of the node is described. Finally, the stiffness and resistance of the basic composite components are defined to determine the moment-rotation curve of a composite joint, either welded or bolted.

2 STEEL-CONCRETE COMPOSITE FRAMES

The behaviour of a steel-concrete composite frame under seismic actions can be very complex, and attention must be given to the nodal behaviour that can be influenced by multiple parameters, such as:

- the sign of the bending moment, the moment-rotation curve is asymmetric;
- the type of steel joint (welded, with extended end-plate, with cleats etc.);
- the presence or not of a concrete cantilever edge strip;
- the node position inside the frame (interior or exterior);
- a bracing system if present.

In a moment resisting frame, the slab is usually in contact with the column (see Figure 1); in this case, interactions between the slab and the column can arise, with the development of different strut-and-tie resistant mechanisms that form between the longitudinal and transverse rebars and the concrete in compression.

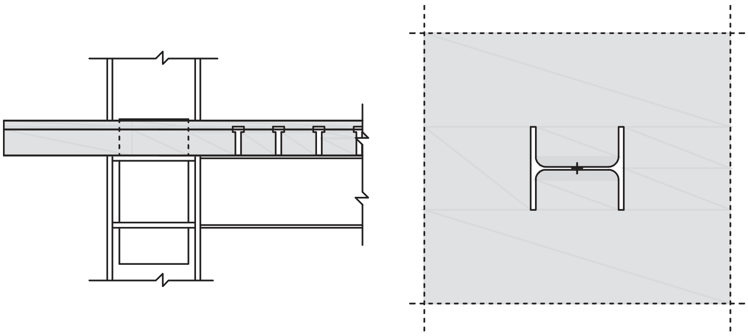


Figure 1. Typical steel-concrete compound joint (slab in contact with the column)

These interactions significantly affect the joint response in terms of ductility, stiffness, and resistance, and therefore a careful evaluation is required to ensure that the principle of hierarchy of resistances is satisfied (Amadio *et al.*, 2016, 2017a).

A design alternative that allows to ignore the composite action near the node is to disconnect the slab, i.e. to avoid, through suitable construction details, the slab-column interactions. Generally, the following arrangements are necessary (Chaudhari *et al.*, 2015) (Seek and Murray, 2008):

- shear studs should not be placed in the beam for a distance from the column face of 1.5 times the height of the composite beam;
- a 2 to 3 centimetres gap between the column and the slab must be made to avoid direct contact and thus the formation of struts;
- the longitudinal rebars must be interrupted at the column.

As for Eurocode 8, interactions can be avoided if the slab is "*totally disconnected from the steel frame in a circular zone around a column of diameter* $2b_{eff}$*, with* b_{eff} *being the larger of the effective widths of the beams connected to that column*".

However, the construction details required to disconnect the slab still need to be studied in depth, particularly as regards the possibility of maintaining the continuity of the longitudinal rebars to realize, through the use of hinged steel joints, composite beams that behave like continuous on multiple supports under vertical loads, and at the same time transfer only shear forces to the column (Amadio *et al.*, 2017b).

In the case of steel-concrete composite structures, the possible construction typologies are numerous. In fact, they vary according to the static scheme that the structure assumes under the action of horizontal and vertical loads, and hence depending on the type of joint. Based on the structural system that opposes the horizontal forces (earthquake, wind, imperfections, etc.), two macro categories are identified:

- braced composite structures;
- moment resisting composite structures.

Below, the main construction choices available for both cases are briefly described.

2.1 Braced structures

The bracing system is considered effective if:

- its stiffness K_{brace} is much greater than that of the frame K_{frame} (usually at least $K_{brace} > 5\ K_{frame}$);
- the seismic base shear V_b is absorbed mainly by the bracing system (for example $V_{b,\ brace} > 80\%\ V_b$).

It is thus clear that a correct evaluation of the joint influence is also required when designing braced structures; when adopting this structural typology, it is appropriate to "isolate" the slab to build pinned frames and entrust the entire horizontal action to the bracing system. For this purpose, some of the possible design choices are:

- pinned beams: the slab is disconnected from the column by means of a gap and there is no rebars continuity between two consecutive spans (Figure 2);

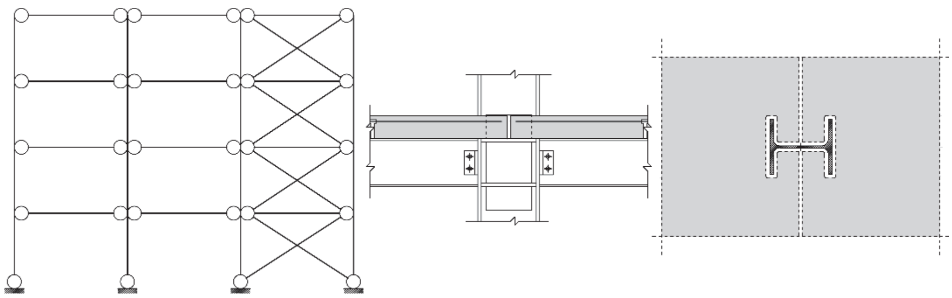


Figure 2. Braced frame with pinned beams

- Continuous beams and pinned columns: the slab is still disconnected but the longitudinal rebars are continuous. The beam behaves as continuous on multiple supports and the negative bending moments at the node are self-balanced (Figure 3);

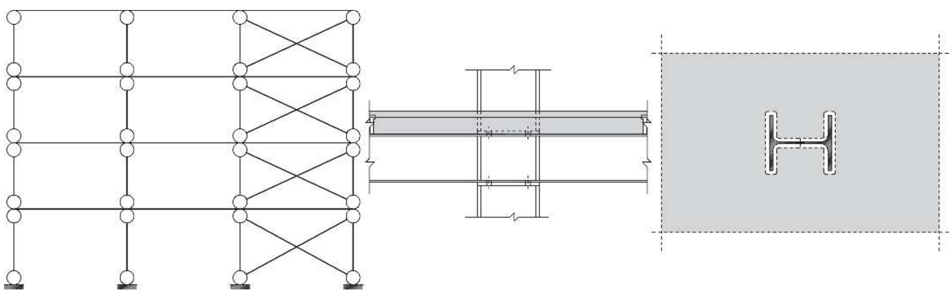


Figure 3. Braced frame with continuous beams and pinned columns

- Frame with disconnected slab and cleated steel joints: the cleats transmit only shears forces to the column while continuity of longitudinal rebars keeps the beam running as a continuous on multiple supports under the action of vertical loads (Figure 4).

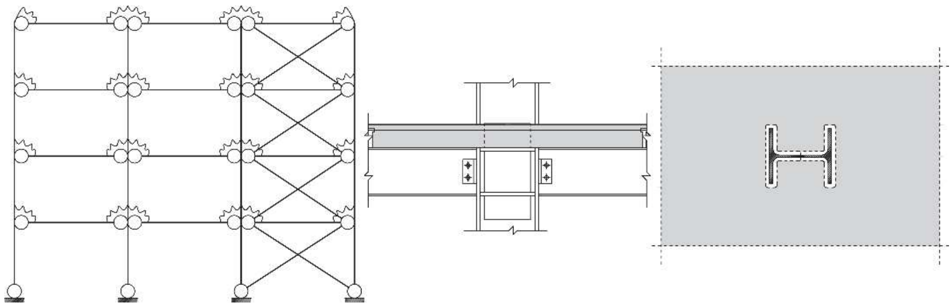
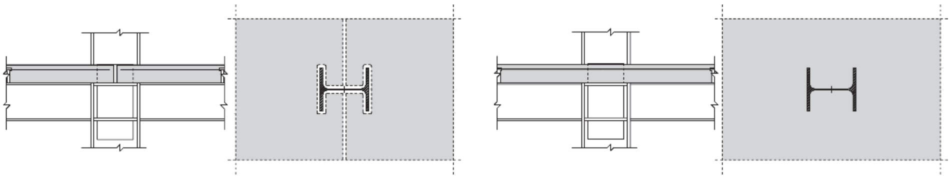


Figure 4. Braced frame with web cleats connection

2.2 Moment-Resisting Frames (MRF)

In MRF both vertical and horizontal loads are entrusted to the framed system, therefore the connections must be able to transfer the bending moments. Two solutions can be adopted for the slab detailing:

- Slab disconnected from the column (isolated slab): only steel components offer resistance at the beam-column (Figure 5a);
- Slab in contact with the column: it is necessary to evaluate the slab-column interactions (Figure 5b).



a) b)

Figure 5. MRF composite structure: Isolated slab (a), Slab in contact (b)

In both cases, the role played by the joint and dissipations near the nodal zone are fundamental to evaluate the structural response (Fasan, 2013), (Pecce and Rossi, 2015). This aspect is deepened in the "*Linee guida per la progettazione sismica di nodi composti acciaio-calcestruzzo*" (Amadio *et al.*, 2016).

3 COMPOSITE JOINTS

In steel-concrete composite joints interactions between the slab and the column lead to creation of different strut-and-tie mechanisms between the rebars in tension (both longitudinal and transversal) and the concrete in compression in contact with the column.
Such interactions significantly affect the joint ductility, stiffness, and resistance, and it is therefore necessary to carefully evaluate them to ensure the principle of hierarchy of resistance is satisfied. At beam-column intersection, depending on the node configuration, several mechanisms can be present simultaneously. In these cases, the maximum compression force that can be transmitted from the slab to the column is given by the sum of the resistances of the individual mechanisms.

3.1 Resistant mechanisms of the slab

3.1.1 Mechanism 1: direct contact between the slab and the column flange

Mechanism 1 consists in a compressed strut in direct contact with the column flange (Figure 6), as proposed in the Eurocode 8, the maximum transmitting force is:

$$F_{Rd,1} = b_b d_{eff} (0.85 f_{ck} / \gamma_c) \tag{1}$$

Where b_b represents the bearing width of the concrete of the slab (equal to the width of the column or that of any plate used to increase the contact area), d_{eff} the effective depth of the slab (equal to its thickness in the case of solid slabs or to the thickness of the slab over the ribs in the case of a slab with a sheet steel), f_{ck} and γ_c the compressive characteristic resistance and the concrete safety coefficient. This mechanism is present either with slab in compression (positive bending moment) or in tension (negative bending moment) (see Figure 6).

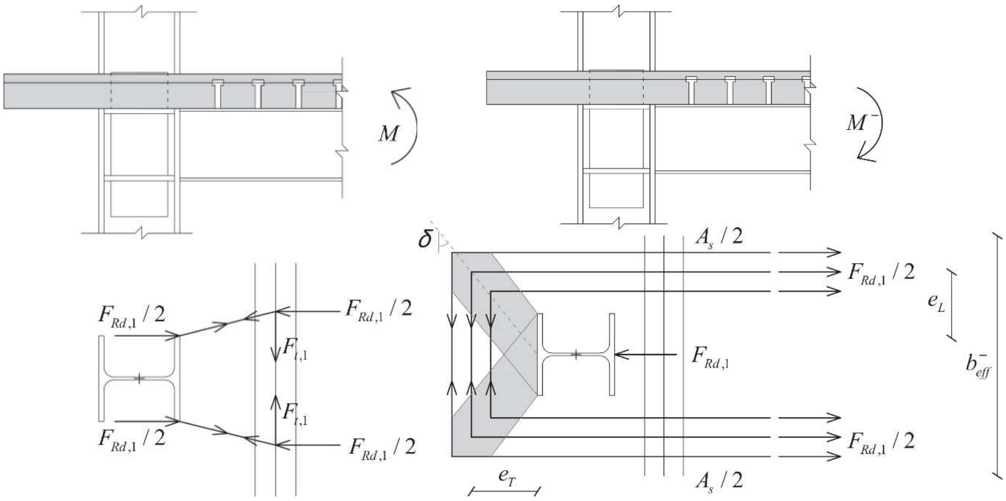


Figure 6. Mechanism 1: a) Positive bending moment; b) Negative bending moment

- Slab in compression

When the slab is in compression (Figure 6a), the force $F_{Rd,1}$ spreads along the beam for a length almost equal to b_{eff}^+ and generates a transversal force $F_{t,1}$ which can be calculated as (Plumier *et al.*, 1998) (Figure 7):

$$F_{t,1} = \frac{F_{Rd,1}}{2} \tan\alpha = \frac{F_{Rd,1}}{4} \frac{\left(b_{eff}^+ - b_b\right)}{b_{eff}^+} \tag{2}$$

Since $b_{eff}^+ = 0.15l$, it follows:

$$F_{t,1} = 0.25 b_b d_{eff}\, 0.85 \frac{f_{ck}}{\gamma_c} \frac{0.15l - b_b}{0.15l} = A_T \left(f_{yk,T} / \gamma_s\right) \tag{3}$$

where A_T represent the transversal rebars area, therefore the mechanism 1 transmit the maximum force if in the slab is placed a total rebar area of:

$$A_T = 0.25 b_b d_{eff} \frac{0.15l - b_b}{0.15l} \left(\frac{0.85 f_{ck} / \gamma_c}{f_{yk,T} / \gamma_s} \right) \quad (4)$$

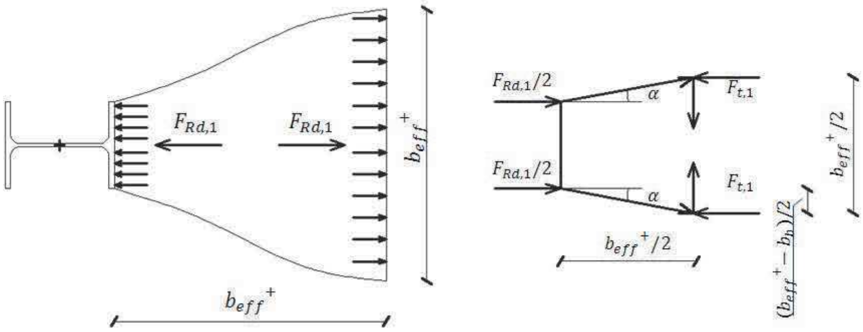


Figure 7. Spread of the concentrated force due to mechanism 1

- Slab in tension

The longitudinal rebars behave like ties and, depending on the nodal configuration, concrete struts might appear in contact with the column on the opposite side (see Figure 6b). To obtain a ductile behaviour the longitudinal rebars should yield before the concrete crushing, hence:

$$A_{s,1} \le \frac{F_{Rd,1}}{f_{yd,l}} = b_b d_{eff} \left(\frac{0.85 f_{ck} / \gamma_c}{f_{yk,l} / \gamma_s} \right) \quad (5)$$

3.1.2 Mechanism 2: inclined concrete struts in contact with the column sides

This mechanism, as proposed in the Eurocode 8, consists in inclined concrete struts in contact with the column sides. The maximum transmissible force is:

$$F_{Rd,2} = 0.7 h_c d_{eff} (0.85 f_{ck} / \gamma_c) \quad (6)$$

Where h_c represents the height of the column. The presence of this mechanism depends on the node configuration.

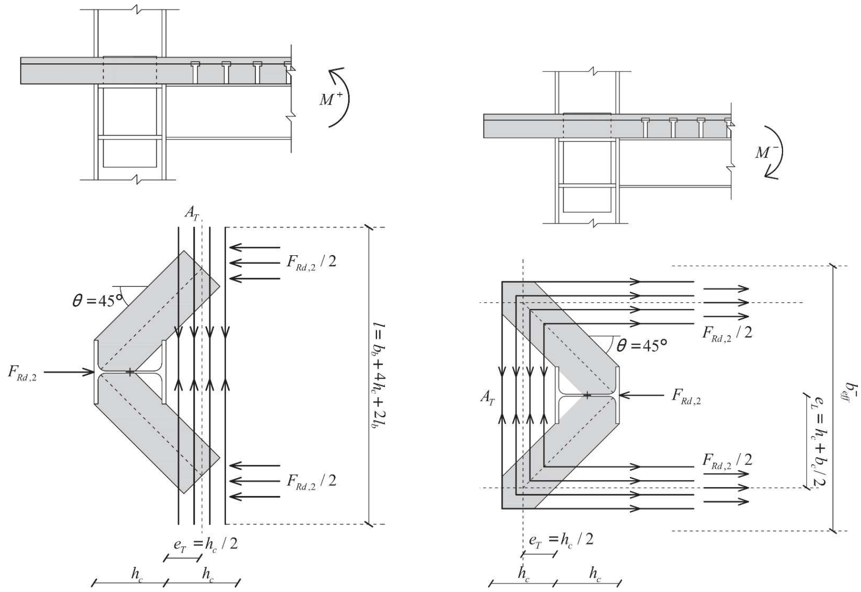


Figure 8. Mechanism 2: a) Positive bending moment; b) Negative bending moment

- Slab in compression (Figure 8a)

In this case the longitudinal rebars do not participate in the activation of the mechanism. On the contrary, the transversal rebars need to be properly designed. Assuming an inclination angle of the struts $\theta = 45°$ and a concrete resistance $0.7f_{cd}$, the maximum resistance of each strut is:

$$F_{c,2} = 0.7b_0 d_{eff}(0.85 f_{ck} / \gamma_c) = 0.7h_c / \sqrt{2} d_{eff} f_{cd} \tag{7}$$

Where b_0 is the width of the strut. The compressive concrete strength has been reduced to consider the presence of transversal tractions. The maximum force transmissible by mechanism 2 is obtained by finding the resultant of the two struts:

$$F_{Rd,2} = 2\frac{F_{c,2}}{\sqrt{2}} = 0.7h_c d_{eff}\left(0.85 f_{ck} / \gamma_c\right) \tag{8}$$

The force in the steel tie is equal to $F_{Rd,2}/2$, therefore the total area of transversal rebars needed is:

$$A_{T,2} \geq \frac{F_{Rd,2}}{2\left(f_{yk,T} / \gamma_s\right)} = 0.35h_c d_{eff}\left(\frac{0.85 f_{ck} / \gamma_c}{f_{yk,T} / \gamma_s}\right) \tag{9}$$

Following this design (suggested by EC8), the concrete crushes before the transversal rebar yielding is reached. However, to increase the ductility the rebars should yield before reaching the crushing of struts. This condition can be achieved dimensioning the transversal rebars as follows (Amadio e*t al*., 2017a):

$$A_{T,2} \leq \frac{F_{Rd,2}}{2\left(f_{yk,T} / \gamma_s\right)} = 0.35h_c d_{eff}\left(\frac{0.85 f_{ck} / \gamma_c}{f_{yk,T} / \gamma_s}\right) \tag{10}$$

Following the last case, the maximum force transmissible by mechanism 2 become:

$$F_{Rd,2} = 2A_{T,2}\, f_{yk,T} / \gamma_s \tag{11}$$

- Slab in tension (Figure 8b)

When the slab is subject to a negative moment, mechanism 2 balances the tension of the longitudinal rebars (Figure 8b). In this case the inclined struts appear on the opposite side to the application of the stress. The maximum force transmissible by each strut should be expressed as a function of its inclination angle θ. Assuming a concrete compressive strength of $0.7f_{cd}$ it can be evaluated as:

$$F_{c,2} = 0.7b_0 d_{eff}(0.85 f_{ck} / \gamma_c) = 0.7(h_c \sin\theta) d_{eff} f_{cd} \tag{12}$$

The maximum force transmissible by mechanism 2 is obtained by finding the resultant of the two struts:

$$F_{Rd,2} = 2F_{c,2}\cos\theta = 1.4h_c d_{eff}(0.85 f_{ck} / \gamma_c)\sin\theta\cos\theta \tag{13}$$

To achieve the yield of the longitudinal reinforcement before the concrete crushing, the maximum rebars area should be determined as:

$$A_{s,2} \le \frac{F_{Rd,2}}{2\left(f_{yk,l} / \gamma_s\right)} = 1.4h_c d_{eff}\sin\theta\cos\theta\left(\frac{0.85 f_{ck} / \gamma_c}{f_{yk,l} / \gamma_s}\right) \tag{14}$$

3.1.3 Mechanism 3: struts in contact with the studs of transversal beam

The maximum transmissible force by this mechanism is:

$$F_{Rd,3} = nP_{Rd} \tag{15}$$

with n number of studs present along the transverse beam for a length equal to the effective width and P_{Rd} the shear strength of the single stud.
When the slab is in tension, to achieve the yield of the longitudinal reinforcement before the concrete crushing, the maximum rebars area should be determined as:

$$A_{s,3} \le \frac{F_{Rd,3}}{f_{yd,l}} \tag{16}$$

3.2 Basic composite joint components and ductility criteria

To evaluate the strength and stiffness of a joint using the component method, it is necessary to identify the basic components that affect its behaviour, characterizing their stiffness k_i and resistance $F_{Rd,i}$. As mentioned above, besides the steel components, a steel-concrete composite joint is influenced:

- by the longitudinal reinforcement placed in the slab;
- by the interactions between the concrete slab and the steel parts.

Consequently, in addition to the bare steel components (defined in EC3 1-8), it is necessary to consider other components: the longitudinal rebars in tension and the concrete slab in compression (as well as to modify the "standard" components influenced by the presence of the concrete).

3.2.1 Longitudinal steel reinforcement in tension

- *Resistance*

Both in an interior joint or in an exterior joint the traction that could be present in the longitudinal rebars is balanced by the strut-and-tie mechanisms occurring in the side of the column opposite to the traction (mechanisms 1 and 2). For an elastic internal joint stressed on both sides, the distribution of stresses within the slab is shown in Figure 9 where the bending moment of the left side $M_{Ed,l}$ is expressed as a function of the bending moment present on the right side $M_{Ed,r}$ (suppose in this case $M_{Ed,r} < M_{Ed,l}$, considering the sign) through the transformation parameter β defined as (neglecting the shear effect):

$$\beta_r = 1 - \frac{M_{Ed,l}}{M_{Ed,r}} \tag{17}$$

$$\beta_l = \frac{M_{Ed,r}}{M_{Ed,l}} - 1 \tag{18}$$

In which the bending moments have positive sign when they compress the slab and negative when they tend the slab.

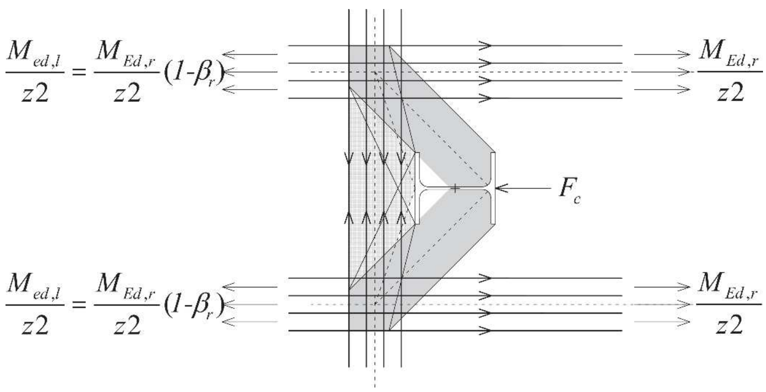


Figure 9. Elastic force distribution in a composite joint ($M_{Ed,r} < M_{Ed,l}$)

The difference between the forces present in the slab (see Figure 9, where z represents the internal lever arm) due to the different value of the internal bending moments acting on the left and right side of the joint generates a compression force F_c which is transmitted to column through the development of the struts. In the state of maximum exploitation of mechanisms 1 and 2 (Figure 10), the traction on the right-hand side ($F_{t,r}$) and left ($F_{t,l}$) of the joint are obtained by imposing the force equilibrium on the basis of the resistances of the mechanisms present in the slab:

$$F_{t,r} = \frac{\sum F_{Rd,i}}{\beta_r} \tag{19}$$

$$F_{t,l} = \frac{\sum F_{Rd,i}}{\beta_r}(1-\beta_r) = \frac{\sum F_{Rd,i}}{\beta_l} \tag{20}$$

An upper limit to this force is represented by the strength of the armature A_s present in the slab within the effective width at negative bending moment b_{eff}^{-}. The maximum tension force transmitted by the slab, defined as the basic component "*longitudinal steel reinforcement in tension*", is thus:

$$F_{t,slab,Rd}^{r} = \min\left(\frac{\sum F_{Rd,i}}{\beta_r}; A_s^r f_{yd}\right) \geq 0 \tag{21}$$

$$F_{t,slab,Rd}^{l} = \min\left(\frac{\sum F_{Rd,i}}{\beta_l}; A_s^l f_{yd}\right) \geq 0 \tag{22}$$

Where A_s^r represents the rebars area on the right side of the joint and A_s^l the one on the left side.

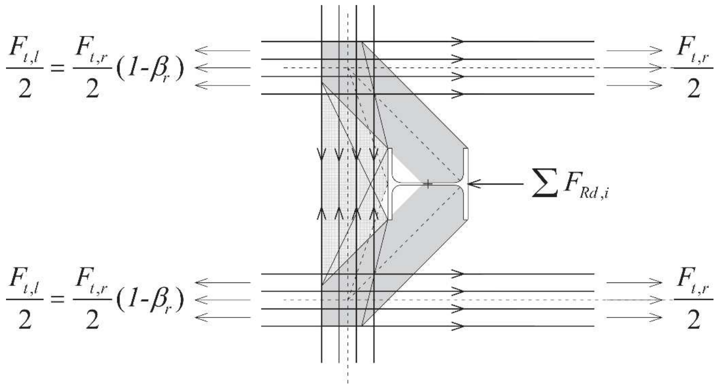


Figure 10. Maximum exploitation condition ($M_{Ed,r} < M_{Ed,l}$)

When the joint is stressed by negative moments, the formation of the mechanisms 1 and 2 is guaranteed only if the rebars in tension can be anchored over the column (cantilever or internal joint). Also when the negative bending moments are equal on both sides ($M_{Ed,r} = M_{Ed,l}$) the mechanisms are not activated because the forces in the slab are in equilibrium. In this case the parameter β is equal to zero and the components " *longitudinal steel reinforcement in tension* " are:

$$F_{t,slab,Rd}^{r} = A_s^r f_{yd} \tag{23}$$

$$F_{t,slab,Rd}^{l} = A_s^l f_{yd} \tag{24}$$

Therefore, the basic component "*longitudinal rebars in tension*" is a function not only of the rebars placed on the slab but also of the nodal configuration (presence or not of different mechanisms) and of the bending moments (via the parameter β). It makes sense to evaluate this component exclusively on the joint side where a negative bending moment is applied. In general, to check the joint resistance, this component should be determined for each load combination. On the other hand, in an external joint, being one bending moments equal to zero, the maximum tolerable traction is independent of β and is equal to:

$$F^{r}_{t,slab,Rd} = \min\left(\sum F_{Rd,i}; A^{r}_{s} f_{yd}\right) \geq 0 \tag{25}$$

$$F^{l}_{t,slab,Rd} = \min\left(\sum F_{Rd,i}; A^{l}_{s} f_{yd}\right) \geq 0 \tag{26}$$

Under seismic loading, in order to provide adequate ductility to the joint, it would be desirable to achieve the yield of the longitudinal reinforcement before the concrete crushing. This condition is assured if, for each seismic load combination, the following inequalities are applied:

$$A^{r}_{s} \leq \left|\frac{\sum F_{Rd,i}}{\beta_r f_{yd}}\right| \tag{27}$$

$$A^{l}_{s} \leq \left|\frac{\sum F_{Rd,i}}{\beta_l f_{yd}}\right| \tag{28}$$

- *Stiffness*

The stiffness coefficient is defined by EC4 [§EC4 1-1 A.2.1.1], it can be expressed in the form:

$$k_{s,r,slip} = \frac{A_{sl}}{l_r} k_{slip} \tag{29}$$

Parameter l_r represents the "effective length" of the reinforcement, defined in the eurocode depending on the node configuration and loading condition. The values of this parameter suggested in the Eurocode are shown in Table 4 (§EC4-1-1 prospectus A1).

Table 1. Effective length of the longitudinal reinforcement

Nodal Configuration	Stresses	Effective Length
Exterior Joint		$l_r = 3.6h_c$
Interior Joint	$M_{Ed,r} = M_{Ed,l}$	$l_r = h_c / 2$
	$M_{Ed,r} > M_{Ed,l}$	• Joint with $M_{Ed,r}$: $l_r = h_c\left(\frac{1+\beta}{2} + k_\beta\right)$ $k_\beta = \beta\left(4.3\beta^2 - 8.9\beta + 7.2\right)$
		• Joint with $M_{Ed,l}$: $l_r = h_c\left(\frac{1-\beta}{2}\right)$

Other formulations have been proposed by various authors. In particular, according to Gil and Bayo (2007), the effective length is independent from the bending moments and is:

- For external joint:

$$l_r = \frac{h_c}{2} + 0.8z \tag{30}$$

- For interior joints:

$$l_r = 2\left(h_c + z\right) \tag{31}$$

where z represents the distance between the axis of the lower flange of the steel beam and the centre of gravity of the reinforcement (internal lever arm).
The k_{slip} stiffness factor considers the effect of the shear deformation of connectors. Its formulation is due to Aribert (1995):

$$k_{slip} = \frac{1}{1 + \frac{E_s\left(A_{sl} / l_r\right)}{k_{sc}}} \tag{32}$$

Where:

$$k_{sc} = \frac{Nk_{sc}}{v - \left(\frac{v-1}{1+\xi}\right)\frac{h_s}{d_s}} \tag{33}$$

$$v = \sqrt{\frac{\left(1+\xi\right)Nk_{sc}d_s^2 l}{E_s I_s}} \tag{34}$$

$$\xi = \frac{E_a I_a}{d_s^2 E_{sl} A_{sl}} \tag{35}$$

and:
h_s is the distance between the centre of gravity of the longitudinal reinforcement and the centre of compression;
d_s is the distance between the centre of gravity of the longitudinal reinforcement and the centre of gravity of the steel section;
I_a is the moment of inertia of the steel beam;
l is the length of the beam subjected to negative bending moment (conventionally the 15% of the span length);
N is the number of studs distributed over the length l;
k_{sc} is the stiffness of a single stud.

3.2.2 Concrete slab in compression

- *Resistance*

Under seismic loading, positive bending moment on one side and negative on the other side of the joint might occur. For each load combination that involves this behaviour, it is necessary to evaluate the positive resistant moment of the joint and hence to define the "*concrete slab in compression*" basic component. This component, defined as $F_{c,slab,Rd}$, is determined by evaluating the force equilibrium at the joint from the knowledge of the basic component "*longitudinal reinforcement in tension*" $F_{t,slab,Rd}$ and the sum of the resistances of Mechanisms $\Sigma F_{Rd,i}$ (Figure 11):

$$F^{r}_{c,slab,Rd} = \sum F_{Rd,i} - F^{l}_{t,slab,Rd} \tag{36}$$

$$F^{l}_{c,slab,Rd} = \sum F_{Rd,i} - F^{r}_{t,slab,Rd} \tag{37}$$

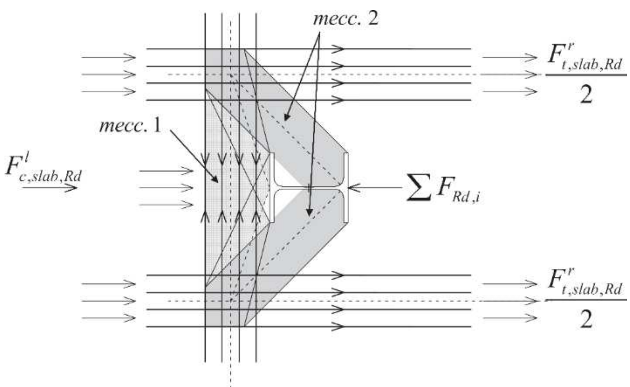


Figure 11. Stresses distribution due to seismic action

The component $F_{c,slab,Rd}$ can be derived directly from equations 19 and 20. In seismic conditions, the stressing bending moments can assume opposite sign and consequently the transformation factor β_r assumes a positive value greater than one. By replacing this value in equations 19 and 20, the forces on the right and left side of the column have the same direction (one induces traction and the other compression). However, such equations are valid

only until $F_{t,slab,Rd}$ is governed by the resistance of the mechanisms. To consider also the possibility that $F_{t,slab,Rd}$ is governed by the reinforcement resistance $F_{c,slab,Rd}$ is then defined by equations 36 and 37.
$F_{c,slab,Rd}$ is a function of the longitudinal reinforcement in tension Therefore, it is also a function of β and must be evaluated for each load combination that provides a positive bending moment at the beam-to-column intersection (on the side where that moment acts).

- *Stiffness*

Codes currently give no indications on the stiffness of the slab mechanisms. It can be assumed equal to (Amadio *et al.*, 2014):

$$k_c = k_{c,1} + k_{c,2} \tag{38}$$

This stiffness coefficient is defined from the stiffness of mechanisms 1 and 2 acting in parallel. The coefficients $k_{c,1}$ and $k_{c,2}$ of the mechanisms are respectively (Amadio *et al.*, 2010) (Bella, 2009):

- Mechanism 1:

The stiffness coefficient is calculated by considering a strut length of h_c (Figure 12). Under this hypothesis, the stiffness coefficient of mechanism 1 is calculated as:

$$k_{c,1} = \frac{A_c E_{cm}}{h_c E_s} = \frac{(b_b h_c) E_{cm}}{h_c E_s} \tag{39}$$

Where E_{cm} is the concrete elastic modulus whereas E_s the steel ones.

- Mechanism 2:

with reference to Figure 13, the stiffness coefficient of the mechanism 2 can be set as:

$$k_{c,2} = \frac{2}{\dfrac{1}{k_t\left(\dfrac{l_y}{l_x}\right)^2} + \dfrac{1}{k_p\left(1+\left(\dfrac{l_y}{l_x}\right)^2\right)}} = \frac{2}{\dfrac{1}{k_t} + \dfrac{1}{2k_p}} \tag{40}$$

Where the stiffness coefficient of the single strut k_p and tie k_t is assumed:

$$k_t = \frac{A_T}{l_y} = \frac{A_T}{1.5h_c} \tag{41}$$

$$k_p = \frac{A_p}{\sqrt{l_x^2 + l_y^2}} \frac{E_{cm}}{E_s} = \frac{0.7 h_c d_{eff}}{\sqrt{1.5h_c^2 + 1.5h_c^2}} \frac{E_{cm}}{E_s} \tag{42}$$

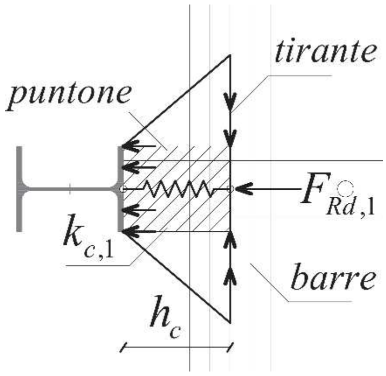


Figure 12. Mechanism 1

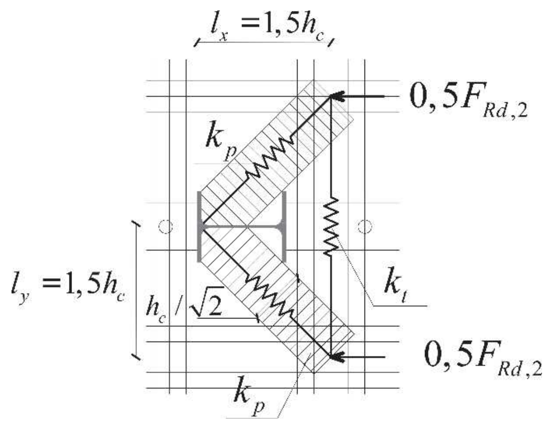


Figure 13. Mechanism 2

3.3 Design of the slab according to EC8

The possible nodal configurations in the case of exterior or interior joint are described in the following, reporting their behaviour according to the bending moments and the necessary checks to ensure the hierarchy of resistance between the beam and the joint.

3.3.1 External joint

The possible configurations that an external composite joint (located on the perimeter of a three-dimensional frame) may have and the design of the slab are summarized in Table 2. As shown in 3.2.2, the maximum compressive force transmitted through the slab in an external joint is given, depending on the nodal configuration, by the sum of the resistances of the various mechanisms since $F_{t,slab,Rd}$ is equal to zero. Considering the case where an external edge strip is present and the absence of a transverse beam (construction detail "b" of Table 2), the maximum compression force is given by mechanisms 1 and 2:

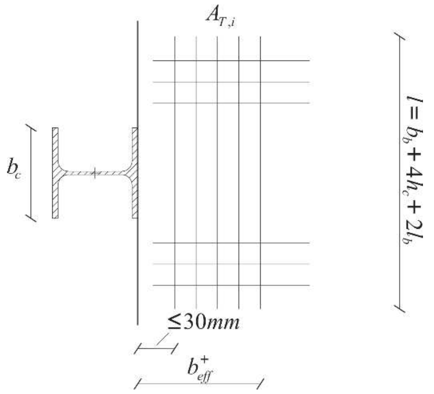
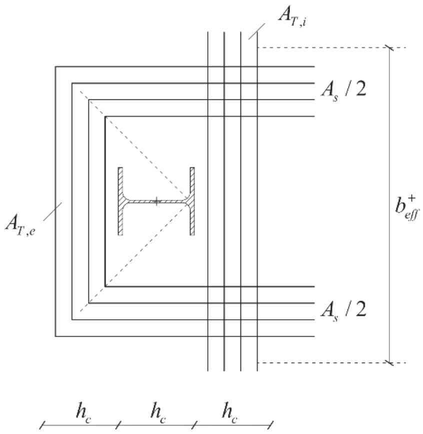
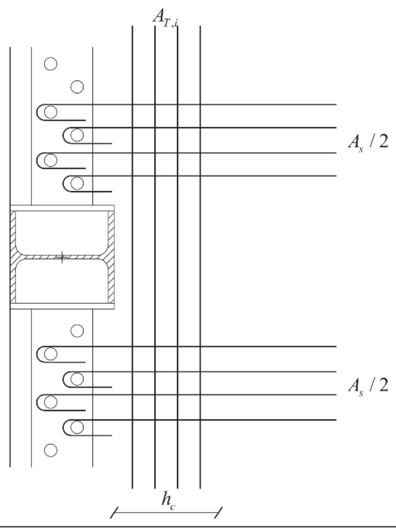
$$F_{Rd,1} + F_{Rd,2} = (b_b + 0.7h_c)d_{eff}(0.85f_{ck} / \gamma_c) \tag{43}$$

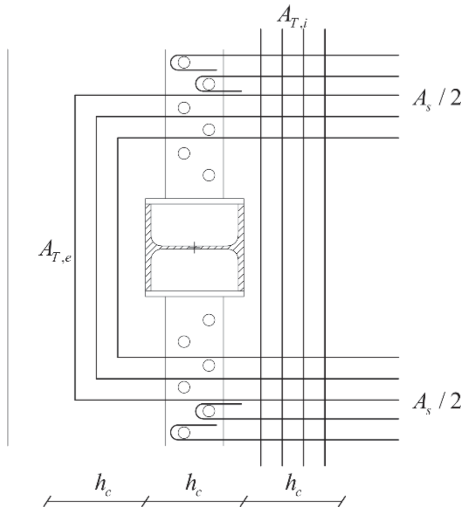
This condition corresponds to an effective connection width of:

$$b^{+}_{eff,conn} = b_b + 0.7h_c \tag{44}$$

Such width corresponds to the maximum effective width at a positive moment for such node configuration. It is thus noted that the maximum compressive force that can be transmitted by the connection at the level of the slab is the same of that transmitted by the beam.

Table 2. Slab design for an external joint

Nodal Configuration	Slab design
a) No transversal beam; No cantilever edge strip  	$F_{c,slab,Rd} = \sum F_{Rd,i} = F_{Rd,1}$ $F_{t,slab,Rd} = 0$ $A_T = A_{T,1} \geq 0.25 b_b d_{eff} \dfrac{0.15l - b_b}{0.15l} \left(\dfrac{0.85 f_{ck} / \gamma_c}{f_{yk,T} / \gamma_s} \right)$
b) Edge strip beam, No transversal beam  	$F_{c,slab,Rd} = \sum F_{Rd,i} = F_{Rd,1} + F_{Rd,2}$ $F_{t,slab,Rd} = \min\left[\left(A_{s1} + A_{s2} \right) f_{yd} ; \left(F_{Rd,1} + F_{Rd,2} \right) \right]$ $A_{T,i} = A_{T,1} + A_{T,2}$ $A_{T,e} \geq \dfrac{A_{sl}}{2}$
c) Transversal beam, No edge strip  	$F_{c,slab,Rd} = \sum F_{Rd,i} = F_{Rd,1} + F_{Rd,2} + F_{Rd,3}$ $F_{t,slab,Rd} = \min\left[A_{s3} f_{yd} ; \sum F_{Rd,i} \right]$ $A_{T,i} = A_{T,1} + A_{T,2}$

Nodal Configuration	Slab design
d) Transversal beam, edge strip 	$F_{c,slab,Rd} = \sum F_{Rd,i} = F_{Rd,1} + F_{Rd,2} + F_{Rd,3}$ $F_{t,slab,Rd} = \min\left[\left(A_{s1} + A_{s2} + A_{s3}\right) f_{yd} ; \sum F_{Rd,i}\right]$ $A_{T,i} = A_{T,1} + A_{T,2}$ $A_{T,e} \geq \frac{A_{s1} + A_{s2}}{2}$

This means that to achieve an overstrength of the joint with respect to the beam, unless there are very short spans, the transverse beam must be introduced (construction detail "c" in Table 2), choosing appropriately the number of connectors *n* to fit within the effective width.

3.3.2 Interior joint

The possible configurations that an interior composite joint can assume, the design of the slab and the hierarchy checks are summarized in Table 3.

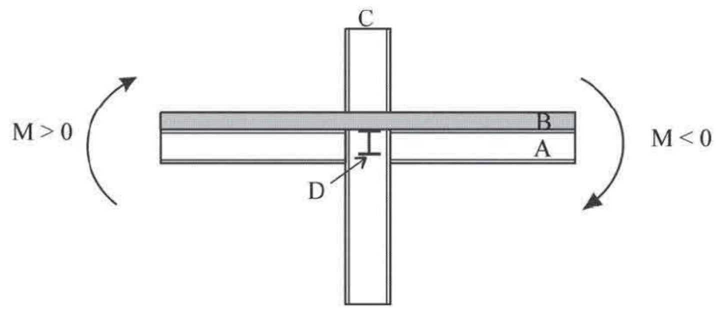


Figure 14. Interior joint subjected to seismic loading

An interior joint subjected to an earthquake can experience at the same time a positive bending moment on one side that generates within the slab a tensile force F_{st} and from a negative bending moment on the other side that generates within the slab a compression force F_{sc}. In an interior node, it is therefore necessary to consider the interaction between the connections on both sides of the column due to the continuity of the slab and the longitudinal reinforcement. In particular, the negative resistant moment on one side depends on the ability of the concrete to resist compression stresses on the other side. The direct consequence of this consideration is that the forces F_{sc} and F_{st} can only be equilibrated by mechanisms 1 and 2 and, when the transverse beam is present, by the mechanism 3. In seismic conditions, the joint can at most be stressed by the bending resistant moments on each side. Under these assumptions the compressive and tensile forces on the slab assume their maximum value which is equal to:

$$F_{sc} = b_{eff}^{+} d_{eff} \left(0.85 f_{ck} / \gamma_c\right) \tag{45}$$

$$F_{st} = A_{sl} \left(f_{yk,l} / \gamma_s\right) \tag{46}$$

Where A_{sl} represents the longitudinal reinforcement placed within b_{eff}^{-}. Such forces must be equilibrated by the mechanisms. To ensure that such forces can be transferred, the maximum compressive force transmitted by the mechanisms should be greater than the sum of the forces F_{sc} and F_{st}. To prevent the concrete crushing and to ensure that the failure occurs with the yield of the reinforcement of the slab on one side and with the yield of the lower flange of the steel beam on the other side, it is necessary to verify that:

- In absence of transversal beam:

$$1.2\left(F_{sc} + F_{st}\right) \le F_{Rd,1} + F_{Rd,2} \tag{47}$$

- With transversal beam:

$$1.2\left(F_{sc} + F_{st}\right) \le F_{Rd,1} + F_{Rd,2} + F_{Rd,3} \tag{48}$$

In the latter, the struts are activated by compression of the concrete on the studs due to the application of a positive bending moment, so the number of studs involved in mechanism 3 is evaluated by considering the effective width under positive bending b_{eff}^{+}. In seismic area, the effective width used to evaluate the resistant positive bending moment of an internal joint is equal to $b_{eff}^{+} = 0.15l$. Considering that in design practice $b_b = b_c \cong h_c$ and that on average $h_c \cong 0.05l$, it results:

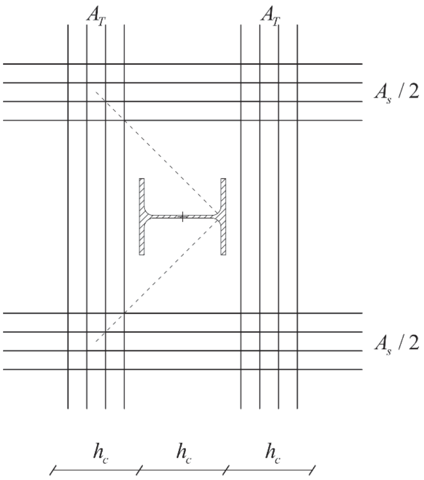
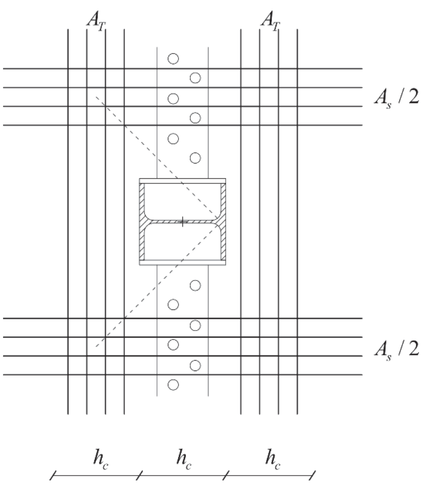
$$b_{eff,conn}^{+} = b_b + 0.7h_c \cong 1.7h_c \cong 0.085l \ll b_{eff}^{+} = 0.15l \tag{49}$$

Therefore, an overstrength of the joint with respect to the beam is possible only in the presence of:

- oversize of the column in very short spans;
- mechanism 3.

It follows that it is usually necessary to introduce the transverse beam and appropriately selecting the number of connectors n to be placed within the effective width.

Table 3. Slab design for an interior joint

Nodal Configuration	Slab design
a) No transversal beam 	1) $M_{Ed,r} \neq M_{Ed,l}$, both negative $F^r_{t,slab,Rd} = \min\left(\frac{F_{Rd,1}+F_{Rd,2}}{\beta_r}; A^r_s f_{yd}\right)$ 2) $M_{Ed,r} = M_{Ed,l}$, both negative $F^r_{t,slab,Rd} = A^r_s f_{yd}$; $F^l_{t,slab,Rd} = A^l_s f_{yd}$ 3) $M_{Ed,r}$ negative and $M_{Ed,l}$ positive $F^l_{c,slab,Rd} = F_{Rd,1}+F_{Rd,2}-F^r_{t,slab,Rd}$ $F^r_{t,slab,Rd} = \min\left(\frac{F_{Rd,1}+F_{Rd,2}}{\beta_r}; A^r_s f_{yd}\right)$ 4) $M_{Ed,r}$ positive and $M_{Ed,l}$ negative $F^r_{c,slab,Rd} = F_{Rd,1}+F_{Rd,2}-F^l_{t,slab,Rd}$ $F^l_{t,slab,Rd} = \min\left(\frac{F_{Rd,1}+F_{Rd,2}}{\beta_l}; A^l_s f_{yd}\right)$ 5) For every case $A_T = A_{T,1}+A_{T,2}$ $1.2\left(F_{sc}+F_{st}\right) \leq F_{Rd,1}+F_{Rd,2}$
b) Edge strip beam, No transversal beam 	1) $M_{Ed,r} \neq M_{Ed,l}$, both negative $F^r_{t,slab,Rd} = \min\left(\frac{F_{Rd,1}+F_{Rd,2}+F_{Rd,3}}{\beta_r}; A^r_s f_{yd}\right)$ 2) $M_{Ed,r} = M_{Ed,l}$, both negative $F^r_{t,slab,Rd} = A^r_s f_{yd}$; $F^l_{t,slab,Rd} = A^l_s f_{yd}$ 3) $M_{Ed,r}$ negative and $M_{Ed,l}$ positive $F^l_{c,slab,Rd} = F_{Rd,1}+F_{Rd,2}+F_{Rd,3}-F^r_{t,slab,Rd}$ $F^r_{t,slab,Rd} = \min\left(\frac{F_{Rd,1}+F_{Rd,2}+F_{Rd,3}}{\beta_r}; A^r_s f_{yd}\right)$ 4) $M_{Ed,r}$ positive and $M_{Ed,l}$ negative $F^r_{c,slab,Rd} = F_{Rd,1}+F_{Rd,2}+F_{Rd,3}-F^l_{t,slab,Rd}$ $F^l_{t,slab,Rd} = \min\left(\frac{F_{Rd,1}+F_{Rd,2}+F_{Rd,3}}{\beta_l}; A^l_s f_{yd}\right)$ 5) For every case should be checked: $A_T = A_{T,1}+A_{T,2}$ $1.2\left(F_{sc}+F_{st}\right) \leq F_{Rd,1}+F_{Rd,2}+F_{Rd,3}$

3.4 *Moment-rotation curve*

The behaviour of a joint is described by its moment-rotation curve (Fasan, 2013), (Pecce and Rossi, 2015), which depends on the stiffness and resistance of its basic components. Generally steel-concrete composite joints are non-symmetric and therefore their active components are function of the bending moment (Figure 15). A composite joint is hence characterized by a different moment-rotation curve according to the sign of the bending moment.

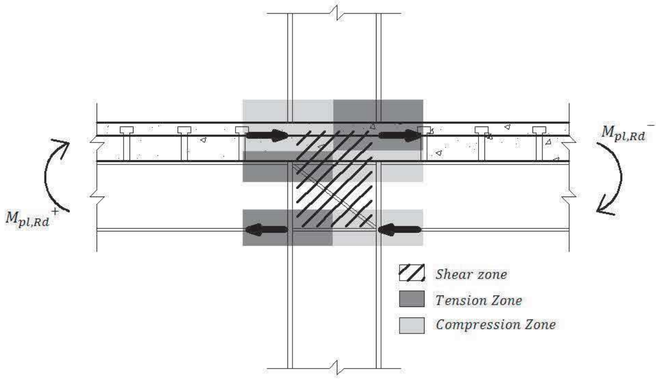


Figure 15. Interior joint subjected to seismic loading

Generally, for each row of basic components, it is possible to obtain its effective strength through equilibrium considerations between the various resistances. The basic components are a function of the node type. The joint resistant moment is equal to the product between the effective strength $F_{r,Rd}$ that can be developed, each with its own sign, and their arm h_r with respect to a point belonging to the joint:

$$M_{j,pl,Rd} = \sum_{i=1}^{r} F_{i,Rd} h_r \tag{50}$$

Stiffness is also asymmetrical. In general, to obtain the moment-rotation curve, a basic mechanical model needs to be defined. The following describes the procedure to be used to design and verify welded and bolted joints. For more details on the component method and other node types refer to EC3 1-8. The general steps to follow are:

- Identification of active components in the tension, shear and compression;
- Identifying the strength and stiffness of each component;
- Assembling the components and defining the moment-rotation curve of the joint via equilibrium considerations;
- Verification of welds (beam-to-column welds in the case of welded joints and between the plate and the beam in the case of bolted joints);
- Checks of bolts resistance.

Bolts, welds, plate and column flange are seized to transfer the stresses deriving from hierarchy criteria. The basic components of a welded and bolted joint with extended end-plate are listed in Table 4 and Table 5 respectively. The procedures to be followed for the determination of the resistant moment differ according to the moment sign.

Table 4. Basic components for a welded composite joint

Zone	Basic Component	Resistance	Stiffness
Tension	Column flange in transverse bending [§EC3 1-8 6.2.6.4]	$F_{fc,Rd}$	-
	Column web in transverse tension [§EC3 1-8 6.2.6.3]	$F_{t,wc,Rd}$	k_3
	Longitudinal reinforcement in tension [sec. 3.2.1]	$F_{t,slab,Rd}$	$k_{s,r,slip}$
Compression	Column web in transverse compression [§EC3 1-8 6.2.6.2]	$F_{c,wc,Rd}$	k_2
	Concrete slab in compression [sec. 3.2.2]	$F_{c,slab,Rd}$	k_c
	Beam flange and web in compression [§EC3 1-8 6.2.6.7]	$F_{c,fb,Rd}$	-
Shear	Column web panel in shear [§EC3 1-8 6.2.6.1]	$V_{wp,Rd}$	k_1

Table 5. Basic components for an extend end plate bolted composite joint

Zone	Basic Component	Resistance	Stiffness
Tension	Bolts in tension [§EC3 1-8 Table 3.4]	$F_{t,Rd}$	k_{10}
	Bolts in shear [§EC3 1-8 Table 3.4]	$F_{v,Rd}$	k_{11}
	Bolts in bearing [§EC3 1-8 Table 3.4]	$F_{b,Rd}$	k_{12}
	Punching shear resistance [§EC3 1-8 Table 3.4]	$B_{p,Rd}$	-
	Column web in transverse tension [§EC3 1-8 6.2.6.3]	$F_{t,wc,Rd}$	k_3
	Column flange in transverse bending [§EC3 1-8 6.2.6.4]	$F_{t,fc,Rd}$	k_4
	End-plate in bending [§EC3 1-8 6.2.6.5]	$F_{t,ep,Rd}$	k_5
	Beam web in tension [§EC3 1-8 6.2.6.8]	$F_{t,wb,Rd}$	-
	Longitudinal reinforcement in tension [sec. 3.2.1]	$F_{t,slab,Rd}$	$k_{s,r,slip}$
Compression	Column web in transverse compression [§EC3 1-8 6.2.6.2]	$F_{c,wc,Rd}$	k_2
	Concrete slab in compression [sec 3.2.2]	$F_{c,slab,Rd}$	k_c
	Beam flange and web in compression [§EC3 1-8 6.2.6.7]	$F_{c,fb,Rd}$	-
Shear	Column web panel in shear [§EC3 1-8 6.2.6.1]	$V_{wp,Rd}$	k_1

3.4.1 Negative bending moment

The design negative resistant moment of the joint is obtained from the limitation imposed by the compression resistance at the lower flange. It is assumed that the neutral axis cannot move closer to the slab as the lower flange instability could prevent it (safety criterion). The steps to follow to define the resistant negative moment of a welded joint are:

a) calculate the resistance of each component;
b) find the maximum resistance that can be developed at each level, namely:

- the maximum compressive strength that can be developed at the lower flange of the beam $F_{c1,Rd}$, given by:

$$F_{c1,Rd} = \min\left(F_{c,wc,Rd}; V_{wp,Rd} / \beta\right) \tag{51}$$

- maximum tensile strength at the rebars level:

$$F_{t1,\max} = \min\left(F_{t,slab,Rd};V_{wp,Rd} / \beta\right) \tag{52}$$

- the maximum tensile resistance that can be developed at the top flange of the beam:

$$F_{t2,\max} = \min\left(F_{t,wc,Rd};F_{fc,Rd};V_{wp,Rd} / \beta\right) \tag{53}$$

the column flange in transverse bending is taken into account only if the joint is not stiffened with continuity plates;

c) define the effective strength of each level $F_{tr,Rd}$ approaching the flange in compression starting from the farthest point and by interrupting the procedure when the sum of the effective resistances equals the maximum compression resistance $F_{c1,Rd}$.

To perform point c), whether it is a welded joint or a bolted joint with end-plate, the first row in tension is at the level of the rebars. Such resistance shall not exceed maximum compressive strength that can be developed at the lower flange:

$$F_{t1,Rd} = \min\left(F_{t1,max};F_{c1,Rd}\right) \tag{54}$$

In order to obtain a ductile behaviour, the rebars at the beam-to-column intersection should be designed in order to yield before the maximum strength of the mechanisms is reached as described in 3.2.1. Moreover, rebars should yield before the maximum compression strength of the joint $F_{c1,Rd}$ is achieved. This is assured if the following inequality is true:

$$F_{t,slab,Rd} = A_s f_{yd} \le F_{c1,Rd} \tag{55}$$

The effective strength of the other rows in tension differs depending on whether the joint is welded or bolted with extend end-plate. In welded joints, the second (and last) row in tension is placed at the upper flange of the beam. The direction and intensity of the force that can be developed by this row $F_{t2,Rd}$ is determined by force equilibrium. $F_{t2,Rd}$ in a welded joint can then be set equal to:

$$F_{t2,Rd} = \min\left(F_{t2,max};\left(F_{c1,Rd} - F_{t1,Rd}\right)\right) \tag{56}$$

In a joint where the ductility criterion expressed in equation 55 is respected, $F_{t2,Rd}$ is always a tensile force or at most null. The procedure described so far can be summarized in Figure 16. With reference to this figure, the negative resistant moment can be set equal to (the centre of the lower flange is chosen as pole but any pole can be used):

$$M^{-}_{j,pl,Rd} = F_{t1,Rd} h_{t1} + F_{t2,Rd} h_{t2} \tag{57}$$

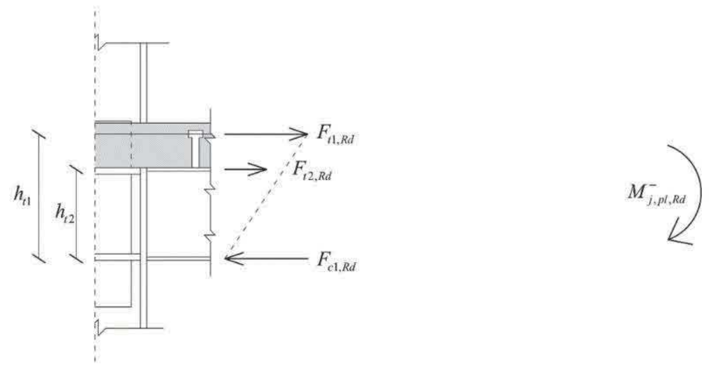


Figure 16. Force distribution in a welded joint under negative moment

If the joint is bolted, it is necessary to identify the effective resistance of each row in tension $F_{tr,Rd}$, equal to the lower between the resistance of the row taken individually $F_{tr,Rd,alone}$ or as part of a group of bolts $F_{tr,Rd,gr}$ (if this mechanism is possible) (EC3 1-8 6.2.7.2). The following basic components should be considered:

- column web in transverse tension $F_{t,wc,Rd}$;
- column flange in transverse bending $F_{t,fc,Rd}$;
- end-plate in bending $F_{t,ep,Rd}$;
- beam web in tension $F_{t,wb,Rd}$;

The procedure for calculating the maximum effective resistances that can be developed is performed in accordance with point c) and can be summarized as follows:

$$\begin{aligned}
&-F_{t1,Rd} = \min\left(F_{t1,max}; F_{c1,Rd}\right);\\
&-F_{t2,Rd} = \min\left(F_{t2,Rd,alone}; \left(F_{c1,Rd} - F_{t1,Rd}\right)\right);\\
&-F_{t3,Rd} = \min\left(F_{t3,Rd,alone}; \left(F_{t,Rd,gr2-3} - F_{t2,Rd}\right); \left(F_{c1,Rd} - F_{t1,Rd} - F_{t2,Rd}\right)\right);\\
&-F_{t4,Rd} = \min\begin{pmatrix} F_{t4,Rd,alone}; \left(F_{t,Rd,gr3-4} - F_{t3,Rd}\right); \left(F_{t,Rd,gr2-3-4} - F_{t2,Rd} - F_{t3,Rd}\right);\\ \left(F_{c1,Rd} - F_{t1,Rd} - F_{t2,Rd} - F_{t3,Rd}\right)\end{pmatrix};\\
&-\ldots;\\
&-F_{tn,Rd} = \ldots;
\end{aligned} \tag{58}$$

The previous procedure leads to a plastic distribution of effective forces. This distribution can only be developed if the joint has adequate ductility (T-stub failure mode 1 or 2). Otherwise, a linear distribution is required. In particular, if the resistance of a row $F_{tx,Rd}$ exceeds the value $1{,}9F_{t,Rd}$ (design tension resistance of the bolts), the further rows resistance must defined using the following inequality:

$$F_{tr,Rd} \le F_{tx,Rd}\frac{h_r}{h_x} \tag{59}$$

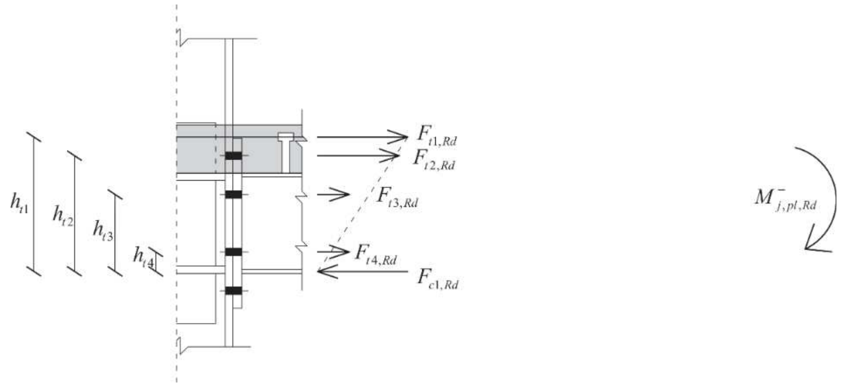


Figure 17. Force distribution in a bolted end-plate joint under negative moment

Referring to Figure 17, the negative resistant moment is equal to (the centre of gravity of the compressed flange is chosen as the pole):

$$M^-_{j,pl,Rd} = F_{t1,Rd}h_{t1} + F_{t2,Rd}h_{t2} + F_{t3,Rd}h_{t3} + F_{t4,Rd}h_{t4} \tag{60}$$

3.4.2 Positive bending moment

When the joint is subject to a positive bending moment the concrete slab is compressed. If there is a full-strength shear connection, this will allow the neutral axis to move towards the metallic components as the moment increases. Thanks to the studs, the instability of the compressed flange is also prevented. Based on these considerations, the positive resistant moment of the joint can be evaluated as follows:

a) calculate the resistance of each component;
b) find the maximum resistance that can be developed at each level, namely:

- the maximum compressive strength of the slab $F_{c1,max}$, given by:

$$F_{c1,\max} = \min\left(F_{c,slab,Rd}; V_{wp,Rd} / \beta\right) \tag{61}$$

- the maximum compressive strength that can be developed at the top flange of the beam $F_{c2,max}$, given by:

$$F_{c2,\max} = \min\left(F_{c,wc,Rd}; V_{wp,Rd} / \beta\right) \tag{62}$$

- maximum compression strength $F_{c,max}$:

$$F_{c,\max} = \min\left(F_{c1,max} + F_{c2,max}; V_{wp,Rd} / \beta\right) \tag{63}$$

- the maximum tensile resistance that can be developed at the lower flange of the beam:

$$F_{t1,\max} = \min\left(F_{t,wc,Rd}; F_{fc,Rd}; V_{wp,Rd} / \beta\right) \tag{64}$$

  the column flange in transverse bending is considered only if the joint is not stiffened with continuity plates;

c) define the effective strength of each level $F_{tr,Rd}$ approaching the flange in compression starting from the farthest point and by interrupting the procedure when the sum of the effective resistances equals the maximum compression resistance $F_{c,max}$;

d) define, via force equilibrium, the effective compression strength of the concrete slab $F_{c1,Rd}$ and of top flange $F_{c2,Rd}$:

$$F_{c1,Rd} = \min\left(\sum F_{tr,Rd}; F_{c1,\max}\right) \tag{65}$$

$$F_{c2,Rd} = \min\left(\sum F_{tr,Rd} - F_{c1,Rd}; F_{c2,\max}\right) \tag{66}$$

In a welded joint $\Sigma F_{tr,Rd}$ coincides with the maximum strength that can be developed at the bottom flange of the beam. Such resistance must not exceed the maximum compressive strength that can be developed:

$$F_{t1,Rd} = \min\left(F_{t1,max}; F_{c,\max}\right) \tag{67}$$

The procedure described so far can be summarized in Figure 18. With reference to this figure, the positive resistant moment can be set equal to (the pole is the centre of the lower flange):

$$M^{+}_{j,pl,Rd} = F_{c1,Rd} h_{c1} + F_{c2,Rd} h_{c2} \tag{68}$$

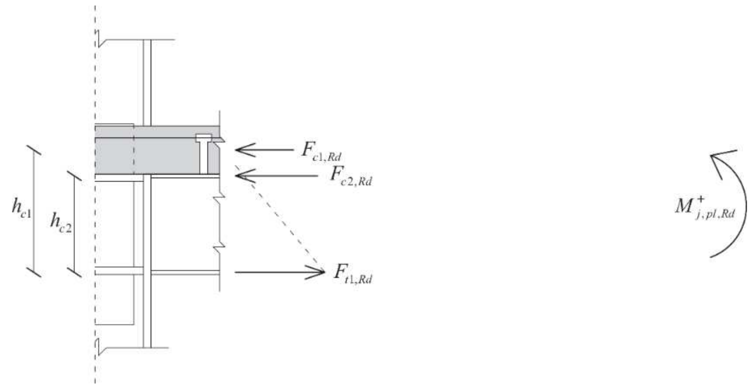


Figure 18. Force distribution in a welded joint under positive moment

If the joint is bolted, it is necessary to identify the effective resistance of each row in tension $F_{tr,Rd}$, equal to the lower between the resistance of the row taken individually $F_{tr,Rd,alone}$ or as part of a group of bolts $F_{tr,Rd,gr}$ (if this mechanism is possible) (EC3 1-8 6.2.7.2). The following basic components should be considered:

- column web in transverse tension $F_{t,wc,Rd}$;
- column flange in transverse bending $F_{t,fc,Rd}$;
- end-plate in bending $F_{t,ep,Rd}$;
- beam web in tension $F_{t,wb,Rd}$;

The procedure for calculating the maximum effective resistances that can be developed is performed in accordance with point c) and can be summarized as follows:

$$
\begin{aligned}
&-F_{t1,Rd} = \min\left(F_{t1,Rd,alone}; F_{c,\max}\right);\\
&-F_{t2,Rd} = \min\left(F_{t2,Rd,alone}; \left(F_{t,Rd,gr1-2} - F_{t1,Rd}\right); \left(F_{c,\max} - F_{t1,Rd}\right)\right);\\
&-F_{t3,Rd} = \min\begin{pmatrix} F_{t3,Rd,alone}; \left(F_{t,Rd,gr2-3} - F_{t2,Rd}\right); \left(F_{t,Rd,gr1-2-3} - F_{t1,Rd} - F_{t2,Rd}\right);\\ \left(F_{c,\max} - F_{t1,Rd} - F_{t2,Rd}\right)\end{pmatrix};\\
&-F_{t4,Rd} = \min\begin{pmatrix} F_{t4,Rd,alone}; \left(F_{t,Rd,gr3-4} - F_{t3,Rd}\right); \left(F_{t,Rd,gr2-3-4} - F_{t2,Rd} - F_{t3,Rd}\right);\\ \left(F_{t,Rd,gr1-2-3-4} - F_{t1,Rd} - F_{t2,Rd} - F_{t3,Rd}\right);\\ \left(F_{c,\max} - F_{t1,Rd} - F_{t2,Rd} - F_{t3,Rd}\right)\end{pmatrix};\\
&-...;\\
&-F_{tn,Rd} = ...;
\end{aligned}
\tag{69}
$$

Referring to Figure 19, the positive resistant moment is equal to:

$$M^{+}_{j,pl,Rd} = F_{c1,Rd}h_{c1} + F_{c2,Rd}h_{c2} - F_{t3,Rd}h_{t3} - F_{t2,Rd}h_{t2} \tag{70}$$

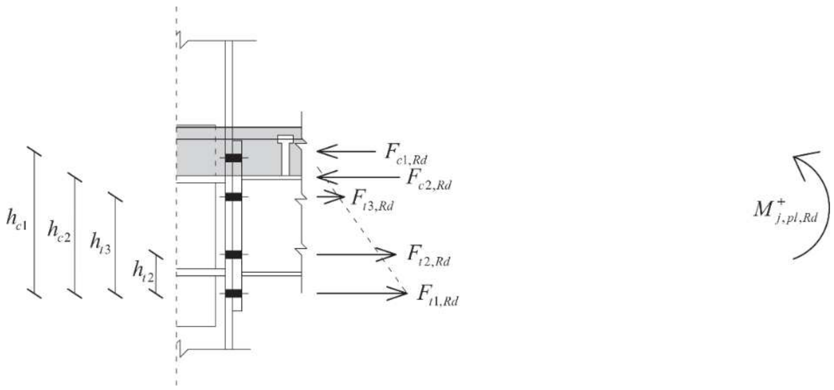


Figure 19. Force distribution in a bolted end-plate joint under positive moment

3.4.3 Stiffness

The stiffness is generally asymmetrical because, as seen so far, different resistant mechanisms intervene with their rigidities. In general, to obtain the moment-rotation curve, a basic mechanical model needs to be defined. The stiffness of the basic components that affect the moment-rotation curve are reported in Table 4 for a welded joint and Table 5 for a bolted joint with extended end-plate.

The basic mechanical models for a welded joint subjected to negative or positive bending moments are shown in Figure 20 and Figure 22 respectively.

The basic mechanical models for a bolted joint with extend end-plate subjected to negative or positive bending moments are shown in Figure 21 and Figure 23 respectively.

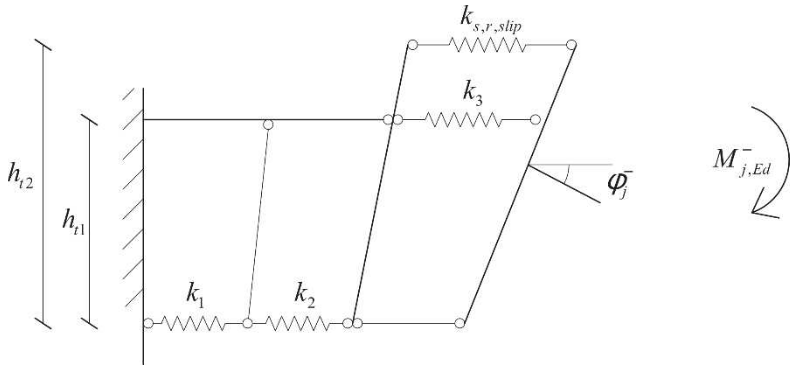


Figure 20. Mechanical model of a welded joint subject to negative moment

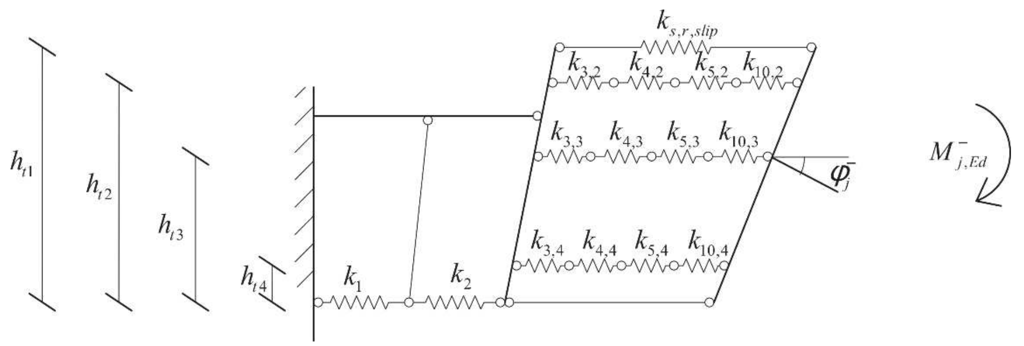


Figure 21. Mechanical model of a bolted joint under negative moment

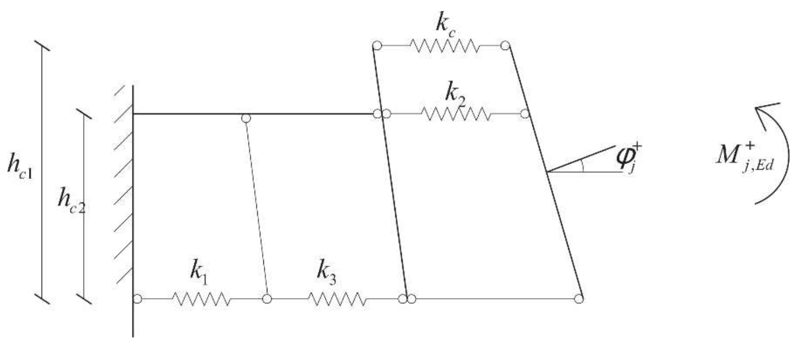


Figure 22. Mechanical model of a welded joint subject to positive moment

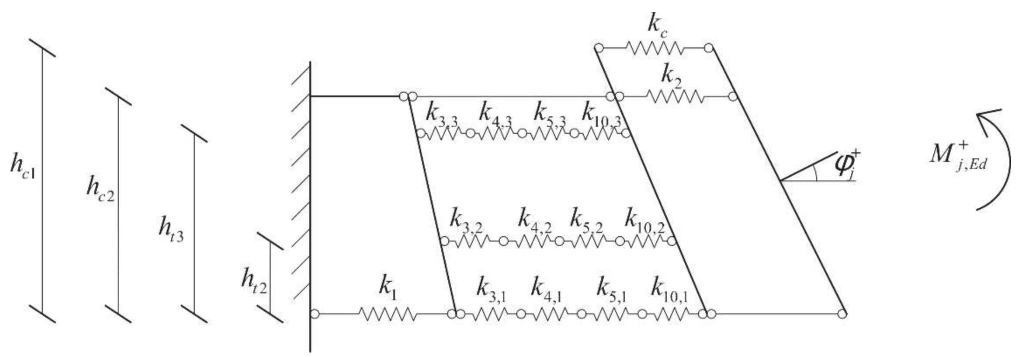


Figure 23. Mechanical model of a bolted joint under potive moment

The mechanical models could be further simplified defining the effective stiffness of each row. To find it, it is necessary to add in series the springs in the row *r* (the springs having the same lever arm). Such stiffness may be equal to:

$$k_{eff,r} = \frac{1}{\sum 1/k_{i,r}} \tag{71}$$

Once the basic mechanical models and the effective stiffness are defined, it is possible to determine the joint initial stiffness such as:

$$S_{j,ini} = E\left(-d_{CR}\sum\nolimits_{r=1}^{n} k_{eff,r}h_r + \sum\nolimits_{r=1}^{n} k_{eff,r}h_r^2\right) \tag{72}$$

Where d_{CR} represents the distance between the calculation pole and the centre of rotation:

$$d_{CR} = \frac{\sum_{r=1}^{n} k_{eff,r}h_r}{\sum_{r=1}^{n} k_{eff,r}} \tag{73}$$

The stiffness is set equal to $S_{j,ini}$ until the urgent moment does not exceed $2/3M_{j,pl,Rd}$. The secant stiffness, which detects the point of intersection between the moment-rotation curve of the joint and the point at which the plastic resistant moment of the joint $M_{j,pl,Rd}$ is reached, is assumed equal to $S_{j,ini}/\mu$. The stiffness ratio μ is defined in EC3 1-8 as:

$$\mu = \left(1.5\frac{M_{j,Ed}}{M_{j,pl,Rd}}\right)^{\psi} \tag{74}$$

The coefficient ψ depends on the type of joint. For a welded or bolted end-plate joint it assumes a value of 2.7. Rotations are obtained by dividing the moments for the corresponding stiffness:

$$\varphi_{j,1} = 2/3M_{j,pl,Rd}/S_{j,ini} \tag{75}$$

$$\varphi_{j,2} = M_{j,pl,Rd}/(S_{j,ini}/\mu) \tag{76}$$

The moment rotation curve is hence represented by a trilinear curve as in Figure 24.

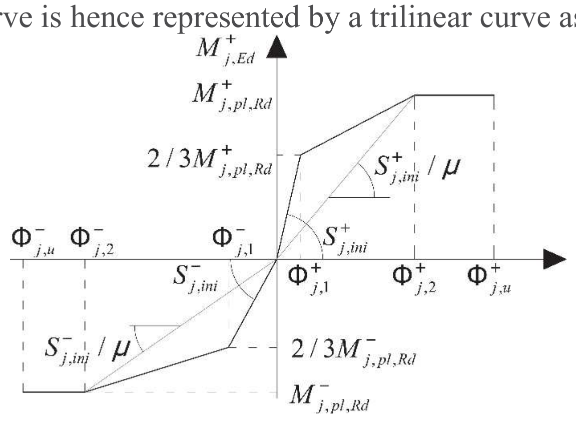


Figure 24. Moment-rotation curve

4 CONCLUSIONS

This paper briefly presents the main aspects concerning the design and behaviour of composite joints subjected to earthquake loading. It describes in detail the strut-and-tie mechanisms that activate in the slab at the beam-to-column intersection. Depending on the nodal configuration, the main design choices that a designer can perform in the cases of braced or moment-resisting frames are presented. In the latter case, the possibility of eliminating the interactions between the column and the slab solution by suitable construction details has also been highlighted. The information presented herein are shown in guidelines that allows to account for the complexity of the composite action.

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