SEISMIC RESPONSE OF MOMENT RESISTING COMPOSITE FRAMES INCLUDING ACTUAL BEHAVIOUR OF BEAM-TO-COLUMN JOINTS

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ABSTRACT

The seismic responses of two composite structures are presented in this paper, considering time-history dynamic analyses based on the Romanian Vrancea 1977 accelerogram and two artificial accelerograms consistent with the European C-soil spectrum. The structures have been pre-designed in a usual manner by elastic push-over analyses, assuming their location in a seismic zone of maximum intensity 0.35g. The dynamic analyses reveal the applicability of the simulated joints for the considered accelerograms, scaled in order to correspond to the design ground acceleration. The results are given in terms of required elasto-plastic rotations, maximum inter-storey drifts, behaviour factor $q$ and the performance factor $\eta$. For the beams and columns, appropriate fibre finite elements are used. The beam-to-column joint behaviour is integrated into the structural modelling by means of a sophisticated finite element which models with good accuracy the experimental behaviour of joints.

INTRODUCTION

Usually, the FE modelling of steel Moment Resisting Frames includes the beam and column modelling, the joints being traditionally modelled as pinned or full-strength and rigid. The recent strong earthquakes revealed a series of undesirable failure modes of beam-to-column welded joints of Moment Resisting Frames (MRF). These zones represent the key-points for a ductile seismic response. That is why the joint behaviour needs a particular attention in the usual design of MRF. The extended end-plate bolted connections are traditionally popular in Europe for steel constructions and may be easily adapted to composite systems. However, in many cases of composite MRF, the joints proved to be in fact partial-strength and/or semi-rigid, due to the increased beam resistance. The recent evolution of steel and composite seismic codes relies on the laboratory tests to be sure of suitable behaviour of beam-to-column joints (Eurocode 8). Unless they are rigid and full strength, their behaviour should be integrated by means of appropriate elements (for instance rotational springs or short length finite elements) into the structural design analysis.

Present paper presents briefly the results of dynamic simulations performed on two regular two and four storey frames. In case of beams and columns, refined bar finite elements with fibres are used. The beam-to-column joint behaviour is integrated into the structural modelling by means of a sophisticated finite element which models with good accuracy the experimental behaviour of joints. The frames were analysed under severe Romanian Vrancea accelerogram and also two artificial accelerograms by non-elastic dynamic analyses, using the DRAIN 2DX (Prakash et al.) computer code where the above mentioned finite elements are included.

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STRUCTURAL MODEL

Structural Design

Although the entire study is more complex (Ciutina, 2003), only two structures are presented in this paper, but representative: four-bay composite Moment Resisting Frames on two-storey and four-storey respectively, as shown in Figure 1. The cross-sectional dimensions of the steel columns and composite beams were deduced from an equivalent push-over static analysis, in accordance with Eurocode 4 (Eurocode 4) and Eurocode 8.

The following assumptions were considered for the design:
- \( g_0 = 0.35g \) – the design ground acceleration;
- \( k_s = 1.15 \) – the soil parameter (soil type C);
- \( G_k = 31.4 \text{kN/m} \) – the characteristic value of the permanent load;
- \( Q_k = 12.0 \text{kN/m} \) – the characteristic value of the live load (reduced by a combination coefficient \( \chi_E = 0.3 \));
- full shear connection between the steel profile and the concrete slab;
- full-strength and rigid beam-to-column joints;
- behaviour factor \( q = 6 \) (for MRF);
- limit value of the elasto-plastic inter-storey drift for Ultimate Limit State: \( \delta \leq 0.02 \text{h} \), where \( h \) is the storey height.

The dimensions of the cross-sections (given in Figure 1) resulted from the elastic static equivalent analysis, as follows:
- in the case of Frame 1, the cross-sectional dimensions were governed generally by the static design under the gravitational loads, the combination of actions for the seismic design situation being not prevailing;
- in the case of Frame 2, the static design under the gravitational loads limited the beams cross-sections, whereas the column cross-sections were governed by the combination of actions for the seismic situation because of the drift limitations (the resistance criterion was largely fulfilled).

FE Modelling of Structural Elements

The above structures were modelled by the help of DRAIN 2DX computer code and subjected to elasto-plastic dynamic analyses with input accelerograms.

The fibre finite element (called “element 15” in DRAIN 2DX) may provide a rather refined behaviour in comparison with traditional bilinear elastic-perfectly plastic elements. Obviously, the accuracy of the response depends on the discretisation adopted for the bar length and on the number of fibres considered. Also, the input stress-strain curve for each fibre may affect the response. As the consequence of step-by-step calculation, plastic zones may develop within the element length and cross-section depth.
Eight and eleven fibres have been used in the case of column and beam elements respectively, as shown in Figure 2. The fibre stress-strain characteristics have been deduced from tensile tests performed on steel samples (beam and column flanges and webs, plus reinforcement) and from compressive tests on concrete cylinders. A general stress-strain diagram may be adopted for both steel and concrete, as shown in Figure 3, provided that appropriate values of elastic limit stress $\sigma$, associated strain $\varepsilon$ and coefficients ($\alpha_i, \mu_i$) given in Table 1 are used. In fact these values result from direct measures on actual materials, in particular the structural steel grades S235 and S355 for beams and columns respectively, and the steel grade S500 for reinforcing bars. The structural steel is assumed to have the same properties in tension and in compression whereas the tensile resistance of concrete is neglected.

![Stress-strain curves for steel and concrete](image)

**Table 1.** Mechanical characteristics of materials for beams and columns.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Column Flange</th>
<th>Column Web</th>
<th>Beam Flange</th>
<th>Beam Web</th>
<th>Reinforcement Flange</th>
<th>Reinforcement Web</th>
<th>Concrete (compression)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon$ [\mu strain]</td>
<td>2380</td>
<td>1943</td>
<td>1252</td>
<td>1390</td>
<td>3148</td>
<td>1390</td>
<td>1390</td>
</tr>
<tr>
<td>$\sigma$ [N/mm²]</td>
<td>500</td>
<td>408</td>
<td>263</td>
<td>292</td>
<td>661</td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>$m_1$</td>
<td>1.14</td>
<td>1.17</td>
<td>1.39</td>
<td>1.34</td>
<td>1.15</td>
<td>1.28</td>
<td>1.28</td>
</tr>
<tr>
<td>$a_1$</td>
<td>33.6</td>
<td>41.2</td>
<td>63.9</td>
<td>57.6</td>
<td>12.71</td>
<td>1.98</td>
<td>1.98</td>
</tr>
<tr>
<td>$m_2$</td>
<td>1.16</td>
<td>1.23</td>
<td>1.54</td>
<td>1.46</td>
<td>1.22</td>
<td>0.896</td>
<td>0.896</td>
</tr>
<tr>
<td>$a_2$</td>
<td>46.9</td>
<td>73.6</td>
<td>153.5</td>
<td>121</td>
<td>30.13</td>
<td>2.88</td>
<td>2.88</td>
</tr>
</tbody>
</table>
The elasto-plastic behaviour of the beams and columns in terms of moment-rotation curves is illustrated in Figure 4, in comparison with the bi-linear elastic-perfectly plastic behaviour computed according to Eurocode analytical models, considering the characteristic strength of materials. It should be underlined that the rotation set in horizontal axis corresponds to the integration of the elasto-plastic curvature within the actual extended plastic zone.

![Figure 4. Resulting moment-rotation curves of beams and columns.](image)

**Beam-to-Column Joints Model**

Generally the modelling of the behaviour of the beam-to-column joints into the structural analysis is a difficult task, because it depends on both the joint typology and material properties. For the presented simulations, a complex “zero-length” finite-element model (Skuber, 1998) is calibrated starting from the actual behaviour of an existing laboratory test (specimen G15). The calibration model of the joint could be found elsewhere (Ciutina, 2003). The experimental test was performed in Laboratory of Structures, at INSA-Rennes, under unsymmetrical cyclic loading, in a series of a testing program (Lachal et al., 2004). The joint consisted in an extended bolted end-plate for the steel part, and had the following characteristics:

- column: HEB 300 (S355);
- steel beam: IPE 360 (S235);
- concrete of class C 25/30;
- composite slab with steel sheeting COFRASTRA 40;
- reinforcement: 10 Φ10 (S500);
- end-plate: t=15 mm (S235) with 6 high-strength bolts of grade 10.9 and of 22mm diameter used with controlled tightening.

Due to welding deficiency, the failure occurred by cracking of the butt weld between beam flange and end-plate under hogging moment, leading to a clear unsymmetrical moment-rotation diagram of the joint.

The comparison of the modelled curve for the structural analysis to the actual joint behaviour is shown in Figure 5. The model represents a quadri-linear envelope behaviour including a discharging branch. Special factors for cyclic evolution are taken into account, such as: (i) the pinching effect; (ii) the stiffness degradation of unloading branch; (iii) the loss of resistance of repeated cycles having the same rotation range. In addition, the model characteristics are different under sagging and hogging bending moments, being able to simulate the unsymmetrical behaviour. The comparison of the envelope curve of Figure 5 (left) with the beam response curve of Figure 4 shows clearly that the actual joint is partial-strength, with resistance ratios of about 0.6 and 0.8 under sagging and hogging bending moments respectively.

A parameter used in the analyses was the resistance degree of the joints, taken into account by the $m$ parameter (see Figure 5 – right). This parameter multiplies the ordinate of the initial curve ($m=1$). The values of $m$ considered in the analyses were 1.0, 1.2, 1.4 and 1.6.
ACCELEROGRAMS AND RESPONSE PARAMETERS

Accelerograms

Dynamic time-history analyses will be performed using the Vrancea 1977 accelerogram (recorded at INCERC Bucharest N-S). The accelerogram and its exact response spectra are shown in Figure 6. For the dynamic analyses, a multiplier $\lambda$ will be applied in order to scale the acceleration to the required levels of seismic intensity. The accelerogram was scaled by Effective Peak Acceleration (EPA) as this factor was considered as the normalizing factor (Lungu, 1998). For example, in order to scale the accelerogram at a seismic intensity of 0.32g, a factor of 1.33 was applied (EPA for Vrancea accelerogram is 0.24g).

Figure 6. Vrancea accelerogram (left) and its elastic response spectra (right).

Figure 7. Response spectra for the artificial accelerograms (5% damping).
As a comparison to above long-period accelerogram two artificial accelerograms determined in such a way that they correspond approximately to Eurocode 8 – soil C spectrum were used. The exact response spectra for these accelerograms are shown in Figure 7.

The Romanian territory is divided from the seismic point of view into seven seismic zones for a return period of 100 years. For the analyses, the two structures were hypothetically placed in three seismic zones, denoted by zone A (seismic intensity 0.32g), zone D (seismic intensity 0.20g) and zone F (seismic intensity 0.12g).

**Response Parameters**

A first parameter which was varied was the resistance degree of beam-to-column joints as explained in the previous section. An ideal case in which the beam-to-column joints are infinitely rigid and totally resistant was also considered. For all the three seismic zones, the following parameters have been monitored:

- the required elasto-plastic rotations for all the structural elements. These values have been integrated for several segments in the case of fibre elements, respectively monitored in the case of beam-to-column joints as a direct result. These values were compared to the values requested by the Eurocode 8, chapter 7, i.e. 35 mrad for the case of Ductility Class High (DCH) and 25 mrad for Ductility Class Medium (DCM), values adopted also in the Romanian seismic norm P100/2004.

- the Inter-storey Drift (ID) values, monitored at each level. They have been computed as a ratio of the relative lateral displacement of the storey and the storey height (expressed in %): \((\text{di} - \text{di-1})/\text{h}\text{storey}\). The ID values could also give information about the structural degradation after an earthquake. The valued of the inter-storey drifts were compared to the limiting value of 0.02h (60mm), required by the P100/2004 norm in the case of the ultimate limit state.

- the output behaviour factor (called also the q factor) and the performance factor (η) of the structures, as described in the corresponding section.

**NUMERICAL RESULTS**

Figure 8 gives information about the structural degradation state in case of the Vrancea accelerogram scaled in order to correspond to the seismic zone A (\(\lambda=1.33\)). There are illustrated the maximum transitory values of (positive and negative) elasto-plastic rotations and inter-storey drifts. The response values (elasto-plastic rotations and ID) of the lower structure – Frame 1 show that even a strong earthquake does not affect much the structure. The joints rotation values of 5 mrad. and ID values under 0.9% could be considered at the limit of the Immediate Occupancy performance criteria according to American Standard FEMA356 (FEMA, 2000) or the Serviceability Limit State according to P100/2004. On the other hand, the higher structure – Frame 2 presents high values of elasto-plastic rotations (of order of 47 mrad – in the joints) and ID values greater than 5%. This means that the frame is far beyond the limit of use, and according to above-mentioned standard it even passes the Collapse Prevention performance criteria.

![Figure 8. Structural response of frame 1 and 2 for zone A (0.32g) – Vrancea accelerogram.](image)

Nevertheless the higher structures are more affected by an earthquake than the lower ones, but for the studied cases another parameter seems to influence their response, namely the
vibration period (1st mode) in regard to the elastic spectra of the accelerogram. The ratio of the elastic spectrum corresponding to the vibration period of the frame 2 (T=0.92s) and the one corresponding to the frame 1 (T=0.47s) is about 1.5 (see Figure 6) fact that explain in part the differences between the two responses in case of Vrancea accelerogram.

On the other hand, in the case of the two artificial accelerograms (A09 and A11) the plastic dissipation is more uniformly distributed for both structures in comparison to the case of Vrancea accelerogram. Although the rotation requirements are moderate under the most severe accelerogram, here A11), their values indicate however that the joints are plastified:
- 12.8 mrad for frame 1;
- 15.5 mrad for frame 2.

The Figure 10 presents globally the maximum values of elasto-plastic rotation and of Inter-storey Drifts for the two structures, separately for zones A, D and F (Vrancea accelerogram). The P100/2004 Ultimate Limit State (ULS) limitation criteria (see paragraph 3.2) have been also figured on the charts.

For Frame 1 the required elasto-plastic rotations remains far under the limits requested by the actual seismic norm, fact that indicate that a $q$ factor not grater than 4 and members of class 2 and 3 corresponding to Ductility Class Medium could be used in design. In the case of the frame 2, the joint required rotations show that Ductility Class High is mandatory at least for zones A to D. The maximum values of the transitory Inter-Storey drifts presented in Figure 10, show also that ULS limitation of 2% criterion is exceeded for the zones A to D for Frame 2, meaning that the deflections given by an equivalent elastic design using perfect rigid and full-strength joints is not safe in regard to the use of partial-resistant and semi-rigid joints.

Figure 9. Structural response of Frame 1 (A09 accelerogram) and Frame 2 (A11 accelerogram) for zone A (0.32g).

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Figure 11 shows the influence of the response parameters in function of the joint type (accounted by the $m$ factor) in the case of the seismic zone A – Vrancea accelerogram. Generally the increase of $m$ factor has a beneficial influence on the required rotation (joints or elements) as well as on the maximum inter-storey drift, and in this trend the structure having Ideal Rigid (IR) joints presents the lower values of response parameters.
Especially in the case of 2nd Frame the values of the elasto-plastic rotation decrease by important amounts, presenting in the ideal case of structure with rigid joints a maximum required rotation of 31 mrad, value that can be easily attained by a usual I steel beam of class 1 or 2. However, this is an ideal case, while the case closest to this situation, having $m = 1.6$ (with the maximum rotation shared between joints and beams) presents also values of required rotations under 35 mrad.

Figure 11 also shows clearly that for the Frame 2 the ultimate limit state requirement for Inter-storey Drift is not satisfied even in the ideal case with rigid joints. However, this case reduces by half the ID requirement of the same structure with $m = 1$. These values confirm however the fact that equivalent elastic design of a structure does not necessarily assure a safer response in the post-elastic domain.

**INVESTIGATION ON THE BEHAVIOUR AND SEISMIC PERFORMANCE FACTOR**

As a reminder, the behaviour factor $q$ characterises the plastic dissipation capacity of a structure globally, allowing a design of the structure by reduced static seismic forces. More exactly, this factor is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic, to the minimum seismic forces that may be used in design with a conventional elastic analysis. When using non-linear dynamic analyses, the $q$ factor may be determined as the ratio - see chapter 4 in (Mazzolani and Piluso, 1996):

$$ q = \frac{(EPA)_u}{(EPA)_e} = \frac{\lambda_u}{\lambda_e} $$

Where:
- $(EPA)_u$ represents the effective peak acceleration corresponding to multiplier $\lambda_u$ for which the ultimate state of the structure is considered (for instance when the local ductility is reached into a structural element or joint);
- $(EPA)_e$ represents the effective peak acceleration corresponding to multiplier $\lambda_e$ for which the first yielding occurs into the structure.

Value $\lambda_d = 1.00$ was considered for the scaling of the accelerograms corresponding to a seismic intensity $a_g=0.35g$.

On the other hand, the seismic performance factor ($\eta$) represents the ability of the structure to resist a certain type of soil motion, as the ratio:

$$ \eta = \frac{(EPA)_u}{(EPA)_d} = \frac{\lambda_u}{\lambda_d} $$

Figure 11. Variation of required elasto-plastic rotation (left) and maximum inter-storey drift (right) in respect to the seismic zone (Vrancea accelerogram).
Where: $EPA_d$ represents the effective peak acceleration corresponding to design (in this case equal to the design ground acceleration $a_G$);
- \( \lambda_e \) the accelerogram multiplier corresponding to $EPA_d$ (in this case equal to 1.00).

Table 2 gives for both frames and for all the accelerograms considered in the dynamic analyses the values of $\lambda_e$, $\lambda_u$ and $q$ considering three different criteria for the structural limit state:
- first, the maximum ground acceleration adopted for the initial design of frames;
- secondly, the attainment of the rotation capacity (under either sagging or hogging bending) in a particular joint of the frame;
- and thirdly, the attainment of the joint rotation capacity equal to 35 mrad (in accordance with Eurocode 8 criterion for structures of high ductility classes) assuming a hypothetical discharging branch under sagging bending up to this value of rotation.

<table>
<thead>
<tr>
<th>Structure and seismic action</th>
<th>$\lambda_e$</th>
<th>$a_g = 0.35g$</th>
<th>Joint rotation capacity</th>
<th>Theoretical joint rotation capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\lambda_u$ ($\eta$)</td>
<td>$q$</td>
<td>$\lambda_u$ ($\eta$)</td>
</tr>
<tr>
<td>Frame n° 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A09</td>
<td>0.35</td>
<td>1.00</td>
<td>2.9</td>
<td>2.71</td>
</tr>
<tr>
<td>A11</td>
<td>0.30</td>
<td>1.00</td>
<td>3.3</td>
<td>1.70</td>
</tr>
<tr>
<td>Vrancea</td>
<td>0.50</td>
<td>1.00</td>
<td>2.0</td>
<td>1.83</td>
</tr>
<tr>
<td>Frame n° 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A09</td>
<td>0.26</td>
<td>1.00</td>
<td>3.9</td>
<td>1.66</td>
</tr>
<tr>
<td>A11</td>
<td>0.22</td>
<td>1.00</td>
<td>4.5</td>
<td>1.46</td>
</tr>
<tr>
<td>Vrancea</td>
<td>0.13</td>
<td>1.00</td>
<td>7.7</td>
<td>0.59</td>
</tr>
</tbody>
</table>

Concerning the first criterion, results confirm that generally, the $q$ factor is smaller than 6 due to the fact already mentioned that the seismic action is not significant enough with regard to the frame resistances. A special case is represented by the second frame under Vrancea accelerogram, for which the first plastic rotation forms very early ($\lambda_e = 0.13$), theoretically earlier than expected by design.

Using the second criterion, the joint rotation capacity is systematically reached under sagging bending (for value of 24 mrad). It should be noted that the $q$ factor exceeds 6 for the artificial accelerograms. This demonstrates the good dissipative performances of the composite frames which may be classified in high ductility despite the use of partial-strength joints with unsymmetrical behaviour. Nevertheless, these deformations imply the acceptance of inter-storey drifts greater than 0.02h. The limit 0.03h appears more appropriate to be consistent with a $q$-factor equal to 6. Here also, the long-control period accelerogram of Vrancea leads to values of $q$ which are under the design value of 6.

Finally, if the hypothetical third criterion is considered, the $q$-factor would range from 7 to 10, provided that the inter-storey drift limitation were increased practically up to 0.05h, as resulted from the results of the analyses. Generally, experimental evidence demonstrates that this latter value of transient inter-storey drift cannot be exceeded without a high risk of structural collapse (FEMA 356 2000).

The values of $\eta$ show generally good abilities of structures to withstand considered ground motions, with values ranging between 1.46 and 2.71 in the case of second limit state criterion, and 1.8 to 3.5 in case of third limit state criterion. Of course, a $\eta$ value greater than 2 may indicate a non-economic design of the structure, but this happens only for the first frame, for which the design was done mainly due to gravitational loads and not due to seismic combination.
As for the $q$ factor, the only exception to above mentions is represented by the first frame under Vrancea accelerogram. In this case $\eta$ takes values smaller than 1, meaning that the structure is not able to overtake the seismic load to which it was designed. Under the circumstances of using partial-resistant and semi-rigid joints, this fact illustrates, very clear that a design made by means of equivalent push-over static analysis is not necessarily safe with regard to the response of the structure under dynamic time-history analyses.

**CONCLUSIONS**

As shown by the previous results, the analysed frames proved a good behaviour under the artificial accelerograms, complying with the Eurocode 8 – soil C spectrum, although the joints were semi-rigid and partial-strength. Despite this good behaviour, the same frames behaved differently under the Vrancea accelerogram, characterised by a higher control period.

Table 3 gathers the performance criteria for the analysed frames function of the seismic zone and joints resistance in regard to lateral deformation and rotational requirements of the P100/2004 norm in case of Vrancea accelerogram. Generally, the following conclusions could be drawn, limited at the element conditions used in this study:

- the degradations in terms of both required elasto-plastic rotation and inter-storey drift, recorded for lower structure (frame 1) are low, even in the case of high seismicity zones, such as the Romanian Zone A. For this case, the values of inter-storey drift remains at the level of serviceability limit state, even using partial-resistant beam-to-column joints.

- in case of medium height structures, as is the case of frame 2, the use of partial-resistance joints should be limited to low seismic zones (zones E-F). High values of $q$-factor (between 4 and 6 according to P100/2004 and Eurocode 8) should be used, in order to have a guaranteed value of the elasto-plastic rotation of the plastic zone of more than 35 mrad.

- in regard to the inter-storey drift criterion, the results show that an equivalent elastic design of a structure does not necessarily assure a safer response in the post-elastic domain. As demonstrated also in Figure 11, the partial-resistant joints could influence by large amounts the lateral displacement of a structure, especially at the ultimate state.

The investigation on the $q$ factor shows that the high values of $q$ specified for the composite frames in Eurocode 8 cannot be adopted without considering the inter-storey drift limitation in the sense of larger tolerance.
In fact, the use of partial-strength joints does not seem to constitute a handicap, contrary to the unfavourable feeling of usual designers in seismic zones.

In addition, these actual joints may lead to a more uniform distribution of the dissipated energy, without requiring a large rotation capacity to classify the structure in high ductility. So, the treated examples have proved that a rotation capacity of about 25 mrad under sagging bending may be sufficient for moderate seismic zones to reach a $q$-factor of about 6 while accepting an inter-storey drift of about 0.03h.

REFERENCES


Lungu D., 1998: Effective Peak Ground Acceleration (EPA) Versus Peak Ground Acceleration (PGA) and Effective Peak Ground Velocity (EPV) Versus Peak Ground Velocity (PGV) for Romanian Seismic Records, Eurocode 8 - Worked examples.

