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Seismic response of EB-frames with inverted Y-scheme: TPMC versus eurocode provisions

R. Montuori[a], E. Nastri* and V. Piluso[b]

Department of Civil Engineering, University of Salerno, Italy

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Abstract. The Theory of Plastic Mechanism Control (TPMC) has been recently extended to the case of Eccentrically Braced Frames (EBFs) with inverted Y-scheme, i.e., EBFs with vertical links. In this paper a further validation of the design procedure, based on TPMC, is provided by means of Incremental Dynamic Analyses (IDA) pointing out the fulfilment of the design goal, i.e., the development of a pattern of yielding consistent with the collapse mechanism of global type where all the links are yielded and all the beams are yielded at their ends while all the columns and the diagonal braces remain in elastic range with the only exception of the base sections of first storey columns. In particular, a study case is designed according to both TPMC and Eurocode 8 provisions and the corresponding seismic performances are investigated by both push-over and IDA analyses.

The results show the different performances obtained in terms of pattern of yielding, maximum interstorey drift, link plastic rotation demand and sharing of the seismic base shear between the moment-resisting part and the bracing part of the structural system. The seismic performance improvement obtained by means of TPMC, compared to Eurocode 8 provisions, is pointed out.

Keywords: thorey of plastic mechanism control; eccentrically braced frames; seismic; steel; Eurocode 8

1. Introduction

EB-Frames constitute a suitable compromise between seismic resistant MR-Frames and CB-Frames, because they exhibit both adequate lateral stiffness, due to the high contribution coming from the diagonal braces, and a ductile behaviour, due to wide and stable hysteresis loops of links constituting the dissipative zones of this structural typology.

As it is universally recognised, one of the primary aims of seismic resistant design is to avoid partial collapse mechanisms and soft storey mechanisms which significantly undermine the energy dissipation capacity of structures. The optimization of the seismic structural response is, conversely, obtained when a collapse mechanism of global type is developed, because, in such case, all the dissipative zones are involved in the corresponding pattern of yielding, leaving all the

*Corresponding author, E-mail: enastri@unisa.it
[a]Ph.D., E-mail: r.montuori@unisa.it
[b]Professor, E-mail: v.piluso@unisa.it

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other structural parts in elastic range. According to the basic principles of capacity design, dissipative zones have to be designed according to the internal actions arising from the load combinations given in code provisions, whereas non-dissipative zones have to be proportioned on the basis of the maximum internal actions which dissipative zones are able to transmit in the fully yielded and strain-hardened state. The hierarchy criteria provided by seismic codes (CEN 2005a, Elghazouli 2010) are often only approximated applications of capacity design principles. As an example, this is the case of the beam-column hierarchy criterion commonly suggested by seismic codes to design the column sections of MR-Frames. It is well known that such hierarchy criterion is able to prevent soft storey mechanisms, but it is not adequate to assure a collapse mechanism of global type (Tirca and Chen 2012, Mazzolani and Piluso 1996). In addition, in case of structural typologies not widely investigated, code provisions are even not able to prevent soft storey mechanisms. This is the case of the structural typology herein investigated, i.e., EB-Frames with inverted Y-scheme, whose design procedure as suggested by Eurocode 8 is not able to avoid partial mechanisms and even soft storey mechanisms.

Therefore, it is apparent that a rigorous application of capacity design principles requires more sophisticated design procedures. This is the case of column design aiming to assure a collapse mechanism of global type, i.e., a collapse mechanism assuring the dissipation of the earthquake input energy by the participation of all the dissipative zones, i.e., all the links and the beam ends close to the beam-to-column connections, while all the non-dissipative zones remain in elastic range with the only exception of base sections of first storey columns. The theory of plastic mechanism control has been developed to assure this ambitious design goal. Such theory was proposed for the first time by Mazzolani and Piluso (1996, 1997), with reference to MRFs with rigid full-strength beam-to-column connections and successively extended to the case of semi-rigid partial-strength connections (Faella *et al*. 1998) and to the case of “dog-bones” (Montuori and Piluso 2000), i.e., RBS (reduced beam section) connections. Further advancements have been obtained with the extension of the theory to the case of eccentrically braced frames with horizontal link elements (Mastrandrea and Piluso 2009), knee-braced frames (Conti *et al*. 2009), dissipative truss-moment frames (Longo *et al*. 2012a, b) and MRF-CBF dual systems (Giugliano *et al*. 2010). In this paper, two goals are achieved. The first one is the conclusion of the validation, by means of incremental dynamic analyses, of the design procedure for EB-Frames with inverted Y-scheme based on the theory of plastic mechanism control, which started in a previous work (Montuori *et al*. 2014a); the second one is the comparison of the seismic response of eccentrically braced frames with inverted Y-scheme designed by means of Eurocode 8 (CEN 2005a) with that occurring when the proposed design procedure is applied. In particular, the structural scheme whose design according to TPMC has been already reported in detail in a previous paper (Montuori *et al*. 2014a) is herein also designed according to Eurocode 8 and the corresponding seismic performances are investigated by means of incremental dynamic analyses which are carried out to validate the design goal of the plastic mechanism control, i.e., the development of a global collapse mechanism, and to underline the differences in terms of ductility, collapse mechanism typologies and energy dissipation for both the designed structures.

Therefore, the present paper is focused on the evaluation of the seismic response of such dual systems designed according to TPMC while the reader interested to the theoretical background of the proposed design methodology and its application can make reference to a recent work of the same authors (Montuori *et al*. 2014a).

2. Failure mode control

The number of possible collapse mechanisms of eccentrically braced frames with inverted Y-scheme is very high, because at each storey yielding can develop in links, beams, columns and diagonal braces depending on the relative flexural strength of members. With reference to one-storey EB-Frames the design conditions to be satisfied to avoid undesired collapse mechanisms, partially or even not involving link members, have been obtained in a previous work (Montuori *et al*. 2014b). The obtained local hierarchy criteria have to be applied at each storey in case of multi-storey structures. They constitute a rigorous application of capacity design principles assuring that yielding involves only the link member while the beam and the diagonal braces remain in elastic range. In case of multi-storey EB-Frames, as soon as the beam and diagonal sections are selected at each storey to satisfy local hierarchy criteria, the problem of plastic mechanism control from the overall point of view needs to be faced to design the column sections.

The presentation of the Theory of Plastic Mechanism Control (TPMC) for EB-Frames with inverted Y-scheme is out of the scope of this work. Therefore, only the basic assumptions and concepts are herein provided. The interested reader can find all the details in previous works where numerical examples are also provided (Montuori *et al*. 2014a). In particular, dealing with the overall behaviour of the structure, the possible collapse mechanisms can be considered as belonging to three main typologies as depicted in Fig. 1 (Montuori *et al*. 2014a).

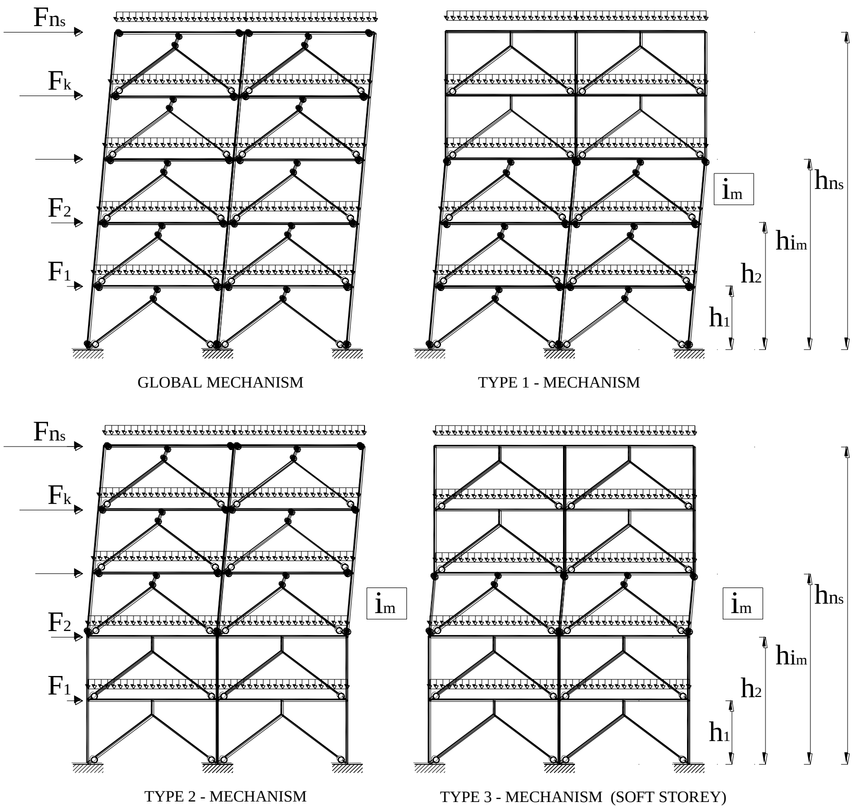

Fig. 1 Collapse mechanism typologies for multi-storey EB-frames

Mechanism type-1 is a partial mechanism involving the links of the first i_m storeys and requires the yielding of the beam ends of the first i_m-1 storeys. In addition, plastic hinges at the base of first storey columns and at the top section of i_m-th storey columns are developed. Mechanism type-2 is also a partial mechanism involving the i_m-th storey and those above it. The yielding of links and beam ends of such storeys also occurs. Column yielding is developed only at the base sections of i_m-th storey. Mechanism type-3 is a soft-storey mechanism involving only the links and the columns of i_m-th storey.

It can be observed that the global mechanism, which represents the design goal of TPMC, is a particular case of type-2 mechanism occurring for i_m=1.

The proposed plastic design procedure for failure mode control is based on the assumption that vertical link elements are preliminarily designed according to the internal actions due to the design seismic forces. Therefore, the link elements are designed according to the following requirements

$$V_{link.k.Ed} = \frac{1}{n_{br}} \sum_{i=k}^{n_s} F_i \leq V_{link.k.Rd} \qquad M_{link.k.Ed} = V_{link.k.Ed} \frac{e}{2} \leq M_{link.k.Rd} \tag{1}$$

where F_i is the storey seismic horizontal force, $V_{link.k.Rd}$ is the link shear resistance at kth storey, $M_{link.k.Rd}$ is the link flexural resistance at k-th storey, e is the link length and n_{br} is the number of braced bays. Beam and diagonal sections are also assumed to be known quantities, because they need to be designed to fulfil local hierarchy criteria assuring, at storey level, that yielding occurs in the link element only (Montuori *et al*. 2014b). Conversely, column sections are the unknowns of the design problem. The theory of plastic mechanism control (TPMC) includes also the influence of second order effects by means of the concept of mechanism equilibrium curve (Mazzolani and Piluso 1997).

In fact, the design requirements are derived by means of the kinematic theorem of plastic collapse extended to the concept of mechanism equilibrium curve (Montuori *et al*. 2014a). Column sections are obtained by imposing that the mechanism equilibrium curve corresponding to the global mechanism has to be located below those corresponding to all the undesired mechanisms within a displacement range compatible with the local ductility supply of dissipative zones (Fig. 2). In fact, the actual behaviour of structures is elasto-plastic and it is characterized by significant displacements before the collapse mechanism is completely developed. These displacements are responsible of second order effects, so that as far as the top sway displacement (δ) increases, the kinematically admissible horizontal force multiplier decreases unless significant strain-hardening occurs. The design process cannot neglect this important issue so that the upper bound theorem of plastic collapse has to be verified for each displacement level. This is assured by extending the kinematic or upper bound theorem of plastic collapse to the concept of mechanism equilibrium curve. Consequently, the design conditions are provided by the following relationships (Montuori *et al*. 2014a)

$$\alpha^{(g)} - \gamma^{(g)}\delta_u \leq \alpha_{i_m}^{(t)} - \gamma_{i_m}^{(t)}\delta_u \qquad i_m = 1,2,3,\ldots,n_s \quad t = 1,2,3 \tag{2}$$

where $\alpha^{(g)}$ and $\gamma^{(g)}$ are, respectively, the kinematically admissible multiplier of horizontal forces and the slope of the softening branch of the α-δ curve corresponding to the global type mechanism, while, $\alpha_{i_m}^{(t)}$ and $\gamma_{i_m}^{(t)}$ have the same meaning of the previous symbols, but they are referred to the i_m-th mechanism of t-th type.

The reader interested to the mathematical details to solve the design conditions given by Eq. (2)

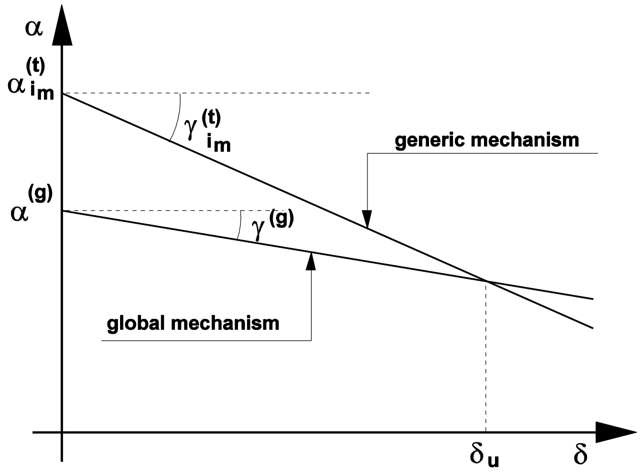

Fig. 2 Design requirements concerning mechanism equilibrium curves

can make reference to a recent paper (Montuori *et al*. 2014a) published by the same authors.

3. Eurocode 8 design provisions

Eurocode 8 design procedure for EBFs is based on simplified hierarchy criteria following the same principle also applied in case of MRFs. In particular, such principle is constituted by the use of an amplifying factor whose aim is the prevention of yielding or buckling of non dissipative elements when the most stressed dissipative zone is yielded and strain-hardened up to its ultimate condition. In the examined case, dissipative zones are constituted by vertical link elements whose stress level is related to the following ratios (CEN 2005a)

$$\Omega_i = 1.5 \frac{V_{p.link.i}}{V_{Ed.i}} \tag{3}$$

in case of short links, and

$$\Omega_i = 1.5 \frac{M_{p.link.i}}{M_{Ed.i}} \tag{4}$$

in case of intermediate and long links, where $V_{p.link.i}$ and $M_{p.link.i}$ are the plastic design resistance under pure shear and bending respectively, 1.5 is an overstrength factor and $V_{Ed.i}$ and $M_{Ed.i}$ are the internal actions, shear and bending moment respectively, occurring in the i-th link element under the seismic load combination.

The most stressed link is identified by means of the minimum value among all the Ω_i ratios computed for each link element. In order to assure a uniform participation of all the link to the dissipation of the earthquake input energy, Eurocode 8 suggests that the difference between the maximum and the minimum value of Ω_i should not be greater than 25% of the minimum value. Regarding non dissipative elements, i.e., columns, beams and diagonal braces the most unfavourable combination of the axial force and bending moments has to be considered to check the following requirement (CEN 2005a)

$$N_{pl.Rd}(M_{Ed}, V_{Ed}) \geq N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} \tag{5}$$

where:

- $N_{pl.Rd}(M_{Ed},V_{Ed})$ is the axial design resistance evaluated considering the interaction with the bending moment, M_{Ed}, and the shear, V_{Ed}, occurring in the seismic load combination;
- $N_{Ed,G}$ is the axial force due to non-seismic loads included in the seismic load combination;
- $N_{Ed,E}$ is the axial force due to seismic loads only;
- γ_{ov} is an overstrength coefficient taking into account random material variability;
- Ω is the minimum value of Ω_i computed among all the links.

It is easy to understand that the above design criterion is able to prevent yielding or buckling of non-dissipative elements before yielding of the most stressed link element, but it cannot assure a pattern of yielding of global type.

4. Study case

The study case herein investigated is constituted by an eight-storey EB-frame with inverted Y-scheme as depicted in Fig. 3. The bay span is L=6.0 m, the interstorey height is H=3.0 m. Regarding the length of the links, it has been changed at each storey aiming to obtain a non-dimensional link length ($\bar{e} = eV_p/M_p$) equal to 1.60 corresponding to the limit value between short and intermediate links. The structural scheme has been extracted from a real building having 18×18 m plan configuration. The characteristic values of the vertical loads are equal to 4 kN/m^2 and 3 kN/m^2 for permanent (G_k) and live (Q_k) loads, respectively. As a consequence, with reference to the seismic load combination provided by Eurocode 8 (CEN 2005a), $G_k + \psi_2 Q_k + E_d$ (where ψ_2 is the coefficient for the quasi-permanent value of the variable actions, equal to 0.3 for residential buildings), the building masses have been computed with reference to a storey vertical load equal to W=18×18×(4+0.3×3)=1587.6 kN; conversely, being the analysed scheme part of a perimeter seismic resistant scheme, the vertical loads acting directly on the beams are given by q=12+0.3×9=14.7 kN/m (considering equal to 3 m the part of the floor deck transmitting the vertical loads to the perimeter scheme).

The structural material adopted for all the members is S275 steel grade (f_{yk}=275 MPa). The design horizontal forces have been determined according to Eurocode 8, assuming a peak ground acceleration equal to 0.35 g, a seismic response factor equal to 2.5, a behaviour factor equal to 6.

Therefore, with reference to an estimated period of vibration equal to 0.085×24$^{3/4}$=0.92 s, the design value of the spectral acceleration is

$$S_{ad}(T) = \frac{0.35 \times 2.5}{6} \cdot \frac{0.40}{0.92} = 0.0633\,\mathrm{g} \tag{6}$$

being T_c=0.40 s the period of vibration corresponding to the beginning of the softening branch of the design spectrum. As a consequence, the design base shear for the whole building is

$$V_d = 8 \times 1587.6 \times 0.0633 \times 0.85 = 683.3\ \text{kN} \tag{7}$$

Being λ=0.85 the factor accounting for the number of storeys according to Eurocode 8.

In Table 1, the seismic horizontal forces for each storey are reported with reference to the single perimeter scheme (two resisting schemes for each direction are assumed).

Regarding the distribution of seismic horizontal forces, reference is made to the triangular distribution suggested in Eurocode 8 independently of the structural typology. However, it is useful to underline that the shape of such distribution is coincident with the shape of the plastic lateral displacements occurring when the global mechanism is developed. This is one of the reasons allowing the success of TPMC, i.e., the attainment of a global mechanism, even when dynamic non-linear analyses are performed.

On the basis of such force distribution, the design shear action of link members has been obtained by assuming that the storey shear is completely entrusted to the link. In the same table, the obtained link, beam and diagonal sections for the designed structure are reported. It has to be underlined that such member sections are the same both for the EB-Frame designed according to TPMC, as briefly outlined in Section 2, and for the EB-Frame designed according to Eurocode 8.

It is useful to point out that by entrusting the whole storey shear to the vertical link is equivalent to entrust the whole storey shear to the braced part of the structural scheme. This choice is justified considering the need to reduce interstorey drifts under the seismic action corresponding to the damage limitation serviceability limit state. In particular, such choice is an economically convenient strategy to fulfil serviceability requirements also (Giugliano *et al*. 2010).

Regarding the link overstrength factors delivered in Table 1, it can be observed that the ratio $\Omega_{max}/\Omega_{min}$ exceeds the value 1.25 suggested by Eurocode 8. This occurs at the top storey even if the smallest available HEB section has been adopted, therefore the Eurocode 8 suggestion has

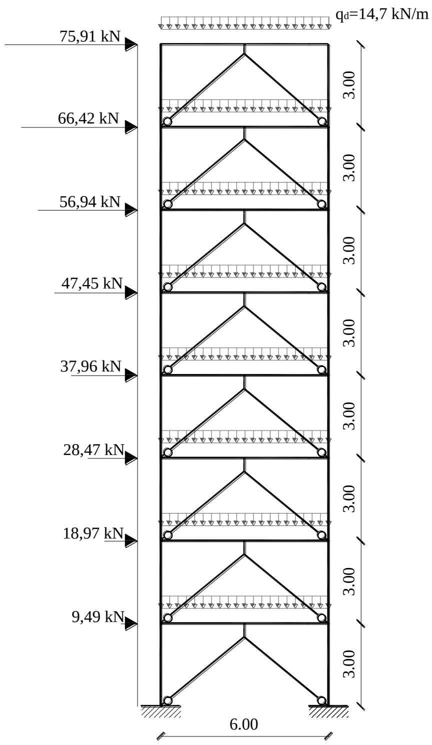

Fig. 3 Structural scheme adopted for the worked example

Table 1 Design seismic forces and link, beam and diagonal sections

STOREY i_m	F [kN]	Ω_i	LINK SECTIONS	BEAM SECTIONS	DIAGONAL SECTIONS
1	9.49	1.85	HE 200 B	IPE 270	CHS 244.5×12.5
2	18.98	1.90	HE 200 B	IPE 270	CHS 244.5×12.5
3	28.47	2.02	HE 200 B	IPE 270	CHS 244.5×12.5
4	37.96	1.87	HE 180 B	IPE 270	CHS 244.5×12.5
5	47.45	1.79	HE 160 B	IPE 270	CHS 244.5×12.5
6	56.94	2.22	HE 160 B	IPE 270	CHS 244.5×12.5
7	66.43	1.85	HE 120 B	IPE 270	CHS 244.5×12.5
8	75.92	2.61	HE 100 B	IPE 270	CHS 244.5×12.5

Table 2 Column sections

STOREY i_m	TPMC		EUROCODE 8	
	COLUMN SECTIONS	$\sum M_{c.Rd}/\sum M_{b.Rd}$	COLUMN SECTIONS	$\sum M_{c.Rd}/\sum M_{b.Rd}$
1	HEB 280	6.34	HEB 240	3.88
2	HEB 280	6.34	HEB 220	3.04
3	HEB 280	6.34	HEB 200	2.06
4	HEB 280	6.34	HEB 160	1.46
5	HEB 280	5.82	HEB 160	1.46
6	HEB 260	4.36	HEB 160	1.46
7	HEB 240	3.04	HEB 160	1.24
8	HEB 200	1.33	HEB 140	0.51

been neglected to avoid the over-sizing of all the links, below the top storey, required to increasethe Ω_{min} value and to reduce the $\Omega_{max}/\Omega_{min}$ ratio.

Regarding the column sections, they are given in Table 2. In particular, the application of the theory of plastic mechanism control leads to bigger columns at all the storeys. As already stated, the numerical details of the procedure to design the column sections are given elsewhere (Montuori *et al*. 2014a).

Regarding the application of Eurocode 8, it has also to be considered that, in the analysed design example, also the beam-to-column hierarchy criterion, usually suggested for MRFs, has been applied in column design, because the analysed scheme is essentially a MRF-EBF dual system. Notwithstanding, as it will be shown in the following, the structural system designed according to Eurocode 8 has led to poor seismic performance.

In the same Table 2, the values of the ratio between the sum of the plastic moment of columns and the plastic moment of the beam converging in the same joint are given. Under this point of view, it is useful to remember the Eurocode 8 general beam-column hierarchy criterion

$$\sum M_{c.Rd}/\sum M_{b.Rd} \geq \gamma_{Rd} \tag{8}$$

where γ_{Rd} is the required overstrength with the only exception of the top storey. The value γ_{Rd}=1.20 has been adopted in the design.

5. Push-over analyses

Push-over analyses have been carried out for the designed EB-Frame by means of SAP2000 computer program (CSI 2007) both for the structure designed by means of the procedure based on TPMC and that designed according to Eurocode 8 design rules. The aim of these analyses is to check the collapse mechanism actually developed to provide a first quick comparison between the plastic performance of the structure designed according to the theory of plastic mechanism control and the structure designed according to the codified rules. Member yielding has been taken in account by modelling the dissipative zones by means of hinge elements, i.e., with a lumped plasticity model. Column, beam, diagonal and link members have been modelled with an elastic beam-column frame element with two rigid-plastic hinge elements located at the member ends. With reference to beams, plastic hinge properties are defined in pure bending (M3 hinge) while in case of columns and diagonals plastic hinge properties are defined to account for the interaction between bending and axial force (P-M3 hinges). Both of them have a rigid plastic constitutive model for the moment rotation behaviour.

Concerning the modelling of yielding of beam elements by means of plastic hinges in pure bending, it is useful to note that the axial force in the beam does not exceed the shear force transmitted by the first storey link. This shear force attains its maximum value when the link is yielded and it is equal to 350 kN about. As a consequence, the maximum axial force in the beam does not exceed 350 kN about. According to Eurocode 3 (CEN 2005b), M-N interaction can be neglected provided that

$$n = \frac{N}{Af_y} \leq 1 - \frac{2bt_f}{A} \tag{9}$$

which is surely satisfied for an IPE 270 section with N=350 kN, A=4594 mm^2, b=135 mm, t_f=10.2 mm and f_y=275 N/mm^2. As a conclusion, M-N interaction in beam elements can be neglected.

Regarding link members, as short links yielding in shear are of concern, plastic hinges in shear have been considered, with a shear force versus shear displacement rigid-hardening constitutive model. The use of a rigid-hardening behaviour for the plastic shear hinges of link elements is justified because of the significant overstrength that link elements are able to exhibit (Bruneau *et al*. 1997, Balendra *et al*. 1991, Hjelmstad and Popov 1983, Kasai and Popov 1986). Even though many doubts have been raised concerning the amount of overstrength arising in short links due to strain-hardening (Dusicka 2004, Dusicka *et al*. 2004, Okazaki *et al*. 2004a, b, Okazaki *et al*. 2009, Itani *et al*. 1998, Mc Daniel *et al*. 2003), the overstrength factor has been assumed equal to 1.50 as suggested in code provisions. The push-over analyses have been led under displacement control taking into account both geometrical and mechanical non-linearities. In addition, out-of-plane stability checks of compressed members have been performed at each step of the non-linear analysis for both the examined structures.

The results provided by the push-over analyses are reported in Fig. 4 where both the push-over curves and the mechanism equilibrium curve corresponding to the global mechanism are depicted. In particular, the results provided by the analysis show that the hardening branch of the push-over curve corresponding to the structure designed by means of the proposed procedure, i.e., TPMC, tends towards the mechanism equilibrium curve obtained by means of second order rigid-plastic analysis. It is also useful to underline that the mechanism equilibrium curve exhibits an hardening behavior, because the occurrence of strain-hardening in shear links counterbalances the softening due to second order effects. A detailed discussion on this issue can be found in a previous work

(Montuori *et al*. 2014a).

Regarding the push-over curve of the structure designed by means of Eurocode 8 it can be observed that the structure has less stiffness and strength compared to the proposed design procedure. However, the most important difference between the two structural solutions is the collapse mechanism typology pointed out by the push-over analyses. In particular, with reference to the proposed design procedure (TPMC), Fig. 5(a) shows the distribution of plastic hinges developed when the design displacement is attained. The result confirms the accuracy of the

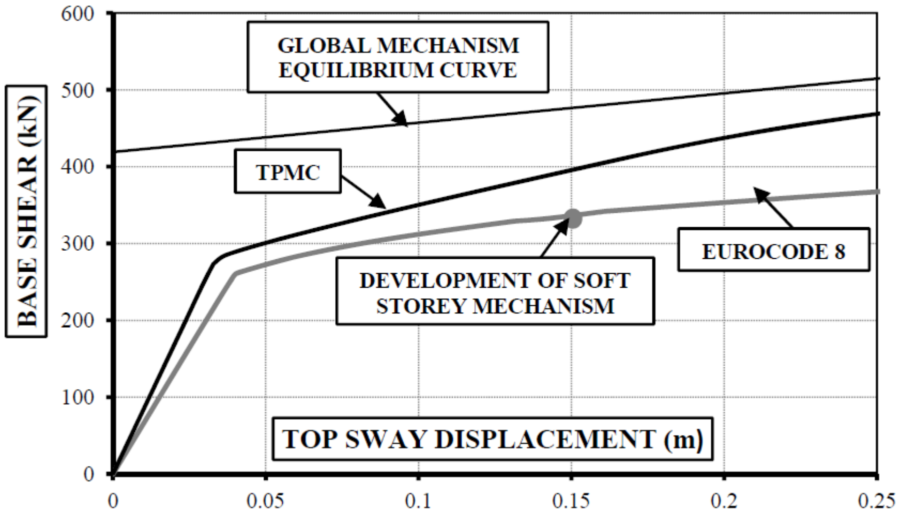

Fig. 4 Push-over curves

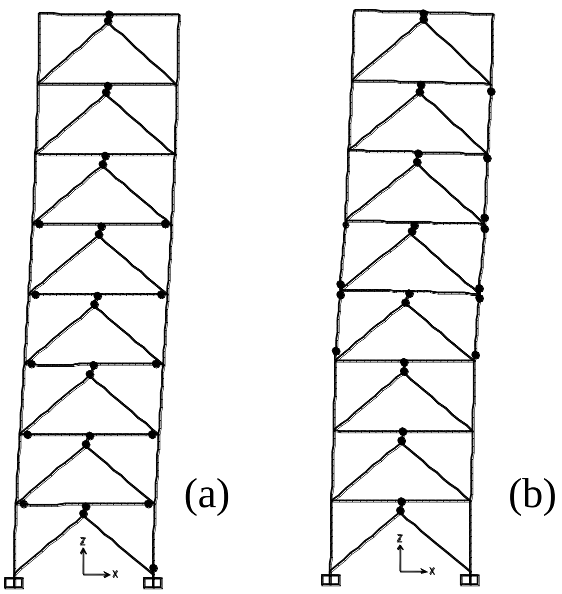

Fig. 5 Plastic hinge distribution at the ultimate design displacement: (a) structure designed according to TPMC (b) structure designed according to Eurocode 8

proposed design procedure, being the mechanism almost completely developed and being the pattern of yielding in perfect agreement with the global mechanism. Conversely, the structure designed by means of Eurocode 8 provisions exhibits a storey mechanism as depicted in Fig. 5(b). The point corresponding to the complete development of the storey mechanism is also depicted in Fig. 4. It occurs for a top sway displacement equal to 0.15 m and the corresponding maximum interstorey drift ratio (MIDR) is equal to 0.011 rad. This value has to be considered for a clear understanding of dynamic analysis results given in the following Section.

In addition, with reference to FEMA 273 (1997), the performance points corresponding to Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) limit state are provided in the same Fig. 4. These points correspond to the values of the top sway displacement leading, according to FEMA 273 provisions for braced steel frames, to a maximum interstorey drift ratio equal to 0.5%, 1.5% and 2%, respectively. It is useful to note that, with reference to the structure designed according to Eurocode 8, the top sway displacement corresponding to the development of the storey mechanism is less than the performance point corresponding to Life Safety (LS) limit state. Conversely, the ultimate behaviour of the structure designed according to TPMC is governed by the link ultimate plastic deformation whose ultimate value (assumed equal to 0.09 rad), leading to Link Fracture (LF), is attained just before the performance point corresponding to collapse prevention (CP).

6. Incremental dynamic analyses

A further validation of the proposed design methodology has been gained by means of incremental dynamic analyses (Vamvatsikos and Cornell 2002) which are aimed, on one hand, to confirm the pattern of yielding actually developed and, on the other hand, to compare the two structural solutions in terms of ductility, collapse mechanism under seismic actions and energy dissipation capacity. Therefore, both the structures designed according to Eurocode 8 and the one designed according to the theory of plastic mechanism control have been subjected to nonlinear dynamic analyses carried out using the Sap2000 computer program (CSI 2007) by means of the same structural model already adopted for push-over analyses, i.e., by means of a FEM model with lumped plasticity. In addition, 5% damping according to Rayleigh has been assumed with the proportional factors computed with reference to the first and third mode of vibration. They are reported in Table 3 for the examined structures.

Record-to-record variability has been accounted for by considering 10 recorded accelerograms selected from PEER data base. In Table 4 the analysed records (name, date, magnitude, ratio between PGA and gravity acceleration, length and step recording) have been reported. The incremental dynamic analyses have been carried out by increasing the $S_a(T_1)/g$ value until the occurrence of structural collapse, corresponding to column or diagonal buckling or up to the attainment of the limit value of the chord rotation which has been assumed equal to 0.04 rad for beam, diagonal and column members and to 0.09 rad for link members.

Table 3 First and third vibration mode period

TPMC		**EUROCODE 8**	
T_1 (s)	T_3 (s)	T_1 (s)	T_3 (s)
1.30	0.23	1.45	0.29

Table 4 Analysed ground motion records

Earhquake (record)	Component	Date	PGA/g	Length (s)	Step recording (s)
Victoria, Mexico (Chihuahua)	CHI102	1980/06/09	0.150	26.91	0.01
Coalinga (Slack Canion)	H-SCN045	1985/05/02	0.166	29.99	0.01
Kobe (Kakogawa)	KAK000	1995/01/16	0.251	40.95	0.01
Spitak, Armenia (Gukasian)	GUK000	1988/12/17	0.199	19.89	0.01
Northridge (Stone Canyon)	SCR000	1994/01/17	0.252	39.99	0.01
Imperial Valley (Agrarias)	H-AGR003	1979/10/15	0.370	28.35	0.01
Helena Montana (Carroll, College)	A-HMC180	1935/10/31	0.150	39.99	0.01
Santa Barbara (Courthouse)	SBA132	1978/08/13	0.102	12.57	0.01
Friuli, Italy (Buia)	B-BUI000	1976/09/15	0.110	26.38	0.005
Irpinia, Italy (Calitri)	A-CTR000	1980/11/23	0.132	35.79	0.0024

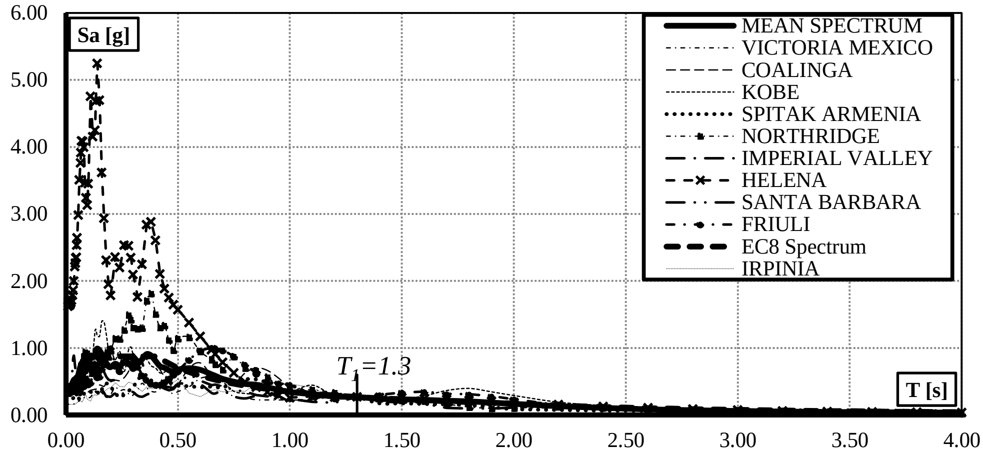

Fig. 6 Response spectra (soil type A, ζ=5%) scaled at the same value of Sa for the period of vibration of the structure designed by means of TPMC

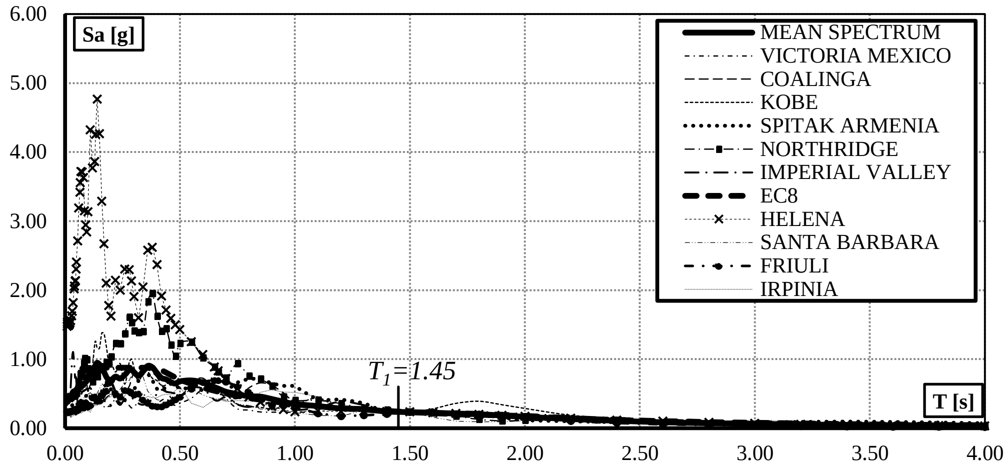

Fig. 7 Response spectra (soil type A, ζ=5%) scaled at the same value of S_a for the period of vibration of the structure designed by means of Eurocode 8

In Fig. 8 and Fig. 9 the maximum interstorey drift ratio (MIDR) (i.e., the maximum value among the peak interstorey drift values computed for all the storeys) versus spectral acceleration is reported, both for the structure designed by means of the proposed procedure and for the structure designed according to Eurocode 8 provisions, respectively. It is important to observe that, while in the case of the structure designed by means of TPMC, MIDR curves appear regular, conversely, in case of the structure designed by means of Eurocode 8 all the curves reach very high MIDR values because, due to the development of a soft storey mechanism, dynamic instability occurs. It is useful to observe that the obtained results are consistent with the behaviour pointed out in Section 5, where push-over curves have been discussed, showing that, in case of Eurocode 8, a soft storey mechanism is completely developed for a MIDR value equal to 0.011 rad. In particular, the

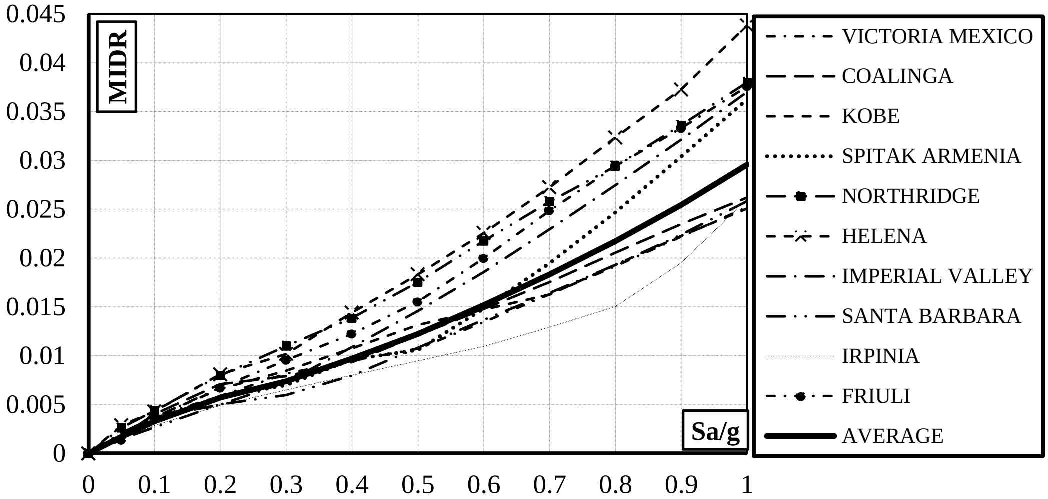

Fig. 8 MIDR versus spectral acceleration for the structure designed by TPMC

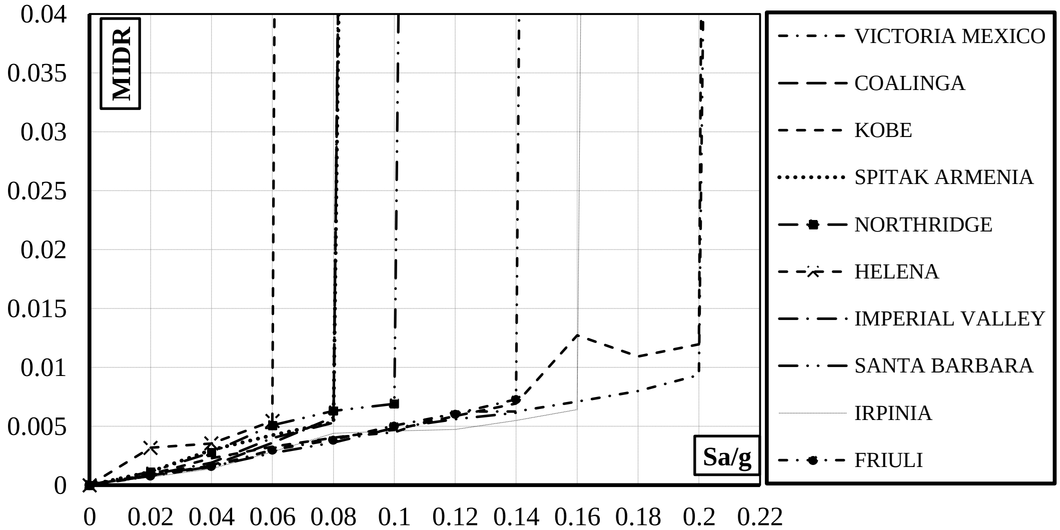

Fig. 9 MIDR versus spectral acceleration for the structure designed according to Eurocode 8

collapse mechanism actually developed under severe ground motions, depicted in Fig. 10 with reference to Coalinga record, is in agreement with the one pointed out by means of the push-over analysis (Fig. 5(b)). As one of the primary goals of the analyses was to gain information on the members involved in the pattern of yielding actually developed, it is important also to underline that the pattern of yielding of the EBF designed according to TPMC is in perfect agreement with the one (Fig. 5(a)) already obtained by means of push-over analysis.

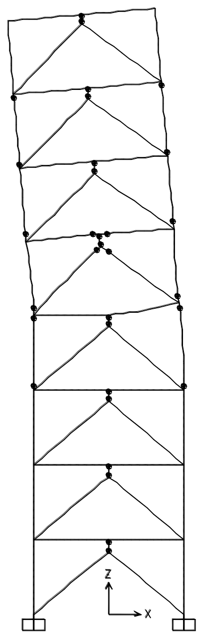

Fig. 10 Collapse mechanism of the structure designed according to Eurocode 8 provisions subjected to Coalinga earthquake record scaled to S_a/g=0.14

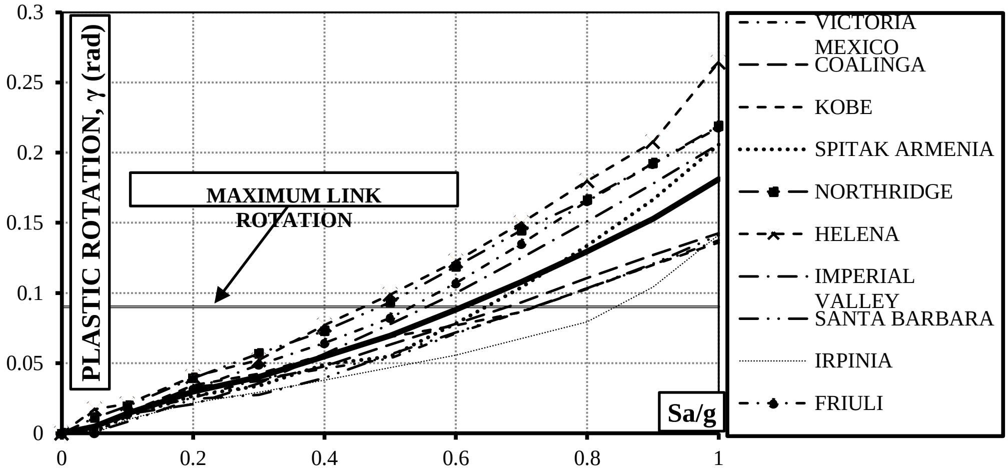

Fig. 11 Maximum plastic rotation of link versus spectral acceleration for the structure designed according to TPMC

In Fig. 11 the maximum plastic rotation of links, for the structure designed by means of TPMC is reported. The $S_a(T_1)/g$ value corresponding to the achievement of the ultimate link plastic rotation, equal to 0.09 rad, can be easily indentified. In particular, the average value of $S_a(T_1)/g$ corresponding to such ultimate limit state is equal to 0.62. Conversely, the IDA curves relating MIDR to the spectral acceleration for the structure designed according to Eurocode 8 (Fig. 9) are characterized by a sudden increase corresponding to the development of a storey mechanism which leads to dynamic instability. In particular, it can be observed that dynamic instability governs the seismic performance of the structure, occurring for low values of the spectral acceleration varying from 0.065 g to 0.20 g with an average value of about 0.14 g.

Downstream of these IDA curves it is possible to observe that, in the case of the structure designed by means of TPMC, the seismic performance is governed by the achievement of the

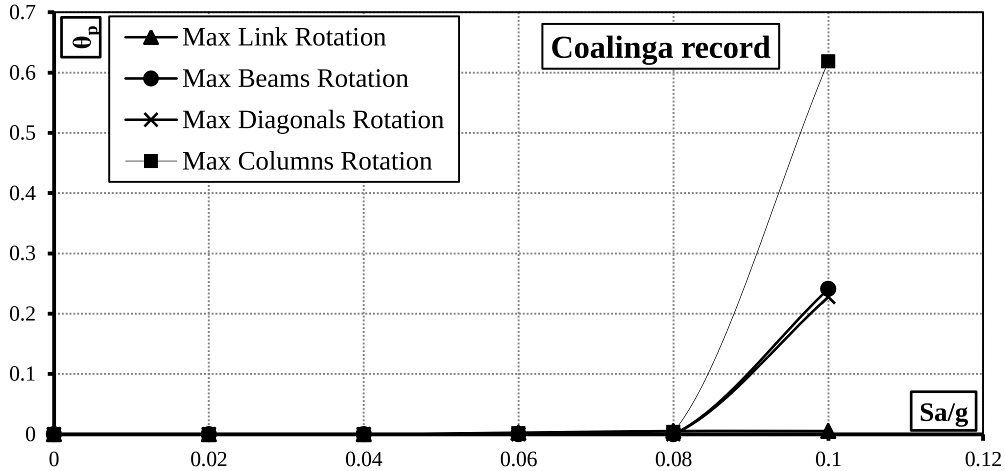

Fig. 12 IDA curves of link, beam, diagonal and column plastic rotations for Coalinga record

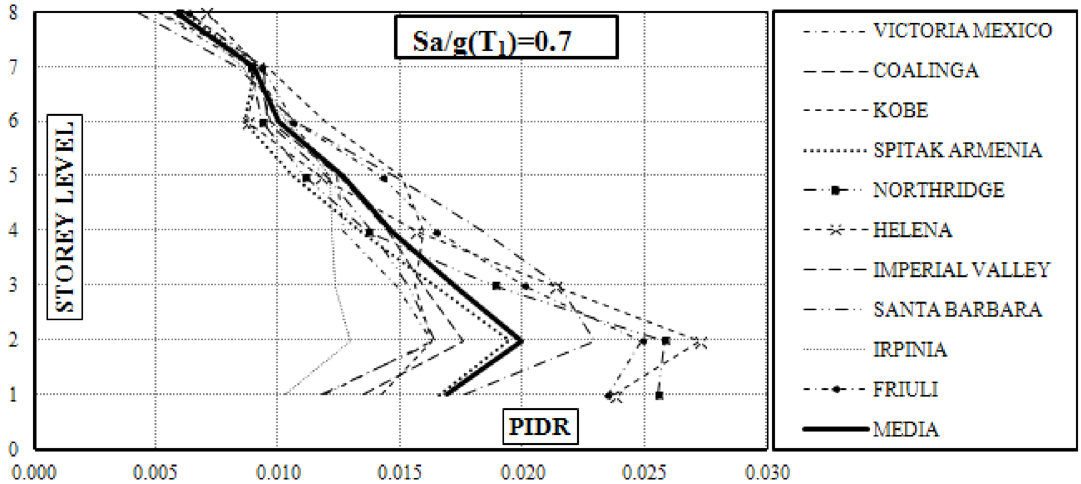

Fig. 13 Storey level versus peak interstorey drift (PIDR) for $S_a(T_1)$/g=0.7

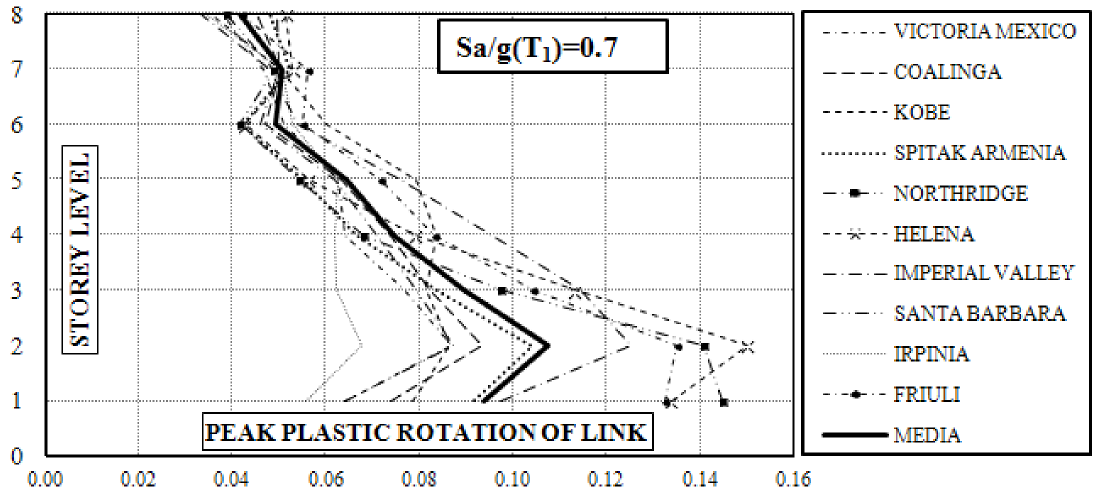

Fig. 14 Storey level versus peak plastic rotation of link for $S_a(T_1)/g$=0.7

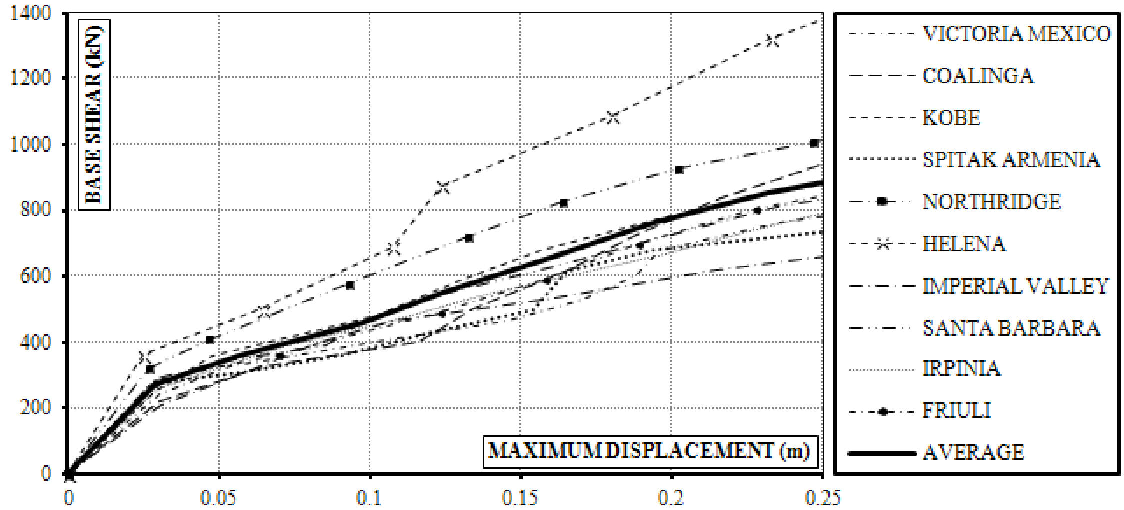

Fig. 15 Base shear versus top maximum displacement for the structure designed according to TPMC

maximum plastic rotation of links. In fact, plastic rotation of the hinges developed at the beam ends and at the base of first storey columns are below the assumed ultimate value (0.04 rad) when the links attain their ultimate conditions. In addition, buckling resistance of both columns and diagonal braces is assured at the achievement of link ultimate plastic rotation. Conversely, as already stated, the seismic performance of the EBF designed according to Eurocode 8 is governed by dynamic instability occurring, on average, for a spectral acceleration value equal to 0.14 g. This is due to the development of the storey mechanism depicted in Fig. 10 with damage concentration at the 5th storey. This damage concentration is well understood by means of Fig. 12 providing the values of the plastic rotation occurring in links, beams, diagonals and columns. Because of the development of a storey mechanism involving also diagonal braces and beam sections close to the link-to-beam connection, plastic rotations are mainly developed in columns which govern the occurrence of collapse, and in diagonal and beam sections close to the link-to-beam connection. Conversely, despite of their yielding, there is no any significant participation of link elements to the energy dissipation capacity of the structure, so that it can be stated that Eurocode 8 design procedures fail in reaching their design goal. In Fig. 13 and Fig. 14 the peak interstorey drift ratio (PIDR) and the peak plastic rotation of links (γ) versus storey level are depicted for the structure designed by means of TPMC. These figures are referred to a spectral acceleration value very close to the one leading, on average, to the attainment of the link ultimate plastic rotation supply in the most engaged storey. It is possible to observe that both the average value of PIDR and peak plastic rotation of links have reached their maximum value at the second storey.

Finally, it is useful to observe that the structure designed according to TPMC exhibits a seismic performance significantly better than the one of the EBF designed according to Eurocode 8, being the ratio in terms of spectral acceleration leading to collapse equal to 4.43 (i.e., 0.62 g/0.14 g).

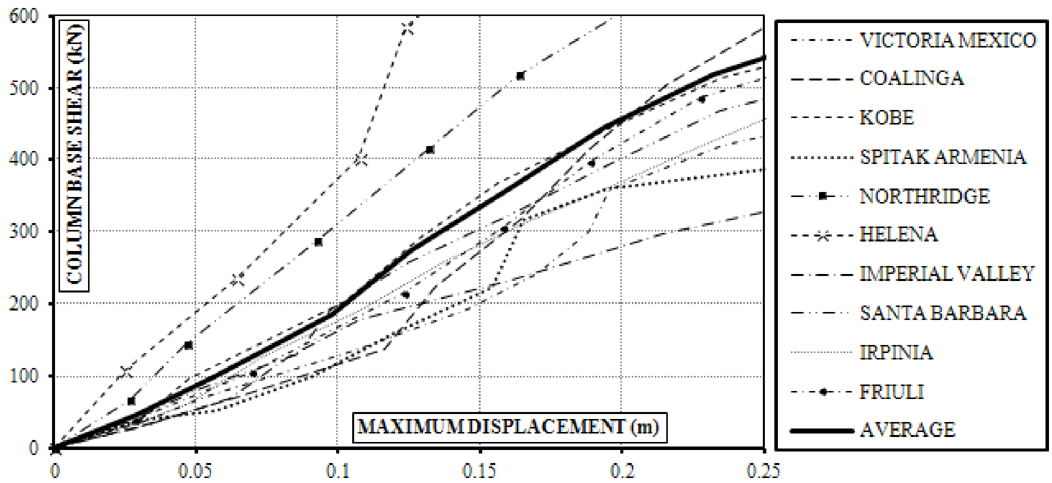

Fig. 16 Column base shear versus top maximum displacement for the structure designed according to TPMC

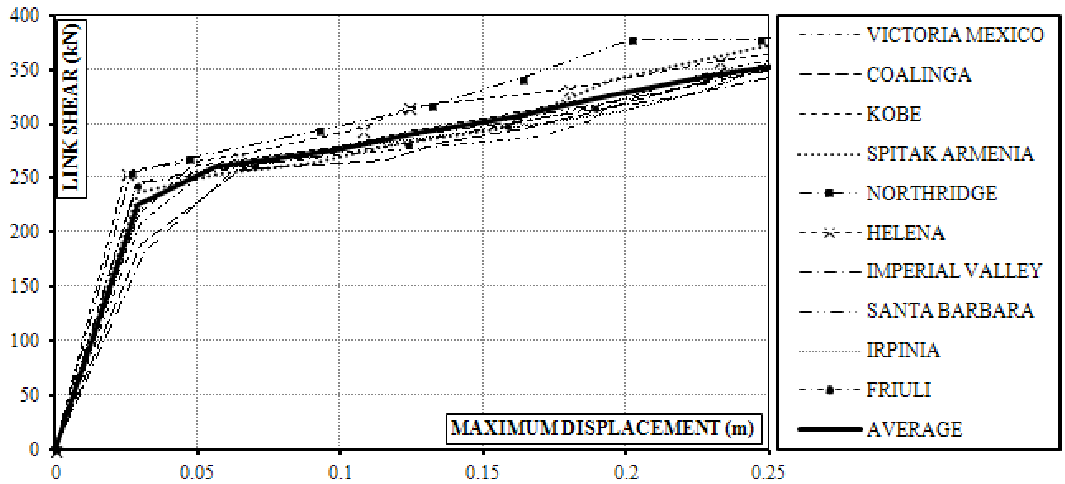

Fig. 17 First storey link shear versus top maximum displacement for the structure according to TPMC

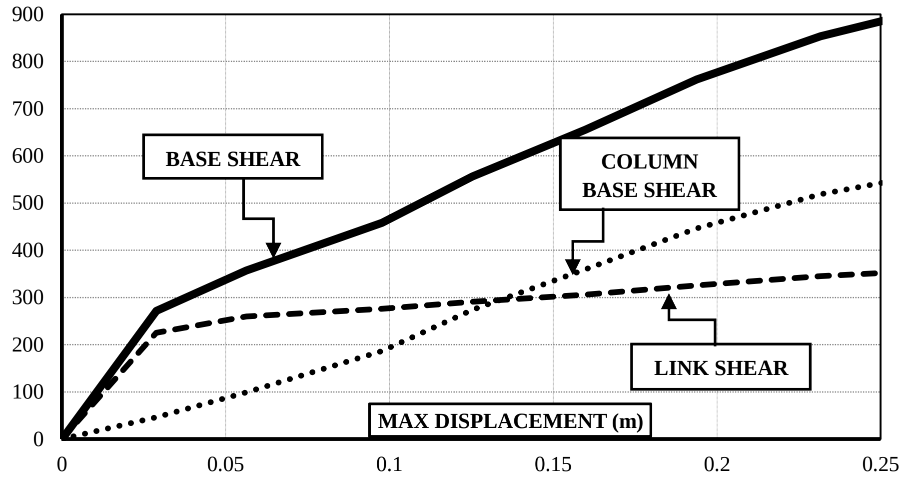

Fig. 18 Dynamic push-over curves for the structure designed according to TPMC

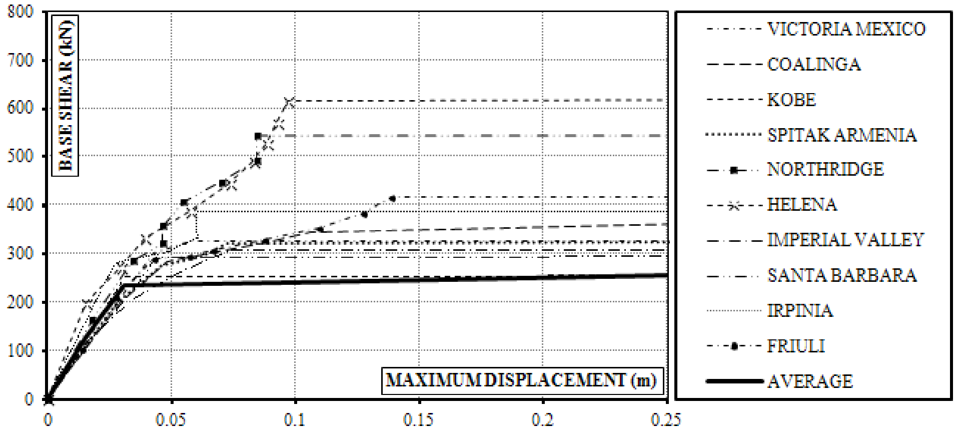

Fig. 19 Base shear versus top maximum displacement for the structure designed according to Eurocode 8

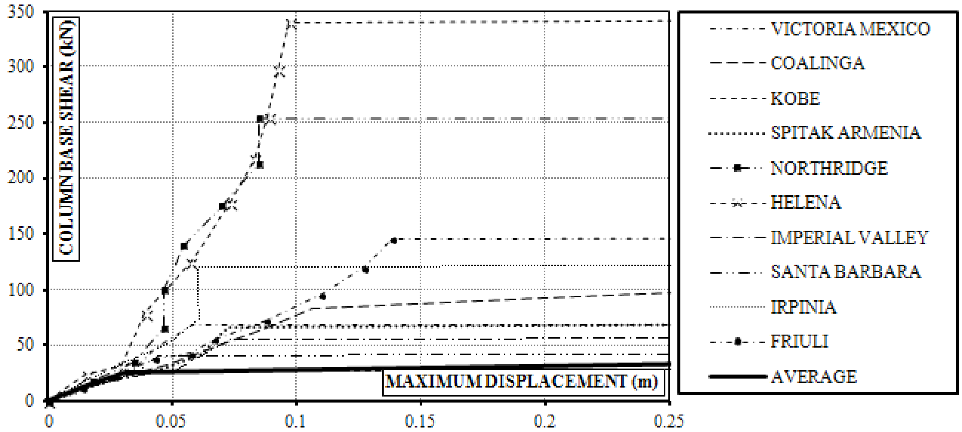

Fig. 20 Column base shear versus top maximum displacement for the structure designed according to Eurocode 8

It is useful to underline that link, beam and diagonal sections are the same for the two designed structure, so that the different seismic performances are due to the increase of column sections which constitute the weak point of Eurocode 8 design provisions. Therefore, under this point of view, it is useful to investigate the distribution of the base shear between the moment resisting part and the bracing part of the structural system.

In Figs. 15 to 17, the total base shear, the column base shear and first storey link shear versus maximum top displacement are depicted for the structure designed according to TPMC. This representation constitutes the results of the so-called dynamic push-over. The average curves

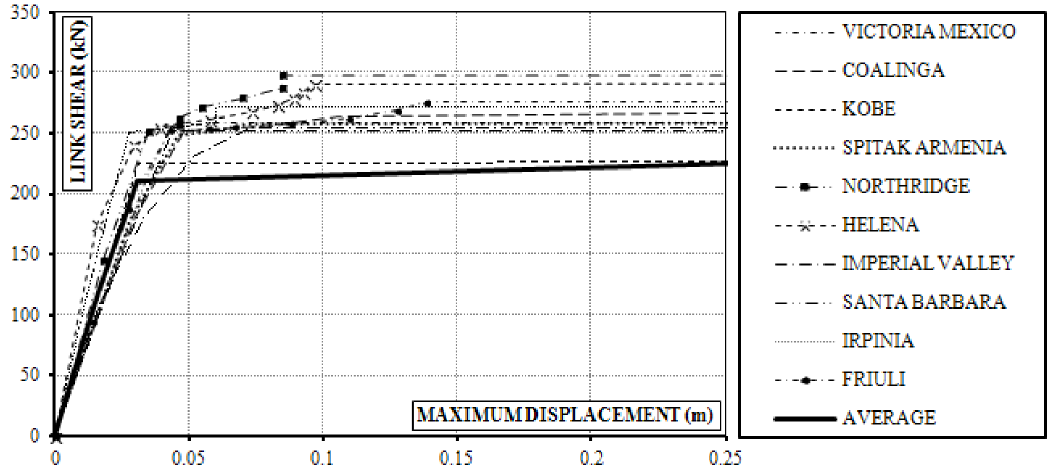

Fig. 21 Link shear versus top maximum displacement for the structure designed according to Eurocode 8

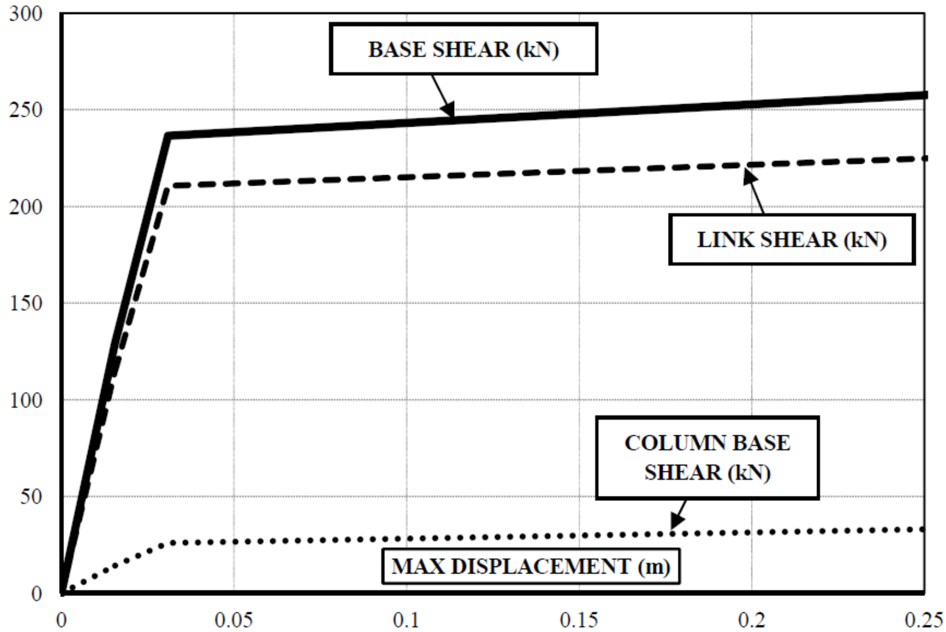

Fig. 22 Dynamic push-over curve for the structure designed according to Eurocode 8

depicted in Fig. 18 show that, for low displacement values, almost all the base shear action is allocated in the first storey link, i.e., the bracing system, while, for increasing displacement values the amount of the base shear withstood by the moment resisting part progressively increases becoming more and more significant. Finally, for a top sway displacement exceeding about 0.13 m, on average, the base shear due to the moment-resisting part exceeds the one due to the bracing part. Therefore, it can be concluded that the EB-Frame with inverted *Y*-scheme designed according to TPMC works like a dual system whose performance is distributed between the bracing system and the moment resisting system which represents a secondary seismic-resistant system whose contribution to the seismic performances of the structure is more and more important as the seismic intensity measure increases.

In Figs. 19 to 22, the same representations are depicted for the structure designed according to Eurocode 8 provisions. In particular, Fig. 22 shows that the behaviour of the structure designed according to Eurocode 8 is completely different from the one designed according to TPMC, because the shear action is allocated almost only in the link, i.e., the bracing system, leaving the moment resisting system, made by columns and beams, almost unloaded.

7. Considerations on economic issues

Aiming to provide a more exhaustive comparison between the two considered design procedures, it is useful to face the topic also from the economic point of view. First of all, it can be assumed that the cost of the Whole Structure is proportional to its weight

$$C_{WS.EC8} = \beta W_{WS.EC8} \tag{10}$$

$$C_{WS.TPMC} = \beta W_{WS.EC8} \tag{11}$$

where the second index denote either the structure designed according to Eurocode 8 or that designed according to TPMC.

It follows that the cost of the whole structure designed according to TPMC can be expressed as

$$C_{WS.TPMC} = C_{WS.EC8} \frac{W_{WS.TPMC}}{W_{WS.EC8}} \tag{12}$$

According to the common design experience, it is important to observe that the cost of the whole structure represents a typical percentage of the Whole Building (WB) cost depending on its destination of use

$$C_{WS.TPMC} = \alpha C_{WB.EC8} \tag{13}$$

so that the cost of Non Structural Components (NSC) is the difference between the cost of the whole building and the one of the whole structure

$$C_{NSC} = (1-\alpha) C_{WB.EC8} \tag{14}$$

Such cost is independent of the design criteria and it can be applied either with reference to Eurocode 8 or to TPMC. As a consequence, the cost of the whole building designed according to TPMC can be related to the one of the building designed according to Eurocode 8 in the following way

$$\frac{C_{WB.TPMC}}{C_{WB.EC8}} = 1 + \alpha\left(\frac{W_{WS.TPMC}}{W_{WS.EC8}} - 1\right) \tag{15}$$

In the examined study case, the weight of the Inverted Y-Scheme Frame (IYSF) whose geometry is depicted in Fig. 3, is equal to 108.35 kN and 88.61 kN for TPMC and EC8 design, respectively. Therefore, by assuming that two IYSF are placed in both the *x* and *y* directions, it results

$$\frac{W_{IYSF.TPMC}}{W_{IYSE.EC8}} = \frac{4 \times 108.35}{4 \times 88.67} = 1.223 \tag{16}$$

It means that TPMC design procedure causes an increase of 22.3% of structural weight with reference to the seismic resistant part only (i.e., IYSFs). However these structural resistant frames are only a part of the whole structure, so that the above cited percentage obviously reduces by considering also the structural weight due to Gravity Load resisting part of the structure (GL). For the examined building (18×18 m in plan) W_{GL} is about 800 kN so that

$$\frac{W_{WS.TPMC}}{W_{WS.EC8}} = \frac{4 \times 108.35 + 800}{4 \times 88.61 + 800} = 1.068 \tag{17}$$

Therefore, the use of TPMC causes an increase of 6,8% of whole structural weight.

However, the most important contribution to the cost of the whole building is the one due non structural components so that Eq. (15) provides

$$\frac{C_{WB.TPMC}}{C_{WB.EC8}} = 1 + 0.3(1.068 - 1) = 1.020 \tag{18}$$

where the coefficient α is assumed equal to 0.3 as a typical value occurring for residential buildings in Italy.

In addition, Eq. (18) points out that, although the use of TPMC leads, in this case, to an increase of 22.3% as of the structural weight of IYSFs, only a small increase (2.0%) of the whole building cost is obtained. Conversely, regarding the seismic performances, the spectral acceleration corresponding to the structural collapse is, for TPMC, on average, about four times higher than the value occurring for the structure designed according to Eurocode 8. As a consequence, it can be concluded that there is a significant advantage in using TPMC, because of the significant reduction of the seismic vulnerability of the structure.

8. Conclusions

The same structural system, i.e., an 8 storey EBF with inverted Y-scheme, has been designed according to two different procedures. The first design procedure is based on the theory of plastic mechanism control. The second one corresponds to the application of Eurocode 8 provisions. Both push-over and dynamic non-linear analyses have pointed out the different seismic performances which can be obtained by means of the investigated design procedures. In particular, the results of both push-over and IDA analyses have pointed out the accuracy of the proposed design methodology, based on TPMC, whose robustness is based on the kinematic theorem of plastic collapse and its extension to the concept of mechanism equilibrium curve. As testified by the obtained pattern of yielding, it allows the control of the failure mode assuring a collapse mechanism of global type. However, as demonstrated in previous works, TPMC can be applied to the most common typologies of seismic resistant structural systems provided that their features are properly accounted for. The application of TPMC has led to the fulfilment of the design goal, i.e., the involvement of all the dissipative zones (links) reaching high values of the spectral acceleration leading to collapse. In particular, the examined EBF with inverted Y-scheme designed by TPMC is able to withstand spectral acceleration values equal to 0.62 g, on average. This performance is due to the control of the failure mode which assures a dual system behaviour where the contribution of the moment-resisting part in the sharing of the seismic base shear increases as far as the seismic intensity measure increases. Conversely, despite the application of hierarchy criteria, the structure designed according to Eurocode 8 does not satisfy the code promises, because the structure does not exhibit a pattern of yielding consistent with the required energy dissipation capacity which the q-factor is based on. In fact, as pointed out both by push-over and IDA analyses, the structure exhibits a storey mechanism which undermines the seismic response as testified by the quite low values of the spectral acceleration leading to dynamic instability, equal to 0.14 g on average.

The comparison between the use of TPMC and Eurocode provisions shows that TPMC gives rise to ultimate values of the spectral acceleration equal, on average, to 4.43 times those of code provisions (0.62 g versus 0.14 g). In addition, despite of the proposed design method leads to a 22.3% increase in the structural weight of the seismic load resisting system, considering that IYSFs are only a part of the whole building structure and accounting for the even more important contribution of non structural components, it can be concluded that the increase of the building cost is about 2.0% only while significantly improved seismic performances in terms of ultimate spectral acceleration are obtained.

Even though the preliminary performance assessment of the designed building is based on IDA analyses limited to only ten records, the obtained results are very encouraging about the performance improvements which can be attained by applying TPMC. However, it has be recognized that seismic response of structures is highly affected by the frequency content of the ground motion, so that, record-to-record variability has to be more accurately considered. Therefore, the future development of the work will require the application of a probabilistic approach aiming to evaluate the seismic reliability of such design criteria in terms of mean annual frequency of exceeding specified limit states and in terms of seismic loss hazard.

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