Seismic design of steel frames with partial strength joints

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Abstract

In the seismic design of steel frames, beam-to-column joints can be designed either as full strength joints, forcing the location of the plastic hinges at the beam ends, or as partial strength joints which have to dissipate the seismic input energy. Seismic codes provide specific design criteria for full strength joints, but there are no detailed recommendations dealing with partial strength connections. Therefore, in this paper, by means of a simplified model, such as a SDOF system, the requirements which partial strength joints have to possess, for designing steel frames characterised by seismic performances equivalent to those of steel frames with rigid full strength joints, are pointed out. Successively, starting from the above requirements, a new method for designing seismic resistant steel frames with extended end plate connections leading to the complete definition of the geometrical and mechanical parameters of the joints is proposed.

Keywords: seismic design, steel frames, partial strength joints.

1 Introduction

It is well known that in the seismic design of steel frames the dissipation of the seismic input energy is provided by the plastic engagement of some zones of structural members, the so-called “dissipative zones” which have to be properly detailed in order to assure wide and stable hysteresis loops. In addition, it also known that, in order to design dissipative structures, it is important to promote the plastic engagement of the greatest number of dissipative zones. Aiming to prevent the premature plastic engagement of the columns which can lead to a non dissipative collapse mechanism, both in the American seismic codes [1, 2] and in Eurocode 8 [3] the member hierarchy criterion is suggested which
imposes that, at each beam-to-column joint, the flexural strength of columns has
to be sufficiently greater than the flexural strength of the beams.

However, the fulfillment of this design criterion, is only able to prevent the
development of storey mechanism, but it is not sufficient to guarantee the
formation of a collapse mechanism of global type. To this scope, a more
sophisticated design procedure can be suggested as, for example, the method

Concerning the location of dissipative zones, beam-to-column joints play a
role of paramount importance. In fact, beam-to-column joints can be designed
either as full or partial strength so that, in the first case, the seismic input energy
is absorbed by means of cyclic excursions of the beam ends in plastic range,
while, in the second case, the plastic engagement of ductile joint components
supplies the required dissipation capacity. In order to design full strength joints,
the recent version of Eurocode 8 [3] states that the degree of overstrength
required to assure the beam end yielding is guaranteed in the case of full
penetration butt welds or satisfying, in the case of other joint typologies, the
following relationship:

$$M_{Rd} \geq 1.1 \gamma_{ov} M_{pb}$$

where $M_{Rd}$ represents the joint design resistance, $M_{pb}$ the plastic resistance of the
connected beams and $\gamma_{ov}$ is an overstrength factor accounting for the random
variability of the steel yield stress, while the coefficient 1.1 covers the effects of
material strain hardening.

Moreover, in [5] it has been underlined that in eqn (1) the coefficient 1.1,
accounting for the steel strain hardening, is generally underestimated while the
coefficient $\gamma_{ov}=1.25$, accounting for the material random variability, is generally
overestimated. Therefore, it is evident that the design of full strength joints
requires both an accurate joint detailing and a properly calibrated design rule
accounting both for overstrength and random material variability.

Within this framework, an alternative design approach consists in the
dissipation of the seismic input energy by means of the plastic engagement of
dissipative joint components. It is important to underline that the last version of
Eurocode 8 [3], has explicitly opened the door to the use of partial strength joints
underlining the possible location of the dissipative zones at the beam ends or in
the joints. In addition, it can be recognized that partial strength joints can lead
also to structural solutions convenient from an economical point of view [6].

The seismic behavior of semirigid steel frames with partial strength joints
has been already examined by many authors and some proposals for the behavior
factor of semirigid partial strength frames have been outlined [7, 8, 9].
Notwithstanding, detailed design procedures for the seismic design of semirigid
partial strength frames still deserves specific investigations.

In order to provide a contribution for covering this gap, in this work a
criterion for designing partial strength joints in seismic resistant steel frames is
identified and a design procedure is outlined. In particular, in the first part of the
work (Section 3) the ductility demand for partial strength joints is investigated
assuming as design condition the equivalence of the performance of the steel
frames with partial strength joints and that of the corresponding steel frames with
full strength joints. In the second part of the work (Section 4), with reference to
steel frames with partial strength joints of the extended end plate typology,
starting from the ductility demand, a new method for designing seismic resistant steel frames satisfying both service and ultimate requirements is proposed. In addition, the suggested approach leads to the complete detailing of the joint geometrical and mechanical properties as pointed out by means of a worked numerical example presented in Section 5.

2 Simplified model

The analysis of the performance which the joints have to supply in seismic design of semirigid steel frames requires the preliminary evaluation of the influence of the joint behavior on the elastic and inelastic behavior of steel frames. This topic has been examined in previous works by means of the model represented in fig. 1 [10, 11]. The model is a substructure extracted by the frame assuming that the beams belong to two storeys, so that they can be characterized by halved mechanical properties.

The basic hypothesis of the model consists in the assumption of a double curvature diagram for the beams with zero moment in the midspan section. The behavior of the simplified model is governed by the following nondimensional parameters:

$$\zeta = \frac{EI_b}{L_b}, \quad \bar{K} = \frac{K_\phi L_b}{EI_b}, \quad \bar{m} = \frac{M_{jRd}}{M_{bRd}},$$

(2)

where $\zeta$ represents the ratio between the flexural stiffness of the beams and that of the column, $\bar{K}$ is a stiffness parameter given by the ratio between the rotational stiffness of the connection $K_\phi$ and the flexural stiffness of the connected beam and, finally, $\bar{m}$ is a resistance parameter given by the ratio between the joint resistance $M_{jRd}$ and the beam plastic moment $M_{bRd}$. In eqn (2), $I_b$ and $L_b$ are the inertia and the length of the beam, respectively, while $I_c$ and $h$ are the inertia and the height of the column, respectively.

![Adopted simplified model](image)

**Figure 1:** Adopted simplified model.
In [10] the elastic and inelastic behaviour of the described model are examined.

First of all, the influence of the joint rotational stiffness on the period of vibration and on the sensitivity of the model to the second order effects has been analysed. In particular, this last influence has been examined by means of the stability coefficient \( \gamma = N/(K_0 h) \), where \( N \) is the axial force in the column and \( K_0 \) is the lateral stiffness of the model.

The analysis of the simplified model provides the following relationships:

\[
\frac{T_k}{T_\infty} = \sqrt{\frac{K(1 + \zeta) + 6}{K(1 + \zeta)}}, \quad \frac{\gamma_k}{\gamma_\infty} = \frac{K(1 + \zeta) + 6}{K(1 + \zeta)},
\]

where \( T_k \) and \( \gamma_k \) are the period of vibration and the stability coefficient of the model with semirigid joints and \( T_\infty \) and \( \gamma_\infty \) are the corresponding values of the model with rigid joints.

Regarding the frame inelastic behaviour, the simplified model allows to clarify the influence of the joint behaviour on the frame global ductility. With reference to rigid full strength joints, the global ductility of the model \( \mu_{FS} \), governed by the plastic rotation capacity of the beams, can be expressed by means of the following relation [10]:

\[
\mu_{FS} = 1 + \frac{3}{2} \frac{R_b}{1 + \zeta}
\]

where \( R_b \) represents the rotation capacity of the beam.

With reference to the model with partial strength joints, the ductility \( \mu_{PS} \) can be expressed by means of the following relation:

\[
\mu_{PS} = 1 + \frac{6}{K + 6 + K\zeta}
\]

where \( R_c \) represents the rotation capacity of the joint given by the ratio between the ultimate plastic rotation of the joint \( \vartheta_{jp} \) and \( \vartheta_{jy} = M_{fy}/K_\varphi \), which is the rotation corresponding to first yielding.

3 Seismic behaviour

3.1 q-factor of the simplified model

The simplified model adopted for evaluating the influence of the joint behaviour on the elastic and inelastic behaviour of steel frames is also able to investigate the seismic response of steel frames. It is equivalent to an elastic-perfectly plastic SDOF system accounting for second order effects [10]. In particular, the q-factor can be evaluated by means of the following relation:

\[
q(\mu, T; \gamma) = \frac{q(\mu, T; \gamma = 0)}{\varphi(\mu, \gamma)}
\]

where \( q(\mu, T; \gamma = 0) \) represents the q-factor of elastic-perfectly plastic SDOF system characterised by ductility \( \mu \) and period of vibration \( T \), while \( \varphi(\mu, \gamma) \) is a coefficient which accounts for the influence of second order effects given by [12]:
\[ \varphi_\Delta = 1 - \psi_1 (\mu - 1)^{\gamma / 2} / (1 - \gamma) \]  
(7)

where the coefficients \( \psi_1 \) and \( \psi_2 \), equal to 0.62 and 1.45 respectively, provide the average value of \( \varphi_\Delta \).

Regarding the q-factor of the system in absence of second order effects, \( q(\mu, T, \gamma = 0) \), the following relationships proposed by Krawinkler and Nassar [13] can be adopted:

\[ q = \left[ c(\mu - 1) + 1 \right]^{1} / c \]  
with \( c = T / (1 + T + 0.42) \)  
(8)

Therefore, with reference to the model with partial strength joints, the q-factor is given by a function of five parameters as follows:

\[ q^K_{PS} = \left[ c\left(\mu^K_{PS} - 1\right) + 1 \right]^{1} \left[ 1 - \gamma_K \right] = f\left(\mu^K_{PS}, T_K, \gamma_K\right) = f\left(R_c, K, \zeta, T_K, \gamma_K\right) \]  
(9)

Taking into account eqn (3) and considering that in the period range of steel structures (typically \( T > 0.8 \) sec), according to the ductility factor theory also Krawinkler and Nassar model provides \( q \approx \mu \) independently of the period of vibration, the q-factor of the model with partial strength joints reduces to a function of four parameters:

\[ q^K_{PS} = f\left(R_c, K, \zeta, \gamma_K\right). \]  
(10)

Similarly, with reference to the model with rigid joints, the following expression of the q-factor can be obtained:

\[ q^K_{FS} = \left[ c\left(\mu^K_{FS} - 1\right) + 1 \right]^{1} \left[ 1 - \gamma_K \right] = f\left(\mu^K_{FS}, T_K, \gamma_K\right) = f\left(R_c, K, \zeta, \gamma_K\right) \]  
(11)

which points out, in this case, the influence of three parameters.

3.2 Seismic performance and design criterion

The capacity of the structure to withstand destructive seismic events depends not only on its capacity of energy dissipation (in other words on its q-factor), but also depends on its resistance. In fact, dissipative structures, characterised by \( q > 1 \), can sustain a seismic event characterised by a peak ground acceleration given by:

\[ V_y = A = \frac{V_y q}{M \cdot R(T)}. \]  
(12)

Therefore, it is clear that the peak ground acceleration (PGA) leading to collapse and the corresponding spectral acceleration are more appropriate than the q-factor to be used as parameters for evaluating the seismic ultimate performance of the structure under severe earthquake, because they take into
account both the effects of the resistance and the effects of the energy dissipation capacity of the structures.

Starting from the above consideration, a criterion for designing semirigid steel frames, can be obtained by imposing that the ultimate value of the peak ground acceleration leading to collapse of the model with semirigid joints is equal to that of the corresponding model with rigid joints:

\[
\frac{A_{u,K}^{PS}}{A_{u,\infty}^{FS}} = 1. \tag{13}
\]

With reference to the simplified model of fig. 1, taking into account that the ratio between the first yielding base shear of the structure with partial strength joint and that of the structure with full strength rigid joints is equal to \(m\) and considering that steel frames are generally characterised by a period of vibration corresponding to the softening branch of the elastic design spectrum (constant spectral velocity branch), the relation between the ultimate PGA corresponding to the model with semirigid-partial strength joints and that of the corresponding model with rigid full strength joints can be expressed by means of the following ratio:

\[
\frac{A_{u,K}^{PS}}{A_{u,\infty}^{FS}} = \frac{V_{y,K}^{PS}}{V_{y,\infty}^{FS}} \cdot \frac{q_{k}^{PS}}{q_{k}^{FS}} \cdot \frac{R(T_{x})}{R(T_{x})} = \frac{m}{\gamma} \cdot \frac{q_{k}^{PS}}{q_{k}^{FS}} \cdot \frac{T_{K}}{T_{x}} = 1. \tag{14}
\]

It can be observed that the ratio \(A_{u,K}^{PS}/A_{u,\infty}^{FS}\) is governed by the following parameters:

\[
\frac{A_{u,K}^{PS}}{A_{u,\infty}^{FS}} = f\left(\bar{m}, R_{c}, R_{b}, \bar{K}, \zeta, \gamma_{x}\right). \tag{15}
\]

Relation (14) can be properly exploited in the seismic design of steel frames with semirigid-partial strength connections. In fact, by assuming as reference parameter the ultimate PGA of the structure with rigid joints (\(A_{u,\infty}^{FS}\)), the design of the corresponding structure (i.e. with the same values of \(R_{b}, \zeta\) and \(\gamma_{x}\)) with semirigid-partial strength joints can be performed by imposing eqn (13).

In particular, taking into account eqs (3), (9) and (11), eqn (14) can be numerically solved, by varying the parameters \(\bar{K}, \bar{m}, \gamma_{x}\) and \(\zeta\) providing the couples of values of \(R_{c}\) and \(R_{b}\) leading to its fulfillment (\(A_{u,K}^{PS} = A_{u,\infty}^{FS}\)). Successively, by means of a regression analyses, the following correlation between the rotation capacity of the connection and that of the beam, assuming the equivalence in terms of ultimate seismic response, is obtained:

\[
R_{c} = a_{1} \cdot R_{b}^{a_{2}} \tag{16}
\]

where the coefficients \(a_{1}\) and \(a_{2}\) are function of \(\bar{K}, \bar{m}, \gamma_{x}\) and \(\zeta\). The values of the coefficients \(a_{1}\) and \(a_{2}\) corresponding to the most significant values of the parameters \(\bar{m}, \gamma_{x}\) and \(\zeta\) are summarised in table 1 and table 2. In addition, fig. 2 shows, as an example, the correlation curves given by eqn (16) for two different combinations of the parameters. It can be observed that, increasing the plastic rotation capacity of the beam, the plastic rotation demand of the connection, required to assure the equivalence, significantly increases reaching very high values in correspondence of the most rigid joints.
Table 1: Coefficients $a_1$ and $a_2$ for $\bar{m} = 0.5$ and $\gamma_\infty = 0.05$.

<table>
<thead>
<tr>
<th>$\bar{K}$</th>
<th>$\zeta = 0.2$</th>
<th>$\zeta = 0.4$</th>
<th>$\zeta = 0.6$</th>
<th>$\zeta = 0.8$</th>
</tr>
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<td>$a_1$</td>
<td>$a_2$</td>
<td>$a_1$</td>
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<tr>
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<td>2.2582</td>
<td>0.5929</td>
<td>2.4059</td>
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<tr>
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<td>4.5609</td>
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<tr>
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</tr>
<tr>
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<td>0.5624</td>
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<tr>
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<td>0.5730</td>
<td>7.9707</td>
<td>0.5430</td>
</tr>
</tbody>
</table>

Table 2: Coefficients $a_1$ and $a_2$ for $\bar{m} = 0.7$ and $\gamma_\infty = 0.05$.

<table>
<thead>
<tr>
<th>$\bar{K}$</th>
<th>$\zeta = 0.2$</th>
<th>$\zeta = 0.4$</th>
<th>$\zeta = 0.6$</th>
<th>$\zeta = 0.8$</th>
</tr>
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<tbody>
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<td></td>
<td>$a_1$</td>
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<td>$a_1$</td>
<td>$a_2$</td>
</tr>
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<tr>
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</tr>
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</table>

Figure 2: Correlation between the plastic rotation demand of joints and beam plastic rotation capacity.
4 Proposed design procedure for steel frames with extended end plate joints

From a practical point of view, the application of the introduced design criterion requires the prediction of the joint plastic rotation capacity and of the beam plastic rotation capacity.

Regarding the parameters affecting $R_{c}$, the first yielding rotation $\vartheta_{jy}$, accounting for eqn (2), can be expressed as a function of nondimensional joint stiffness:

$$\vartheta_{jy} = \frac{M_{j}\cdot M_{pb}\cdot L_{b}}{K_{\varphi} \cdot E \cdot I_{b}} = \left(\frac{m}{K}\right) \left(\frac{Z_{b}L_{b}}{I_{b}}\right) \left(\frac{f_{y}}{E}\right)$$  \hspace{1cm} (17)

where $K_{\varphi}$ represents the secant stiffness equal to 1/3 of the initial stiffness, according to the elastic-perfectly plastic joint model of Eurocode 3.

With reference to extended end-plate beam-to-column joints, by means of a wide number of numerical analyses, the following relation between the joint flexural resistance and the joint rotational stiffness, has been provided [14]:

$$\frac{m}{K} = c_{1} \cdot c_{2} \cdot \left(\frac{L_{b}}{h_{b}} \cdot \frac{1}{K}\right)^{c_{2}} = c_{1} \cdot \left(\frac{L_{b}}{h_{b}}\right)^{c_{2}} \cdot \left(\frac{1}{K}\right) \cdot \left(\frac{f_{y}}{E}\right)$$  \hspace{1cm} (18)

where the coefficients $c_{1}$ and $c_{2}$ are dependent on the type of column profile, on the steel grade, on the bolt class and on the ratio $m/db$ between the distance $m$ from the bolt axis and the T-stub web, and the bolt diameter $d_{b}$.

In addition, taking into account that for IPE profiles, frequently adopted for the beams, the ratio $I_{b}/Z_{b}$ is approximately given by $0.539 \cdot h_{b}$, eqn (16) can be rewritten in the following form:

$$\vartheta_{jy} = 1.855 \cdot c_{1} \cdot \left(\frac{L_{b}}{h_{b}}\right)^{1-c_{2}} \cdot \left(\frac{f_{y}}{E}\right)$$  \hspace{1cm} (19)

With reference to the plastic rotation of the connection, it can be observed that, in the case of end plate connections, the main sources of deformability are represented by the end plate in bending and by the column flange in bending. In the following, the joint plastic rotation capacity is predicted by considering only the deformability contribution of these two components, while the additional contributions of the others joint components are neglected providing a safe side design value.

The ultimate plastic deformation of the end plate in bending and of the column flange in bending can be predicted by means of the method proposed by Piluso et al. [15, 16] which is based on the plastic analysis of an equivalent T-stub. Assuming that the thickness of the end plate is comparable with the thickness of the column flange, which represents a frequent situation in practice, the following relation can be adopted for a simplified design approach:

$$\vartheta_{ip} \approx \frac{2 \delta_{p}}{h_{t}}$$  \hspace{1cm} (20)

where $h_{t}$ is the lever arm given by the distance between the mid thickness lines of the beam flanges. Accounting for eqn (19) and (20) and adopting the same class
of material for both the end plate and the column profile, the joint ductility $R_c$ can be expressed as:

$$R_c = \left( \frac{C \cdot E}{3.71 \cdot c_1 \cdot f_y} \right) \times \left( \frac{m^2}{t_{ep} \cdot h_t} \right) \times \left( \frac{L_b}{h_b} \right)^{c_2-1} \times \bar{K}^{1-c_2}$$  \hspace{1cm} (21)

In fig. 3 the curves expressed by eqn (21) for two combinations of the possible values of the parameters, are depicted.

It is important to underline that fig. 2 provides, through $R_c$, the plastic rotation demand; conversely, fig. 3 provides the plastic rotation supply.

Regarding the prediction of the rotation capacity of the beams, in the present work the formulation proposed by Sedlacek and Spangemacher [17], which also accounts for the post-buckling behaviour, has been adopted.

Starting from the previous analyses, the following design algorithm is herein proposed:

a) In order to design dissipative structure, beams and columns can be designed assuring the development of a collapse mechanism of global type by means of the procedure suggested by Mazzolani and Piluso [4] and extended to semirigid frames by Faella et al. [6]. Therefore, taking into account that reducing the joint stiffness $K_\phi$ also the joint flexural resistance $M_{j,Rd}$ reduces, by means of an iterative procedure it is possible to evaluate the non dimensional joint stiffness $\bar{K}$ and the joint resistance $m$ in order to reduce the oversizing of the column sections with respect to those strictly necessary for satisfying deformability and resistance checks. In particular, the iterative procedure starts from $m=1$ and develops progressively reducing $m$.

Preliminary, it is necessary to design the beam sections considering the vertical loads corresponding to the load combination $q=1.35G+1.5Q$, with $G$ and $Q$ equal to dead and live loads, respectively. The beam plastic moment required to withstand vertical loads can be defined, on safe side, greater than $qL^2/8$, where $L$ is the beam span. Successively, the column sections can be designed taking into account the selected beams and the current value of $m$ and using a procedure assuring a collapse mechanism of global type. In addition, by means of eqn (18) the value of $\bar{K}$ can be obtained for a chosen design value for the parameter $m/d_b$ (generally in the range 2-4) and global elastic analyses can be performed with reference to the relevant load combinations. Therefore, the fulfilment of resistance and stability checks for beams, columns and joints, according to the code provisions, are controlled and the compatibility of the maximum interstorey drift with the imposed design limit is verified. The iterative procedure is stopped when at least one check is not satisfied. Therefore, the result of this step is the identification of the minimum $m$ value and the corresponding $\bar{K}$ value leading to a structural solution fulfilling the global mechanism goal, the stability and resistance requirements and, finally, the interstorey drift limitation requirement.

b) The thickness of the end plate required to obtain the desired connection rotational response, described by the parameters $\bar{K}$ and $m$, obtained in previous step, can be evaluated by means of the following relation proposed by Faella et al. [14] on the basis of a wide number of numerical analyses:
where the coefficients $c_3$, $c_4$ and $c_5$ are dependent on the type of column profile, on the steel grade, on the bolt class and on the ratio $m/d_b$.

c) The plastic rotation capacity of the beam can be evaluated by means of formulations proposed in the technical literature, such as that suggested by Sedlacek and Spangemacher [17] and, therefore, by means of eqn (17), the joint plastic rotation demand required to assure the equivalence between the performance of rigid and semirigid steel frames, can be predicted.

d) By assuming as further design condition the equality between the joint plastic rotation demand, exploited in the previous step, and the joint plastic rotation supply given by eqn (21), the value of the parameter $m$ can be evaluated.

e) Finally, it is possible to control that the designed joints really provide a ductile behaviour by checking that the involved joint components (i.e. end plate in bending and column flange in bending) are characterised by a dissipative collapse mechanism of Type 1. To this scope the fulfilment of the following relation [11] is required:

$$\beta_u = \frac{4 \cdot M_u}{2 \cdot B_u \cdot m} < \frac{2\lambda}{1 + 2\lambda}$$  \hspace{1cm} (23)

where $\beta_u$ is the ultimate resistance of the bolt, $\lambda$ is the ratio between $m$ and the distance $n$ between the bolts axis and the free edge of the T-stub and $M_u$ is the ultimate moment of the single T-stub flange given by:

$$M_u = \frac{b_{eff} \cdot t_p^3 \cdot f_y}{6}$$  \hspace{1cm} (24)

in which $b_{eff}$ is the effective width chosen, according to EC3, in order to minimize the collapse load provided by the three possible yield line mechanisms of circular pattern, non-circular pattern and beam pattern [11].

Figure 3: Correlation between the plastic rotation supply of extended end plate joints and the parameter $m/(\text{tepht})$. 
5 Application of the proposed method

The design procedure proposed in the previous section has been applied for designing the three bay-six storey steel frame depicted in fig. 4.

By means of the proposed approach, the beam-to-column joints have been detailed as shown in fig. 4 with M20 bolts. In particular, assuming $m/d_b=2$, the bolt class 10.9, the interstorey drift limit equal to $1/175$, the proposed approach leads to beam-to-column joints characterized by nondimensional resistance $m=0.7$, nondimensional stiffness $K=0.9$ and plastic rotation capacity $R_c=5.2$.

Figure 4: Analysed frame and geometrical properties of designed joints.

The accuracy of the proposed procedure has been verified by means of incremental dynamic inelastic analyses, IDA, using the general purpose DRAIN-2DX software package. In particular, these analyses have been performed both for the frame with semirigid joints and for the corresponding frame with rigid joints considering six simulated accelerograms generated to match the elastic design spectrum given by Eurocode 8 [3] for soil type A. For each accelerogram, the peak ground acceleration has been progressively increased up to the value corresponding to the attainment of the plastic rotation capacity in the most stressed plastic hinge. In fig. 5 the results of the dynamic inelastic analyses for the six simulated accelerograms are depicted. In particular, fig. 5 shows the values of the peak ground acceleration corresponding to the attainment of the plastic rotation capacity both for frames with rigid joints ($A_{uKFS}$) and for frames with semirigid joints ($A_{uKPS}$) and, in addition, the values of the ratio $A_{uKPS}/A_{uKFS}$. It can be observed that the value of the ratio $A_{uKPS}/A_{uKFS}$ varies between 0.84 and 1.07 with an average value equal to 0.95 testifying the sufficient accuracy of the proposed approach.

6 Conclusions

In the present work, with reference to the seismic design of steel frames with semirigid joints, by means of a SDOF simplified model, the criteria for designing
partial strength joints are provided. In particular, assuming as design goal the attainment of steel frames with partial strength joints having the same seismic performance of steel frames with full strength joints, in terms of maximum value of the peak ground acceleration leading to collapse, a relation for evaluating the plastic rotation demand of joints has been obtained.

In addition, starting from the prediction of the joint plastic rotation demand, with reference to extended end plate joints, a method leading to the complete definition of joint structural details assuring a plastic rotation supply equal to the previously predicted demand has been proposed.

![Figure 5: Results of dynamic inelastic analyses.](image)

Finally, the accuracy of the proposed approach has been testified by comparing the inelastic performances of steel frames with full strength and partial strength joints resulting from IDA analyses.

**References**


