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SEISMIC BEHAVIOUR OF EBFS COUPLED TO MOMENT RESISTING FRAMES

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SUMMARY

Eccentrically Braced Frames (EBFs) are structural systems able to provide both the large ductility typical of Moment Resisting Frames (MRFs) and the stiffness which characterizes the Concentrically Braced Frames (CBFs). In the EBFs only links can yield, while other members must remain in the elastic range. Unfortunately, because of the low redundancy of EBFs such desirable behavior might be impossible to catch. In particular, in some cases (mostly in buildings having a large number of stories) the frame may fail with story mechanisms that do not allow the full development of the high energy dissipation expected by these systems. Further studies have shown that dual systems, obtained by coupling schemes having different structural behavior, may provide a seismic response better than that given by each component, separately considered. Thus, a possible way for improving the seismic response of EBFs is to take into account the presence of the other structural elements, necessary to bear vertical loads. In the paper the seismic performances of a twelve-stories building, in which horizontal forces are mainly sustained by eccentrically braced frames, are analyzed. Several structural configurations are considered. In the first one only the EBFs sustain seismic forces. Instead, in the other configurations also the contribution of the resisting elements designed to sustain the vertical loads to the global lateral stiffness of the building is taken into account. Results of the numerical analyzes show that the additional stiffness provided by the resisting elements bearing the vertical loads may avoid the story mechanism and grant the desirable seismic performances.

INTRODUCTION

As it is well known, buildings located in seismic areas have to be designed so as to fulfill specific requirements. In particular during frequent and low earthquakes all structural elements should remain in the elastic range, while non-structural elements should be only slightly damaged; this aim is in practical applications attained by limiting the inter-story drift. During strong earthquake, instead, structure may undergo large deformation in plastic range; it is therefore necessary in such conditions to assure to the structure the capacity to dissipate large amounts of energy by means of a stable hysteretic behavior (i.e. granting widespread plastic deformations distribution in the structure and appropriate values of available ductility to the members).

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Moment Resisting Frames (MRFs), if designed by means of proper methods, show a really high dissipative behavior, thanks to the large number of plastic hinges developed when the structure fails in a global mechanism, but they are at the same time very flexible systems. For such a reason their design is generally addressed by the limits imposed to the displacements for low seismic events and, consequently, their cross-sections result oversized with respect to those strictly required to bear the ultimate limit state loads. On the contrary, *Concentrically Braced Frames* (CBFs) are very stiff systems but, unfortunately, they are characterized by a quite poor inelastic behavior, because of the buckling of the braces. This imposes the use of larger design forces in order to counterbalance the low levels of available ductility.
In order to overcome the deficiencies of MRFs and CBFs, *Eccentrically Braced Frames* (EBF) have been proposed. In such structural typology braces worklines do not intersect the beams at a single point. The beam segment between these two points is called *Link*. In these systems, the necessary lateral stiffness is granted by the braces. Furthermore, links, which are characterized by a stable hysteretic behavior, should provide EBFs with a large dissipative capacity. Unfortunately, because of the low redundancy of EBFs such desirable behavior might be impossible to catch. In particular, after the yielding of the first link large plastic deformation has to be expected in the yielded element while the other links remain in elastic range. Such a tendency, which leads the structure to unfavourable mechanisms in which inelastic deformations occur only in a few elements, has been pointed out in previous studies, Popov [1], Lu [2] and Ghersi [3].
Nevertheless eccentrically braced frames are almost never isolated, but are generally coupled with resisting elements devoted to sustain the vertical loads. These structural elements provide an additional stiffness which may be sufficient to avoid the story mechanism and to fully develop available ductility of EBF. In order to investigate such issue the seismic performances of a twelve-stories building, in which horizontal forces are mainly sustained by eccentrically braced frames, are evaluated according to FEMA 356 [4]. Several structural configurations are considered. In the first only the EBFs sustain seismic forces. Instead, in the other configurations also the contribution of the resisting elements designed to sustain the vertical loads to the global lateral stiffness of the building is taken into account.

STRUCTURAL SYSTEMS ANALYZED

The analyzed eccentrically braced frame belongs to the structure of a twelve-stories symmetric building. The decks of the building, square in shape and rigid in their own plane, are supported by steel frames arranged along two orthogonal directions. Each frame has three spans 8.0 m long. Seismic actions are mainly sustained by the outside frames, which are endowed with eccentric braces in the central span. Length of the links is equal to 0.8 m. Wide-flange shapes are used for all the members. Figure 1 shows the plan layout of the building and the geometrical scheme of the eccentrically braced frames located on the perimeter.

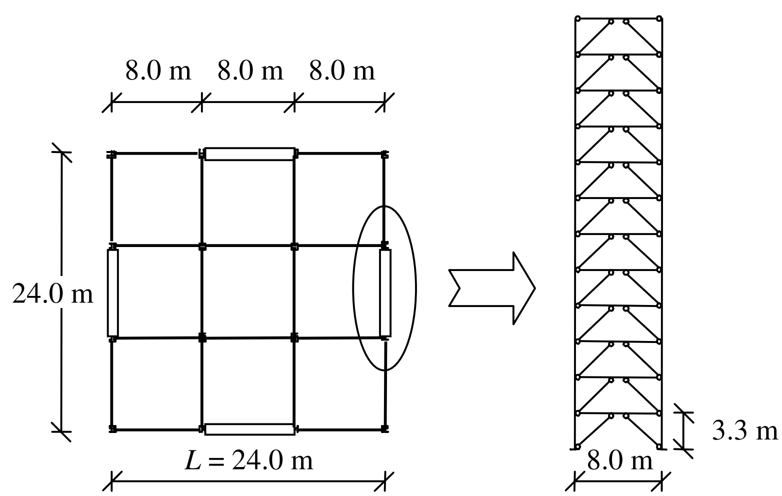
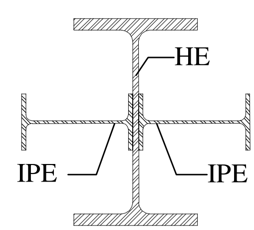


Figure 1. Plan layout and geometrical scheme of the eccentrically braced frames.

The eccentrically braced frames are designed to sustain the whole seismic action. The design seismic forces have been evaluated by means of the design spectrum proposed by Eurocode 8 [5] for subsoil type C, characterized by a peak ground acceleration $a_g = 0.35\ g$, a behavior factor $q = 5$ and a 5% viscous damping factor. Story masses have been calculated by taking into account the presence of a global weight equal to 5.0 kN/m^2, representative of the whole dead gravity load G_k and the rate of the live gravity load $\psi\ Q_k$ present in the occurrence of the earthquake. The design internal actions of the links, which represent the dissipative members of the structure, have been evaluated by applying an inverted triangular forces distribution. A different cross-section is used at each floor in order to minimize the overstrength. Columns and braces, which should remain in the elastic range of their behavior until the attainment of the ultimate plastic rotation of the links, have been designed according to the capacity design criterion. In particular, the shear force transmitted by each link and considered in the design of the non dissipative members has been obtained by multiplying their ultimate shear strength V_u for a factor equal to 1.2. Because in the examined frames only short links are used, ultimate shear strength V_u has been assumed equal to 1.5 times the plastic shear V_p. The factor 1.2 takes into account uncertainties related to geometrical and mechanical characteristics.

Members which do not belong to the eccentrically braced frames are designed to sustain only the vertical loads. The design value of the internal actions in beams and columns is evaluated in consideration of the tributary areas. The unit weight of the deck, dead gravity load G_k and live gravity load Q_k increased respectively by the coefficients γ_g and γ_q (according to the ultimate limit state approach stipulated in Eurocode 3 [6]), is assumed equal to 10.5 kN/m^2. Beams sustain a vertical load q equal to 84.0 kN/m and their design bending moment has been evaluated as $q\ l^2 / 8$ ($M_{sd} = 672$ kNm), where l is the length of the span. A IPE450 cross-section is used for the beams at each story. Columns have been designed in such a way that their buckling strength, evaluated according to Eurocode 3, results not smaller than their design axial force. Columns of the lower stories, in order to optimize their buckling strength, are constituted by two IPE shape welded to the web of the HE shape. The cross-sections obtained for the different structural elements (except for the beams) are summarized in Table 1.

Table 1. Cross Section of members.

Story	Eccentrically braced frame			Other	Composed
	Links/Beams	Braces	Columns	Columns	Cross-sections
12	HEA180	HEA200	HEA180	HEB220	
11	HEA220	HEA200	HEA180	HEB220	
10	HEA300	HEA260	HEB260	HEB300	
9	HEA340	HEA260	HEB260	HEB300	
8	HEA360	HEA300	HEB400 [1]	M1	
7	HEA400	HEA300	HEB400 [1]	M1	
6	HEA400	HEA320	HEB500 [2]	M2	M1 = HEB400 + 2 IPE100
5	HEA450	HEA320	HEB500 [2]	M2	M2 = HEB450 + 2 IPE200
4	HEA450	HEA320	M3 [2]	M4	M3 = HEB500 + 2 IPE240
3	HEA 450	HEA 320	M3 [2]	M4	M4 = HEB500 + 2 IPE270
2	HEA 500	HEA 360	M6 [2]	M5	M5 = HEB550 + 2 IPE360
1	HEA 500	HEA 360	M6 [2]	M5	M6 = HEM500 + 2 IPE240

1 steel grade Fe430, 2 steel grade Fe510, all other sections: steel grade Fe360

Although the lateral stiffness of the eccentrically braced frames is certainly prevalent, also the resisting elements designed to sustain the vertical loads are generally provided with an own stiffness. The additional stiffness of such resisting elements, linked to the eccentrically braced frames, may change the plastic hinges pattern and limit soft-story mechanism. In order to investigate about the beneficial effect on the seismic performances of the eccentrically braced frames provided with such additional lateral stiffness, four different configurations of the resisting elements designed to sustain the vertical loads have been considered. In the first instance, which corresponds to the reference system, pinned connections are supposed in the columns at each floor and, therefore, any additional stiffness is provided. In the other three cases, instead, columns are continuous from the bottom to the top of the building. Beams to columns connections are supposed pinned in the second configuration and rigid in the others two. The fourth configuration is different from the third only in the shapes adopted for the columns belonging to the last six stories of the building. The cross-sections of such members are replaced with M2 composed shape in order to increase the additional stiffness in the upper stories where the first yielding is expected. The following acronyms are used: R for the reference configuration and C1, C2, C3 for the 2nd, 3rd and 4th configurations.
The lateral stiffness distribution along the height has been evaluated for the eccentrically braced frames (EBF) and for the backup frames (represented by all the vertical resisting elements designed to sustain only vertical loads) corresponding to the C1, C2 and C3 structural configurations. For the generic systems, the story lateral stiffness is defined as the ratio between the story shear force provided by an inverted triangular force distribution and the corresponding inter-story drift. In Figure 2 the four stiffness distributions are compared. The additional lateral stiffness given by the backup frame corresponding to the C1 structural configuration is negligible with respect to that of the eccentrically braced frames. Conversely, both backup frames corresponding to the C2 and C3 structural configurations provide a significantly additional stiffness. In particular, the stiffness of the C2 and C3 backup frames is, with respect to that of EBF system, about 25% in the first story and becomes very close in the upper stories.

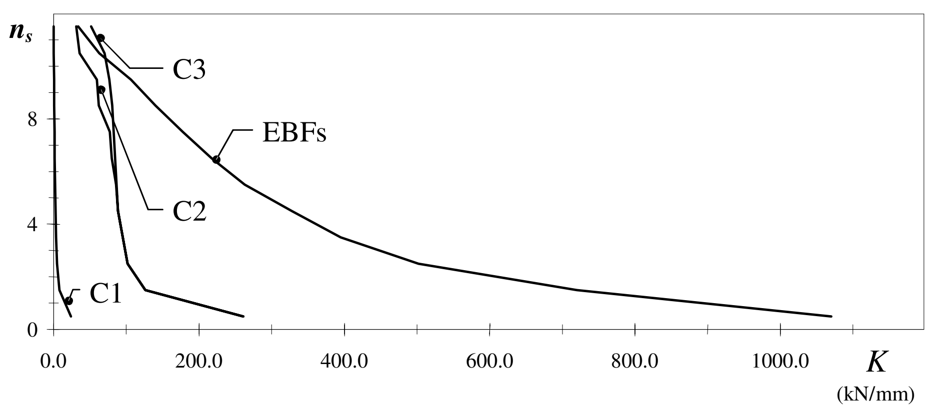


Figure 2. Lateral stiffness distribution along the height.

NUMERICAL ANALYZES

Numerical models

Because of the symmetric configuration of the analyzed structures, the numerical models considered are representative of half building. The geometrical schemes of the analyzed systems, which will be called EBF-R, EBF-C1, EBF-C2 and EBF-C3, are showed in Figure 3. The system named EBF-R corresponds to the structural configuration in which only the eccentrically braced frame is able to sustain horizontal forces. Therefore the presence of the other vertical resisting elements has been neglected. In the numerical model of the system EBF-C1, instead, the eccentrically braced frame is linked to a continuous gravity column by means of a rigid diaphragm. Finally in EBF-C2 and EBF-C3 numerical models the additional stiffness is taken into account by means of an equivalent shear type frame. At each story, the moment of inertia of the columns of the coupled systems (EBF-C1, EBF-C2, EBF-C3) is fixed in such a way that their lateral stiffness results equal to that corresponding to the backup frame in C1, C2 and C3 configurations. No yielding is allowed in the members of the backup frames.

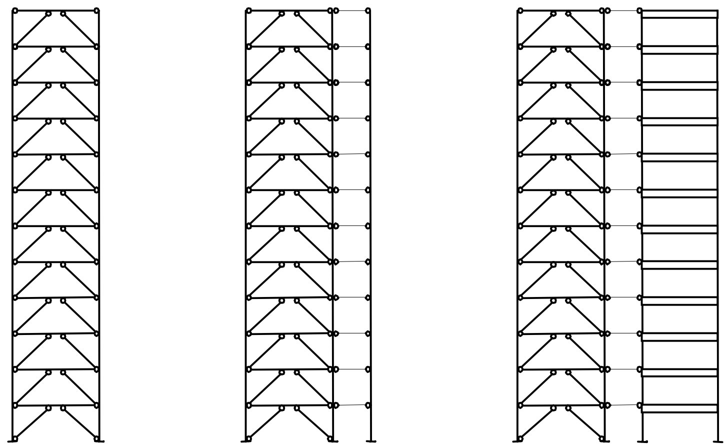


Figure 3. Geometrical schemes of analyzed systems

The columns, beams and braces of the eccentrically braced frame have been schematized by means of mono-dimensional members. A very high strength is assigned to the end cross-sections of such elements in order to avoid their yielding. Such a choice arises from the adoption of the capacity design criterion. According to such criterion, in fact, in eccentrically braced frames only seismic links are devoted to dissipate energy, while other elements have to remain in the elastic range of their behavior.
Links' seismic behavior has been simulated by means of two elements disposed in series. The former schematizes whole shearing deformability of links and yields when plastic shear force V_p, evaluated as follows, is attained:

$$V_p = h_w t_w \frac{f_y}{\sqrt{3}} \tag{1}$$

In Eq. (1) h_w is the distance from inside of compression flange to inside of tension flange, t_w is the web thickness and f_y the expected yield strength of the material.
The latter simulates only flexural behavior of links and yields when plastic bending moment, determined by means of the following formula, is achieved:

$$M_p = Z f_y \tag{2}$$

In Eq. (2) Z represents the cross-section plastic modulus.
The hardening ratio of the shearing element has been determined so as to reach the ultimate shear force V_u (equal to 1.5 V_p) when the plastic rotation 0.09 rad is reached, Kasai and Popov [7]. The hardening of the flexural element, instead, has been fixed so as to reach the ultimate bending moment M_u (equal to 1.5 M_p) when the plastic rotation 0.03 rad is reached. Such choice, however, does not affect the results because plastic hinges almost never occur in the flexural elements.
Rayleigh damping has been considered. The related coefficients have been fixed so as to obtain a 5% damping factor in correspondence of the first and the third periods of vibration of the structures.

Methodology of analysis

The seismic behavior of the eccentrically braced frames corresponding to the aforementioned four structural configurations has been analyzed considering three performance objectives stipulated in FEMA 356 [4]:

- in case of frequent earthquakes (probability of exceedance 50% in 50 years) the structure should exhibit the immediate occupancy performance level (post-earthquake damage state in which vertical and lateral resisting elements have suffered negligible damage and retain nearly all their pre-earthquake strength and stiffness);
- in case of rare earthquakes (probability of exceedance 10% in 50 years) the system should achieve the life safety performance level (post earthquake damage state including significant damage in structural components but retains margin against partial or global collapse);
- in occurrence of the strongest earthquake (probability of exceedance 2% in 50 year) the achievement of the collapse prevention structural performance level is allowed (post-earthquake damage state corresponding to structural damage such that the structure continues to support gravity loads but retains no margin against the collapse.

The inelastic response of the systems has been evaluated by means of the non-linear dynamic procedure stipulated in FEMA 356 [4]. The dynamic non linear analyzes have been carried out by means of the program DRAIN-2DX [8, 9].

Three sets of accelerograms, corresponding to the frequent, rare and very rare seismic events, have been considered. Each set is composed by ten accelerograms artificially generated by means of the procedure proposed by Falsone and Neri [10]. They match with the elastic response spectra proposed by Eurocode 8 [5] for soil C with 5% viscous damping factor having peak ground acceleration equal to 0.10 g, 0.35 g and 0.525 g respectively. The accelerograms are enveloped by a trapezoidal intensity function and are characterized by a total duration of 35 s and by a stationary part of 22.5 s. In Figure 4, with reference to the peak ground acceleration equal to 0.35 g, the elastic spectra of the considered accelerograms are superimposed to the Eurocode 8 elastic response spectrum.

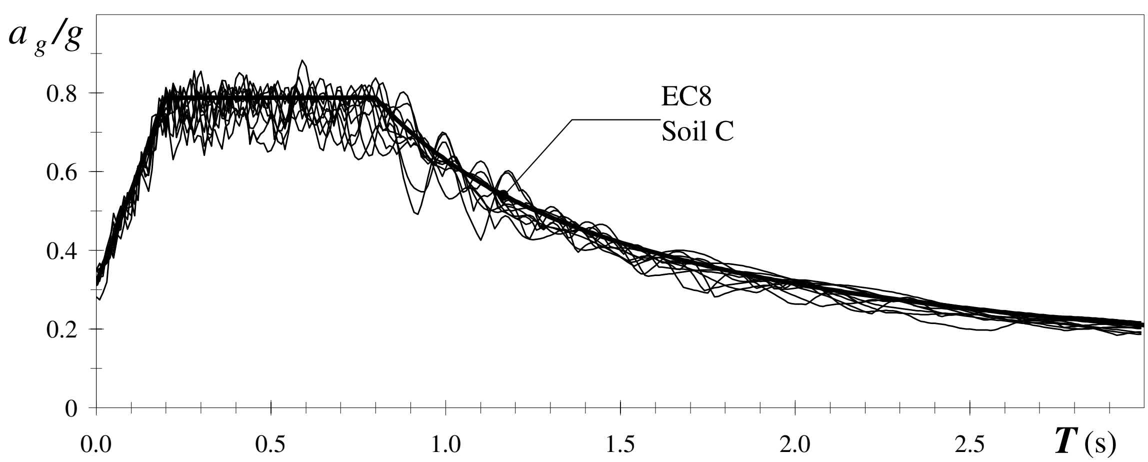


Figure 4. Elastic spectra of the considered earthquakes, PGA = 0.35 g.

According to the provisions stated in FEMA 356, seismic links have been considered deformation controlled. Therefore, for each of the aforementioned seismic levels, plastic rotation in the links has to satisfy specific limit values. Table 2 summarizes the maximum values of the plastic rotation allowed in the links for each performance objective. Other structural elements (beams, columns and braces) have been considered force controlled. Therefore their internal actions, in occurrence of the strongest earthquake, should not exceed their strength. The strength of columns and braces has been evaluated by means of AISC 1999 provisions [11]. Seismic behavior of the EBF-R system has been also analyzed by means of adaptive push-over analysis. In order to follow the abrupt modification of dynamic properties of the system caused by the yielding of the links, the stiffness matrix of the structure is updated when links yield and the response within the generic step is evaluated by means of modal analysis.

Table 2. Links requirements according to FEMA 356.

Seismic event Level	Performance Level	Maximum Plastic Rotation of Link (rad)
Frequent	Immediate Occupancy (IO)	0.005
Rare	Life Safety (LS)	0.110
Very rare	Collapse Prevention (CP)	0.140

SEISMIC BEHAVIOR OF THE REFERENCE SYSTEM

In order to estimate the seismic performances of the reference system (EBF-R), in which only eccentrically braced frames sustain seismic action, three response parameters are examined: plastic rotation in the links, bending moment in the columns of the bracings bents and axial force in the braces.

By means of the pushover analysis the height distributions of the plastic rotation of the links corresponding to the reaching of the three performance levels (immediate occupancy, life safety and collapse prevention) are evaluated and reported in Figure 5a. We assume that each performance level is reached when the corresponding limit plastic rotation (Table 2) is attained in any of the links.

With regard to the dynamic analyzes, the maximum value of the plastic rotation θ_p of the links has been evaluated for each seismic action level and for each accelerogram. Hence, the ratio of the maximum plastic rotation θ_p to the limit value θ_{lim}, stipulated by FEMA 356 for the considered seismic action level, has been evaluated. Such normalized parameter indicates if the performance levels required for the considered seismic action levels are obtained or not. Finally, at each story, minimum, maximum and mean values over the ten considered records have been computed and plotted in Figure 5b ~ d for each seismic action level.

The internal actions in columns and braces corresponding to the accelerograms representative of very rare earthquakes (PGA equal to 0.525 g) have been evaluated at each step. Hence, with reference to the columns, the maximum ratio M_{CE} / M_{CL} (bending moment over flexural strength reduced taking into account the interaction with the axial force) is calculated. Instead, with reference with braces, the ratio between the maximum axial force N_{CE} and the buckling strength N_{CL} is determined. Both M_{CL} and N_{CL} are evaluated according to AISC 1999 provisions [11]. In Figure 5e and Figure 5f height distributions of minimum, maximum and mean values over the ten considered accelerograms are plotted for normalized bending moment in the columns and axial force in the braces respectively.

Figure 5a shows clearly that EBF-R system tends to develop a soft story after the first yielding. In particular, the height distribution of links plastic rotations corresponding to the immediate occupancy performance level shows that when the 11th floor link develops a plastic rotation equal to 0.005 rad other links are still in elastic range. Therefore the first yielding occurs at the 11th floor. This is consistent with the design approach used. In fact, in tall buildings, the inverted triangular forces distribution leads to underestimate seismic internal actions in the upper part of the structures and to oversize structural members in the lower floors. Furthermore, because structural elements not belonging to the eccentrically braced frame do not provide any additional stiffness against lateral forces, further increase in seismic actions provides, after the first yielding, a large increase in plastic rotations of the 11th link but not significant increase in internal actions of the other links. Consequently, when collapse prevention performance level is achieved (Figure 5a), plastic deformation are concentrated in the 11th floor link while other links have exploited partially their plastic deformation capacity. Results in terms of plastic rotation of the links obtained by means of dynamic analysis (Figure 5b ~ d) confirm the trend observed in Figure 5a. Because results at each story are generally scattered in a narrow range, the mean value has been assumed as representative of the structural behavior. In occurrence of frequent earthquakes (Figure 5b) plastic rotation at 11th floor is more that 10 times the value stipulated in FEMA 356 for immediate occupancy performance level and almost zero elsewhere. Concentration of plastic deformation at 11th floor is less evident when rare and very rare earthquakes (Figure 5c ~ d) are considered. However limit values of plastic rotation stipulated in FEMA 356 for life safety and collapse prevention performance levels are exceeded again in the link of the 11th floor. Furthermore, plastic rotation of the other links are always well below FEMA 356 limit values (rarely plastic rotation of the other links reaches the 50% of their plastic deformation capability).

Results regarding force controlled members (columns and braces) are showed in Figure 5e ~ f. Braces exhibit a good seismic behavior. In fact, also for very rare earthquakes their axial force results smaller than their buckling strength (Figure 5f). Conversely, seismic performance of the columns does not meet FEMA 356 requirements. In particular, in the 11th story columns bending moment exceeds the flexural strength. Such bad performance comes directly from the story collapse mechanism: in the upper part of the building, it leads to large inter-story drifts and therefore also large bending moments in the columns.

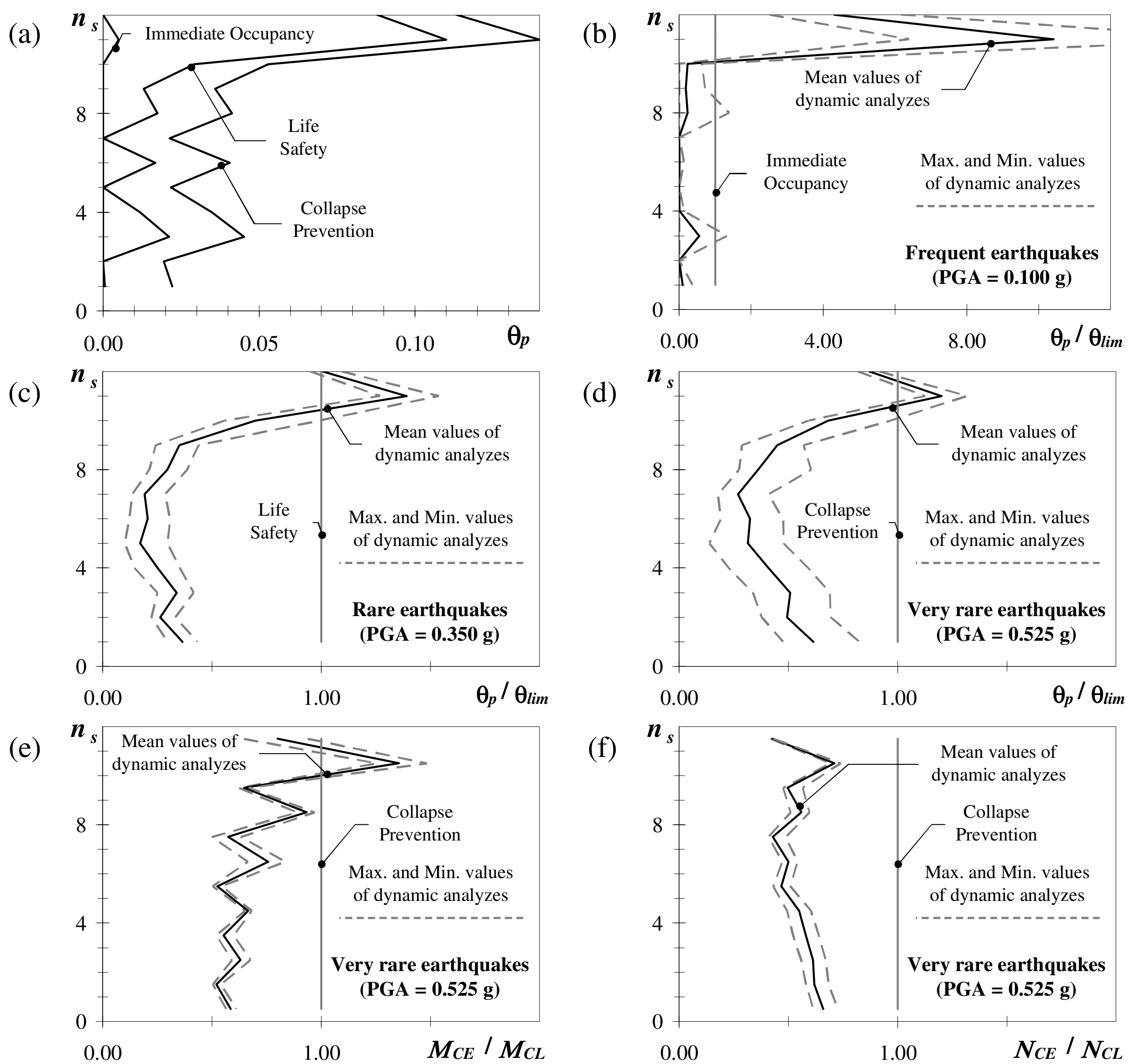


Figure 5. Height distribution of the response parameters. (a) Link plastic rotation by pushover analysis, (b, c, d) links normalized plastic rotation, (e) columns normalized bending moment, (d) braces normalized axial force.

SEISMIC BEHAVIOR OF THE COUPLED SYSTEMS

In order to assess the beneficial effect of the additional stiffness, provided by resisting elements designed to bear vertical loads, on the seismic performances of the EBF's members, dynamic analyzes have been carried out on coupled systems. In the following, seismic performance of EBF-C1, EBF-C2 and EBF-C2 systems, expressed in terms of normalized parameters (plastic rotation of links, bending moment of columns and axial force of braces), are compared with those of the reference system EBF-R.

When the eccentrically braced frame is coupled with a continuous column (EBF-C1 scheme), whatever is the seismic level considered (frequent, rare and very rare earthquakes), none appreciable improvement in the seismic performances may be observed. In particular EBF-C1 system, similarly to EBF-R scheme, results prone to develop soft-story mechanisms. At 11^{th} story plastic deformation of the link exceeds limit values stated in FEMA 356, especially with reference to frequent earthquakes (Figure 6b) but also when rare and very rare earthquakes are considered (Figure 7b and Figure 8b). Also columns exhibit poor performances. Because the formation of the soft story, bending moment results about 20% larger than the flexural strength in the columns belonging to the 11^{th} story (Figure 9b).

On the contrary, relevant beneficial effects, with respect to all the seismic event levels considered and all the members analyzed apart the braces, may be observed when beams bearing vertical loads are jointed by means of rigid connections to the columns of the backup frames (EBF-C2 system). In particular, in EBF-C2 system, the additional stiffness provided by the backup frame seems to be sufficient to give an alternative path to the seismic internal actions after the first yielding. It allows the plasticization of all links and, consequently, avoids the formation of soft-story mechanisms. Figure 7c ~ Figure 9c show that limits stipulated by FEMA 356 on plastic deformation of the links, in occurrence of rare and very rare earthquakes, and on bending moment of the columns, in occurrence of very rare earthquakes, are everywhere met. Seismic performance of the links exhibited in occurrence of frequent ground motions, instead, does not satisfy the requirement stated in FEMA 356, nevertheless are significantly improved with respect to the reference system. Normalized plastic rotation at 11^{th} floor is larger than unity, however, is about 60% smaller than that of the EBF-R system (Figure 6c).

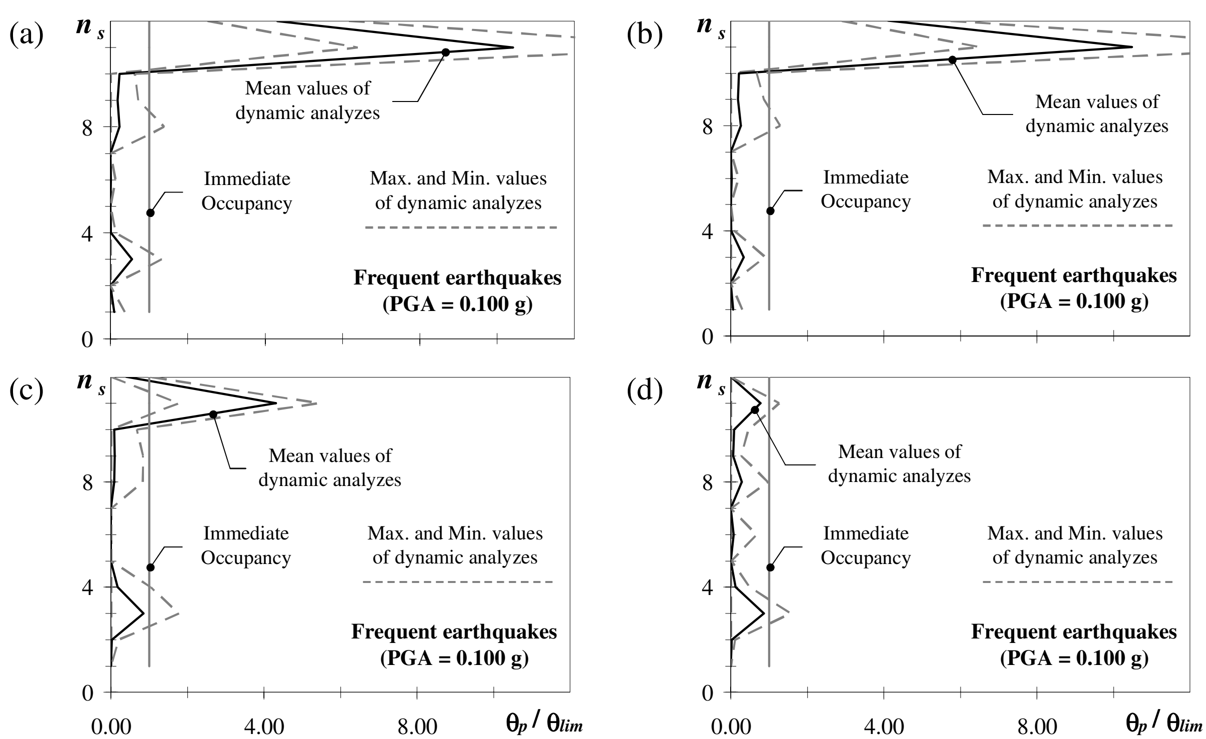


Figure 6. Height distribution of links normalized plastic rotation provided by frequent earthquakes.
(a) EBF-R, (b) EBF-C1, (b) EBF-C2, (b) EBF-C3.

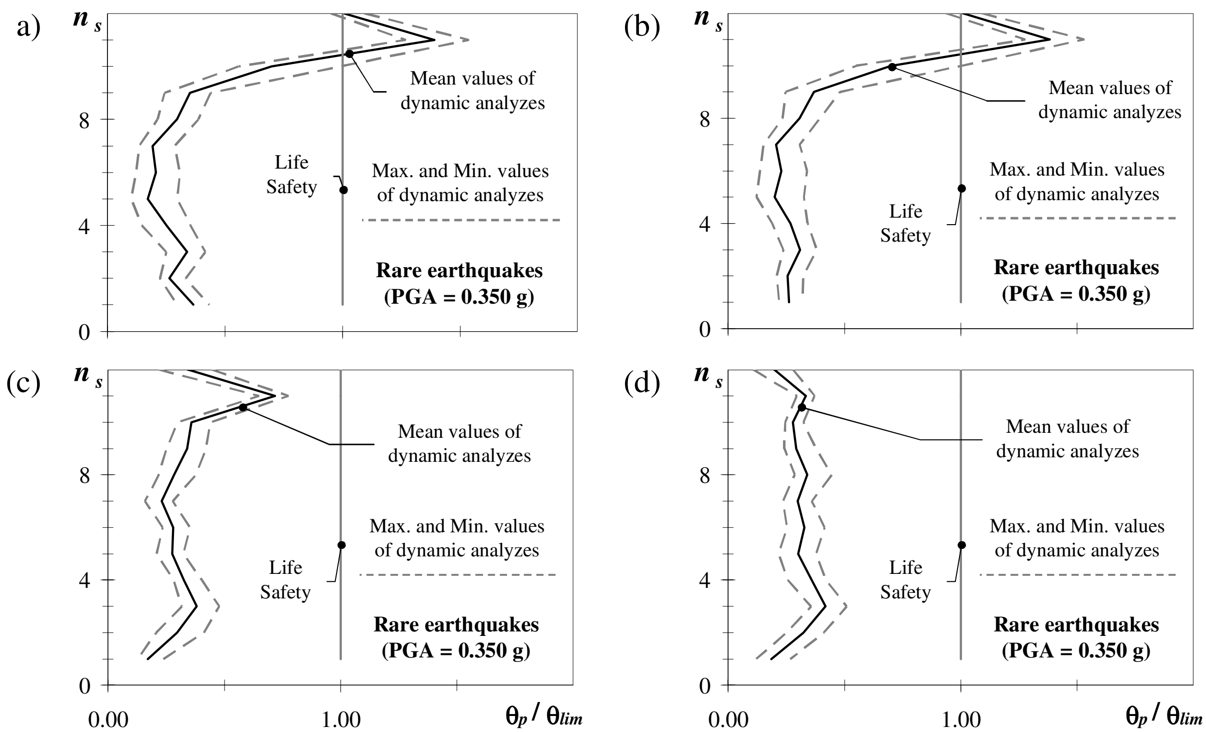


Figure 7. Height distribution of links normalized plastic rotation provided by rare earthquakes.
(a) EBF-R, (b) EBF-C1, (b) EBF-C2, (b) EBF-C3.

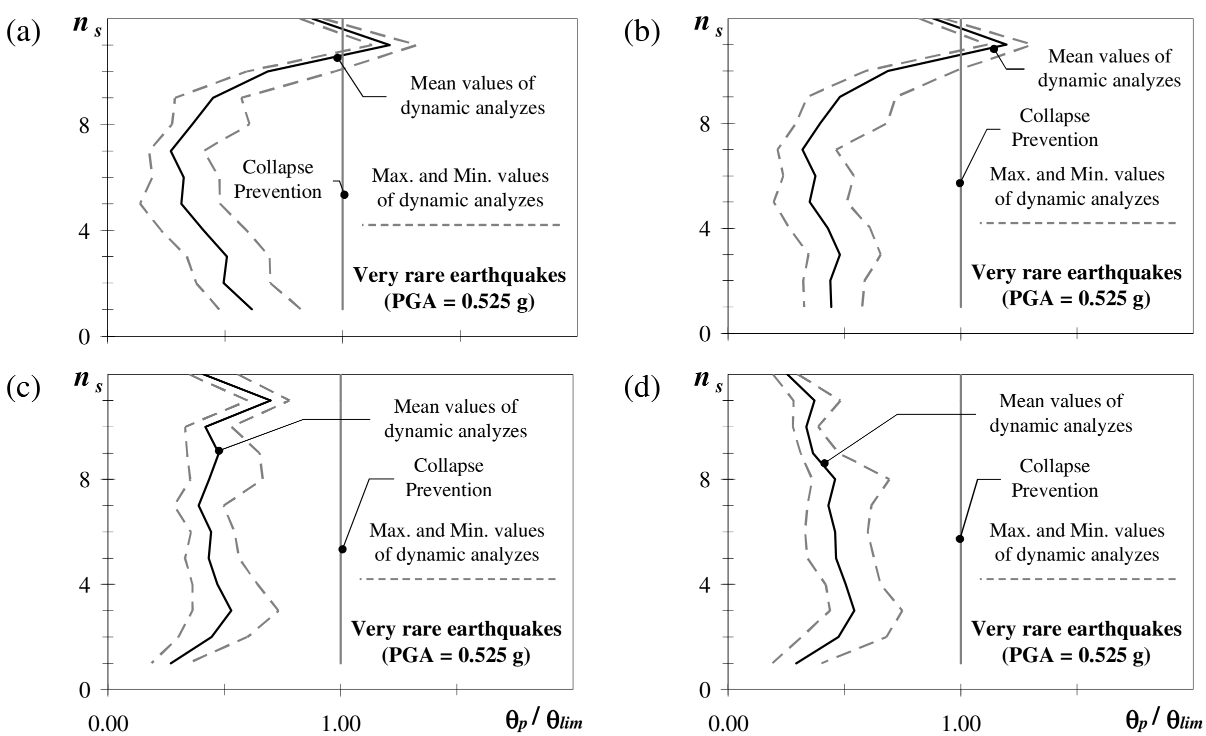


Figure 8. Height distribution of links normalized plastic rotation provided by very rare earthquakes.
(a) EBF-R, (b) EBF-C1, (b) EBF-C2, (b) EBF-C3.

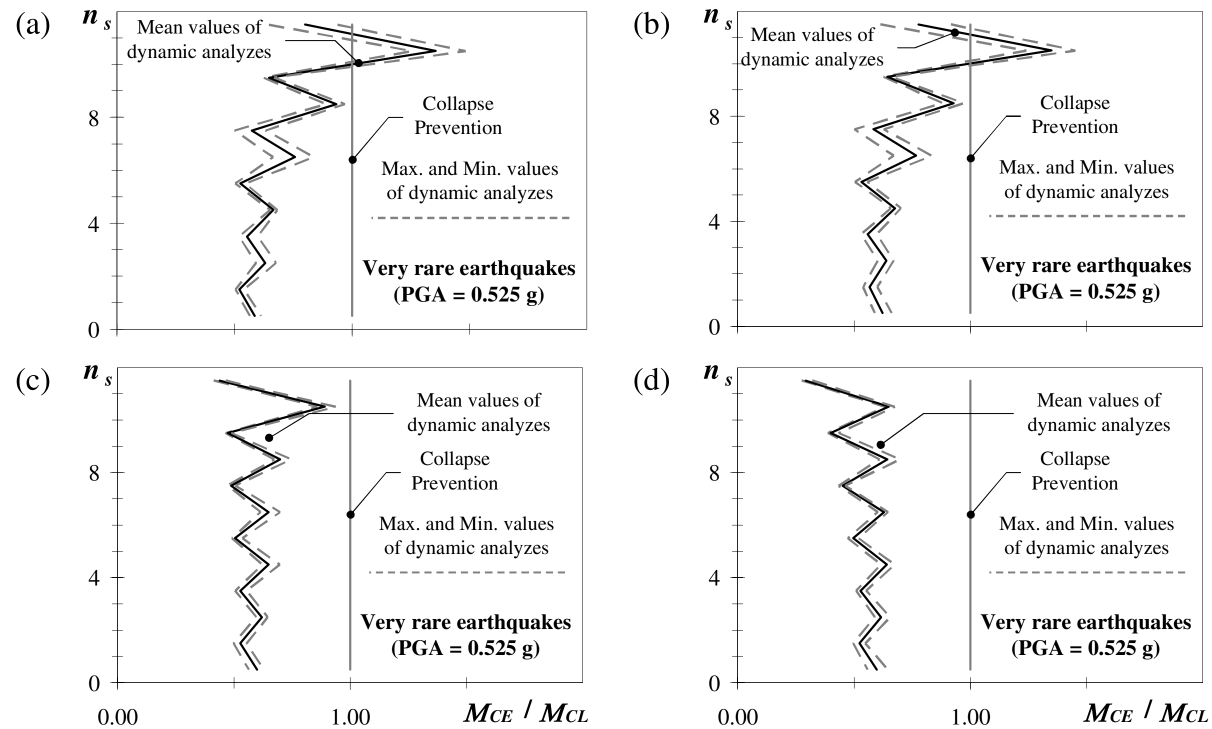


Figure 9. Height distribution of columns normalized bending moment provided by very rare earthquakes. (a) EBF-R, (b) EBF-C1, (b) EBF-C2, (b) EBF-C3.

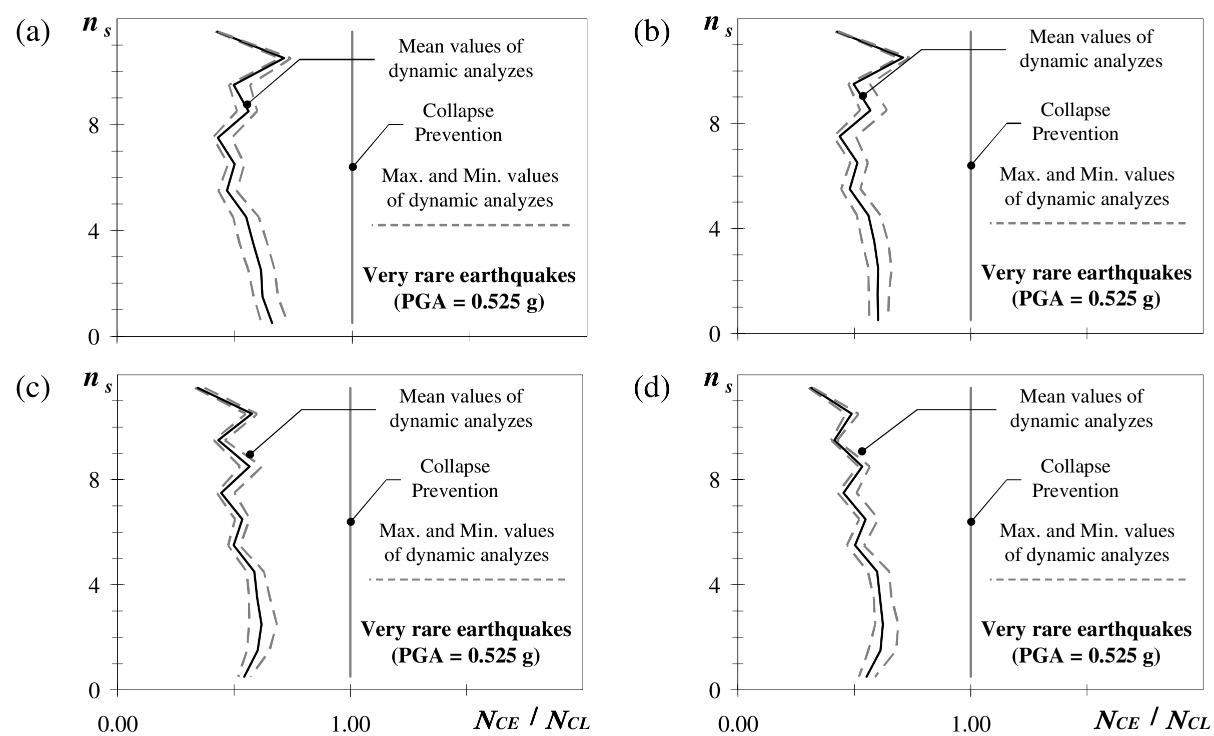


Figure 10. Height distribution of braces normalized axial force provided by very rare earthquakes. (a) EBF-R, (b) EBF-C1, (b) EBF-C2, (b) EBF-C3.

Figure 6d ~ Figure 9d, referring to the EBF-C3 system, show that, if the stiffness of the backup frame is increased in the upper stories, FEMA requirement on plastic deformation links in occurrence of frequent ground motions is also satisfied. However, no further improvements on the other performance parameters are observed.
Seismic performance of braces (Figure 10) seems to be not influenced by the additional stiffness provided by the backup frame. In braces, never axial force exceeds the buckling strength, as required by FEMA 356. Braces exhibit excellent seismic performances for the coupled systems as well as for the reference system.
One of the aims of the seismic design is to reach a uniform structural behaviour. Plastic deformations and internal actions uniformly distributed within the structure allow the better exploitation of the structural resources (in terms of plastic deformation capability and strength) and, therefore, to sustain stronger earthquakes. In order to compare the structural behaviour of the analyzed structures from this point of view, for each performance parameter, the coefficient of variation (COV) of the corresponding mean distribution along the height reported in Figure 6 ~ Figure 10 has been evaluated and showed in Figure 11. Performance parameters have been considered as lognormally distributed variables. The closer to zero is the COV and the more the structural behaviour is uniform along the height. The COV of normalized plastic rotation of the links (Figure 11a) shows that severe concentration occurs for both the EBF-R and EBF-C1 systems, especially with reference to the frequent earthquakes. Yielding of the links, conversely, is significantly more widespread when the EBF-C2 system is considered. A further, but less relevant, improvement is obtained with the EBF-C3 system. Analogous consideration may be repeated with reference to the columns behavior (Figure 11b). Results regarding the braces shows that normalized axial force is always uniformly distributed along the height, regardless the stiffness of the backup frame (Figure 11b).

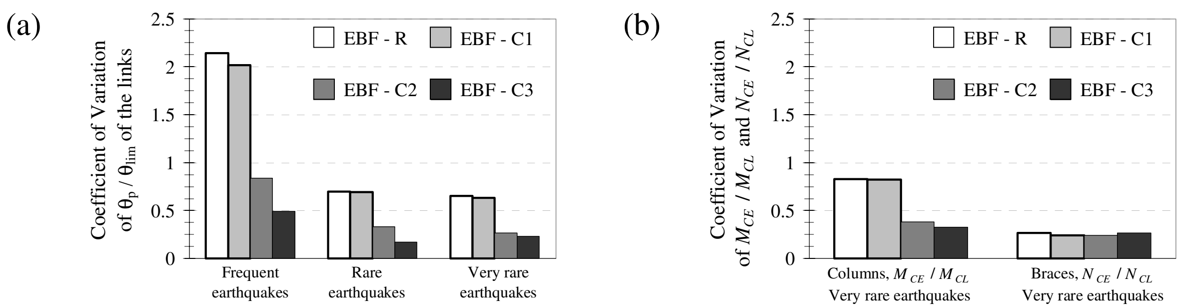


Figure 11. Coefficient of variation of the response parameters. (a) Normalized plastic rotation of the links, (b) normalized bending moment of the columns and normalized axial force of the braces.

CONCLUSIONS

In the paper the seismic performances of several structural configurations, in which horizontal forces are mainly sustained by eccentrically braced frames, have been evaluated according to FEMA 356 specifications and analyzed. Several performance parameters and seismic design levels have been considered.
Results regarding the reference system (EBF-R system) confirm that, if no additional stiffness is provided by the backup frame, high-rise eccentrically braced frames are prone to develop soft-story mechanisms. It results in large plastic deformation and bending moment demand, respectively in links and columns, concentrated in a few stories. The additional stiffness provided by continuous gravity columns (EBF-C1 system) does not provide any appreciable improvement of the seismic performances.
When beams bearing vertical loads are jointed to the gravity columns by means of rigid connection (EBF-C2 system), instead, the additional stiffness provided by the backup frame is sufficient to avoid the large degradation in story shear strength and stiffness characterizing the EBF-R and EBF-C1 systems. Plastic

deformations of links and seismic internal actions of columns and braces become widespread along the height of the building. As a consequence, the EBF-C2 system meets all the FEMA 356 requirements except that regarding plastic deformation of the links in occurrence of frequent earthquakes.
Finally, the EBF-C3 system, in which a larger additional stiffness is provided in the upper stories with respect to the EBF-C2 system, shows the best seismic performances. Nevertheless seismic behavior of EBF-C3 is close to that of the EBF-C2 system.

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