# A new design approach for braced frames with extended end plate connections

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# A NEW DESIGN APPROACH FOR BRACED FRAMES WITH EXTENDED END PLATE CONNECTIONS

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## Summary

The main difficulty to be faced in designing semirigid frames is that the design internal actions of joints depend on the joint rotational behaviour which, in turn, can be determined provided that the joints have been completely detailed. As a consequence, any design method requires many iterations to achieve safe solutions. With reference to braced frames, a new design procedure able to overcome this difficulty is presented in this paper.
The proposed design approach is based on the use of design abaci, developed by the same authors in previous works, relating the joint rotational behaviour to the main geometrical parameters of the structural detail. The innovative feature of the proposed design procedure is its ability to guide the designer up to the complete detailing of beam-to-column connections. Finally, a design example is presented to show the practical application of the proposed design procedure.

## 1. Introduction

Even though the semirigidity concept has been introduced many years ago, steel structures are usually designed by assuming that beam-to-column joints are either pinned or rigid. This design assumption allows a great simplification in structural analysis, but it neglects the true behaviour of joints.
The economic and structural benefits of semirigid connections are well known and much has been written about their use in braced frames. The main advantages they provide over pinned frames are the reduction of the mid-span moments and of the column effective length. Notwithstanding, they are seldom used by designers, because most semirigid connections have highly nonlinear behaviour so that the analysis and design of frames using them is difficult. In particular, the design problem becomes more difficult as soon as the true rotational behaviour of beam-to-column joints is accounted for, because the internal actions that members and joints have to withstand, depend on the joint rotational stiffness. As the joint flexural resistance is strictly related to its rotational stiffness, the design problem requires some attempts to achieve a safe and economical design.
In the case of braced frames, the beam line method is commonly used to face the design problem, but it does not provide any indication regarding the detailing of beam-to-column joints. In other words, as it is difficult to design joints having predetermined values of rotational stiffness and flexural resistance, the most important point in designing semirigid frames is practically still to be solved. For this reason, within a strategic programme

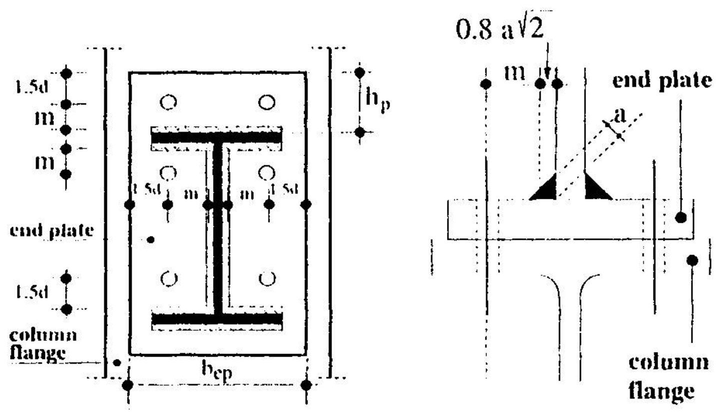



**Fig.1** - Geometrical detail of the analysed joints

«SPRINT» of the European Community [1], tables giving the rotational stiffness and the flexural strength of a great number of joints for different connection typologies have been prepared to provide designers with an important help in designing semirigid frames, allowing the exploitation of the benefits of this structural typology.

Despite the great number of considered cases, these tables do not represent an exhaustive solution of the design problem. For this reason, with reference to extended end plate connections a new design procedure is herein proposed with the aim to guide the designer up to the complete detailing of beam-to-column joints.

## 2. Design of extended end plate connections

The rotational behaviour of extended end plate connections can be predicted by means of the procedure suggested by Eurocode 3 in its Annex J [2]. The reliability of Annex J procedure for predicting the rotational behaviour of extended end plate connections has been statistically investigated by the authors [3,4,5] on the basis of comparison with a great number of experimental data collected in the technical literature [6-11]. In addition, some proposals have been developed to improve the codified approach leading to a better agreement with the experimental data.

Starting from these results, in order to stress the role of the main geometrical parameters defining the structural detail of extended end plate connections, a wide parametric analysis has been carried out [12].

The end plate of the analysed joints is extended at the beam tension flange side (Fig.1). At the tension flange level, the fastening action is assured by two bolt rows with two bolts for each row. Unstiffened joints (i.e. without continuity plates), both external and internal, have been considered by varying the column section, the beam section, the $m/d$ ratio (Fig.1) (where d is the bolt diameter), the end-plate thickness and the bolt class.

In order to assure an adequate rotation capacity and to simplify the design procedure, the bolts have been designed to withstand the axial forces corresponding to a bending moment equal to 1.20 times the beam plastic moment.

Concerning the joint components affected by the state of stress of the column (column web in shear, column web in compression and column web in tension), some assumptions have been made. In particular, as the aim of the work [12] is to provide a design tool for detailing beam-to-column joints, simplified values of the coefficients taking into account the above state of stress have been considered [2]:

- the coefficient $\xi$ taking into account the influence of the shear force in the column has been assumed equal to 1.0 in the case of external joints and equal to 0 in the case of internal joints, as suggested in Annex J;
- the coefficient $k_{wc}$ taking into account the influence of the normal stress in the web (adjacent to the root radius), duę to axial force and bending moment, has been assumed equal to 0.75, i.e. the most severe design condition has been considered.

In addition, as the aim of the work is to provide the designer with operative tools to quickly evaluate to joint resistance rather than the resistance of the joint-beam system, the limitation to the resistance given by the beam web and beam flange in compression has not been considered. This allows to classify the joints as full strength joints when the design

flexural resistance exceeds that of the connected member or as partial strength joints in the opposite case.
The flexural strength and the rotational stiffness of the examined joints have been computed by a modified version of Annex J, according to the authors' proposals [3,4,5].
The first outcome of this parametric analysis is the relationship between the rotational stiffness and the flexural resistance of joints. To this scope, it is useful to adopt the concept of equivalent beam length [13]. The equivalent beam length $L_e$ represents the value of the beam length which corresponds to the equality between the joint rotational stiffness and the beam flexural stiffness:

$$K_\varphi = \frac{E I_b}{L_e} = \frac{E I_b}{\eta\, d_b} \qquad (1)$$

where the equivalent beam length has been expressed as $\eta$ times the beam depth $d_b$ (where $K_\varphi$ is the joint rotational stiffness and $I_b$ is the beam inertia moment).
According to this definition, the parameter $\eta$ can be used to represent the joint rotational deformability:

$$\eta = \frac{L}{d_b\, K} \qquad (2)$$

where $K$ is the nondimensional rotational stiffness of the joint, defined as the ratio between the joint rotational stiffness $K_\varphi$ and the beam flexural stiffness $EI_b/L$ (where $L$ is the beam length). The parameter $\eta$ can be conveniently used, because it provides through $1/\eta$ a nondimensional stiffness independent of the beam length as it is immediately recognized considering that $1/\eta = K_\varphi\, d_b/E\, I_b$.
In addition, the joint flexural resistance can be expressed through the nondimensional parameter:

$$\overline{M} = \frac{M_{j.Rd}}{M_{b.Rd}} \qquad (3)$$

which represents the ratio between the design flexural resistance of the joint and that of the connected beam.
Starting from the consideration that the joint flexural resistance increases as the rotational deformability decreases and accounting for the results of a wide parametric analysis [12], the following mathematical structure has been chosen for the $\overline{M} - \eta$ relationship:

$$\overline{M} = C_1\, \eta^{-C_2} \qquad (4)$$

where $C_1$ and $C_2$ are two constants which can be computed by regression analysis.

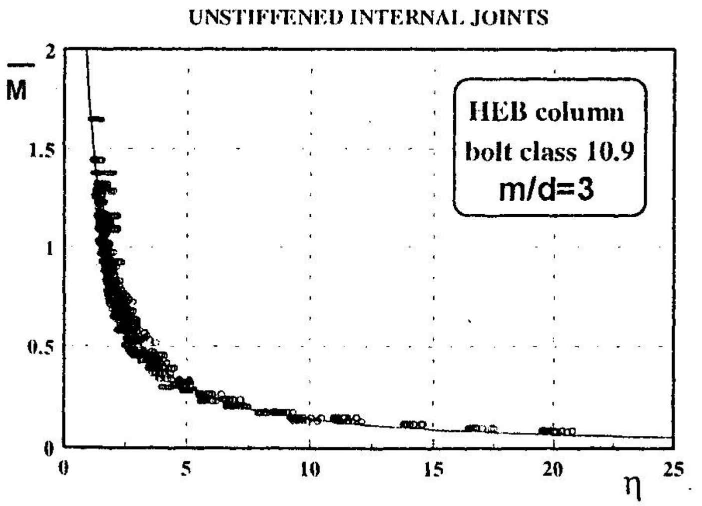



**Fig.2** - $\overline{M} - \eta$ relationship for unstiffened internal joints and $m/d = 3$

The regression analyses of the results of the numerical simulations have confirmed the validity of the above relationship provided that the influence of the spacing between the bolts and the beam section is taken into account. In other words, it is possible to obtain a relationship of type (4) for any given value of the parameter $m/d$ [12].
With reference to unstiffened internal joints, the relationship $\overline{M} - \eta$ is presented in Fig.2 for $m/d$=3, where the points represent the data of the numerical analyses. As an example, the coefficients $C_1$ and $C_2$, the

**Table 1** - Coefficients of $M-\eta$ regressions (HEB columns, IPE beams, bolt class 10.9)

| GROUP | $m/d$ | $C_1$ | $C_2$ | $n$ | $r$ |
|---|---|---|---|---|---|
| UNSTIFFENED INTERNAL JOINTS | 2 | 2.1421 | 1.6825 | 1239 | 0.91 |
| | 3 | 1.7691 | 1.0955 | 1463 | 0.97 |
| | 4 | 1.6080 | 0.8482 | 1309 | 0.98 |
| | 5 | 1.5167 | 0.7164 | 1232 | 0.98 |
| UNSTIFFENED EXTERNAL JOINTS | 2 | 3.6069 | 1.7982 | 1239 | 0.94 |
| | 3 | 2.2169 | 1.1569 | 1463 | 0.97 |
| | 4 | 1.8309 | 0.8817 | 1309 | 0.98 |
| | 5 | 1.6416 | 0.7351 | 1232 | 0.98 |

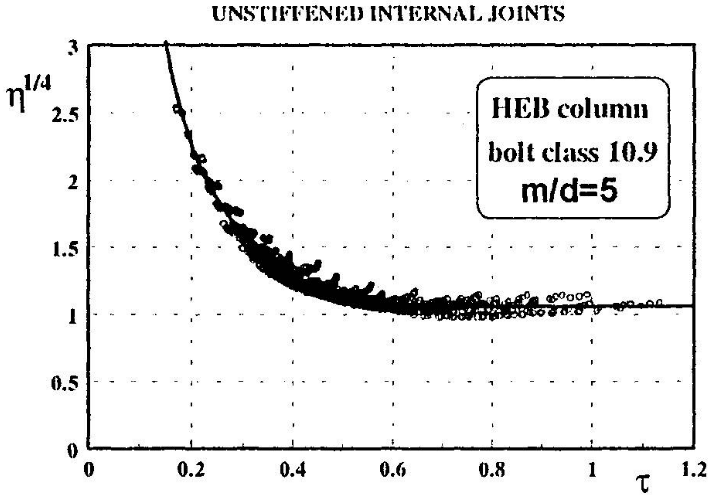



**Fig.3** - $\eta-\tau$ relationship for unstiffened internal joints and $m/d=5$

correlation coefficient $r$ and the data number $n$ are given in Table 1 with reference to HEB columns, IPE beams and bolt class 10.9. The complete series of results for HEA and HEM columns and for bolt class 8.8 are presented in reference [12].

The correlation coefficients are always very close to 1 confirming the accuracy of the proposed relationship (4).

The examination of the failure modes of the designed extended end plate joints have evidenced that the column flange and the end plate in bending are generally involved [12]. Therefore, it can be stated that, for any given $m/d$ ratio, the most important geometrical parameters governing the joint behaviour are the column flange thickness and the end plate thickness. For this reason, in order to account for the fact that the column flange in bending and the end plate in bending behave as a series of springs, the following parameter $t_{eq}$ has been introduced:

$$\frac{1}{t_{eq}^3} = \frac{1}{t_{fc}^3} + \frac{1}{t_{ep}^3} \tag{5}$$

where $t_{fc}$ and $t_{ep}$ are the thicknesses of the column flange and of the end plate, respectively.

This parameter has been properly nondimensionalized according to the following relationship:

$$\tau = \left(t_{eq}^3\, d_b/I_b\right)^{1/4} \tag{6}$$

The relationship between the joint rotational deformability, expressed by means of the parameter $\eta$, and the thickness of the connected elements, expressed by $\tau$, can be investigated through the numerical data of the parametric analysis.

Starting from the consideration that, obviously, the joint rotational deformability increases as the thickness of the connected elements decreases and from the observation of the numerical analysis data, the following mathematical structure has been selected for the $\eta-\tau$ relationship:

$$\eta^{0.25} = \frac{C_3}{\tau - C_4} + C_5 \geq C_6 \tag{7}$$

**Table 2** - Coefficients of $\eta - \tau$ regressions (HEB columns, IPE beams, bolt class 10.9)

| GROUP | $m/d$ | $C_3$ | $C_4$ | $C_5$ | $C_6$ | $s$ | $n$ |
|---|---|---|---|---|---|---|---|
| UNSTIFFENED INTERNAL JOINTS | 2 | 0.081 | 0.035 | 0.850 | 1.128 | 0.032 | 1239 |
| | 3 | 0.172 | 0.024 | 0.655 | 1.111 | 0.043 | 1463 |
| | 4 | 0.248 | 0.027 | 0.535 | 1.089 | 0.047 | 1309 |
| | 5 | 0.310 | 0.029 | 0.459 | 1.054 | 0.056 | 1232 |
| UNSTIFFENED EXTERNAL JOINTS | 2 | 0.060 | 0.047 | 1.034 | 1.182 | 0.032 | 1239 |
| | 3 | 0.146 | 0.032 | 0.797 | 1.148 | 0.032 | 1463 |
| | 4 | 0.204 | 0.044 | 0.681 | 1.104 | 0.035 | 1309 |
| | 5 | 0.296 | 0.031 | 0.526 | 1.070 | 0.046 | 1232 |

where the coefficients $C_3$, $C_4$, $C_5$ and $C_6$ can be computed through a nonlinear regression by means of the least squares method.

With reference to unstiffened internal joints and to $m/d = 5$, the relationship $\eta - \tau$ and the corresponding data are presented in Fig.3, where the double square root of the parameter $\eta$ has only been used to improve the readability of the figure. As an example, the coefficients $C_3$, $C_4$, $C_5$ and $C_6$ corresponding to HEB columns, IPE beams and bolt class 10.9 are given in Table 2 where the standard deviation $s$ and the data number $n$ are also presented. The complete series of results is given in reference [12] where HEA and HEB columns and bolt class 8.8 are also considered.

It is interesting to point out the physical meaning of the limitation provided to the connection deformability parameter $\eta$ by the coefficient $C_6$. In fact, the influence of the joint components depending on the column section (i.e. the column web in shear, the column web in compression, the column flange in bending and the column web in tension) becomes more and more important as the end plate thickness increases. As a consequence, when the end plate thickness is sufficiently great so that its deformability is negligible, the joint deformability becomes almost constant being a feature of the beam-column coupling, of the $m/d$ ratio and of the bolt class.

## 3. Design abaci

The results of the parametric analysis have pointed out that the behavioural parameters of extended end plate connections ($\overline{M}$, $\eta$), are strictly related. In addition, the deformability parameter $\eta$ is strictly related to the parameter $\tau$ which accounts for the influence of the thickness of the connected elements.

From the design point of view, it has to be pointed out that, according to Annex J, nonlinearity arises before the design resistance of beam-to-column joints is completely developed (Fig.4). As, for economy, joints have to be designed to obtain a flexural resistance $M_{j.Rd}$ close to the design bending moment $M_{j.Sd}$, this means that elastic structural analyses can be carried out on the basis of the secant rotational stiffness of the joints [14], corresponding to $M_{j.Rd}$. According to Annex J, this secant stiffness is given by (Fig.4):

$$K_{\varphi_{sec}} = 0.335\, K_{\varphi} \tag{8}$$

The corresponding nondimensional secant rotational stiffness is given by:

$$K_{sec} = \frac{K_{\varphi_{sec}}\, L}{E\, I_b} = 0.335\, K \tag{9}$$

Obviously, the corresponding secant deformability parameter can be defined according to the relationship:

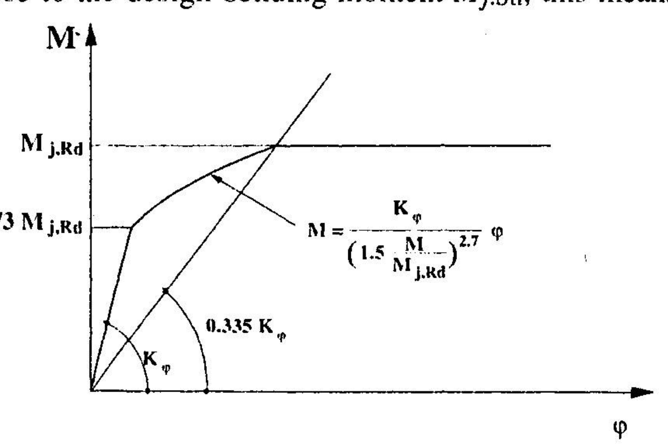



**Fig.4** - Moment-rotation curve according to Annex J

$$\eta_{sec} = \frac{L}{d_b K_{sec}} = \frac{1}{0.335}\eta \approx 3\eta \qquad (10)$$

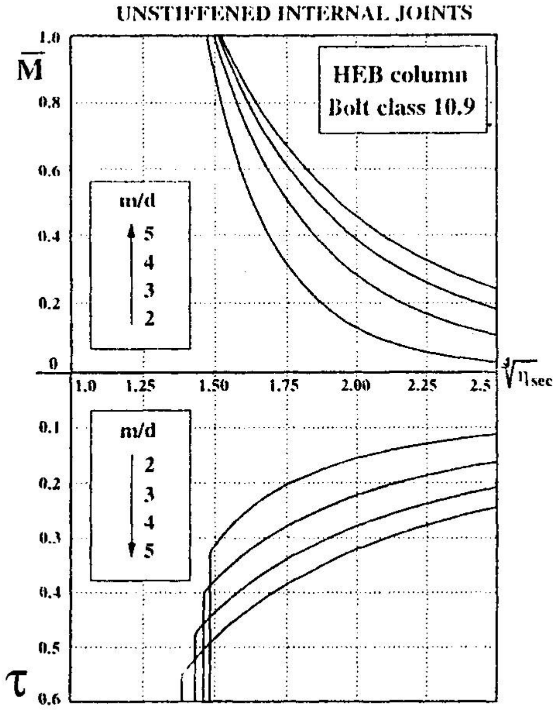



**Fig.5** - Design abacus for unstiffened internal joints

As the joint design has to be based on the secant stiffness $K_{\varphi sec}$, it is clear that, for design purposes, the previous correlations $\overline{M}$ versus $\eta$ and $\eta$ versus $\tau$ have to be rearranged using the secant deformability parameter $\eta_{sec}$.

The relationships obtained by the regression analyses provide the designer with an operative tool for detailing beam-to-column connections. In fact, for each group of joints, it is possible to provide the design abacus presented, as an example, in Fig. 5 where reference is made to the joint secant deformability parameter $\eta_{sec}$.

The structural analysis requires the knowledge of the joint rotational stiffness, whose value can be chosen on the basis of different design requirements such as the limitation of the beam deflections imposed by service conditions in braced frames. For a given beam and a given column, the lower part of the design abaci provides the end-plate thickness required to assure the desired value of the joint rotational stiffness, for different values of the $m/d$ ratio. In addition, by means of the upper part of the abaci the flexural resistance of the joint can be evaluated as a function of the $m/d$ ratio which, therefore, can be selected on the basis of the design internal actions obtained from elastic analysis.

However, it must be stressed that any design approach generally requires an iterative procedure, because the internal actions that joints have to withstand depend on the joint properties. A method to overcome this difficulty will be presented in Section 4 with reference to semirigid braced frames and uniform loads acting on the beams. The procedure can be easily extended to other loading conditions.

# 4. Design of braced frames

## 4.1 Design conditions

Braced frames are usually designed assuming that beams are pin-jointed to the columns. In other words, the beam-to-column joints are designed to transmit the shear forces only and the beams are designed to withstand a bending moment equal to $q_t L^2/8$, where $q_t$ is the total vertical uniform load (including the partial safety factors, i.e. $q_t = 1.35\, g_k + 1.5 q_k$ where $g_k$ and $q_k$ are the characteristic values of the permanent and live load, respectively) acting on the beams whose span is $L$.

The use of semirigid joints, such as extended end plate connections, allows to reduce the maximum bending moment and the midspan deflection that the beam has to sustain so that a smaller section can be adopted.

The design procedure of braced frames can be based on a very simple model represented by a beam partially restrained at its ends. With reference to this model, five design conditions have to be taken into account. The first two conditions are the check of beam resistance against the sagging and hogging moment, respectively. Other two conditions concern the serviceability limit state requiring the limitation of the beam deflection under both live loads and total loads. The last design condition is the check of the resistance of the joints subjected to the hogging moments.

The check of the beam resistance against the sagging moment requires the fulfilment of the following relationship:

$$K_{sec} \geq \frac{6(1-\alpha)}{3\alpha - 1} \tag{11}$$

where:

$$\alpha = \frac{M_{b.Rd}}{q_t L^2/8} \tag{12}$$

The check against the hogging moment is given by:

$$K_{sec} \leq \frac{6\alpha}{2 - 3\alpha} \tag{13}$$

It is important to underline that both in equation (11) and in equation (13) reference has been made to the secant stiffness. This is justified taking into account that an economic design of joints requires a joint flexural resistance very close to the design hogging moment.

With reference to the serviceability limit state, according to Eurocode 3 [15], the maximum beam deflection under live loads has to be less than 1/350 times the beam span. This requirement can be expressed by the relationship:

$$K \geq \frac{2\beta_l}{1 - \beta_l} \tag{14}$$

The parameter $\beta_l$ is given by:

$$\beta_l = \frac{5}{4} - \frac{f_l\, 96\, E I_b}{q_k L^4} \tag{15}$$

where $q_k$ is the characteristic value of the uniform live load and $f_l = L/350$ is the limit deflection under live loads.

With reference to the secant stiffness, equation (14) gives:

$$K_{sec} \geq \frac{6\beta_l}{1 - \beta_l} \tag{16}$$

In addition, according to Eurocode 3 [15], the maximum beam deflection under the total loads has to be less than $f_t = L/250$. This design condition leads to the relationship:

$$K_{sec} \geq \frac{6\beta_t}{1 - \beta_t} \tag{17}$$

where:

$$\beta_t = \frac{5}{4} - \frac{f_t\, 96\, E I_b}{q_t L^4} \tag{18}$$

It must be stressed that the use of the initial nondimensional rotational stiffness $K$ (instead of $K_{sec}$) in equation (14) is due to the reduced load levels for serviceability limit states (i.e in this case $q_l = g_k + q_k$) which leads to a significant reduction of the bending moment $M_{j.Sd}$. This justifies the use of the initial rotational stiffness of joints in evaluating the beam deflection.

Therefore, according to the first four design conditions, the nondimensional rotational stiffness of the joint has to be designed so that the secant stiffness lies in the range $K_{sec_{min}} - K_{sec_{max}}$ defined by:

$$K_{sec_{min}} = \max\left\{\frac{6(1-\alpha)}{3\alpha - 1}\ ;\ \frac{6\beta_l}{1-\beta_l}\ ;\ \frac{6\beta_t}{1-\beta_t}\right\} \tag{19}$$

$$K_{sec_{max}} = \frac{6\alpha}{2 - 3\alpha} \tag{20}$$

When the parameter $\alpha$ exceeds ⅔ there is not any limitation to the joint rotational stiffness.

The last design condition regards the check of resistance of the joints. This condition defines the minimum strength that the joints have to develop, through the relationship:

$$\frac{q_t L^2/12}{M_{b.Rd}} \frac{K_{sec}}{K_{sec} + 2} \leq \overline{M} \tag{21}$$

which, through equations (12) and (10), gives:

$$\overline{M} \geq \frac{1}{3\,\alpha} \frac{2}{1 + \dfrac{2\,\eta_{sec}}{L/d_b}} \tag{22}$$

### 4.2 Design algorithm

The previous design conditions and the relationships relating the joint rotational behaviour to its geometrical parameters allow to develop a design algorithm which provides simultaneously the beam section and the geometrical parameters of the joints. The design algorithm is given by the following steps:

a) select the beam section, according to the most economical solution, to withstand a bending moment equal to $q_t L^2/16$ which corresponds to a nondimensional secant stiffness $K_{sec}$ of the joint equal to 6;

b) as, in general, the design resistance of the selected beam section exceeds $q_t L^2/16$, compute the range of stiffness $K_{sec_{min}} - K_{sec_{max}}$, given by equations (19) and (20), and the corresponding range of the joint rotational deformability $\eta_{sec_{min}} - \eta_{sec_{max}}$ defined by:

$$\eta_{sec_{min}} = \frac{L}{d_b\, K_{sec_{max}}} \qquad \eta_{sec_{max}} = \frac{L}{d_b\, K_{sec_{min}}} \tag{23}$$

where $\eta_{sec_{min}} = 0$ when $\alpha$ exceeds 2/3.

c) for the selected $m/d$ ratio, compute the coordinates ($\eta^*_{sec}$,$\overline{M}^*$) of the point A (Fig.6) corresponding to the intersection between the continuous curve representing the flexural resistance which the joint is able to provide (i.e. equation (4) rearranged as $\overline{M} - \eta_{sec}$ taking into account that $\eta_{sec} \approx 3\,\eta$) and the dashed curve, given by equation (22), representing the design value of the bending moment (for a given $\alpha$ value). This figure refers to the practical application of the proposed design method, corresponding to the example given in the following Section;

d) control the location of the intersection point. If for the selected $m/d$ ratio the intersection point is outside the range $\eta_{sec_{min}} - \eta_{sec_{max}}$ the joint cannot be designed for the chosen beam. In such a case, select the next beam section from the standard shapes and return to point b). On the contrary, if for a selected $m/d$ ratio the intersection point lies within the above range, design the beam-to-column joints according to the following steps;

e) for the selected $m/d$ ratio, compute the $\tau$ parameter which, according to equation (7), is given by:

$$\tau = \frac{C_3}{\eta^{0.25} - C_5} + C_4 \tag{24}$$

f) compute the parameter $t_{eq}$ through equation (6);

g) for a given column section, compute the end plate thickness through equation (5) which provides:

$$t_{ep} = \frac{t_{eq}\, t_{fc}}{\left(t_{fc}^3 - t_{eq}^3\right)^{1/3}} \tag{25}$$

Equation (25) can be applied provided that $t_{fc} > t_{eq}$. If the above condition is not satisfied then select the next beam section from the standard shapes or, if any other design restraint exists, increase the column size and return to point b).

In fact, it is important to underline that, for a given beam section, the requirement $t_{fc} > t_{eq}$ shows that it is not always possible to design joints having a fixed rotational behaviour, i.e. strength and stiffness, with an arbitrary column section. This is justified by the fact that the joint behaviour is also governed by some components depending on the column section. Typical cases are those of high beams which cannot be combined with small columns due to the collapse of one of the joint components belonging to the column.

## 5. Application

In order to show the practical application of the proposed procedure, the design of a braced frames has been developed and a comparison, from the economical point of view, between the solution with pin-joints (as an example double web angle connections) and that with semirigid joints is carried out.

The bay span of the examined frame is equal to 7.0 m and the interstorey height is equal to 3.5 m (Fig.8). All members are in Fe360 steel. The uniform loads acting on the beams are 28.5 kN/m and 19 kN/m for permanent and live loads respectively, including the partial safety factors equal to 1.35 and 1.5 respectively.

In the solution with pinned joints the beams have an IPE450 section, while the use of semirigid joints allows to reduce the beam section up to an IPE360. The beam-to-column joints have been designed according to the method previously described. Reference has been made to an $m/d$ ratio equal to 2.

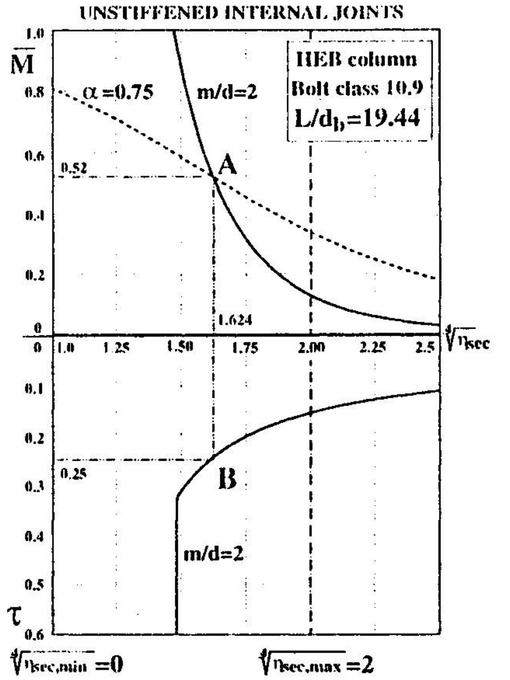



**Fig.6** - Design procedure for braced frames

The graphical representation of the design procedure is given in the already mentioned Fig.6, with reference to internal joints. For the given loading condition and the selected beam (IPE360) the parameter α is equal to 0.75 and the required joint flexural resistance, as a function of the joint rotational deformability, is represented by the dashed curve. The intersection (A) with the continuous curve, representing for $m/d = 2$ the resistance that joint is able to develop, provides $\overline{M} = 0.52$ and $\eta_{sec}^{0.25} = 1.624$. This solution lies within the range defined by equations (23), being $\eta_{sec_{min}} = 0$ and $\eta_{sec_{max}} = 16$, therefore it satisfies resistance and deformability requirements. The value of the parameter τ defining the end plate thickness is equal to 0.25. For each column section, the corresponding minimum value of the required end plate thickness $t_{ep_{min}}$, computed through equations (6) and (25), is given in Fig.7 where the adopted design value $t_{ep}$ is also shown. Furthermore, the design results concerning external joints, obtained with the same method, are also indicated. In addition, for each designed joint, this figure provides the values of the nondimensional secant and initial rotational stiffness ($K_{sec}$ and $K$, respectively) and flexural resistance computed by the modified version [3,4,5] of Annex J for the adopted values of the end plate thickness.

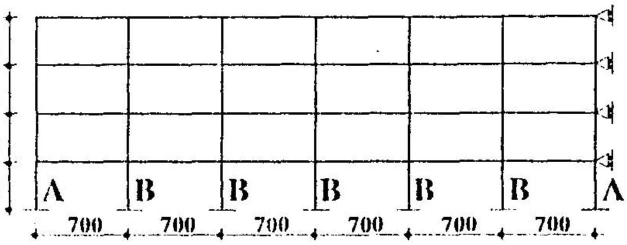
On the basis of the computed joint rotational stiffness, the elastic analysis of the designed semirigid frame has been carried out and the stability and resistance of members has been checked according to Eurocode 3 [15].

With reference to the examined structural scheme, the use of semirigid joints has led to a significant economy in structural weight (18.1%).

## 6. Conclusions

In this paper, the relationships between the parameters describing the rotational behaviour of extended end plate connections, i.e. nondimensional strength and stiffness, have been recalled evidencing how they can be predicted on the basis of some important geometrical parameters, such as the $m/d$ ratio, the end plate thickness and the column flange thickness.

Starting from these results, effective design tools have been suggested and their use in a rational design procedure has been presented for braced frames. The innovative feature of the proposed design procedure consists in its ability to guide the designer up to the complete detailing of beam-to-column joints.




| | | PINNED | SEMIRIGID | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | profile | $t_{ep,min}$ (mm) | $t_{ep}$ (mm) | K | $K_{sec}$ | $\overline{M}$ |
| COLUMNS | A | HE 180 B | HE 180 B | 16.0 | 16 | 4.61 | 1.54 | 0.37 |
| | B | HE 220 B | HE 220 B | 14.0 | 15 | 8.98 | 3.01 | 0.59 |
| BEAMS | | IPE 450 | IPE 360 | | | | | |
| TOTAL WEIGHT | | 19.50 t | 15.97 t (-18.10 %) | | | | | |

**Fig.7** - Application of the proposed design procedure and comparison with the pinned solution

Finally, the design example presented in this paper has shown the economical convenience of using semirigid joints. Taking into account that, as suggested by some authors [16], the increase of cost with respect to pinned frames due to the detailing of beam-to-column joints is about 5%, the economy, from the point of view of the overall cost of the structure, can reach 10% and more.

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