INNOVATIVE BOLTED BEAM-TO-COLUMN JOINTS FOR SEISMIC RESISTANT BUILDING FRAMES

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ABSTRACT

In the present paper, innovative joint configurations leading to more economical solutions for full-strength beam-to-column joints for seismic building frames are detailed. The specificities of these joints are due to the fact that (i) the column is made of high strength steel while the beam are made of mild carbon steel and (ii) the design of some components allows partially neglecting the overstrength factor. Also, methods for the characterisation of specific joint components not directly covered by the Eurocode recommendations are proposed.

1. INTRODUCTION

According to Eurocode 8, earthquake resistant steel building frames shall be designed following either the "low dissipative structural behaviour concept" or the "dissipative structural behaviour concept". In the second concept, the ability of parts of the structure to resist earthquake actions through inelastic behaviour is taken into account: energy is dissipated in plastic mechanisms. In such a design, it has to be ensured that the dissipative zones form where they are intended to and that they yield before other zones leave the elastic range. In particular, moment resisting frames are designed in such a way that plastic hinges develop at the extremities of the beams. These dissipative zones can be located either in the beams or in the beam-to-column joints. In this paper, non-dissipative bolted beam-to-column connections are considered. They must be sufficiently resistant to remain in elastic range while cyclic yielding develops in the dissipative zones located in the beams. Besides, the possibility that the actual yield strength of the beam is higher than the nominal value has to be taken into account by a material overstrength factor. Such an approach generally leads to very strong and thus expensive joints.

In the present paper, a design strategy leading to more economical solutions for full-strength beam-to-column joints is detailed. This study is conducted within the framework of an RFCS project called HSS-SERF (High Strength Steel in Seismic Resistant Building Frames). The considered moment-resisting joints are part of seismic resistant building frames made of high strength steel composite columns and mild carbon steel beams. The columns are either partially-encased wide-flange columns (H columns) or concrete-filled rectangular hollow-section columns (RHS columns).

The proposed joint configuration uses hammer-heads extracted from the beam profile. To fulfil the resistance requirement taking account of the possible overstrength of the beam, the resistant moment of the joint is decomposed in the contributions of the different components involved. Then, no overstrength factor needs to be considered for the components related to the beam itself and to the hammer-heads. This approach is in full accordance with the basic principles of Eurocode 8 and can decrease much the required resistance of the joints provided some conditions are fulfilled, meaning lower costs.

Also, the chosen joint configurations involved joint components not directly covered by the Eurocode recommendations. Methods for the characterisation of these components are proposed and discussed within the present paper.

2. PROPOSED JOINT CONFIGURATIONS

2.1 Wide-flange column

In the present approach, the joints are designed to be non-dissipative, which means they have to be full-strength in such a way that the plastic hinge at a beam extremity will form in the beam itself while the joint remains elastic. Besides, the possible overstrength of the beam material has to be taken into account. This approach thus leads to very strong joints.

The proposed joint configuration when partially-encased H columns are used is represented in Figure 1. Hammer-heads and lateral plates welded from one flange to the other both sides of the column at the joint level are required to ensure a sufficient joint resistant moment. The hammer-heads have the effect of increasing the lever arm between the compression and tension forces within the joint and of reinforcing the end-plate submitted to bending. The lateral plates act as reinforcement for the following components: the column web panel in shear, the column flange in bending, the column web in tension and the column web in compression.

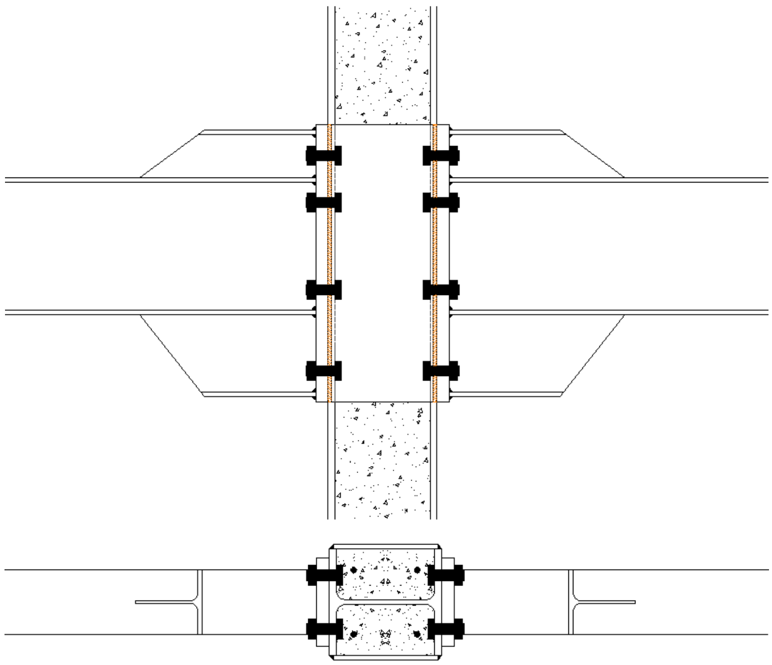

Figure 1. Joint configuration for a H column

In order to design as economical as possible full-strength joints fulfilling the resistance requirements of Eurocode 8 for non-dissipative connections, the hammer-heads have to be extracted from the same profile as the beam. The reason why this is important is explained in section 3.

The selection of this joint configuration results from a long process in which several other designs were investigated and appeared to be unsuitable, as explained in [4]. Two particular joints designed for the project HSS-SERF using the chosen configuration are also detailed in that document.

2.2 Rectangular hollow-section column

For concrete-filled RHS columns, the following joint configuration is proposed (Figure 2), in which the beam is fixed to the column via a U-shaped piece welded to the RHS column side walls. The bolted connection between the beam end-plate and the U front face is similar to the one proposed in 2.1 for H columns, and hammer-heads extracted from the beam profile are used.

This joint configuration as well as two particular joints designed for the project HSS-SERF are described in [5].

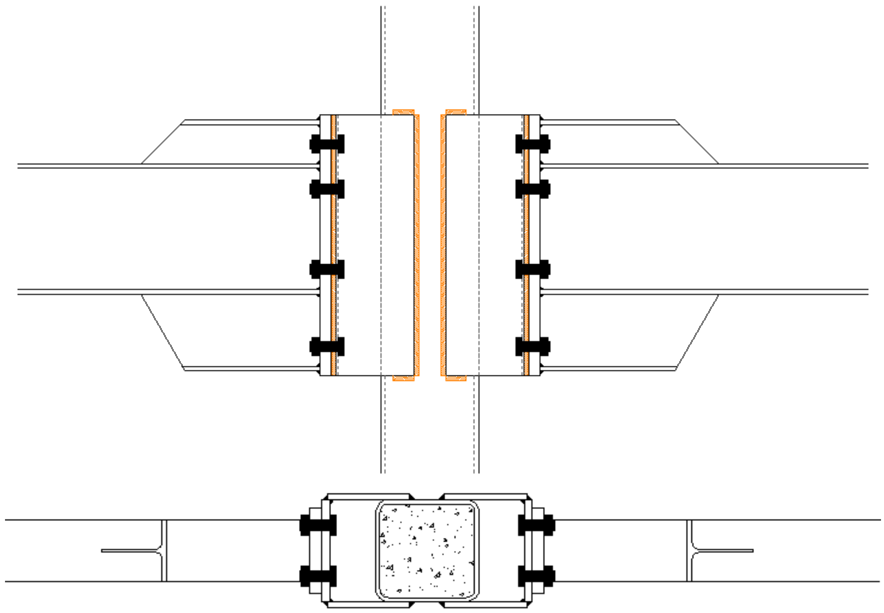

Figure 2. Joint configuration for a RHS column

3. DESIGN STRATEGY

In case of a seismic design in which it has to be ensured that the plastic hinges appear in the beams and not in the joints, the latter have to be over-resistant compared to the beams, taking account of the possible overstrength of the beams. Indeed, the actual resistance of the beam material may be higher than its nominal value. Accordingly, the following check has to be fulfilled (EN 1998-1 6.5.5 (3)):

$$M_{\mathrm{Rd,joint}} > 1{,}1 \cdot \gamma_{ov} \cdot M_{pl,beam} \tag{1}$$

Eurocode 8 suggests that the overstrength factor γ_{ov} be considered equal to 1,25.

Actually, this inequality is only valid provided the plastic hinge forms just next to the column flange so that the joint is subjected to approximately $M_{pl,beam}$. But it will not be the case for the joint configurations that are under consideration here due to the hammer-heads reinforcing the beam in the vicinity of the joint. Consequently, it has to be taken into account that the moment in the joint is greater than the one acting in the beam cross section after the hammer-heads, where the plastic hinge is meant to appear (see Figure 3).

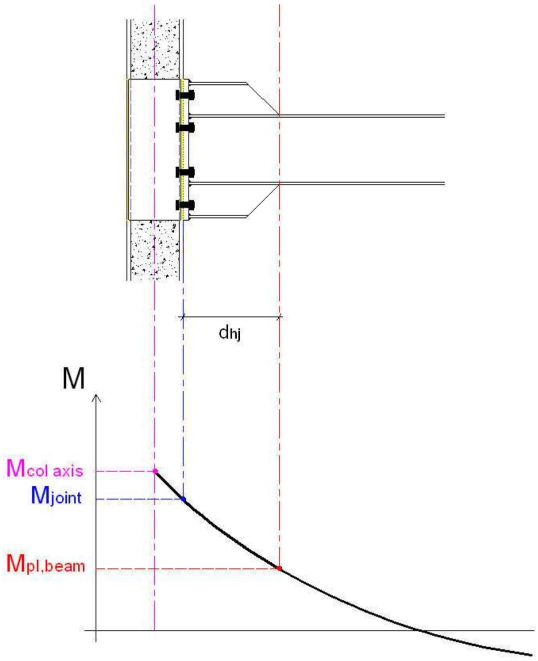

Figure 3. Moment in the joint and at the column axis when the plastic hinge appears in the beam section after the hammer heads

So, when the plastic hinge forms in the beam, the moment the joint is subjected to is greater than $M_{pl,beam}$. Then, in Eq.(1), "$M_{pl,beam}$" should be replaced by the moment $M_{\text{j - pl hinge in beam}}$ acting in the joint when the plastic hinge has formed in the beam section after the hammer-heads:

$$M_{\text{Rd,joint}} > 1{,}1 \cdot \gamma_{ov} \cdot M_{\text{j - pl hinge in beam}} \tag{2}$$

$M_{\text{j - pl hinge in beam}}$ is computed as follows as far as seismic circumstances are concerned (see Figure 4):

- maximum hogging moment in the joint:

$$M_{\text{j - pl hinge in beam,HOG}} = M_{\text{pl,beam}} + V_1 \cdot d_{hj} + p_{\max} \cdot \frac{d_{hj}^2}{2} \text{ , with } \quad V_1 = \frac{2 \cdot M_{pl,beam}}{l} + \frac{p_{\max} \cdot l}{2} \tag{3}$$

- maximum sagging moment in the joint:

$$M_{\text{j - pl hinge in beam,SAG}} = M_{\text{pl,beam}} + V_2 \cdot d_{hj} - p_{\min} \cdot \frac{d_{hj}^2}{2} \text{ , with } \quad V_2 = \frac{2 \cdot M_{pl,beam}}{l} - \frac{p_{\min} \cdot l}{2} \tag{4}$$

where:

- $M_{pl,beam}$ is the plastic moment of the beam cross section (based on the nominal value of the yield stress)
- V_1 is the shear force in the beam cross section after the hammer-heads when the plastic hinge appears, next to the joint subjected to hogging moment
- V_2 is the shear force in the beam cross section after the hammer-heads when the plastic hinge appears, next to the joint subjected to sagging moment
- d_{hj} is the distance between the plastic hinge and the joint connection
- l is the distance between the two plastic hinges developing at the extremities of the beam

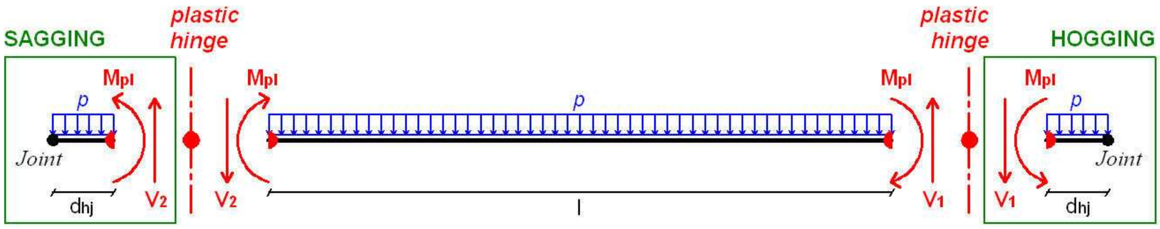

Figure 4. Internal forces at the beam extremities once plastic hinges have formed under seismic actions

Actually, the inequality of Eq. (2) is not totally right because, as shown in Eqs. (3) and (4), $M_{\text{j - pl hinge in beam}}$ does not only depend on the mechanical characteristics of the beam, but also on the external loads and there is no reason why the over-strength factor should multiply these loads. Consequently, using Eqs. (3) and (4) in Eq. (2) and applying the overstrength factor only to the terms which are related to the beam material strength, the resistance requirements for the joint become:

$$M_{\text{Rd,joint,HOG}} > M_{\text{pl,beam}} \cdot 1{,}1 \cdot \gamma_{ov} + \left(\frac{2 \cdot M_{pl,beam} \cdot 1{,}1 \cdot \gamma_{ov}}{l} + \frac{p_{\max} \cdot l}{2} \right) \cdot d_{hj} + p_{\max} \cdot \frac{d_{hj}^2}{2} \tag{5}$$

$$M_{\text{Rd,joint,SAG}} > M_{\text{pl,beam}} \cdot 1{,}1 \cdot \gamma_{ov} + \left(\frac{2 \cdot M_{pl,beam} \cdot 1{,}1 \cdot \gamma_{ov}}{l} - \frac{p_{\min} \cdot l}{2} \right) \cdot d_{hj} - p_{\min} \cdot \frac{d_{hj}^2}{2} \tag{6}$$

The bending resistance of the joint is calculated using the component method in accordance with EN 1993-1-8. The resistant moment of the joint depends on the resistance of the different components involved. Amongst them, the component "beam web in tension" is part of the beam itself and so, obviously, no overstrength factor has to be taken into account to determine the required resistance of this component. If the hammer-heads are made from the same profile as the beam, then the same remark applies for the corresponding components ("hammer-head flange and web in compression", "hammer-head web in tension" and "hammer-head web in shear"). Indeed, if the yield stress of the beam material is higher than its nominal value considered in the computation of $M_{pl,beam}$, then the resistance of these four components will automatically increase in the same way.

To be able to take this beneficial effect into account, the resistant moment of the joint has to be decomposed into the contributions of the different components in Eqs. (5) and (6). The resistant moment of the joint is:

$$M_{\text{Rd,joint}} = \sum_{rowsr} F_{Rd,r} \cdot h_r \tag{7}$$

where:

- $F_{Rd,r} = \min_{components\ \text{k}} \left\{ F_{Rd,r,k} \right\}$ is the resistance of row "r"
- $F_{Rd,r,k}$ is the resistance of component "k" in row "r"
- h_r is the vertical distance from row "r" to the compression centre

Consequently, defining a "reduced" resistant moment as:

$$M_{\text{Rd,j,REDUCED}} = \frac{M_{Rd,\text{joint}}}{1{,}1 \cdot \gamma_{ov}} \tag{8}$$

It comes:

$$M_{\text{Rd,j,REDUCED}} = \sum_{rows\ \text{r}} \frac{F_{Rd,r}}{\gamma_{ov}} \cdot \frac{h_r}{1{,}1} \tag{9}$$

with:

$$\frac{F_{Rd,r}}{\gamma_{ov}} = \min_k \left\{ \frac{F_{Rd,r,k}}{\gamma_{ov,k}} \right\} \tag{10}$$

where the overstrength factor associated to component "k", $\gamma_{ov,k}$, depends on the considered component (i.e. it is equal to 1,0 for the components related to the beam or to the hammer-heads if they are made from the same profile as the beam, and to 1,25 for the other components). Then a reduced resistance can be computed for each component using the proper value of the overstrength factor; and the reduced resistant moment of the connection is deduced from the reduced resistances of the different components involved.

Finally, the inequalities to fulfil are the following ones, for hogging and sagging moment respectively:

$$M_{\text{Rd,j,REDUCED,HOG}} > M_{\text{pl,beam}} + \left(\frac{2 \cdot M_{pl,beam}}{l} + \frac{p_{\max} \cdot l}{2 \cdot 1{,}1 \cdot \gamma_{ov}} \right) \cdot d_{hj} + \frac{p_{\max} \cdot d_{hj}^2}{2 \cdot 1{,}1 \cdot \gamma_{ov}} \tag{11}$$

where γ_{ov} is taken equal to 1,0 (safe side); and

$$M_{\text{Rd,j,REDUCED,SAG}} > M_{\text{pl,beam}} + \left(\frac{2 \cdot M_{pl,beam}}{l} - \frac{p_{\min} \cdot l}{2 \cdot 1{,}1 \cdot \gamma_{ov}} \right) \cdot d_{hj} - \frac{p_{\min} \cdot d_{hj}^2}{2 \cdot 1{,}1 \cdot \gamma_{ov}} \tag{12}$$

in which γ_{ov} is taken equal to 1,25 (safe side).

It is also important to note that, as far as the resistance check of the component "column panel in shear" is concerned, the possible overstrength of the beam has not to be taken into account according to Eurocode 8. Consequently, the inequality to fulfil is simply:

$$V_{\text{wp,Rd}} \geq V_{\text{wp,Ed}} \tag{13}$$

where:

- the resistance of the column panel in shear $V_{wp,Rd}$ is computed according to EN 1993-1-8 6.2.6.1 and EN 1994-1-1 8.4.4.1, taking also account of the prescriptions of Eurocode 8 regarding the resistance of the column panel in shear in composite columns (EN 1998-1 7.5.4 (3));
- the shear force the column panel is subjected to is $V_{wp,Ed} = \beta \cdot M_{col\ \text{axis,Ed}} / z$ (EN 1993-1-8 5.3), where $M_{col\ \text{axis,Ed}}$ is the moment applied to the considered joint, computed at the intersection of the beam and the column centrelines (*Figure 3.*), and z is the forces lever arm.

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4. Computation of the component resistances

Both joint configurations introduced in Section 2 have the same "beam-part" which includes the beam itself, the hammer-heads and the end-plate. The corresponding components are exactly the same in both cases: end-plate in bending (see 4.1), hammer-head flange and web in compression (4.2), beam-web in tension (4.3), hammer-head web in tension (4.4), bolts in tension (4.5) and hammer-head in shear (4.6). The other components are specific to the column which is used. For the wide-flange column: column panel in shear (4.7), column in transverse compression (4.8), column flange in bending (4.9), column web in tension (4.10). And for the rectangular hollow-section column: column panel in shear (4.11), lateral faces of the U in transverse compression (4.12), front face of the U in transverse bending (4.13), lateral faces of the U in transverse tension (4.14).

Amongst all these components, some are directly covered by the Eurocode [2] or very similar to components directly covered in such a way the corresponding formulae can be extended. On the other hand, some components are not covered by [2] and need particular attention. All the components are listed below and explanations on how their resistances can be evaluated are given.

4.1 End-plate in bending

The case of bolt rows 2 and 3 (between the beam flanges) is covered by EN 1993-1-8, §6.2.6.5. The case of the first bolt row is particular because it is between the beam flange and the hammer-head flange. This situation is similar to the case where there is an intermediate stiffener between bolt rows. This problem is addressed in [7], section 3.2.1.3.

4.2 Hammer-head flange and web in compression

The resistance of this component is computed on the basis of EN 1993-1-8, §6.2.6.7. It is given by:

$$F_{Rd,6} = \frac{M_{c,Rd}}{h_b + h_1 + h_2 - t_{fh}}$$

where $M_{c,Rd}$ is the design moment resistance of the cross-section including the beam and the hammer-heads, neglecting the beam flanges; h_b is the beam height; h_1 and h_2 are the heights of the upper and lower hammer-heads respectively.

4.3 Beam-web in tension

This component is covered by EN 1993-1-8, §6.2.6.8.

4.4 Hammer-head web in tension

This component is computed exactly the same way as the beam web in tension as recommended in EN 1993-1-8.

4.5 Bolts in tension

This component is covered by the Eurocode.

4.6 Hammer-head in shear

The compression force acting in the hammer-head flange and web has to be transferred to the beam, essentially by shear. However, when the shear resistance of the hammer-head web is reached, the system can still resist, until the flexural mechanism represented at Figure 5 below is formed. The component named here “hammer-head in shear” actually involves both the hammer-head web in shear and the end-plate and the hammer-head flange in bending. Its resistance is evaluated by:

$$F_{Rd} = P_{pl} + F_{shear}$$

where F_{shear} is the plastic shear resistance of the hammer-head web, and P_{pl} is the force corresponding to the formation of the two plastic hinges (once the hammer-head web is already yielded).

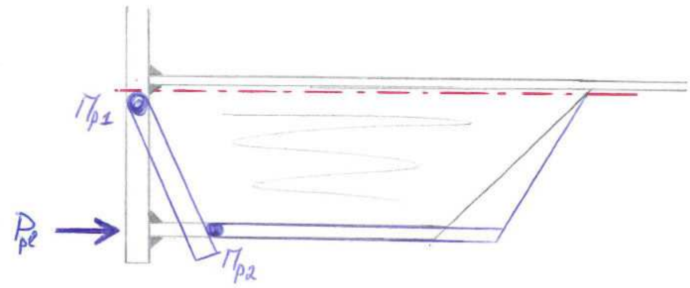

Figure 5. Plastic flexural mechanism including the end-plate and the hammer-head flange

4.7 Column panel in shear (for wide-flange column)

The resistance of the column panel in shear is computed considering the contributions of the steel profile web, the encasing concrete and the lateral plates, taking also account of the prescriptions of Eurocode 8 regarding the resistance of the column panel in shear in composite columns (EN 1998-1, §7.5.4 (3)).

The contribution of the steel profile web is covered by EN1993-1-8, §6.2.6.1 while the contribution of the encasing concrete is dealt with in EN1994-1-1, §8.4.4.1. The contribution of the lateral plates is evaluated the same way as the profile web (the formula given in EN1993-1-8 6.2.6.1 can easily be extended).

4.8 Column in transverse compression (for wide-flange column)

The resistance of the column in transverse compression includes the contribution of the steel profile web (covered by EN1993-1-8 6.2.6.2) and the contribution of the encased concrete (EN1994-1-1 8.4.4.2). No contribution of the lateral plates is considered (safe side).

4.9 Column flange in bending (for wide-flange column)

The column cross section made up of the H-profile and the lateral plates is equivalent to two hollow sections next to each other. Then, the formulae related to a face of a rectangular hollow cross section in transverse tension are used. These formulae are detailed in [8]. They correspond to those developed in [7], section 3.3, for minor axis beam-to-column joints.

4.10 Column web in tension (for wide-flange column)

Both the steel profile web and the lateral plates contribute to the resistance of this component. The resistance of the profile web is given in EN1993-1-8, §6.2.6.3. The formula can easily be extended to evaluate the resistance of the lateral plates in a similar way.

4.11 Column panel in shear (for RHS column)

The resistance of the column panel in shear is computed considering the contributions of the steel column webs and the U lateral faces (the contribution of the encased concrete is neglected, which is on the safe side). The resistance of the column webs and the U side faces is computed extending the formula given in EN1993-1-8, §6.2.6.1 for the web of a wide-flange column.

4.12 Lateral faces of the U in transverse compression (for RHS column)

The resistance of the U lateral faces in transverse compression can be evaluated based on EN1993-1-8, §6.2.6.2 which gives the resistance of a wide-flange column web in transverse compression. Indeed, the formula can be easily adapted.

4.13 Front face of the U in transverse bending (for RHS column)

The formulae related to a face of a rectangular hollow cross section in transverse tension are used to compute the resistance of the U front face in bending. These formulae are detailed in [8]. They correspond to those developed in [7], section 3.3, for minor axis beam-to-column joints. Three possible local failure modes are considered: flexural mechanism, punching shear mechanism and combined flexural and punching shear mechanism.

As the length of the U above or below the tension or compression zone is quite short, the possibility of an "edge" flexural mechanism (Figure 6) has to be taken into account for the compression zone as well as for the tension zone if row 1 is involved. No formula exists for a combined flexural and punching shear "edge" mechanism; so this effect couldn't be taken into account.

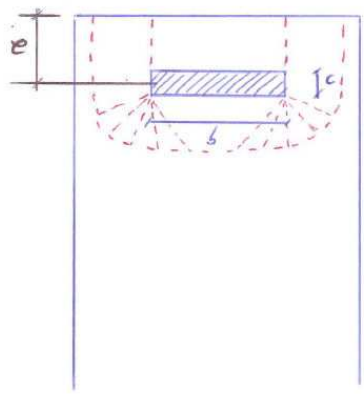

Figure 6. *Edge mechanism*

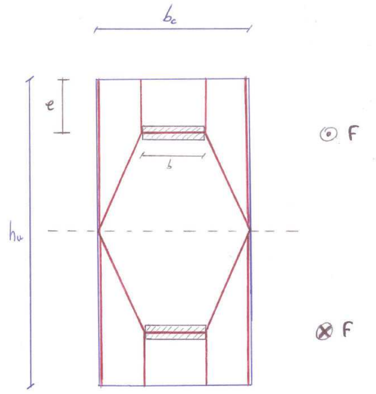

Figure 7. *Global mechanism*

Besides, the possibility of a global failure mechanism involving both the tension and the compression zones has also to be considered (Figure 7).

4.14 Lateral faces of the U in transverse tension (for RHS column)

The formula given in EN1993-1-8, §6.2.6.3 for the resistance of a wide-flange profile web in transverse tension can easily be extended to evaluate the resistance of the U lateral faces in a similar way.

5. CONCLUSIONS AND GENERAL RECOMMENDATIONS

Moment resisting frames designed according to the "dissipative structural behaviour concept" of Eurocode 8 have to dissipate seismic energy through cyclic yielding of plastic hinges located at the extremities of the beams. These dissipative zones can be either part of the beams or the beam-to-column joints. If the connections are meant to be non-dissipative and thus to remain in elastic range while plastic hinges develop in the beams next to the joints, they have to be full-strength, taking account of the possible overstrength of the beam material. This requirement customarily leads to very strong and expensive joints.

In this paper, particular joint configurations were proposed for such non-dissipative bolted joints, associated with a design strategy which can reduce the joint costs while in full accordance with both Eurocode 8 and the component method. The proposed design procedure is based on the principle that no overstrength factor needs to be taken into account for components that are part of the beam itself or of an element which is extracted from the same profile (e.g. the hammer-heads in the considered joint configuration). This method permits the use of a particular value of the overstrength factor for each component, through the concept of reduced resistance. Extending the fundamental principles of Eurocode 8, the proposed design procedure leads to less severe resistance requirements. Consequently, less strong and thus less expensive joints can be used provided they are designed in such a way that the weakest component, causing the failure of the connection (in terms of full resistance), is part of the beam itself or of an element extracted from the beam profile (for which the overstrength factor can be taken equal to 1,0).

ACKNOWLEDGMENTS

The present work was supported by the funds of European Project HSS-SERF: "High Strength Steel in Seismic Resistant Building Frames", Grant N0 RFSR-CT-2009-00024.

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