Hammer-head beam solution for beam-to-column joints in seismic resistant building frames

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Key words: bolted joints; hammer-head beams; experimental tests; component method; design guidelines

Highlights

- An innovative solution for bolted beam-to-column joints in seismic areas is proposed.
- Hammer heads are proposed to be used instead of the traditional haunches.
- A test campaign was realised demonstrating the good behaviour of the joints.
- A design procedure founded on the component method has been developed.
- A good agreement between the analytical and experimental results is found.

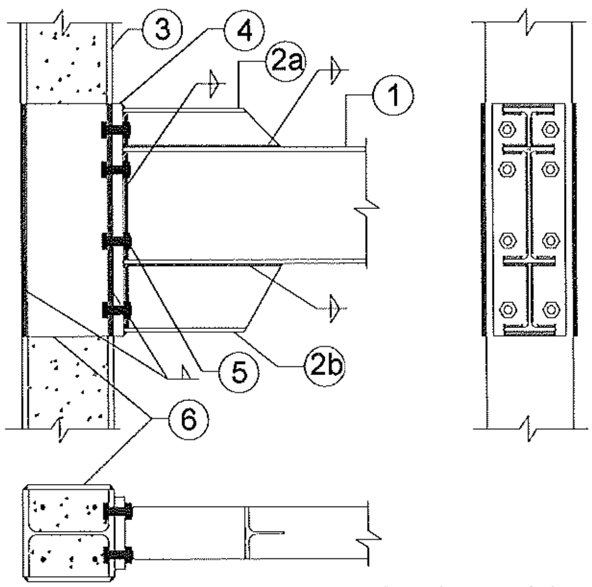
Abstract: this paper presents investigations conducted on an innovative stiffened extended end-plate joint, used to connect I-shaped beams to partially-encased composite wide flange columns; the objective is to propose an economical full-strength and a fully-rigid joint solution for buildings in high seismicity regions, respecting the requirements from EN1998-1-1 dedicated to the seismic design of buildings. In the investigated joint configuration, T-shaped hammer heads extracted from the same I-profiles than the beams are used instead of using traditional haunches. At the joint level, the column web is strengthened by two lateral plates welded to the column flanges; these plates also reinforce the column flanges. Within the present paper, a test program carried out within a RFCS European project entitled HSS-SERF "High Strength Steel in Seismic Resistant Building Frames", 2009-2013, will be first presented. Then, analytical developments based on the component approach and aimed at predicting the joint response will be described; their validity will be demonstrated through comparisons with the test results. Moreover, a new design concept for full strength joint accounting for the actual

position of the plastic hinge and the possible individual over-strength factors for each component is proposed, respecting the requirements of EN1998-1-1.

1. Introduction

In order to obtain a full-strength and a fully-rigid solution for bolted extended end-plate beam-to-column joints to be used in seismic resistant building frames, two directions may be foreseen: (i) reducing the beam section near the joint (dog-bone beam) or (ii) using stiffeners to reinforce joint components. If the second solution is considered, the use of haunches (with or without flanges) are also generally required.

In this paper, a new economical joint configuration is proposed to connect I-shaped beams to partially-encased composite wide flange columns (Fig.1). In the proposed joint configuration, T-shaped hammer heads extracted from the same I-profiles as the beams are used, instead of using the traditional haunches. At the joint level, the column is also strengthened by two lateral plates welded to the column flanges (Fig.1); the use of these plates allows increasing the resistance of the column web components (in shear, tension or compression) but also the column flange in bending component.



Elements		*Steel materials*
1	Double-T steel beam	Mill steel
2a, 2b	Top and bottom hammer- heads	Extracted from the beam profiles
3	Partially-encased wide-flange column	High strength steel may be used
4	End-plate	Mill steel
5	Bolts	High strength bolts (8.8 or 10.9)
6	Lateral plates	Same grade with the column profiles

Fig.1. Proposed joint configuration

In comparison with the joint solutions using haunches, the following advantages can be pointed out for the hammer-head joint solution: (1) the use of hammer-head allows a good load transfer from the beam to the joint zone and so avoids local compression in the beam web which appears with haunches (at the intersection between the haunch flange and the beam); (2) the use of hammer-heads directly extracted from the beam profile simplifies the fabrication procedure and lead to a cost saving; (3) no overstrength coefficient between the hammer-heads and the steel beam as to be considered as they are coming from the same profile, which will induce some economies in the design process. The observation reported in point (1) regarding the load transfer at the joint level has been demonstrated through the experimental tests conducted within the HSS-SERF project [1]; these tests will be presented in Section 2. Also, regarding the remark reported in point (2) on the economical fabrication process, a technical and economic evaluation was carried out for several types of joints in [1]: joint using long bolts, joint with external diaphragm, joint with rib stiffeners, and joint with hammer head beams. The conclusion was that the hammer head joint is the best solution. Finally, regarding point (3), detailed explanations will be given in Section 4 of the present paper.

However, the design of the proposed joint is not presently covered in Eurocodes and in literature, as the joint involves some new components. Therefore, analytical developments were realised in order to propose a full design procedure useful for practitioners and in full agreement with the component method which is the design method recommended in Eurocodes for the characterisation of joints.

The present paper summarizes the researches on the proposed joint configuration, from the experimental tests to the development of the design procedure. In Section 2, the results of the tests on the proposed joint configuration will be reported. Section 3 will deal with the analytical development based on the component method. Section 4 is dedicated to the validation of the proposed models through comparisons to the experimental results. How to take into account for the actual position of the plastic hinges and individual component over-strength factors to satisfy the full-strength requirement from EN1998-1-8 dedicated to the seismic design of buildings will be the content of Section 5. Section 6 is finally devoted to the concluding remarks.

2. Experimental results

A test program was defined and performed on the proposed joint configuration within the HSS-SERF project; details about the performed tests and the obtained results can be found in [2]. All the joints were designed to be full strength ones, meaning that the plastic hinges should develop in the beam, more precisely in the cross-sections close to the hammer head ends. Within the test program, two categories of tests were defined: (1) prequalification tests for which the "actual" specimen configuration, i.e. the configuration which would be met in a building structure, were used and for which the plastic hinges occurred at the beam sections close to the hammer head ends; and (2) joint characterization tests for which the beams were strengthened so as to force the failure at the joint level and to obtain the complete behaviour of the joint. Within the present paper, the joint characterization tests will be described as only these tests are used to validate the joint design procedure.

The specimen geometries and materials are presented in Table 1 and Fig.2. Test A1 was defined to evaluate the resistance of the hammer head zone while Tests A2 and B1 aim at characterizing the connection resistance under hogging and sagging moments respectively. Obviously, the elastic stiffness of the specimens can be recorded from the three tests. The HEB320 columns used for Specimens A1 and A2 are made of S460 steel while the column HEB260 column in Specimen B1 is made of high strength steel S690 to investigate the possibility of using high strength steel in seismic resistant building frames which is one of the main objective of the HSS project.

Table 1: Description of the tested specimens (Fig.2)

Tests	**Column**	**Beam**	**Lateral plates**	**Reinforcement degree**	**Loading type**
A1	HEB320	IPE400	800x290x15	Partial reinforcement (a=350 mm – Fig.2)	Hogging moment
A2	HEB320	IPE400	800x290x15	full reinforcement (a=50 mm – Fig.2)	Hogging moment
B1	HEB260	IPE400	800x230x15	full reinforcement (a=50 mm – Fig.2)	Sagging moment

- C30/37 concrete is used for all specimen; S355 steel is used for the beams and the end-plates; S460 steel is used for the HBE320 column and the associated lateral plates; S690 steel is used for the HEB260 column and the associated lateral plates; M30 10.9 bolts are used.
- The filet welds of 5 mm is used to connect the hammer head web to the beams while the beam and the hammer head flanges are attached to the end-plate through filet welds of 8

mm.

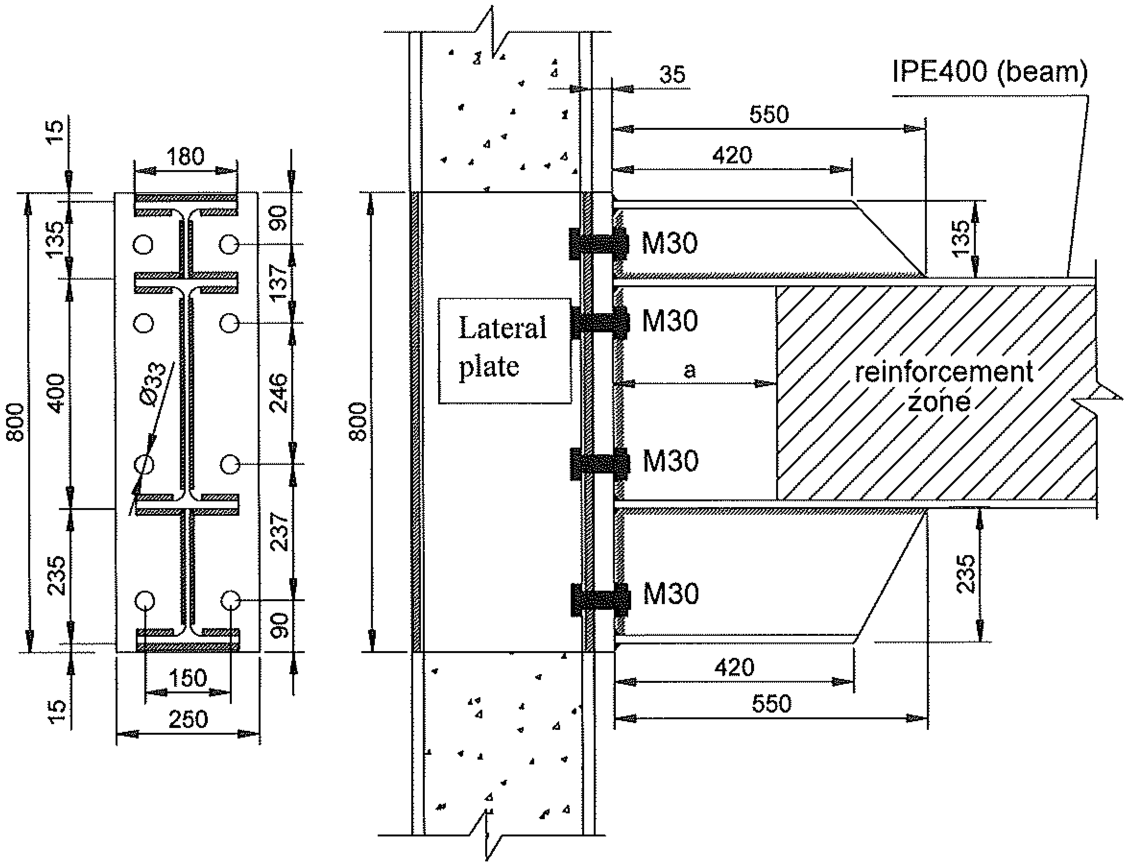


Fig.2. Geometry of the tested specimens

The used testing set-up is presented in Fig.3**Erreur ! Source du renvoi introuvable.**. A fixed hinge at the bottom and a hinge allowing a vertical displacement at the top are used at the column extremities. Possible displacements of the hinges have been anyway recorded during the tests. A vertical load is applied at the free end of the beam introducing a bending moment and a shear force in the joints. Lateral supports on the beam length have been placed to avoid the lateral torsional buckling of the beam during the tests.

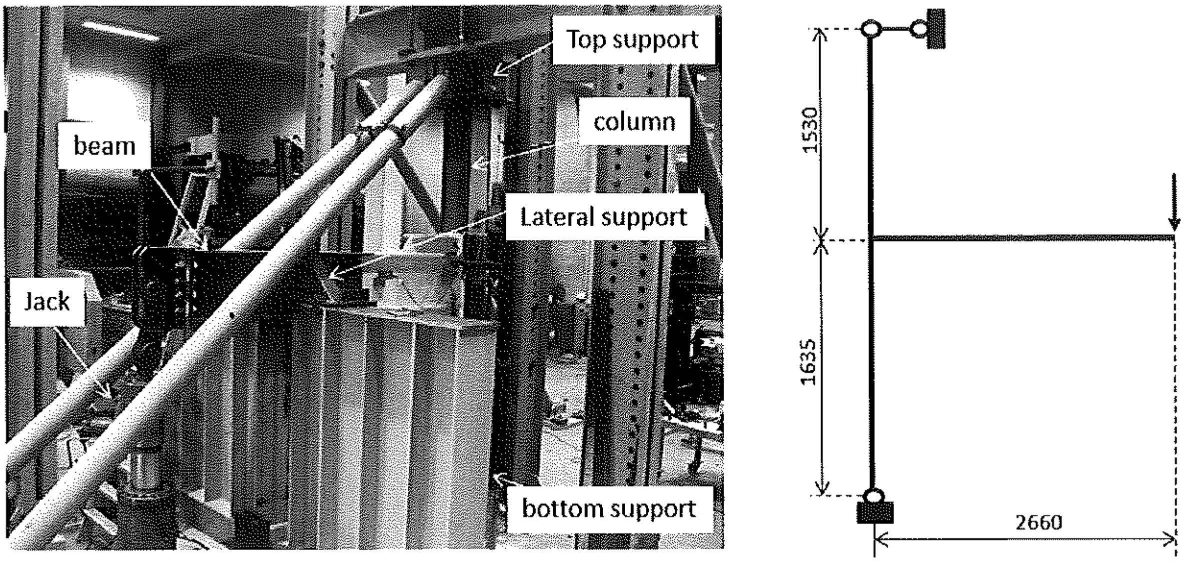


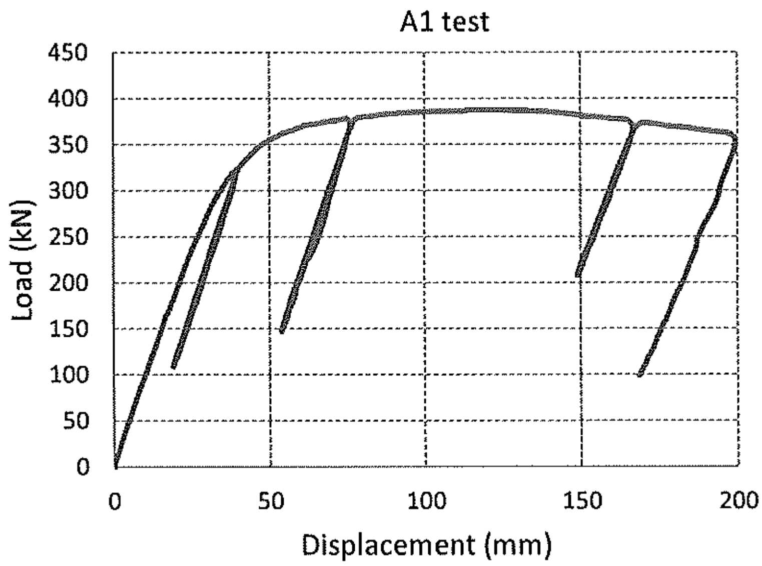
Fig.3. Testing set-up

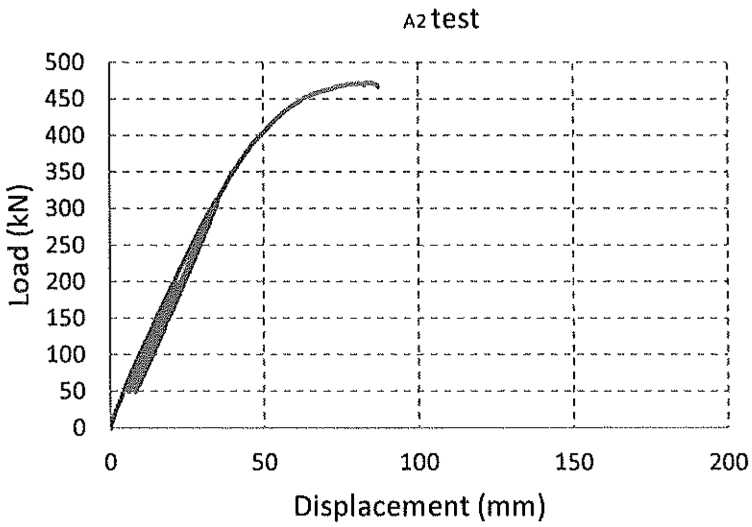
Displacement and rotational transducers were used to records the kinematics of the specimens during the tests, i.e.: the column panel rotation, the connection rotation, the plastic hinge rotation and the displacement of the load application point.

The load-displacement curves of the tests are presented in Fig.4. It is shown that Specimen A1 presents a better ductility than Specimens A2 and B1. This observation can be explained from the different failure modes observed during the tests. Indeed, as expected, a plastic hinge occurred at the hammer head zone, at the end of the reinforcement (Fig.5), in Specimen A1 while the two bolt rows in the tension zone simultaneously failed in Specimens A2 and B1 (see the arrows in Fig.5). Also, some plastifications can be observed in the hammer head webs (in both compression and tension zones) and in the beam end section (see the dashed lines in Fig.5). The yielding of the hammer head webs in tension may be associated to a plastic redistribution between the two bolt rows in tension, so explaining why the two bolt rows failed at the same time. Through the test observation, it can be shown that the critical section for Specimen A1 is in the hammer head zone, close to the end of the beam reinforcement, while the critical sections for Specimens A2 and B1 is the column face. The stiffness, the maximum moment at the critical sections and the maximum moment at the hammer head end (i.e. where the plastic hinge should developed in the "actual" specimens without the beam reinforcement) are reported in Table 2.

Through Tests A2 and B1, it is also possible to demonstrate the full strength character of the studied joints (see Table 2). Indeed, the ultimate bending resistance of the joints is equal to 900 kNm, while the actual ultimate bending capacity of the beam section is equal to 613.3 kNm (determined through the prequalification tests not described within the present paper).

The joints have a very high stiffness, the coefficient k_b as defined in EN1993-1-8 [3] (i.e. ratio between the joint stiffness and the bending rigidity of the beam) is equal to 29.8, 28.9 and 23.8 for Specimens A1, A2 and B1 respectively, assuming a beam span of 7.5 m (corresponding to the span of the beam of the reference building from which the joints were extracted). According to EN1993-1-8 [3], the tested joints may be classified as fully-rigid ones for all types of frames, i.e. unbraced frames ($k_b \geq 25.0$) or braced frames ($k_b \geq 8.0$) except for Specimen B1 in case of unbraced frames.





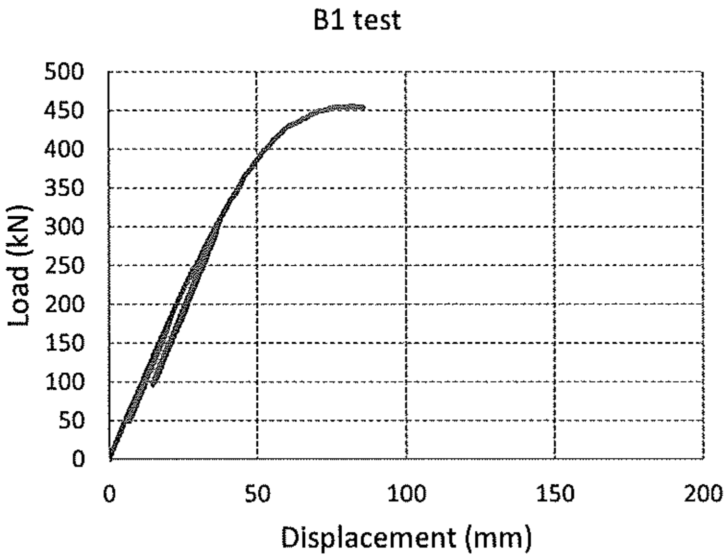
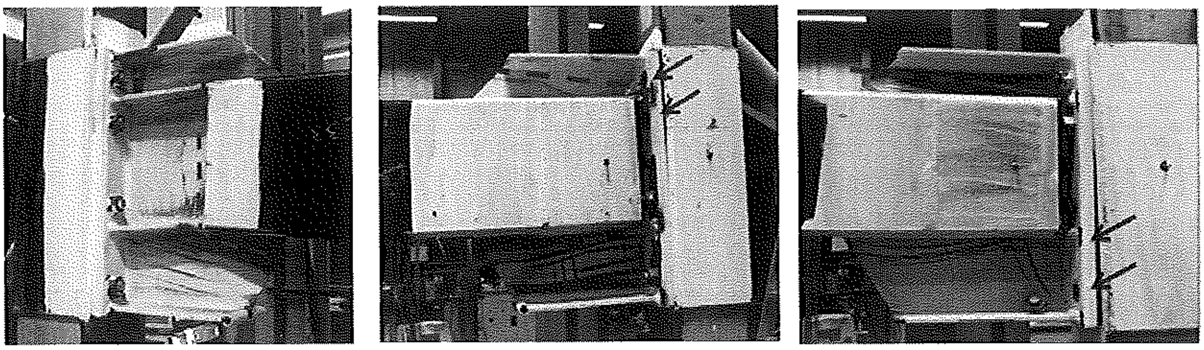


Fig.4. Load – point load displacement curves of the tests

Table 2: Stiffness and resistance of the specimens

Test	Joint stiffness (kNm/rad)	Moment at hammer head ends (kNm)	Moment at the critical sections (kNm)
A1	193000	742.4	820.0 (at the end of the reinforcement)
A2	187000	909.2	1187.0 (at the column face)
B1	154500	894.1	1160.0 (at the column face)
Remark: the yielded and ultimate strength of the beam section are 500.0 kNm and 613.3 kNm, respectively (from the coupon test results, see Table 5).			



A1 test A2 test B1 test

Fig.5. Tested specimens at failure

3. Application of the component method to the investigated joint configuration

In this section, the joint resistance and stiffness calculations using the component method is presented. Table 3 lists the basic components which are met in the investigated joints which are covered by the Eurocodes while Table 4 identifies all the specific components of the investigated joint and explain how to calculate the resistance and stiffness of these components. There are some components which are directly covered by Eurocodes while additional rules are required for some other components. The rules available in the Eurocodes are not reminded in this section which only focuses on the new proposed rules (as detailed from Section 3.1 to Section 3.5).

Table 3: Basic component met in the investigated joint

Components		Associated rules in the Eurocodes	
		resistance	stiffness
1	Steel column web in shear		
2	Column web in compression		
3	Column web in tension		
4	End-plate in bending	EN-1993-1-8, §6.1.3[3]	
5	Beam flange and web in compression		
6	Bolts in tension		
7	Beam web in tension		
8	Encased concrete in shear[(a)]	EN-1994-1-1, §8.4.4.1[4]	EN-1994-1-1,A.2.3.2 [4]
9	Encased concrete in compression	EN-1994-1-1, §8.4.4.2[4]	EN-1994-1-1,A.2.3.2 [4]
10	Lateral plates [(b)]	EN-1993-1-8, §6.1.3[3]	

[(a)] The conditions to take into account the contribution of the encased concrete in the calculation of the column panel in shear is indicated in EN1998-1-1, § 7.5.4(7) [5].

[(b)] The calculation of the lateral plates in shear/tension/compression is not explicitly covered in the Eurocodes but can be easily extrapolated from the rules proposed for the column web component.

Table 4: Identification of the specific components for the investigated joints and proposed design rules for their characterisation and assembly

Considered components	Resistance/stiffness		Proposed rules
Column panel in shear	$F_{Rd,1}$	k_1	Involved basic components [(a)]: *steel column web*, *lateral plates* and *encased concrete*.
Column in transverse compression	$F_{Rd,2}$	k_2	Involved basic components: *steel column web*, *lateral plates* and *encased concrete.*
Column in transverse tension	$F_{Rd,3}$	k_3	Involved basic components: *steel column web and lateral plates.*
End-plate in bending	$F_{Rd,4}$	k_4	Rules are proposed in Section 3.1
Beam flange and web in compression [(b)]	$F_{Rd,5}$	k_5	Involved basic component: *beam flange and web in compression.*
Beam web in tension [(b)]	$F_{Rd,6}$	k_6	Involved basic component: *beam web in tension.*
Bolts in tension	$F_{Rd,7}$	k_7	Involved basic component: *bolts in tension*
Column flange in bending	$F_{Rd,8}$	k_8	Rules are proposed in Section 3.2
Hammer heads in compression [(b)]	$F_{Rd,9}$	k_9	Rules are proposed in Section 3.3
Hammer heads in tension [(b)]	$F_{Rd,10}$	k_{10}	
Hammer head zone in bending [(c)]			Rules are proposed in Section 3.4
Component assembly	$M_{RD,j}$	$S_{j,ini}$	Rules are proposed in Section 3.5

[(a)]The resistance/stiffness of the considered components is calculated as the sum of the contributions of the listed basic components.

[(b)] These components are made of the beam material, this remark will be used in Section 5.

[(c)] This concerns the resistance of the beam in the hammer head zone which is not directly involved in the component assembly.

3.1. End-plate in bending component

The formulas to estimate the resistance and stiffness of the bolt rows inside the beam flanges are given in EN1993-1-8, §6.2.6.5[3], they can be directly applied to the present configuration. However, with respect to the bolt rows between the beam flanges and the hammer head flanges, the situation is different because these bolt rows presents a specificity which is their proximity to two flanges (Fig.6), bolt row configuration not yet covered in EN1993-1-8.

The proximity of the bolt row to two flanges affects the development of the yielding lines within the end-plate and so affects the effective length to be considered for the T-stub model which is the model recommended in EN1993-1-8 for the characterization of the joint component in bending. In [6], a method for the estimation of an appropriate effective length with account for the presence of two flanges close to the considered bolt row is given. This method is summarized here below and is recommended for the investigated joint configuration.

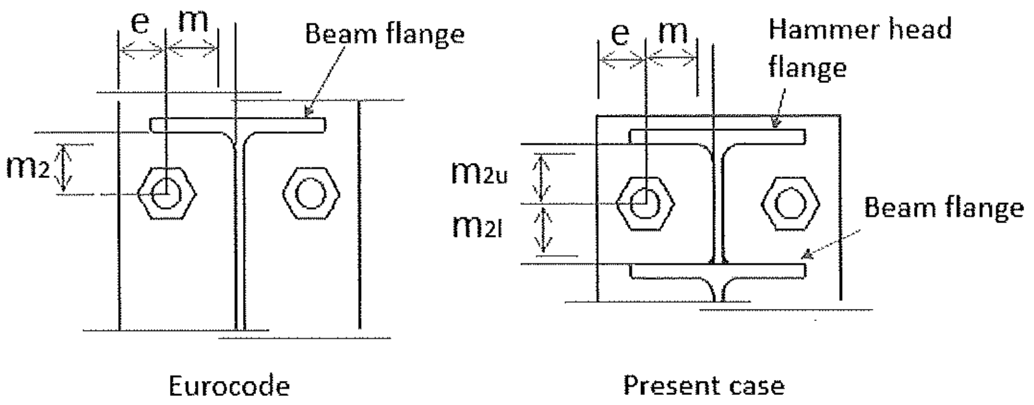


Fig.6. End-plate in bending component

The possible effective lengths to be considered for the T-stub model are the minimum the followings:

$l_{eff,c} = 2\pi m$ for circular pattern

$l_{eff,nc} = (\alpha_u + \alpha_l)m - (4m+1.25e)$ for non-circular pattern

in which the parameters m and e are shown in Fig.6, taking into account the welds as described in EN1993-1-8, §6.2.4.1 [3]; and α_u (u for "upper") and α_l (l for lower) are computed in agreement with Fig. 6.11 of EN1993-1.8, §6.2.6.5 [3] using the following parameters λ_1, λ_{2u}, and λ_{2l}:

$$\alpha_u = f(\lambda_1, \lambda_{2u}) \quad \text{with} \quad \lambda_1 = \frac{m}{m+e}, \lambda_{2u} = \frac{m_{2u}}{m+e}$$

$$\alpha_l = f(\lambda_1, \lambda_{2l}) \quad \text{with} \quad \lambda_1 = \frac{m}{m+e}, \lambda_{2l} = \frac{m_{2l}}{m+e}$$

With the so-calculated effective length, the formulas as given in Table 6.2 of EN1993-1-8, §6.2.4.1 [3] can be used for the prediction of the resistance of the T-stub and so, of the bolt row.

3.2. *Column flange in bending component*

The column cross-section made of a H-profile and lateral plates as illustrated in Fig.1 may be considered as two hollow sections connected to each other. Accordingly, half of this component may be seen as a face of a rectangular hollow cross-section in transverse tension, with only one bolt on one horizontal row. In the Eurocodes and in literature, such a component is not explicitly covered. However, the calculation of the "column face"/or "column web" components in bending (Fig.7) can be found in many works (e.g. [7], [8], [9]). For the investigated joint configuration, these developments may be applied assuming the distance between two bolts as equal to zero. The formulation which are proposed are summarized here after.

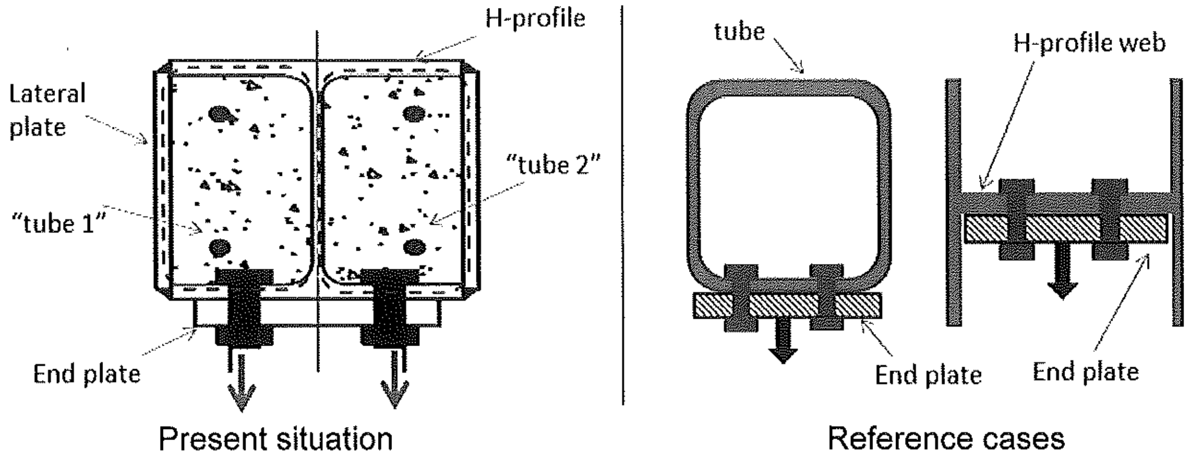


Fig.7. Column flange in bending component

The resistance of the column flange under bending is defined as the minimum value given by the bending and punching mechanisms as described in [8]:

$$\begin{cases} F_{Rd,4,bending} = \beta \dfrac{4\pi m_{pl,fc}}{1-\dfrac{0.9d_m}{L}} \left(\sqrt{1-\dfrac{0.9d_m}{L}} + \dfrac{1,8d_m}{\pi L} \right) \\ F_{Rd,4,punching} = \pi d_m \dfrac{t_{fc} f_{y,fc}}{\sqrt{3}} \end{cases} \tag{1}$$

while the stiffness of the considered component can be determined using the following formula [9]:

$$k_4 = \frac{\pi t_{fc}^3}{12(1-\nu^2)0.18(L_{stiff}/2)^2} \tag{2}$$

In which, $m_{pl,fc}$ is the unit plastic resistant moment of the column flange; d_m is the mean diameter of the bolt head/nut; t_{fc} is the thickness of the column flange; $f_{y,fc}$ is the yield strength of the column flange; ν is the Poisson coefficient; $L = 0.5(b_c - t_{wc}) - 0.75r_c$ (Fig.8), $L_{stiff} = 0.5(b_c - t_{wc}) - 0.5r_c + 0.5t_l$ (Fig.9) (with b_c the column flange width, t_{wc} the thickness of the column web, r_c the corner radius of the column and t_l the thickness of the lateral plates); the coefficient β is given by:

$$\begin{cases} \beta = 1 \quad \text{if} \quad d_m \geq 0.28L \\ \beta = 0.7 + 1.08 d_m / L \quad \text{if} \quad d_m < 0.28L \end{cases}$$

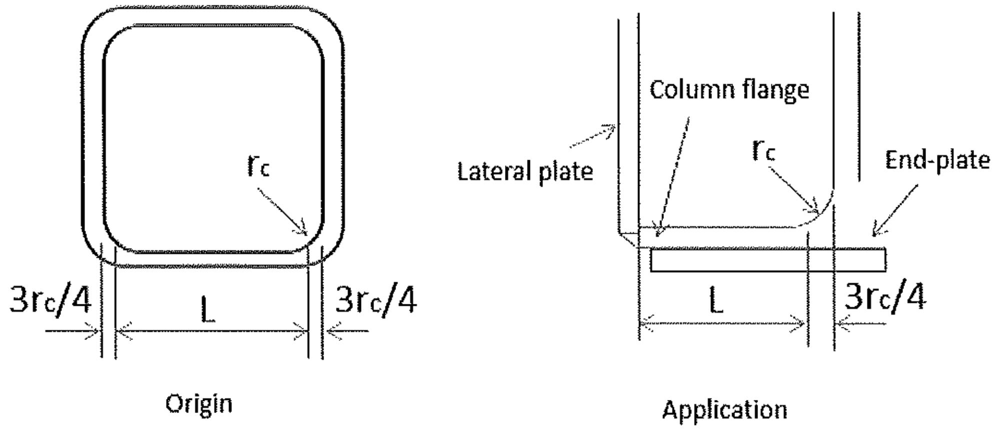


Fig.8. Span of the column flange in the resistance determination

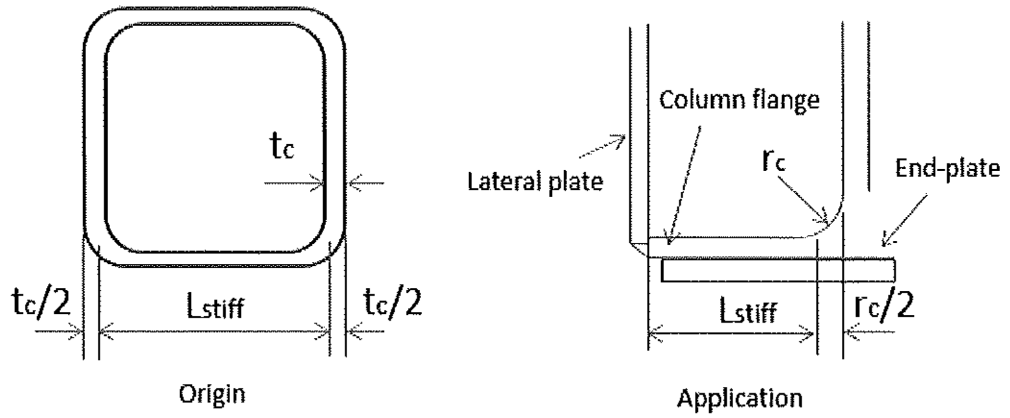


Fig.9. Span of the column flange in the stiffness determination

3.3. Hammer-heads in compression/tension component

In terms of resistance, three mechanisms shown in Fig.10 should be considered for the "hammer head in compression/tension" component. The shear mechanism is considered for the hammer heads in the compression or the tension zone while the compression and tension mechanisms are respectively adopted for the hammer heads in the compression or tension zone.

Even if the compression and tension mechanisms developing in the hammer-heads are not directly covered by the Eurocodes, the rules given in EN1993-1-8 for "hanched beam" and "beam web in tension" components can be easily adapted to the compression and tension mechanisms respectively.

The resistance of the shear mechanism is taken as equal to the resistance in shear of the hammer head web added to the resistance of the end-plate and the hammer head flange in bending (see Fig.10) at the image of what is done for a column web panel in shear stiffened by transverse horizontal plates. However, in most of the cases, the contribution of the hammer head web in shear is preponderant and therefore the contribution of plastic hinges forming in the end-plate and the hammer head flange may be neglected. So, neglecting the contribution from the plastic hinges, the resistance of the shear mechanism can be formulated as:

$$F_{Rd,9,shear} = l_{h1} t_w f_{yb} / \sqrt{3} \tag{3}$$

with l_{h1} the length of the hammer head web (Fig.10); t_w the thickness of the hammer head web; f_{yb} the yield strength of the hammer heads (equal to the yield strength of the beam).

The resistance of the hammer-heads in compression or tension is taken as the minimum between the resistance in shear and the resistance in compression or in tension respectively.

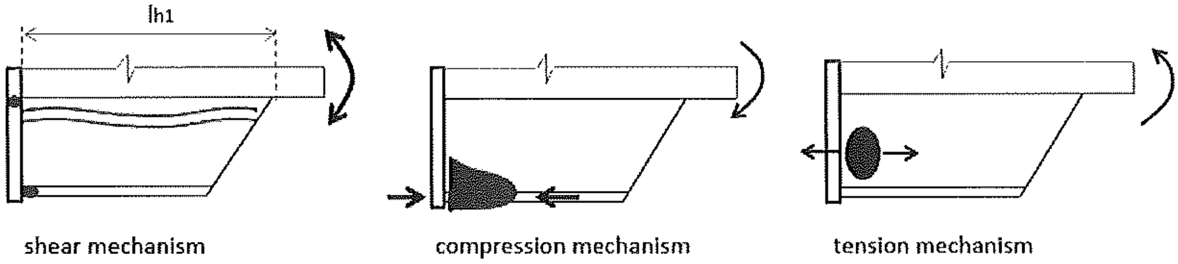


Fig.10. Considered mechanisms for the hammer head component

In terms of stiffness, the formula recommended EN1993-1-8, 6.3.2 [3] for the stiffness of the column web panel in shear can be applied to the hammer heads in compression/tension components:

$$k_{9,shear} = \frac{0.38 A_{vh}}{Z_{vh}} \tag{4}$$

This formula is valid for a rectangular plate while the shape of the hammer head is trapezoidal; accordingly, an equivalent rectangular panel has to be defined as illustrated in Fig.11. So, the parameters A_{vh} and Z_{vh} can be computed as follows:

$Z_{vh} = h_{hw}$ in case of compression

$Z_{vh} = n$ in case of tension

$A_{vh} = t_{hw}(l_{h1} + l_{h2})/2$ in case of compression

$A_{vh} = t_{hw}\left(l_{h1} + l_{h2} + \frac{h_{hw} - n}{h_{hw}}(l_{h1} - l_{h2})\right)/2$ in case of tension

where t_{hw} is the thickness of the hammer head web; the other parameters are defined in Fig.11.

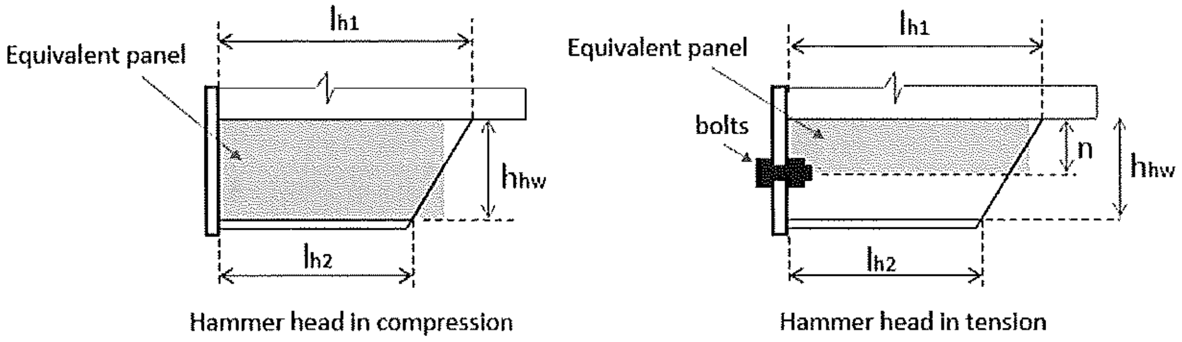


Fig.11. Equivalent rectangular panel to estimate the stiffness of the hammer head

3.4. *Resistance of the beam in the hammer head zone*

The resistance of the beam in the hammer head zone should be verified to avoid the development of a plastic hinge in this part.

For a section at a distance s from the hammer head end (see Fig.12), two possible critical sections (1-1 and 2-2) are identified. The plastic resistance of Section 1-1 can be easily estimated. For Section 2-2 combining the bending resistance of the beam and the shear resistance of the hammer head web, the resistance can be estimated as follows:

$$M_{Rd,hammerhead\ zone} = M_{Rd,beam} + f_{yw} t_w s h_b / \sqrt{3} \tag{5}$$

where $M_{Rd,beam}$ is design resistance of the beam I-profile; f_{yw} is the yield strength of the hammer head web material (equal to the yield strength of the beam web); s is the distance represented in Fig.12; h_b is the beam height.

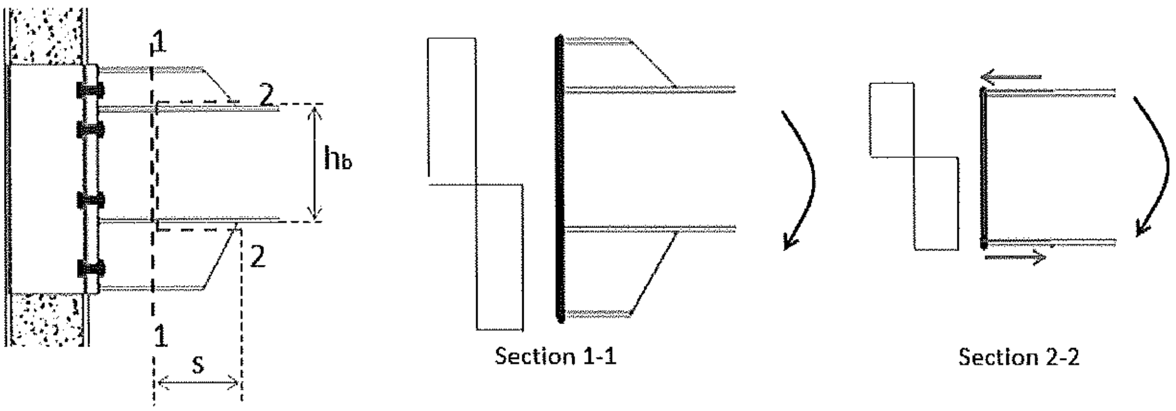


Fig.12. Resistance of the beam in the hammer head zone

3.5. *Component assembly*

The assembly rule recommended in EN-993-1.8 [3] can be applied for the investigated joints but the two following specificities should be considered.

Firstly, a plastic redistribution in the compression zone may be adopted for the investigated joints, redistribution which is not considered in the present draft of the Eurocodes. Indeed, at the beginning, the hammer head flange may be considered as the compression point of the joint, identified as compression zone 1 (Fig.13). With the increase of the load, the compression zone 1 may yield but additional compression forces can be supported by activating a second compression zone made of the beam flange and web component (compression zone 2 in Fig.13). In reality, the compression zone spreads from the hammer head flange to the beam flange, but, for sake of simplicity with the application of the component method, the compression zone is splitted into two zones. Obviously, the force developing in the two compression zones must be in equilibrium with the tension forces in the two bolt rows in the tension zone.

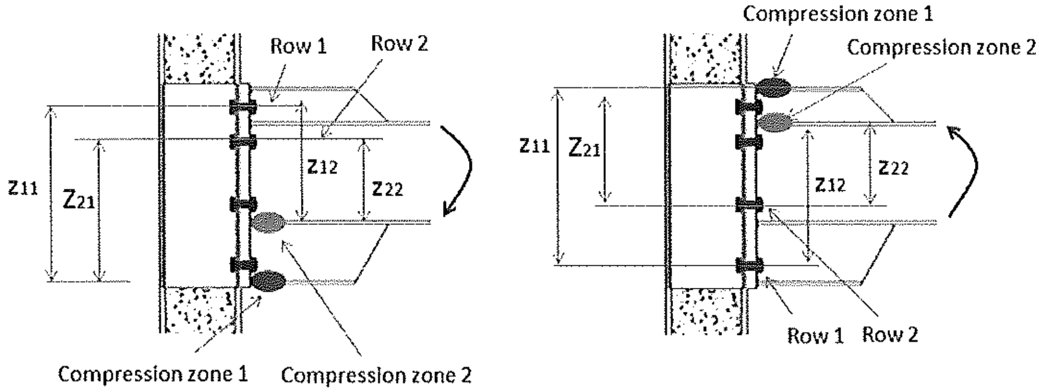


Fig.13. Definition of the compression zones

Secondly, the plastic redistribution in the two bolt rows in the tension zone may be considered when at least one of the following components in the tension zone is activated at yielding: the hammer head web (in the tension zone), the column web in tension, the end-plate in bending or the column flange in bending. In the contrary, if another component is activated, the elastic distribution between the two bolt rows should be used.

When the above plastic redistribution can be activated, the resistance of the joint can be computed as follows:

$$\begin{aligned}
&\text{If } F_{Rd,1}+F_{Rd,2}\le F_{Rd,zone1} \text{ then}\\
&\quad M_{Rd,j}=F_{Rd,1}.Z_{11}+F_{Rd,2}.Z_{21}\\
&\text{If } F_{Rd,1}\le F_{Rd,zone1}\le F_{Rd,1}+F_{Rd,2} \text{ then}\\
&\quad M_{Rd,j}=F_{Rd,1}.Z_{11}+\min[F_{Rd,2};F_{Rd,zone1}-F_{Rd,1}].Z_{21}+\\
&\qquad\qquad \min[F_{Rd,2}-\min(F_{Rd,2};F_{Rd,4}-F_{Rd,1});F_{Rd,zone2}].Z_{22}\\
&\text{If } F_{Rd,zone1}\le F_{Rd,1} \text{ then}\\
&\quad M_{Rd,j}=F_{Rd,zone1}.Z_{11}+\min[F_{Rd,zone2};F_{Rd,1}-F_{Rd,zone1}].Z_{12}+\\
&\qquad\qquad \min[F_{Rd,zone2}-\min(F_{Rd,zone2};F_{Rd,1}-F_{Rd,zone1});F_{Rd,2}].Z_{22}
\end{aligned} \tag{7}$$

with $F_{Rd,zone1}$ and $F_{Rd,zone2}$ the resistances of the governing components in compression zones 1 and 2, respectively; Z_{11}, Z_{12}, Z_{21} and Z_{22} the level arms shown in Fig.13.

Remark: above rule is applied for estimating the joint resistance, while only the compression zone 1 should be used for calculating the stiffness, because in the elastic domain, only this zone is assumed to be activated. The formula given in EN1993-1-8, 6.3.3 can be directly applied for the present joint.

4. Validation of the proposed models

The proposed analytical models for the joint characterisation provided in Section 3 are validated through comparisons to the experimental results presented in Section 2. In order to make the comparison, the actual material characteristics obtained through coupon tests are used; the main actual characteristics of materials are given in Table 5, more detail information can be found in [2]. Moreover, all partial safety factors are taken as equal to 1,0.

The comparisons of analytical predictions to the experimental resistances of for Specimens A1, A2 and B1 are reported in Table 6, 7 and 8 respectively. In these tables, the resistances of the non-critical components are not presented. With respect to the stiffness estimations, Table 8 summarize the stiffness factors of the all components of the specimens, and Table 9 makes the stiffness assembly and compares the so-obtained stiffness's with the experimental ones.

Good agreements are observed demonstrating the accuracy of the proposed models. Indeed, less than 5% of difference is observed for the resistance estimations and about 15% for the stiffness evaluations.

Table 5: Coupon test results

Elements	***Yielded strength***	***Ultimate strength***
Bolts	-	606,0 kN/bolt
Beam/hammer head flange	396,0 N/mm^2	490,0 N/mm^2
Beam/hammer head web	430,0 N/mm^2	512,0 N/mm^2
Using the actual strengths, the plastic and ultimate capacities of the IPE400 beam are respectively: $M_{\text{yield,beam}}$ = 500,0 kNm; $M_{\text{ultimate,beam}}$ = 613,3 kNm		

Table 6: Bending resistance of the beam in the hammer head zone (A1 test)

Section position (Fig.12)	s=0,2 (A1 specimen)
IPE400 ultimate capacity (kNm)	613,3 (Table 5)
Hammer head contribution (kNm)	203,7 (Eq.(5))
Estimated ultimate resistance (kNm)	**817,0**
Experimental ultimate resistance for A1 test (kNm)	**820,0** (Table 2)
Model-test difference	0,36%

Table 7: Ultimate strength of the joint under sagging moment (A2 test)

Critical components and resistances (kN)	***Compression forces (kN)***
Row 1: bolt in tension, $F_{\text{Rd,row1}}$=1212 (Table 5)	F_{zone1}=1175 (Eq.(6)) F_{zone2}=1249 (Eq.(6))
Row 2: bolts in tension, $F_{\text{Rd,row2}}$=1212 (Table 5)	
Zone 1: hammer head in shear, $F_{\text{Rd,zone1}}$=1175 (Eq.(3))	
Zone 2: beam flange and web in compression, $F_{\text{Rd,zone2}}$=1295	
Lever arms (m): z_{11}=0,688; z_{12}=0,553; z_{21}=0,451; z_{22}=0,316 (Fig.2)	
Predicted Bending resistance of joint - Eq.(7): $F_{zone1}z_{11}+(F_{Rd,row1}-F_{zone1})z_{12}+F_{Rd,row2}z_{22}$ =**1212** kNm	
Experimental Bending resistance:	**1187** kNm
Model-test difference:	2,1%

Table 8: Ultimate strength of joint under hogging moment (B1 test)

Critical components and resistances (kN)	***Compression forces (kN)***
Row 1: bolt in tension, $F_{\text{Rd,row1}}$=1212 (Table 5)	F_{zone1}=1175 (Eq.(6)) F_{zone2}=1249 (Eq.(6))
Row 2: bolts in tension, $F_{\text{Rd,row2}}$=1212 (Table 5)	
Zone 1: hammer head in shear, $F_{\text{Rd,zone1}}$=1175 (Eq.(3))	

Zone 2: beam flange and web in compression, $F_{Rd,zone2}$=1295	
Lever arms (m): z_{11}=0,688; z_{12}=0,453; z_{21}=0,551; z_{22}=0,316 (Fig.2)	
Predicted Bending resistance of joint - Eq.(7): $F_{zone1}z_{11}+(F_{Rd,row1}-F_{zone1})z_{12}+F_{Rd,row2}z_{22}$ =**1208** kNm	
Experimental bending resistance of joint:	**1160** kNm
Model-test difference:	4,1%

Table 9: Component stiffness factors (mm)

Considered components	**Specimens**		
	A1	**A2**	**B1**
Column panel in shear (one side) [(a)]	k_1=10.972	k_1=11.498	k_1=7.264
Column in transverse compression [a)]	k_2=24.235	k_2=24.235	k_2=21.942
Column in tension [(a)]	$k_{3,r1}$=18.097 $k_{3,r2}$=17.348 $k_{3,r3}$=20.623	$k_{3,r1}$=20.462 $k_{3,r2}$=17.924 $k_{3,r3}$=20.623	$k_{3,r1}$=19.220 $k_{3,r2}$=19.220 $k_{3,r3}$=21.073
End plate in bending [(a)]	$k_{4,r1}$=54.707 $k_{4,r2}$=44.185 $k_{4,r3}$=44.185	$k_{4,r1}$=50.349 $k_{4,r2}$=44.185 $k_{4,r3}$=44.185	$k_{4,r1}$=54.707 $k_{4,r2}$=44.185 $k_{4,r3}$=44.185
Beam flange and web in compression	$k_5=\infty$	$k_5=\infty$	$k_5=\infty$
Beam web in tension	$k_6=\infty$	$k_6=\infty$	$k_6=\infty$
Bolts in tension [(a)]	$k_{7,r1}$=10.317 $k_{7,r2}$=10.317 $k_{7,r3}$=10.317	$k_{7,r1}$=10.317 $k_{7,r2}$=10.317 $k_{7,r3}$=10.317	$k_{7,r1}$=10.317 $k_{7,r2}$=10.317 $k_{7,r3}$=10.317
Column flange in bending (Eq.(2))	$k_{8,r1}$=5.760 $k_{8,r2}$=5.760 $k_{8,r3}$=5.760	$k_{8,r1}$=5.760 $k_{8,r2}$=5.760 $k_{8,r3}$=5.760	$k_{8,r1}$=6.359 $k_{8,r2}$=6.359 $k_{8,r3}$=6.359
Hammer heads in compression (Eq.(4))	k_9=7.160	k_9=13.050	k_9=7.160
Hammer heads in tension (Eq.(4))	$k_{10,r1}$=28.210 $k_{10,r2}=\infty$ [(b)] $k_{10,r3}=\infty$ [(b)]	$k_{10,r1}$=10.270 $k_{10,r2}=\infty$ [(b)] $k_{10,r3}=\infty$ [(b)]	$k_{10,r1}$=28.210 $k_{10,r2}=\infty$ [(b)] $k_{10,r3}=\infty$ [(b)]
Level arms (Fig.2)	z_1=688.250 z_2=551.250 z_3=305.250	z_1=688.250 z_2=451.250 z_3=205.250	z_1=688.250 z_2=551.250 z_3=305.250

[(a)] The detail calculation can be found in [10].

[(b)] As no hammer head component is existing at the level of bolt row 2 and 3, these coefficients are taken as equal to infinite.

Table 10: Joint stiffness estimation and comparison

Quantities and formulas	**Specimens**		
	A1	**A2**	**B1**
Effective stiffness of each bolt row (mm)			
$k_{eff,r1}=\left(\frac{1}{k_{3,r1}}+\frac{1}{k_{4,r1}}+\frac{1}{k_{7,r1}}+\frac{1}{k_{8,r1}}+\frac{1}{k_{10,r1}}\right)^{-1}$	2.639	2.293	2.783

$k_{eff,r2}=\left(\frac{1}{k_{3,r2}}+\frac{1}{k_{4,r2}}+\frac{1}{k_{7,r2}}+\frac{1}{k_{8,r2}}+\frac{1}{k_{10,r2}}\right)^{-1}$	2.851	2.866	3.046
$k_{eff,r3}=\left(\frac{1}{k_{3,r3}}+\frac{1}{k_{4,r3}}+\frac{1}{k_{7,r3}}+\frac{1}{k_{8,r3}}+\frac{1}{k_{10,r3}}\right)^{-1}$	2.927	2.948	3.089
Effective stiffness of compression zone (mm)			
$k_{eff,c}=\left(\frac{1}{k_2}+\frac{1}{k_9}\right)^{-1}$	5.527	8.482	5.398
Equivalent lever arm (mm)			
$z_{eq}=\frac{k_{eff,r1}z_1^2+k_{eff,r2}z_2^2+k_{eff,r3}z_3^2}{k_{eff,r1}z_1+k_{eff,r2}z_2+k_{eff,r3}z_3}$	558.031	516.418	557.952
Equivalent stiffness factor (mm)			
$k_{eq}=\frac{k_{eff,r1}z_1+k_{eff,r2}z_2+k_{eff,r3}z_3}{z_{eq}}$	7.672	6.724	8.132
Joint stiffness (models) (kNm/rad)			
$S_{J,ini}=\frac{Ez_{eq}^2}{\frac{1}{k_1}+\frac{1}{k_{eff,c}}+\frac{1}{k_{eq}}}$	**162500**	**158360**	**146620**
Joint stiffness (test –Table 1) (kNm/rad)	**193 000**	**187 000**	**154 500**
Model-test differences (%)	**15.8**	**15.3**	**5.1**

5. Joint classifications

In Section 3, the analytical tools to estimate the resistance and stiffness of the joints were presented. Now, the question is how to classify the joints in terms of stiffness and resistance.

On one hand, for the stiffness classification (i.e. as pinned, semi-rigid or rigid), the rule as given in EN1993-1-8, 5.2.2 [3] can be directly applied. On the other hand, the resistance classification (i.e. pinned, partially resistant or fully resistant) needs to be clarified, in particular when considering the specific seismic design requirement as given in EN1998-1-1 [5]. The detailed discussion about this question have been dealt with in [11]; a summary is given here below.

According to EN1998-1-1 [5], it is required to take into account of the possible over-strength effects to classify a joint as fully resistant when the capacity design is considered. The objective is to ensure that the plastic hinges developed in the beam sections, and not in the joints, in case of over-strength of the beam material. Accordingly, the following condition as given in EN1998-1-1, 6.5.5 (3) [5] as to be respected:

$$M_{Rd,joint}\geq 1.1\gamma_{ov}M_{pl,beam} \tag{8}$$

where $M_{Rd,joint}$ is the required resistance of the joint; $M_{pl,beam}$ is the plastic moment of the beam section; γ_{ov} is the over-strength factor, equals to 1.25.

The condition as given in Eq.(8) does not take into account of the fact that (i) for some joint configurations as the one investigated here, the beam plastic hinge may form at a certain distance from the joint and that (ii) for some components linked to the beam properties, a possible over-strength effect should not be considered as they are made from the same material as the beam one. Therefore, the condition (8) should be revised in order to take into account the aspects. The proposal is the rewrite the "full strength" condition as follows:

$$\begin{cases} M_{Rd,j} \geq M_{Ed,j} \\ M_{Rd,j} = f_1\left(F_{Rd,beam\,components}, F_{Rd,other\,components}/(1.1\times\gamma_{ov})\right) \\ M_{Ed,j,HOG} = M_{pl,beam} + \left(\frac{2M_{pl,beam}}{l} + \frac{p_{\max}l}{2\times1.1\gamma_{ov}}\right)d_{hj} + \frac{p_{\max}d_{hj}^2}{2\times1.1\gamma_{ov}} \\ M_{Ed,j,SAG} = M_{pl,beam} + \left(\frac{2M_{pl,beam}}{l} - \frac{p_{\min}l}{2\times1.1\gamma_{ov}}\right)d_{hj} - \frac{p_{\min}d_{hj}^2}{2\times1.1\gamma_{ov}} \end{cases} \tag{9}$$

In Eq.(9), $M_{Ed,j}$ is the moment at the joint level when a plastic hinge appears in the beam which can be computed through the equilibrium equation (under hogging or under sagging bending moment – $M_{Ed,j,HOG}$ and $M_{Ed,j,SAG}$ respectively). p_{max} and p_{min} are respectively the maximum and minimum applied loads on the beam. The geometric parameters d_{hj} and l are represented in Fig.14. $M_{Rd,j}$ is the joint resistance computed through the component method, represented by the function f_1 in Eq.(9). In this new so-defined criteria, it is proposed to report the over-strength coefficient on the estimation of the joint resistance instead of applying it to the loading as it was the case in Eq. (8). Accordingly, it can be seen in Eq.(9) that the resistance of the components which are not made of the same material of the beam ($F_{Rd,other\ components}$) are divided by the over-strength coefficient while the resistance of the components made of the same material as the beam ($F_{Rd,beam\ components}$) are not affected by this coefficient. For the investigated joint configuration, the component with "b" in Table 4 are the components defined here as "beam components". Also, in the equations of $M_{Ed,j}$, it can be seen that the over-strength coefficient is applied to some terms associated to the applied load p_{max} or p_{min}. It is due to the fact that the applied load should not be affected by the over-strength effects; accordingly when comparing $M_{Ed,j}$ to $M_{Rd,j}$ in which the overstrength coefficient, the over-strength coefficient applied to the terms associated to the applied load are simplified.

The fact that the over-strength coefficient is not applied to the components made of the same material than the beam and in particular to the hammer-head components which are directly extracted from the beams allows an economical design of the proposed joint configuration as no over-resistance has to be considered for these components.

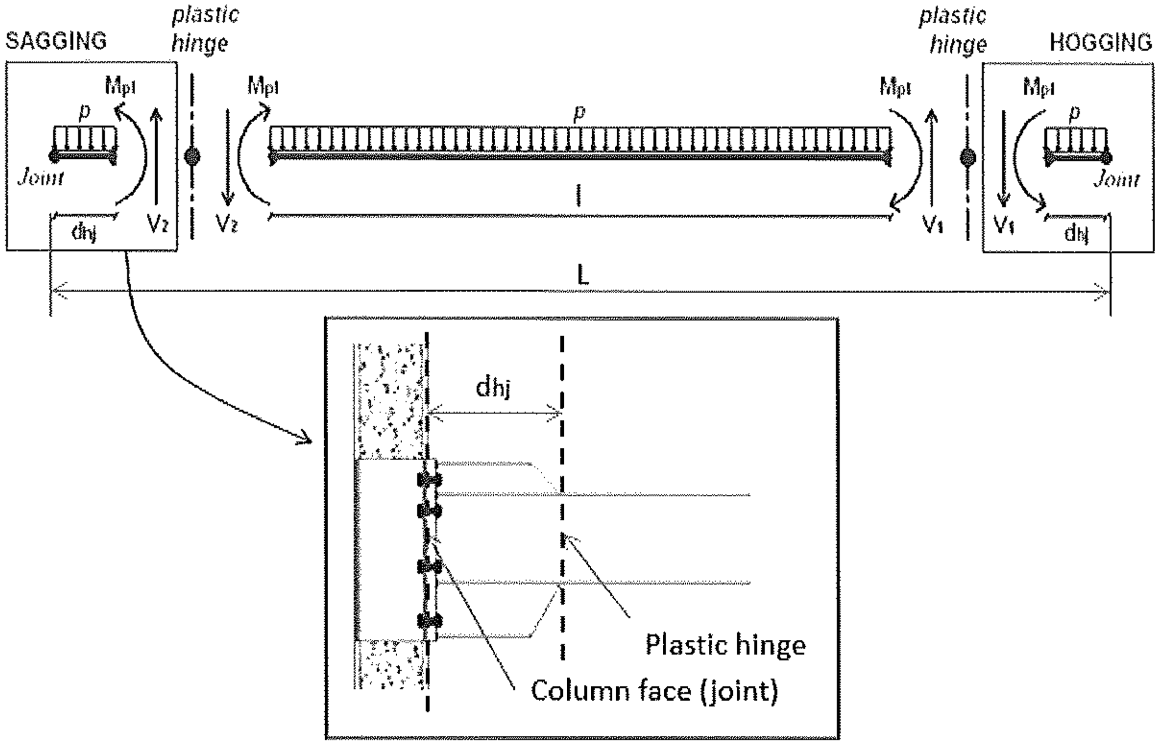


Fig.14. Internal force in joint at the seismic situation

6. Conclusion

A new type of bolted stiffened end-plate beam-to-column joint has been studied in this paper. The proposed joint configuration uses hammed heads extracted from the beam profiles, instead of using traditional haunches. It has been pointed out that the proposed configuration is a consistent/economic solution for beam-to-column joints used in seismic resistant building frames. The economic interest has been drawn from both theoretical and practical evaluations while the good mechanical behaviour of the joint has been demonstrated by the experimental tests.

Analytical tools to characterize the proposed joint in terms of resistance and stiffness have been developed, in full agreement with the component method philosophy as recommended in the Eurocodes. The proposed analytical methods have been validated through comparison to experimental tests. Moreover, an innovative method to take into account the actual position of the plastic hinge and the over-strength factor according to EN1998-1-1 dedicated to the seismic design of buildings has been proposed and presented.

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References

[1] HSS-SERF project “High Strength Steel in Seismic Resistant Building Frames”, final report, 2013.

[2] HSS-SERF project “High Strength Steel in Seismic Resistant Building Frames”, Deliverable D4: Prequalification tests on bolted beam-to-column joints in moment-resisting dual-steel frames report, 2013.

[3] EN1993-1-8-2005. European Committee for Standardization – CEN. Eurocode 3: Design of steel structures. Part 1.8: Design of joints, Brussels.

[4] EN1994-1-1-2005. European Committee for Standardization – CEN. Eurocode 4: Design of composite steel and concrete structures. Part 1.1: General rules and rules for buildings.

[5 EN1998-1-1-2005. European Committee for Standardization – CEN. Eurocode 8: Design of structures for earthquake resistance - Part 1.1: General rules, seismic actions and rules for buildings.

[6] Jaspart J.-P., “Recent advances in the field of steel joints – Column bases and further configurations for beam-to-column joints and beam splices”, Thesis, University of Liege, 1997.

[7] Gome Gomes FCT. Etat limite ultime de la résistance de l’ame d’une colonne dans un assemblage semi-rigide d’axe faible (in french). Rapport interne 203, Technical Report, University of Liege, 1990.

[8] Jaspart J.P. Pietrapertosa C., Weynand K., Busse E., Klinkhammer R (2005). Development of a full consistent design approach for bolted and welded joints in building frames and trusses between steel members made of hollow and/or open sections: application of the component method. Volume 1: practical guidelines. CIDECT Report 5BP – 4/05.

[9] Mágala-Chuquitaype C., Elghazouli A.Y. (2010b) “Component-based mechanical models for blind-bolted angle connections” Engineering Structures Vol. 32, (pp. 3048-3067).

[10] Comeliau L., Demonceau J.F., Jaspart J.P (2012a). “Computation note on the design on bolted beam-to-column joints within HSS-SERF project”, internal report, University of Liege, 2012

[11] Comeliau L., Demonceau J.F., Jaspart J.P (2012b). “Innovative bolted beam-to-column joints for seismic resistant building frames”, in Dubina, Dan; Grecea, Daniel (Eds.) Connection VII proceedings, 2012.