Seismic response of tied and trussed eccentrically braced frames

A. Ghersi, F. Neri, P.P. Rossi
Dipartimento di Ingegneria Civile e Ambientale, Faculty of Engineering, Catania (Italy)

A. Perretti
Dipartimento di Analisi e Progettazione Strutturale, Faculty of Engineering, Napoli (Italy)

ABSTRACT: This paper analyses the inelastic behaviour of eccentrically braced frames by means of pushover and dynamic response analyses. The structural typologies considered include standard eccentrically braced frames and schemes provided with vertical or diagonal elements, which connect the ends of each link to the upper storey (tied or trussed braced frames). The length of the links has been varied so as to examine both short links (which yield for shear) and long links (which yield for bending moment). The number of storeys has been varied from 4 to 12, so as to point out its influence on the behaviour of the scheme. Pushover analyses have shown some differences in the behaviour of schemes with short, intermediate or long links, together with an increase of ultimate displacements when ties or trusses are added, independently of the number of storeys. On the contrary, dynamic analyses have pointed out the strong influence of the number of storeys in the case of standard eccentrically braced frames, which are dramatically penalised by the arising of a soft-storey behaviour. The adoption of ties or trusses is really effective in preventing such behaviour and in granting the eccentrically braced frames a highly dissipative response also in the case of tall buildings.

1 INTRODUCTION

As it is well known, buildings located in seismic areas have to be designed in such a way to fulfil specific requirements. Although nowadays more detailed sets of structural performances are under discussion, the basic behavioural aspects to be analysed are those related to the occurrence of low or moderate seismic events and of very strong earthquakes. In the first case all structural elements should remain in the elastic range, while non-structural elements should be only slightly damaged; this aim is in practical applications achieved by limiting the storey drift, i.e. checking the stiffness. In the second case the structure may undergo large inelastic deformations; it is therefore fundamental in such conditions to assure to the structure the capacity to dissipate large amounts of energy by means of a stable hysteretic behaviour (i.e. granting proper values of local and global ductility). A Moment Resistant Frame (MRF) shows a really good dissipative capacity, thanks to the large number of plastic hinges developed when the structure fails in a global mechanism, but it is at the same time very flexible. For this reason its design is often governed by the limits imposed to the maximum displacements for low seismic events and its cross-sections are usually oversized respect to those strictly necessary in presence of the ultimate limit state loadings. On the contrary, a Concentrically Braced Frame (CBF) is very stiff but it is characterised by a quite poor inelastic behaviour: in occurrence of strong earthquakes this imposes the use of larger design forces in order to counterbalance the low levels of available ductility. Aiming at obtaining at the same time stiffness and ductility, a new typology has been proposed about 25 years ago: the Eccentrically Braced Frame (EBF). The presence of bracings, although not converging in the same point, provides the scheme enough stiffness (e.g. see Hjelmstad and Popov 1984), while the beam segment between the braces, named *link*, is able to undergo large plastic deformations and to dissipate a conspicuous amount of energy. The link is subjected to constant shear forces and linearly varying bending moments. When it is short, i.e. its length $e < 1.6\, M_p / V_p$, being M_p and V_p the limit values of bending moment and shear respectively, the shear yielding of the whole link dominates the inelastic response. When it is long, i.e. $e > 2.6\, M_p / V_p$, flexural yielding arises at its ends; in intermediate cases, the inelastic response is governed by a combination of shear and flexural yielding (see Kasai & Popov 1986a, AISC 1997).

The cyclic behaviour of short links is really stable, provided that the web buckling is prevented by means of proper web stiffeners (e.g. see Hjelmstad and Popov 1983). A very large monotonic plastic deformation (even 0.2 radiant) can be resisted without

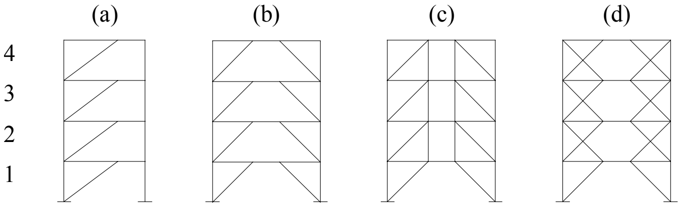

Figure 1. D-braced frames (a); split K-braced frames (b); tied braced frames (c); trussed braced frames (d)

a significant loss of capacity (Malley and Popov 1984), although a lower value (0.09 radiant) is suggested in order to limit strength deterioration in cyclic deformation (NEHRP 1994). Many tests have furthermore showed that, mainly because of steel hardening, the maximum shear force can be 1.5 times the plastic capacity V_p (Kasai & Popov 1986b, Ricles & Popov 1989).

Some technological aspects have to be considered in designing eccentrically braced frames. First of all, link-to-column connections in a D-braced scheme (Fig. 1a) may be subjected to deformation demands even larger than those of beam-to-column connections in a MRF, because the deformation is confined in a shorter portion of the beam. This requires more complex and costly connections, together with the necessity to carry out cyclic tests to confirm the inelastic behaviour. Furthermore the lack of symmetry of D-braced schemes may give rise to strongly different bending moments at the ends of the links, with the possibility of an early flexural yielding of the most stressed end. All these problems are bypassed when Split-K-braced schemes are used (Fig. 1b), which grant symmetry and require very simple beam-to-column pinned connections.

A second aspect to be considered is the influence of the connection of links to the floor slab. As a matter of fact, the vertical loads acting on the link modify both the value and distribution of the internal actions; this may give rise to an unexpected inelastic behaviour (flexural instead of shear yielding; partial shear yielding of the link). Furthermore, the large inelastic displacements of the link induce considerable deformation and damage in the floor slab. A suggested way to avoid this is to disconnect the floor slab from the lateral load resisting system, introducing, when necessary, an additional beam parallel to the one of the link (Perretti 1999).

The dissipative capacity of EBFs has been highlighted in many studies, mainly on the basis of pushover analyses but in some cases also by means of inelastic response analyses. Nevertheless it has been also shown that links may undergo very large deformation at a single floor, often the first one (Foutch 1989, Popov et al. 1989) or an intermediate or upper storey (Lu et al. 1997). AISC (1997) remarks that "in extreme cases this may result in a tendency to develop a soft storey" but in spite of this it gives no particular relevance to the negative consequences of such a possibility. A reliable method for obtaining uniform link deformation at all storeys has been devised by Ricles & Bolin (1991), who suggest to introduce vertical elements (ties) to connect the corresponding ends of the links of contiguous floors. This scheme (Fig. 1c) is here referred as Tied eccentrically Braced Frame (TBF). For the same purpose, an alternative (Fig. 1d) has been recently proposed by Perretti (1999), who suggests to connect the ends of each link to the beam-to-column node of the upper storey. It is thus obtained a scheme that in some way recalls a truss and which is named for this reason TRussed eccentrically Braced Frame (TRBF). Such a solution allows furthermore, respect to the previous ones, larger architectonical flexibility in placing windows within the frame.

This study analyses EBF, TBF and TRBF schemes by means of both pushover and inelastic dynamic response analyses, in order to compare their dissipative capacity and to find out how much it may be penalised by large inelastic deformations at single storeys and by soft storey mechanisms.

2 STRUCTURAL SYSTEMS AND DESIGN CRITERIA

This paper analyses the seismic behaviour of three steel buildings, having a square plan (24×24 m²) and 4, 8 and 12 storeys. Their structure is constituted by pinned frames arranged along an orthogonal grid, with span length L equal to 8 m. The horizontal actions are withstood by eccentrically braced frames located along the perimeter of the system (Fig. 2).

Decks are made by sheetings and light concrete so as to limit the mass of the structure. For the same reason high quality and light non-structural elements have been used. Dead and live loads of the deck are therefore supposed to be in total 5 kN/m².

The structure firstly examined encloses standard split K-braced frames with a link length e=0.1 L; all the columns are pinned at the base. The seismic design actions have been evaluated according to Eurocode 8, using static analysis with the design response spectrum proposed for subsoil class C with a peak ground acceleration $a_g = 0.35$ g, a behaviour factor $q = 5$ and a damping factor of 0.05. At this stage the

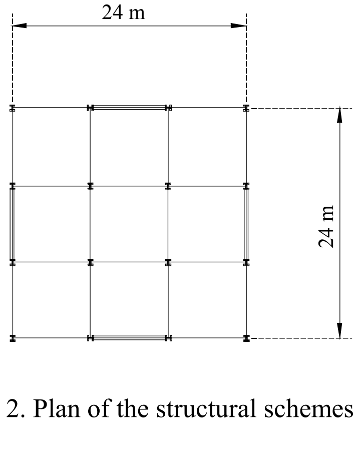

Figure 2. Plan of the structural schemes

Table 1. Parameters of design

n. storeys	Total seismic weight (kN)	Design period (s)	Design acceleration	Base shear for each EBF (kN)	Actual period (s)
4	11520	0.5	0.158 g	907.2	0.656
8	23040	1.0	0.136 g	1563.6	1.108
12	34560	1.5	0.104 g	1789.9	1.779

fundamental periods of vibration of the structures under analysis have been estimated in 0.5, 1.0 and 1.5 s for the systems having 4, 8 and 12 storeys respectively. Their actual values, calculated at the end of the phase of design, do not show great differences (Table 1).

The first step of the design consists in defining the cross-section of the links at the generic k^{th} storey. In presence of horizontal actions only, the vertical translation equilibrium of half of the frame from the top to the k^{th} floor gives the vertical action N_k transmitted to the floor below

$$N_k = \sum_{j=k}^{n_s} V_j \tag{1}$$

being n_s the number of storeys and V_j the shear in the link at the j^{th} storey. The rotational equilibrium of the portion of the frame above the level k gives the sum of the shear of the links from the top to the k^{th} floor

$$\sum_{j=k}^{n_s} V_j = \frac{\sum_{j=k}^{n_s} F_j \ (h_j - h_{k-1})}{L} \tag{2}$$

where F_j is the horizontal action and h_j is the height of the floor j respect to the base. This formulation may be used to design their shear strength, proceeding from the top to the base of the scheme. If the strictly necessary strength is given to each link, the value of the design shear at each storey is obtained by the formula

$$V_k = \frac{\left(\sum_{j=k}^{n_s} F_j \right) h_k^s}{L} \tag{3}$$

being h^s_k the storey height at the storey under consideration.

The necessity of using commercial cross-sections implies anyway some overstrength for the links. Economic reasons, which suggest adopting the same cross-section at many storeys, lead to further increases in the overstrength. In the examined cases, aiming at limiting at the same time the overstrength and the number of different cross-sections of the links, the same cross-section has been adopted every two floors.

All the other elements (columns, bracings and ties) shall be defined according to the capacity design criterion, i.e. basing on the internal actions corresponding to the maximum capacity of the link. As previously mentioned a realistic relationship between the ultimate shear V_u and the yielding value V_y may be given by the following expression

$$V_u = 1.5\, V_y \tag{4}$$

In order to take into account all other uncertainties in geometrical and mechanical characteristics, this value is furthermore increased by means of a factor 1.2 (i.e. multiplying V_y by 1.8) so as to effectively grant that yielding or buckling will never occur in bracings and ties before links have reached their ultimate shear.

The design value of the axial force in columns is sum of the share due to the vertical loads and of that due to the presence of the plastic shear in the links. The low probability that in tall buildings the links contemporarily reach the ultimate shear value at all the storeys allows the designer to decrease the amplification factor previously described, used for bracings and ties. In the analysed schemes, the overall amplification factor of the axial force deriving from the yielding shears has been assumed equal to 1.8 for the upper four storeys; it has been differently fixed equal to 1.5 and 1.2 for the stories starting from the top from the fifth to the eighth and from the ninth to the twelfth respectively. The cross-section chosen according to this procedure are shown in Table 2.

In order to analyse the influence of the link length on the seismic behaviour of the system, eccentrically braced frames with different values of the link length (0.1 L, 0.15 L, 0.2 L, 0.3 L and 0.4 L) have been considered. The same cross-sections previously selected have been used in all these structures. Furthermore, analogous schemes with the addition of tie or truss elements have been studied. Thus, a total of 3×5×3 schemes have been considered.

Table 2. Cross-sections

storey	link			bracing	column
	section	V_y (kN)	M_y (kNm)	section	section
4 storey scheme					
3-4	HEA240	221.3	158.9	HEA160	HEA160
1-2	HEA320	357.9	347.8	HEA180	HEA240
8 storey scheme					
7-8	HEA240	221.3	158.9	HEA160	HEA160
5-6	HEA360	449.0	446.1	HEA220	HEA240
3-4	HEA400	550.3	547.3	HEA240	HEA340*
1-2	HEA450	649.1	687.1	HEA260	HEA400**
12 storey scheme					
11-12	HEA220	188.6	121.3	HEA160	HEA160
9-10	HEA320	357.9	347.8	HEA180	HEA240
7-8	HEA400	550.3	547.3	HEA240	HEA300*
5-6	HEA400	550.3	547.3	HEA240	HEA500*
3-4	HEA500	754.3	843.4	HEA280	HEA500**
1-2	HEA500	754.3	843.4	HEA280	HEA500**

* steel grade Fe430   all other sections:
** steel grade Fe510   steel grade Fe360

Table 3. First period of vibration of the analysed schemes (s)

Scheme; n. storeys		e/L 0.10	0.15	0.20	0.30	0.40
EBF	4	0.656	0.768	0.899	1.196	1.518
	8	1.108	1.206	1.329	1.630	1.980
	12	1.779	1.883	2.018	2.361	2.775
TBF	4	0.649	0.754	0.878	1.158	1.465
	8	1.103	1.196	1.312	1.590	1.928
	12	1.775	1.873	2.000	2.322	2.716
TRBF	4	0.616	0.726	0.854	1.138	1.447
	8	1.073	1.167	1.284	1.569	1.900
	12	1.739	1.839	1.967	2.291	2.683

In order to synthesise their dynamic characteristics, the values of their first period of vibration are reported in table 3.

3 PUSHOVER ANALYSES

The inelastic behaviour of all the above-mentioned structures has been firstly tested by means of pushover analyses. Vertical loads are directly applied to the columns, assuming that a proper framing of the deck or an additional beam are used to avoid to apply vertical loads to the link. Horizontal forces are linearly variable along the height, i.e. proportional to the design actions.

Both shear and flexural yielding have been checked at the ending cross-sections of the links. According to the experimental results previously referred to, the shear deformation of the link is described by a bilinear elastic-plastic model. The relationship between V_u and V_y is given by Equation (4) and the ultimate rotation is equal to 0.09 radiant; the plastic modulus of elasticity is supposed to be approximately 1/150 of the elastic one. The flexural yielding is schematised by means of plastic hinges; their ultimate flexural rotation is assumed equal to 0.06 radiant because of the presence of stiffeners, which reduce the effects of local buckling.

The possibility of flexural yielding of the columns has not been expressly taken into account in the analysis. Anyway a final check has confirmed that the columns would quite always remain in the elastic range up to the collapse of the structure, consistently with the capacity design criterion used in design.

The intensity of the lateral forces has been increased up to the failure of the frame, conventionally defined as the achievement of the above listed values of the ultimate shear or flexural rotation. The second order effects of the vertical loads have been not taken into account at this stage of the research.

Figures 3, 4 and 5 show the base shear versus top displacement relationship for EBF, TBF and TRBF respectively. In each figure the curves corresponding to different values of *e/L* are clearly distinct, because the strength and the stiffness of the schemes decrease as the length of the link increases.

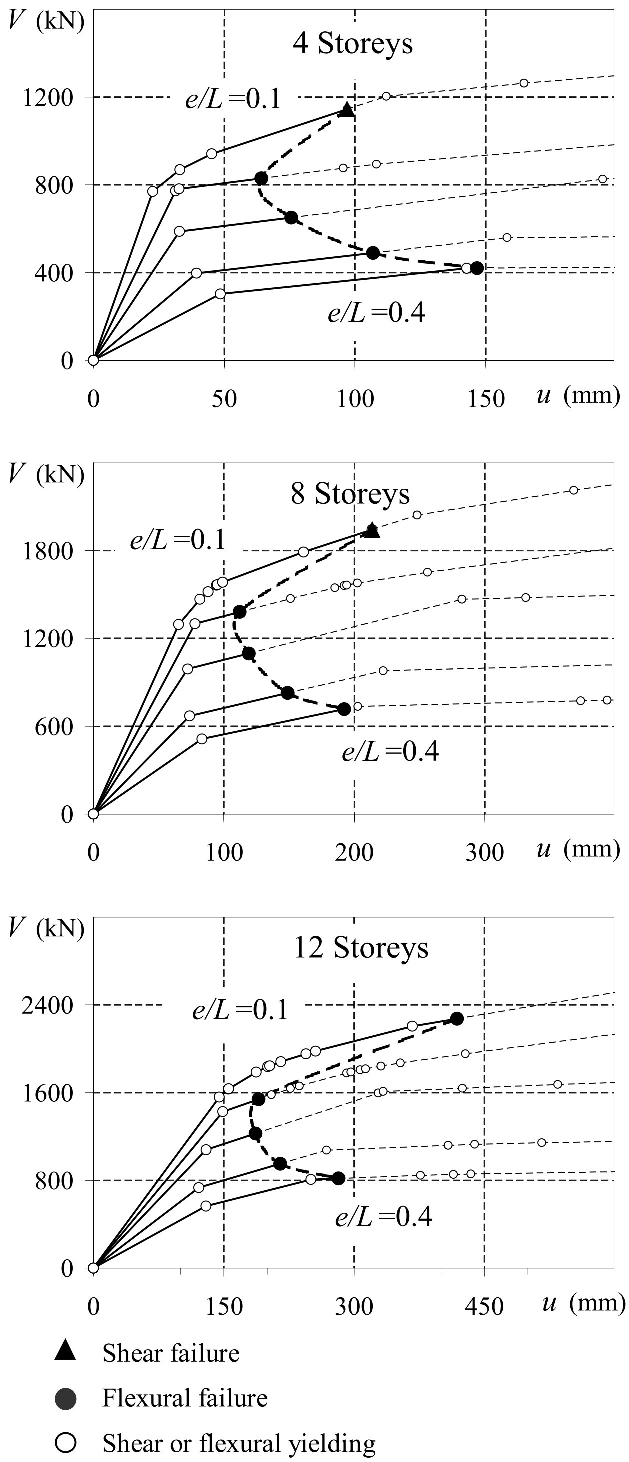

Figure 3. Pushover analyses of eccentrically braced frames

The presence of ties or trusses gives a moderate increase to the stiffness and in some cases also to the strength of scheme.

In all the EBFs (Fig. 3) the links do not yield in correspondence of a unique value of the multiplier of the lateral forces, because of the overstrength of some links respect to the design actions.

In the frames with long links (*e/L*=0.30-0.40), which experience flexural yielding, the increase of flexibility at first plastifications is so large as to bring the structure to collapse with only one or two links in the plastic range.

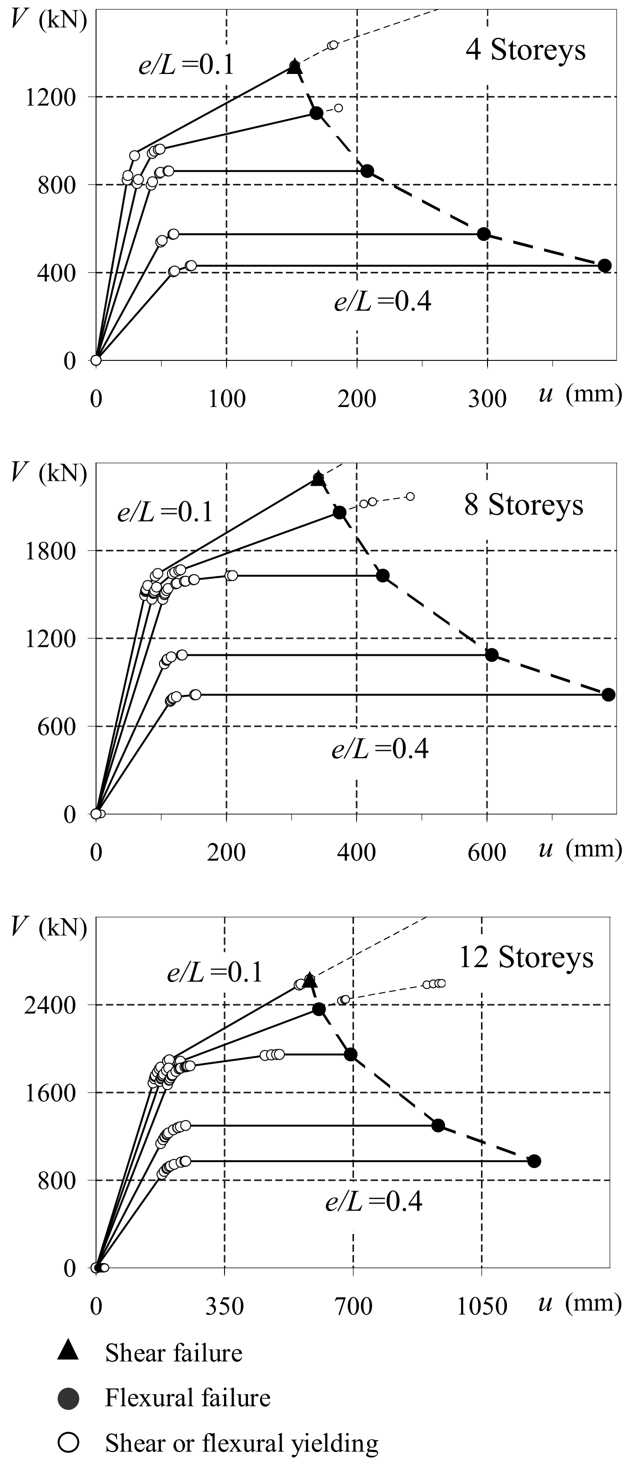

Figure 4. Pushover analyses of tied braced frames

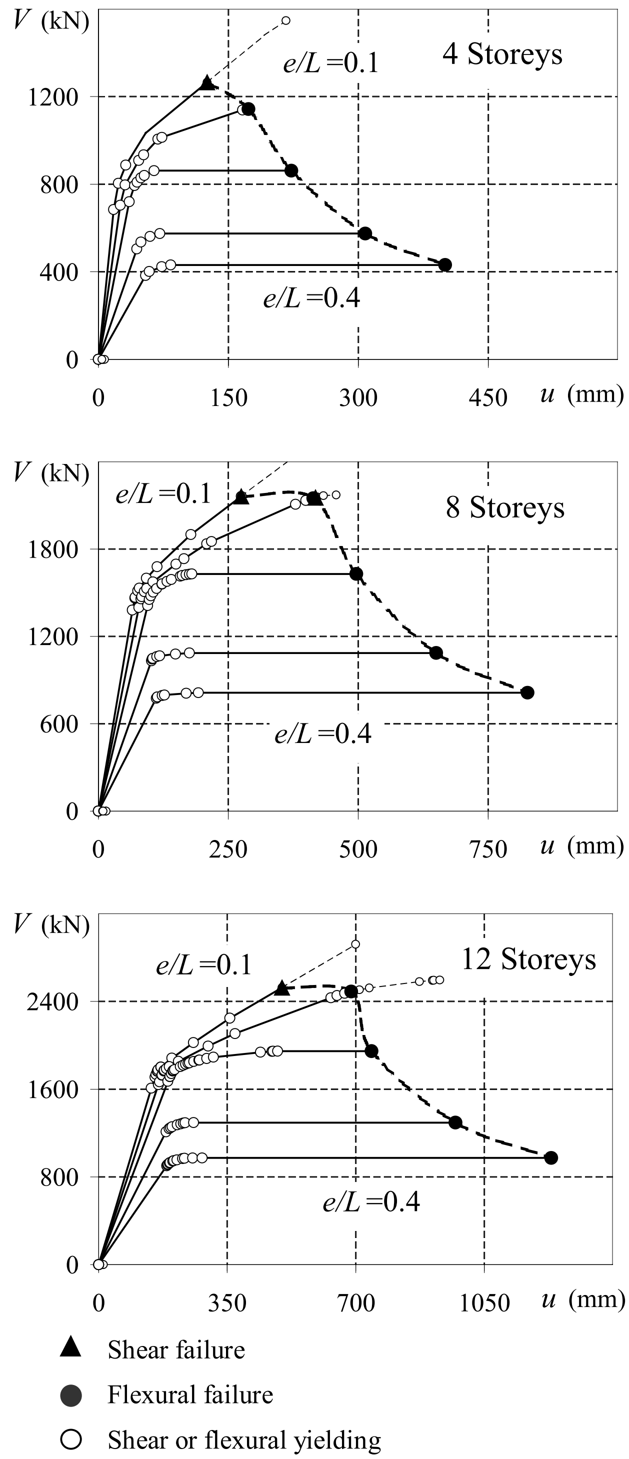

Figure 5. Pushover analyses of trussed braced frames

This may be partially owed to the fact that flexural hardening is neglected in the model, because the stiffness given by it might in some cases modify the described behaviour. On the contrary, in the frames with short links (*e/L*=0.10) nearly all links yield for shear before failure. The ultimate horizontal displacement is influenced by the length of the links, being larger in systems with very short or long links. The number of storeys does not seem to substantially modify the above-mentioned observations.

Differently from standard eccentrically braced frames, the presence of ties (Fig. 4) or truss elements (Fig. 5) force links to present quite equal deformations at each level of the lateral load. Because of that in tied and trussed braced frames the first plastic hinge occurs at values of the lateral forces slightly higher than those of standard eccentrically braced frames. For the same reason, most plastifications of the links occur in correspondence of a narrow range of values of the horizontal forces. Also the collapse load is slightly higher than that of EBFs and it is achieved with a larger number of yielded sections.

Anyway, the most important differences may be noted in the values of horizontal displacements at collapse. In the case of schemes with short links (*e/L*=0.10) they are about 50% larger than the corresponding values of EBFs. The differences are even more relevant in the case of systems with longer links and seem to be emphasised in the buildings with a larger number of storeys. The P-Δ effect, neglected in the analyses, could slightly reduce the maximum displacements, but it would not change the overall behaviour of the schemes.

4 DYNAMIC ANALYSES

Each one of the systems described in section 2 has been subjected to a set of ten accelerograms, artificially generated by means of the procedure proposed by Falsone & Neri (1999). The mean value of their spectral pseudo-accelerations matches the response spectrum proposed by EC8 for soft soil (class C) and for a damping ratio equal to 5%. 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.

The shear and flexural yielding of the links has been modelled as described in the previous section; no strength or stiffness deterioration has been considered in the cyclic behaviour of the elements. The non-linear behaviour of the ending cross sections of the columns has been schematised by means of an elastic-perfectly plastic moment-curvature relationship, taking into account the influence of the axial force on the yielding value of the bending moment.

The inelastic response analyses, performed by means of the well-known DRAIN-2D code, have been repeated scaling each accelerogram to different values of the peak ground acceleration, so as to detect the intensity $a_{g,y}$ which causes within the structure the first plastic hinge and that one $a_{g,u}$ which provokes the structural failure. For each accelerogram, has thus been possible to evaluate the ratio

$$q = \frac{a_{g,u}}{a_{g,y}} \tag{5}$$

The mean of the values computed for the ten accelerograms has been assumed as representative of the actual behaviour factor q of the scheme.

As already noticed for the pushover analyses, the structural failure generally occurs for achievement of the shear or flexural deformation capacity in systems with short and long links respectively.

The mean value of the behaviour factor in eccentrically braced systems is always lower than the values noticed in tied and trussed braced frames, both in presence of short and long links (Fig. 6). Furthermore, it is strongly conditioned by the number of storeys. In particular, it is quite comparable to the values suggested by Eurocode 8 in the case of low buildings (4 storeys), ranging from 4 to 5.5, but it rapidly decreases in the case of taller buildings, ranging from 2.4 to 3.6.

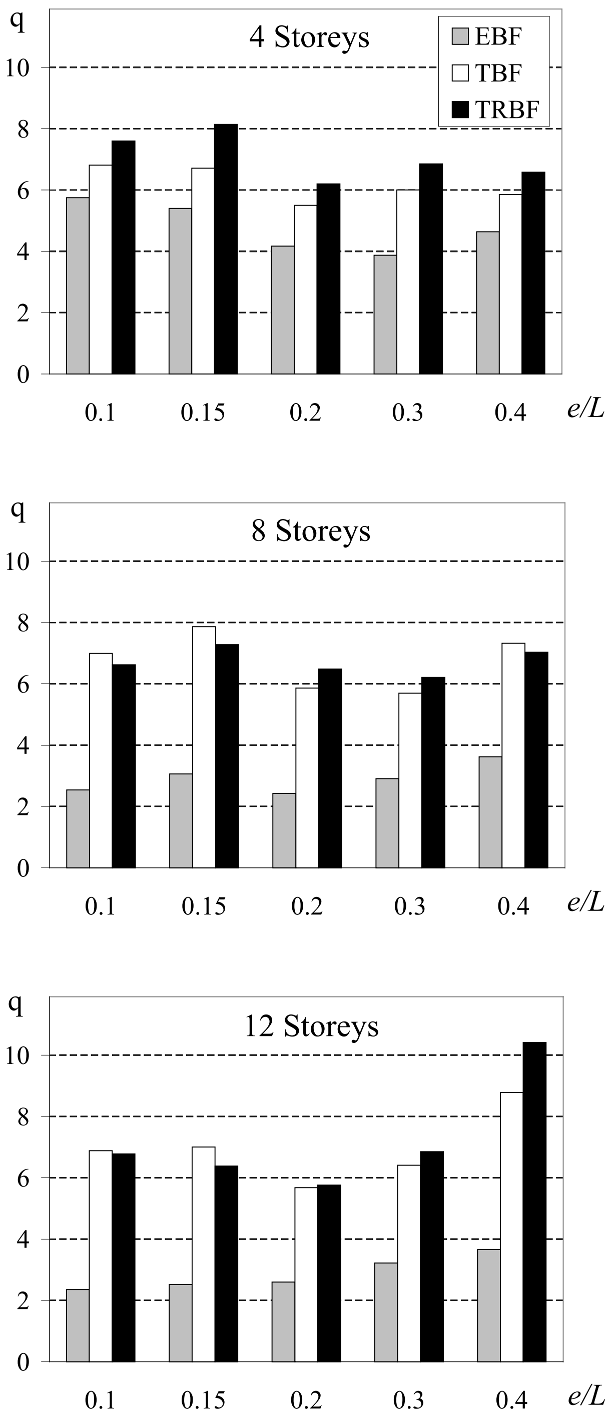

Figure 6. Behaviour factors of standard eccentrically braced frames, tied and trussed braced frames

On the contrary the values in tied and trussed frames are quite stable on varying the number of storeys, ranging from 5.5 to 8 in frames with four storeys and from 6 to 10 in systems with 12 storeys.

The difference between standard EBFs and tied or trussed braced frames and the influence of the number of storeys on it is confirmed by the amount of energy dissipated by the different schemes in the collapse configuration (Fig. 7). As expected, the dissipated energy is mostly owed to shear yielding in the case of short links and to flexural yielding in the case of long links; the amount of damping energy is always negligible. In the case of low buildings (4 storeys), the hysteretic energy dissipated by standard

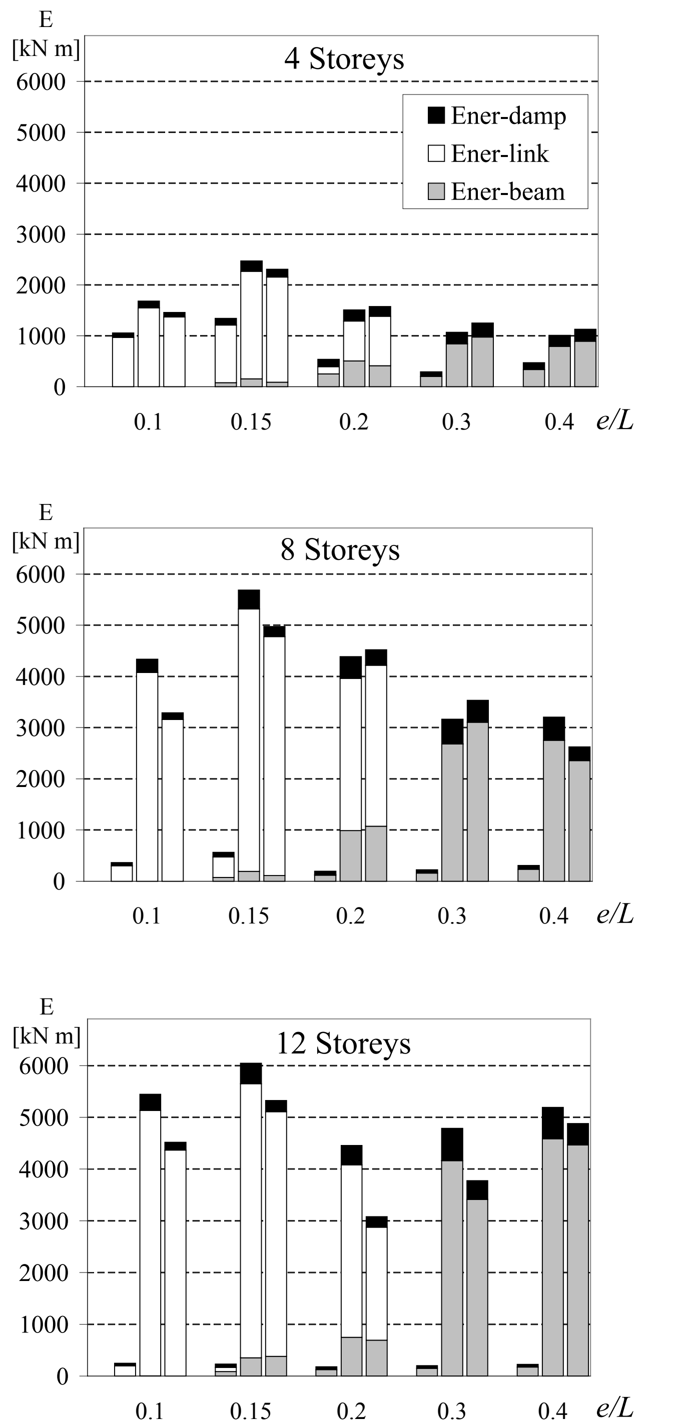

Figure 7. Dissipated energy of standard eccentrically braced frames, tied and trussed braced frames

EBFs is always smaller than that of TBFs or TRBFs, but the difference is not extremely large. As the number of storeys increases, the first one decreases while the other increase; the difference thus becomes really conspicuous.

Such different behaviour of standard eccentrically braced frames and tied or trussed braced frames may be understood by analysing in detail the step-by-step response of the schemes. As already noticed in pushover analyses, the deformation of the links is quite equal at all the storeys in tied and trussed frames while it is appreciably different in standard EBFs. Consequently, in tied and trussed schemes the links reach the maximum shear deformation (or flexural plastic deformation) at the same time and show quite the same value at every storey (Fig. 8 b, 9 b). On the contrary, in EBFs the links present different values of the maximum plastic deformation, reached at different times. Anyway, in frames having a low number of storeys most links can reach a large plastic deformation (Fig. 8 a), thus providing a good dissipation, while in higher frames very few links are significantly yielded (Fig. 9 a) and the inelastic behaviour is very poor.

Such a behaviour, which is not dependent on the length of the link, may be explained by the fact that when a link is yielded the flexibility at that storey increases dramatically. This has minor importance in pushover analyses, in which horizontal forces increase proportionally independently of the plastification of any cross-section, while it has enormous influence on the dynamic behaviour of the scheme (soft storey) where the inertial forces change at different times according to the stiffness of the structure.

5 CONCLUSIONS

The performed analyses have first of all pointed out that the dynamic response of standard eccentrically braced frames may be really worst than what considered by many researchers and assumed by seismic codes. The large increase of flexibility at a storey when a link yields gives rise to a ”soft-storey” behaviour, which modifies the dynamic response and does not allow the other links to develop their dissipative capacity. The suggestion of providing some overstrength to the links of lower storeys, given by AISC (1997), appears to be ineffective because in most cases this problem arises at intermediate or upper storeys. The location of possible soft storeys cannot be easily foreseen, because it depends on the distribution of strengths and masses and on the random characteristics of the seismic input.

On the contrary, the analyses have confirmed the good inelastic behaviour of eccentrically braced frames with vertical connections between links (tied braced frames). Independently of the number of storeys, the presence of ties forces all the links to the same deformation, sweeping away the risk of soft storey behaviour.

Finally, the analyses have shown that eccentrically braced frames with diagonal connections between links and nodes (trussed braced frames) have a dynamic inelastic behaviour quite similar to that of tied braced frames. This new typology thus appears to be a possible alternative for designing structures able to sustain strong seismic events.

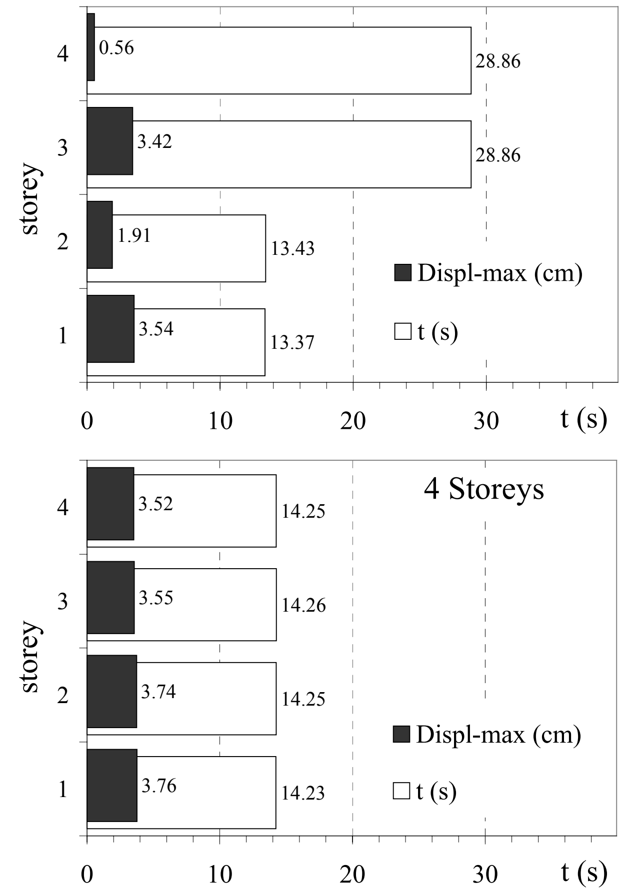

Figure 8. Maximum vertical displacement of the end of the link and time at which it is reached, for a standard EBF (a) and a tied braced frame (b) having short links and 4 storeys.

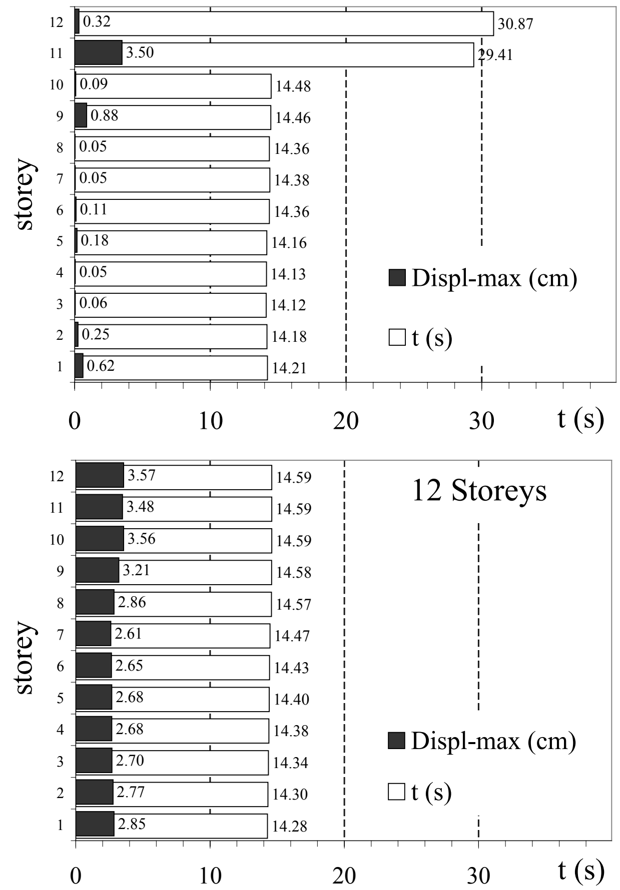

Figure 9. Maximum vertical displacement of the end of the link and time at which it is reached, for a standard EBF (a) and a tied braced frame (b) having short links and 12 storeys.

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