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SEISMIC PERFORMANCES OF MRF-EBF DUAL SYSTEMS: INFLUENCE OF BRACE GEOMETRY

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Abstract

The work herein presented is devoted to the evaluation of the influence of the brace geometry on the seismic performances of Moment Resisting Frames-Eccentrically Braced Frames (MRF-EBF) dual systems designed by means of the Theory of Plastic Mechanism Control (TPMC). Even though TPMC design approach is still not introduced in modern seismic code, it has earned the reputation of being a very robust method, based on kinematic theorem of plastic collapse, able to assure collapse mechanism of global type. Conversely, the design approach proposed by Eurocode 8 (EC8), following the rules given by the other modern seismic codes, promotes the application of the so-called beam-column hierarchy criterion which is able to avoid soft-storey mechanisms, but is not able to assure the yielding of all the dissipative zones, because a collapse mechanism of global type is usually not attained. The main purpose of the present work is to compare, given the design approach, the different performances affecting structures with four different brace geometry of the eccentrically braced part of the seismic resistant scheme. For this reason, 5 bays structures with 4, 6 and 8 storeys have been considered for the four link configurations for a total number of 12 structural schemes. The seismic performances have been evaluated by means of both push-over and Incremental Dynamic Analyses (IDA) carried out until the achievement of the structural collapse.

Keywords: Theory of Plastic Mechanism Control, MRF-EBF dual systems, Seismic Design, Steel, IDA analyses

1. Introduction

Eccentrically braced frames constitute an efficient seismic resistant scheme, because they combine the lateral stiffness of braced frames with the energy dissipation capacity due to the links. As a result, such structural system is able to satisfy both the serviceability requirements, reducing the interstorey drifts occurring under frequent or moderate earthquakes, and the ultimate limit state requirements, providing the energy dissipation capacity needed to prevent collapse under rare destructive seismic events. Even though such structural typology is codified since many years, modern seismic codes [1] do not provide any information regarding the influence of the brace geometry on the seismic performances, thus proposing the use of an unique value of the q-factor (behavior factor) independently of the brace geometry. Regarding this last issue, it is well known that eccentrically braced frames can be configured with horizontal links or with vertical links. In the first case, i.e. horizontal links, different schemes can be obtained: K-scheme, D-scheme and V-scheme (Fig. 1 a), b) and c)). In the second case, an inverted Y-scheme is obtained (Fig. 1 d)).

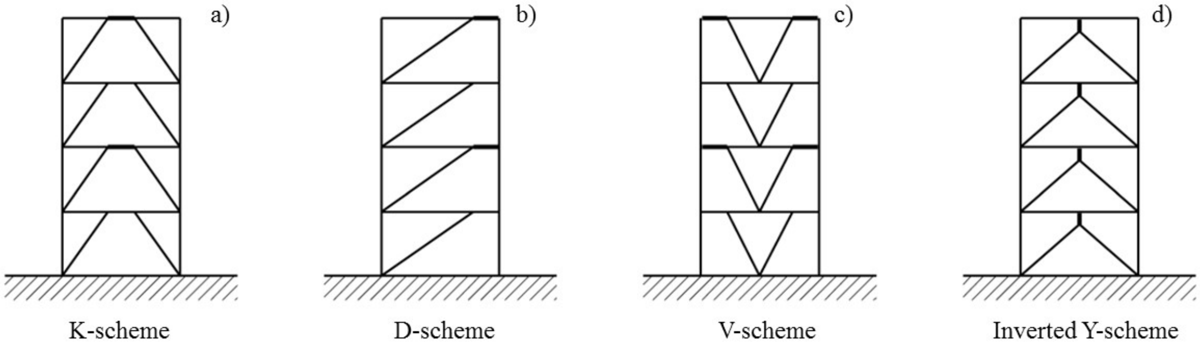

Fig. 1 – Brace geometry of EBFs in seismic codes

Despite the geometry clearly affects the relationship between the interstorey drift ratio and the link plastic deformation demand, code provisions do not predict any difference in their seismic behavior. Therefore, the research work herein presented is devoted to the evaluation of the influence of the brace geometry on the seismic performances of Moment Resisting Frames-Eccentrically Braced Frames dual systems (MRF-EBF dual systems) designed by means of the same approach. In particular, the Theory of Plastic Mechanism Control (TPMC) approach has been selected, because it assures the design of structures able to involve at collapse the largest number of dissipative zones. In fact, it guarantees, the development of a collapse mechanism of global type where all the link members, beams ends and first storey column bases are involved in plastic range.

TPMC provides a design procedure with a strong theoretical background. In fact, it is based on the extension of the kinematic theorem of plastic collapse to the concept of mechanism equilibrium curve [2]. From a historical point of view, TPMC has gained relevance in the last decades, because it has been recognized as the only procedure which assures the development of a collapse mechanism of global type. Starting from the first application to MRFs [2]-[3] the procedure has been extended to all the seismic resistant typologies [4]-[9] and also to MRF-EBFs dual systems constituting the main topic of the present paper [10]. In this case, being the structures designed to assure the same collapse mechanism typology, it is possible to investigate how the brace geometry affects their seismic performances. To this scope, different schemes have been designed starting from the same plan configuration, by changing the number of storeys that have been selected equal to 4, 6 and 8 and, obviously, the brace geometry. The seismic response of the resulting structures has been analyzed by means of both push-over and IDA analyses.

2 Design approach

TPMC is based on a rigorous theoretical approach assuring a collapse mechanism of global type [3], which exploits the kinematic theorem of plastic collapse extended to the concept of mechanism equilibrium curve:

$$\alpha = \alpha_0 - \gamma\delta \tag{1}$$

Where, following the theory of rigid-plastic analysis, α_0 is the first order collapse multiplier of horizontal forces, γ is the slope of the linearized mechanism equilibrium curve due to second order effects and δ is the plastic top sway displacement. TPMC states that the mechanism equilibrium curve corresponding to the global mechanism has to be located below those corresponding to all the undesired mechanisms until a design displacement δ_u compatible with the local ductility supply (Fig. 2). The column sections at each storey needed to assure a collapse mechanism of global type are the unknowns of the design problem, while link, beam and diagonal members are preliminarily designed according to the first principle of capacity design. More details about this design procedure devoted to EBFs are reported in previous works, [10], [11].

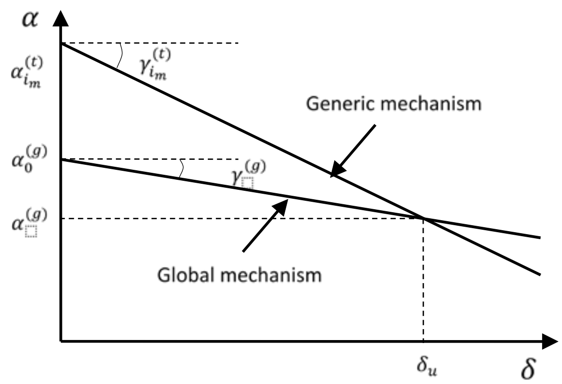

Fig. 2 –TPMC statement

2. Analyzed schemes

Different schemes have been designed starting from the same plan configuration reported in Fig. 2, by changing the number of storeys that have been selected equal to 4, 6 and 8. In addition, the four brace geometry proposed by codes have been considered leading to a total number of 12 analyzed schemes. For sake of shortness, the seismic response of the buildings is herein analyzed with reference to seismic actions in the longitudinal direction only. The corresponding seismic resistant schemes are depicted in Fig. 3, for the K-scheme, D-scheme, V-scheme and inverted Y-scheme and only for the 6-storey structures. In addition in such figures also the leaning column adopted in structural modelling to account for second order effects due to the internal gravity load resisting system is reported. In fact, gravity loads acting on the leaning part of the structure significantly contribute to the structural seismic masses and to second order effects.

It is also important to observe that only short links have been considered for the design of structures because they present many advantages with respect to long and intermediate links. In fact, the main parameter governing the seismic response of such structural typology, both in elastic and post-elastic range, is the length *e* of the links, constituting the dissipative zones. This parameter influences the lateral stiffness of the structure, the ability to dissipate the seismic input energy and the link plastic rotation capacity. In particular, the lateral stiffness of the bracing system increases as far as the link length decreases [13]. Due to their performance in terms of both stiffness and ductility, short links are in several cases the most suitable choice for seismic-resistant EBFs. In fact, the cyclic behaviour of short links is characterised by wide and stable cycles allowing the development of high energy dissipation capacity, provided that adequate web stiffeners are adopted along the element length to prevent web local buckling. In such a case, high rotation capacity has been experimentally exhibited by short links when compared to intermediate and long ones. Conversely, long links are characterised by a completely different failure mechanism where large flexural deformations lead to the fracture of tensile flanges or to the buckling of compressed ones. As a result, maximum plastic rotations up to ±0.02 rad only have been experimentally measured. Finally, intermediate links are characterised by a behaviour in-between those of short and long links.

The characteristic values of the vertical loads are equal to 4.0 kN/m^2 and 2.0 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 [1], $G_k + \psi_2 Q_k + E_d$ (where ψ_2 is the coefficient for the quasi-permanent value of the variable actions, equal to 0.3 for residential buildings), the vertical loads acting on the floor are equal to 4.6 kN/m^2. The structural material adopted for all the members is S355 steel grade ($f_{yk} = 355\, MPa$). The beams of the moment-resisting part have been designed to withstand vertical loads accounting also for serviceability requirements (Fig. 3). The design horizontal forces have been determined according to Eurocode 8, assuming a peak ground acceleration equal to 0.35g, a seismic response factor equal to 2.5, a behaviour factor equal to 6 [1]. 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. The link length has been selected in order to assure that the design ultimate displacement, δ_u, according to TPMC design procedure, is the same for all the examined brace configurations.

In particular, the link length has been selected as follows:

$$\delta_u = \gamma_u\left(e/L_j\right)h_{n_s} = 0.08(1.2/6)h_{n_s} = 0.016h_{n_s} \tag{2}$$

for K-scheme and D-scheme,

$$\delta_u = \gamma_u\left(2e/L_j\right)h_{n_s} = 0.08(2\cdot 0.6/6)h_{n_s} = 0.016h_{n_s} \tag{3}$$

for V-scheme,

$$\delta_u = \gamma_u(e/h_i)h_{n_s} = 0.08(0.7/3.5)h_{n_s} = 0.016h_{n_s} \tag{4}$$

for inverted Y-scheme, where γ_u is the target link plastic rotation, e is the link length, L_j is the braced bay length, h_i is the interstorey height and $h_{n_s}$is the building height. In particular, the short link length adopted for the analyzed schemes herein presented is equal to 1.20 m for the K-scheme and D-scheme, 0.60 m for V-scheme buildings and 0.70 m for the inverted Y-scheme, at each storey. In Table 1 the link and diagonal sections for the designed structures are reported. In addition, in the same table, the values of the overstrength factor, Ω_i, of link elements computed according to Eurocode 8 are reported. It can be observed that the ratio between the maximum Ω_i value and the minimum one is not always less than the limit value suggested by Eurocode 8 (equal to 1.25) to promote the yielding of all the link elements. Notwithstanding, the results of push-over analyses, presented in the following, have pointed out that all the links are subjected to yielding. Moreover, the column sections obtained by applying TPMC are delivered in Table 2.

2. Push-over analyses

With reference to the analyzed seismic resistant systems, push-over analyses have been carried out by means of SAP2000 computer program for all the four structural schemes. The aim of these analyses is to check the collapse mechanism actually developed in order to validate the accuracy of the TPMC application. Member yielding has been accounted for 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. In addition, an axial hinge has been located in the mid-span of the each column with the scope to check the out of plane buckling. Regarding link members, as short links yielding in shear are of concern, plastic hinges in shear have been considered. The shear force versus shear displacement rigid-hardening constitutive model has been selected assuming a rigid-hardening behavior with an overstrength factor equal to 1.50. 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 all the examined structures.

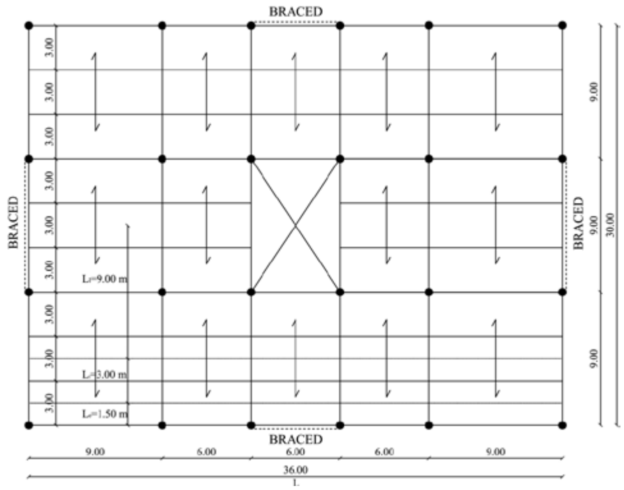

Fig. 3 – Plan configuration of the analysed strucutres

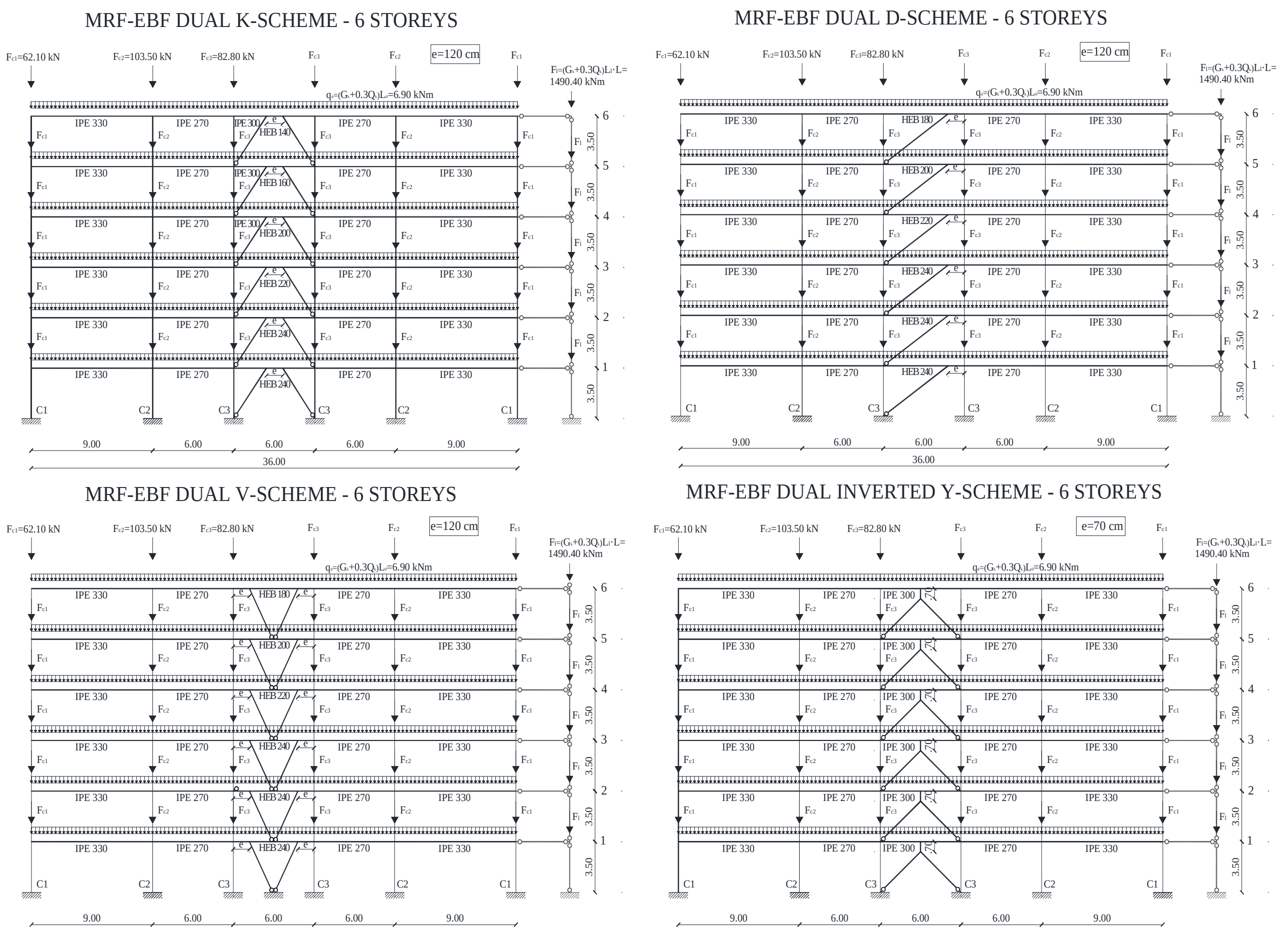

Fig. 4 – Structural schemes of the longitudinal seismic resistant system

The results provided by the pushover analyses are reported in Fig. 5, Fig. 4 and Fig. 5 for the 4-storey, 6-storey and 8-storey buildings, respectively. The results show that the softening branch of the push-over curve of structures designed by means of TPMC tends towards the mechanism equilibrium curve obtained by means of second order rigid-plastic analysis. It is also useful to underline that, in the examined cases, push-over curves exhibit a softening behaviour, because the occurrence of strain-hardening in shear links does not counterbalance the softening due to second order effects.

Table 1 – Design seismic forces, link and diagonal sections for the designed structures

			K-SCHEME			D-SCHEME			V-SCHEME			INVERTED Y-SCHEME		
4-STOREY	STOREY i_m	F [kN]	LINKS	Ω_i	DIAGONAL SECTIONS	LINKS	Ω_i	DIAGONAL SECTIONS	LINKS	Ω_i	DIAGONAL SECTIONS	LINKS	Ω_i	DIAGONAL SECTIONS
	1	96.097	HEB 240	1.72	CHS 323.9x20	HEB 240	1.71	CHS 323.9x20	HEB 160	1.89	CHS 323.9x20	HEB 300	1.14	CHS 244.5x20
	2	192.195	HEB 200	1.73	CHS 323.9x20	HEB 200	1.85	CHS 323.9x20	HEB 140	1.94	CHS 323.9x20	HEB 300	1.15	CHS 244.5x20
	3	288.293	HEB 180	1.81	CHS 323.9x20	HEB 180	1.96	CHS 323.9x20	HEB 140	2.02	CHS 323.9x20	HEB 240	1.21	CHS 244.5x20
	4	384.391	HEB 140	1.88	CHS 323.9x20	HEB 140	2.20	CHS 323.9x20	HEB 140	2.08	CHS 323.9x20	HEB 180	1.19	CHS 244.5x20
6-STOREY	STOREY i_m	F [kN]	LINKS	Ω_i	DIAGONAL SECTIONS	LINKS	Ω_i	DIAGONAL SECTIONS	LINKS	Ω_i	DIAGONAL SECTIONS	LINKS	Ω_i	DIAGONAL SECTIONS
	1	50.643	HEB 240	1.53	CHS 355.6x16	HEB 240	1.34	CHS 355.6x16	HEB 180	1.43	CHS 355.6x16	HEB 200	1.20	CHS 244.5x20
	2	101.285	HEB 240	1.24	CHS 355.6x16	HEB 240	1.28	CHS 355.6x16	HEB 180	1.25	CHS 355.6x16	HEB 200	1.16	CHS 244.5x20
	3	151.928	HEB 220	1.24	CHS 355.6x16	HEB 220	1.29	CHS 355.6x16	HEB 160	1.27	CHS 355.6x16	HEB 200	1.19	CHS 244.5x20
	4	202.571	HEB 200	1.29	CHS 355.6x16	HEB 200	1.39	CHS 355.6x16	HEB 140	1.42	CHS 355.6x16	HEB 180	1.16	CHS 244.5x20
	5	253.214	HEB 160	1.38	CHS 355.6x16	HEB 160	1.46	CHS 355.6x16	HEB 140	1.37	CHS 355.6x16	HEB 160	1.29	CHS 244.5x20
	6	303.856	HEB140	2.01	CHS 355.6x16	HEB140	2.68	CHS 355.6x16	HEB 140	2.05	CHS 355.6x16	HEB 160	1.44	CHS 244.5x20
8-STOREY	STOREY i_m	F [kN]	LINKS	Ω_i	DIAGONAL SECTIONS	LINKS	Ω_i	DIAGONAL SECTIONS	LINKS	Ω_i	DIAGONAL SECTIONS	LINKS	Ω_i	DIAGONAL SECTIONS
	1	31.745	HEB240	1.74	CHS 406.4x32	HEB 240	1.36	CHS 406.4x32	HEB 220	1.25	CHS 406.4x12.5	HEB 340	1.26	CHS 406.4x32
	2	63.489	HEB 240	1.21	CHS 406.4x32	HEB 240	1.15	CHS 406.4x32	HEB 220	1.01	CHS 406.4x12.5	HEB 340	1.22	CHS 406.4x32
	3	95.234	HEB 240	1.22	CHS 406.4x32	HEB 240	1.24	CHS 406.4x32	HEB 220	1.08	CHS 406.4x12.5	HEB 320	1.17	CHS 406.4x32
	4	126.978	HEB 220	1.20	CHS 406.4x32	HEB 220	1.22	CHS 406.4x32	HEB 200	1.05	CHS 406.4x12.5	HEB 300	1.19	CHS 406.4x32
	5	158.723	HEB 200	1.23	CHS 406.4x32	HEB 200	1.24	CHS 406.4x32	HEB 180	1.07	CHS 406.4x12.5	HEB 280	1.28	CHS 406.4x32
	6	190.467	HEB 180	1.34	CHS 406.4x32	HEB 180	1.37	CHS 406.4x32	HEB 180	1.37	CHS 406.4x12.5	HEB 240	1.28	CHS 406.4x32
	7	222.212	HEB 160	1.64	CHS 406.4x32	HEB 160	1.79	CHS 406.4x32	HEB 140	1.43	CHS 406.4x12.5	HEB 200	1.37	CHS 406.4x32
	8	253.956	HEB 140	2.63	CHS 406.4x32	HEB 140	3.73	CHS 406.4x32	HEB 140	4.07	CHS406.4x12.5	HEB 140	1.91	CHS 406.4x32

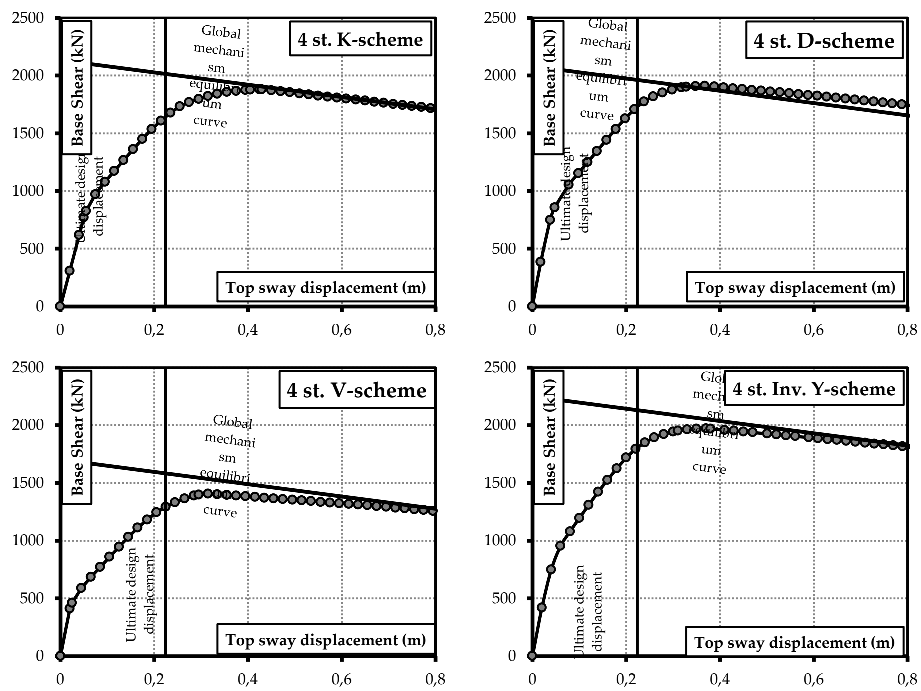

Fig. 5 – Push-over curves for 4-storey structures

Table 2 – Column sections

	n_s	K-scheme			D-scheme			V-scheme			Inv. Y-scheme		
		C1	**C2**	**C3**	**C1**	**C2**	**C3**	**C1**	**C2**	**C3**	**C1**	**C2**	**C3**
4-STOREY	1	HEB 280	HEB 280	HEB 360	HEB 260	HEB 260	R=HEB 400 L=HEB 340	HEB 260	HEB 260	HEB 360	HEB 260	HEB 280	HEB 400
	2	HEB 280	HEB 280	HEB 360	HEB 260	HEB 260	R=HEB 400 L=HEB 340	HEB 260	HEB 260	HEB 360	HEB 260	HEB 260	HEB 360
	3	HEB 280	HEB 280	HEB 300	HEB 260	HEB 260	R=HEB 340 L=HEB 280	HEB 260	HEB 260	HEB 360	HEB 260	HEB 260	HEB 340
	4	HEB 280	HEB 280	HEB 260	HEB 260	HEB 260	R=HEB 340 L=HEB 240	HEB 260	HEB 260	HEB 360	HEB 240	HEB 260	HEB 260
	n_s	**K-scheme**			**D-scheme**			**V-scheme**			**Inv. Y-scheme**		
		C1	**C2**	**C3**	**C1**	**C2**	**C3**	**C1**	**C2**	**C3**	**C1**	**C2**	**C3**
6-STOREY	1	HEB 300	HEB 300	HEB 300	HEB 280	HEB 280	R=HEB 550 L=HEB 500	HEB 300	HEB 300	HEB 450	HEB 300	HEB 300	HEB 500
	2	HEB 300	HEB 300	HEB 300	HEB 280	HEB 280	R=HEB 500 L=HEB 450	HEB 300	HEB 300	HEB 450	HEB 300	HEB 300	HEB 450
	3	HEB 300	HEB 300	HEB 300	HEB 280	HEB 280	R=HEB 450 L=HEB 400	HEB 300	HEB 300	HEB 450	HEB 300	HEB 300	HEB 450
	4	HEB 300	HEB 300	HEB 300	HEB 280	HEB 280	R=HEB 450 L=HEB 400	HEB 300	HEB 300	HEB 450	HEB 300	HEB 300	HEB 450
	5	HEB 300	HEB 300	HEB 300	HEB 280	HEB 280	R=HEB 450 L=HEB 340	HEB 300	HEB 300	HEB 450	HEB 300	HEB 300	HEB 360
	6	HEB 240	HEB 240	HEB 260	HEB 280	HEB 280	R=HEB 400 L=HEB 260	HEB 280	HEB 280	HEB 400	HEB 240	HEB 260	HEB 260
	n_s	**K-scheme**			**D-scheme**			**V-scheme**			**Inv. Y-scheme**		
		C1	**C2**	**C3**	**C1**	**C2**	**C3**	**C1**	**C2**	**C3**	**C1**	**C2**	**C3**
8-STOREY	1	HEB 320	HEB 340	HEB 650	HEB 300	HEB 320	R=HEB 700 L=HEB 700	HEB 320	HEB 320	HEB 650	HEB 320	HEB 340	HEB 650
	2	HEB 320	HEB 340	HEB 600	HEB 300	HEB 320	R=HEB 650 L=HEB 650	HEB 320	HEB 320	HEB 600	HEB 320	HEB 340	HEB 600
	3	HEB 320	HEB 340	HEB 600	HEB 300	HEB 320	R=HEB 600 L= HEB 600	HEB 320	HEB 320	HEB 600	HEB 320	HEB 340	HEB 600
	4	HEB 320	HEB 340	HEB 550	HEB 300	HEB 320	R=HEB 600 L=HEB 550	HEB 320	HEB 320	HEB 600	HEB 320	HEB 340	HEB 550
	5	HEB 320	HEB 340	HEB 500	HEB 300	HEB 320	R=HEB 550 L=HEB 550	HEB 320	HEB 320	HEB 550	HEB 320	HEB 340	HEB 500
	6	HEB 320	HEB 320	HEB 450	HEB 300	HEB 320	R=HEB 550 L=HEB 500	HEB 320	HEB 320	HEB 550	HEB 320	HEB 320	HEB 450
	7	HEB 300	HEB 300	HEB 360	HEB 300	HEB 320	R=HEB 500 L=HEB 400	HEB 300	HEB 300	HEB 450	HEB 300	HEB 300	HEB 360
	8	HEB260	HEB260	HEB260	HEB 300	HEB 300	R=HEB 400 L=HEB 280	HEB 280	HEB 280	HEB 400	HEB260	HEB260	HEB260

4 IDA Analyses

The scope of IDA analyses is the evaluation of the actual seismic performances of the structures considering the four brace layouts proposed by seismic codes. To this scope IDA analyses have been carried out on the same structural model already adopted for push-over analyses. 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 (Table 3). Despite of the different brace geometry it is useful to note that, because of the common design method, the dynamic properties of the structural schemes are very close to each other. The only significant difference is due to the number of storeys and not to the brace geometry. For sake of shortness, only the main results of IDA analyses are herein presented while a wide discussion is provided in [12].

Record-to-record variability has been accounted for by considering 10 recorded accelerograms selected from PEER data base. These recorded accelerograms (Table 4) have been selected to approximately match the linear elastic design response spectrum of Eurocode 8, for soil type A. In addition, each ground motion has been scaled to obtain the same value of the spectral acceleration, $S_a(T_1)$, corresponding to the fundamental period of vibration T_1 of the structure under examination (Table 3); successively $S_a(T_1)$ values have been progressively increased. The IDA 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 the attainment of the limit value of the chord rotation of structural members. The $S_a(T_1)/g$ values corresponding to the structural collapse are reported in

Table 5, Table 6 and Table 7 for each ground motion and for the 4-storey, 6-storey and 8-storey structures, respectively.

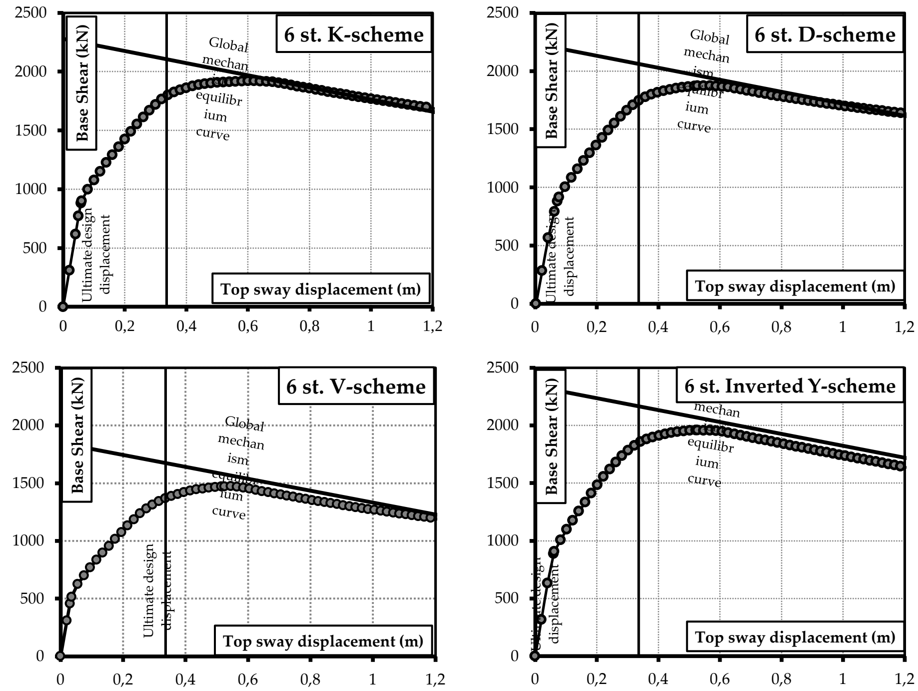

Fig. 6 – Push-over curves for 6-storey structures

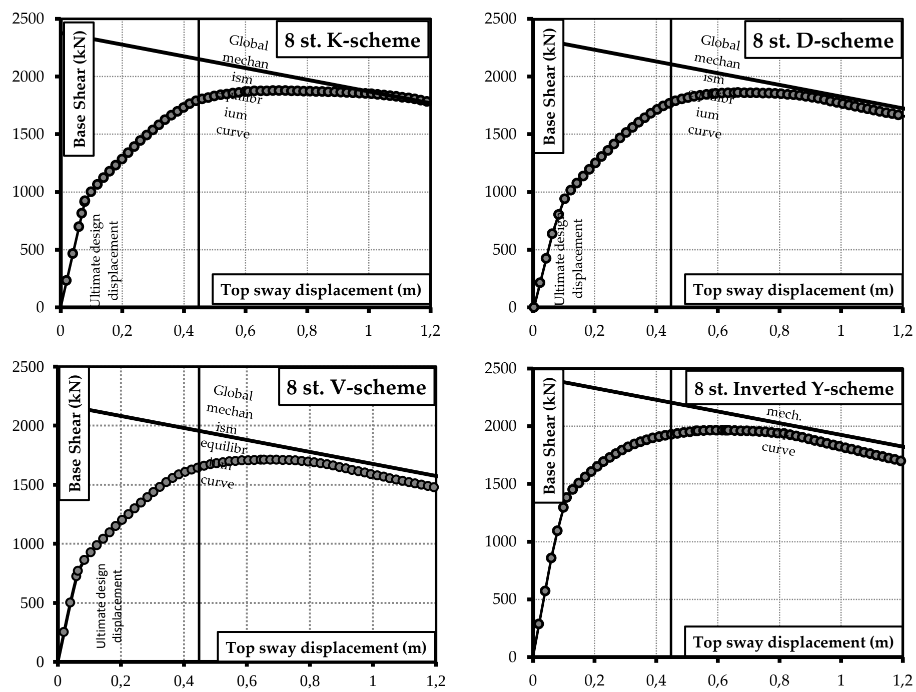

Fig. 7 – Push-over curves for 8-storey structures

By comparing the average value of the collapse value of $S_a(T_1)/g$, it is possible to observe that, given the number of storeys, the collapse condition is always achieved, first by the V-scheme structures followed by the D-scheme structure and K-scheme structure. Inverted Y-scheme structures provide the best performances and this result becomes more significant as far as the structural height increase. Conversely, structures whose brace geometry is configured with horizontal links show decreasing performances for increasing number of storeys. It is important to observe that, the structural collapse is always achieved due to the attainment of the ultimate rotation of link members while beam ends and first storey column base sections are far from their ultimate state. This outcome is a benefit of the use of dual systems where the moment-resisting part works as a survival secondary structural system which is engaged in plastic range after the spreading of yielding in the primary structural system constituted by the braced part. Given the above, it is possible to conclude that all the brace layout exhibit almost the same performances for low number of storeys while the differences becomes more relevant as the number of storeys increases. For this reason, a different behavior factor for the different brace configurations should be assumed and also proposed in seismic codes. However, in this paper only 5 bays structures, with 4, 6 and 8 storeys, have been investigated so that additional analyses on different structural schemes with different number of bays should be carried out in order to provide more general conclusions. However, these preliminary results show that the spectral acceleration leading to collapse reduces of about 30% compared to the other schemes so that a similar reduction of the design value of the q-factor could be prudentially proposed for the V-scheme.

Table 3 – First and third vibration mode period of buildings designed

	4 STOREYS		**6 STOREYS**		**8 STOREYS**	
	T_1 (s)	**T_3 (s)**	**T_1 (s)**	**T_3 (s)**	**T_1 (s)**	**T_3 (s)**
K-scheme	1.00	0.45	1.38	0.56	1.80	0.67
D-scheme	1.01	0.44	1.42	0.56	1.87	0.67
V-scheme	1.00	0.42	1.40	0.51	1.74	0.59
Inverted Y-scheme	1.01	0.44	1.39	0.54	1.62	0.60

Table 4 – Selected Accelerograms

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
Palm Springs (San Jacinto)	PALMSPR/H08000	1986/07/08	0.250	26.00	0.005
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

Table 5 – Sa (T_1) values corresponding to the attainment of the collapse condition for the 4-storey structures

	4 st. K-scheme	**4 st. D-scheme**	**4 st. V-scheme**	**4-storey Inv. Y**
	Sa(T_1)	**Sa(T_1)**	**Sa(T_1)**	**Sa(T_1)**
Coalinga	0.90 g	0.70 g	0.70 g	0.80 g
Friuli, Italy	0.90 g	0.90 g	0.60 g	1.00 g
Imperial Valley	0.60 g	0.60 g	0.40 g	0.60 g
Irpinia, Italy	1.60 g	1.40 g	0.90 g	1.60 g
Kobe	0.80 g	0.90 g	0.50 g	0.90 g
Northridge	0.70 g	0.60 g	0.50 g	0.80 g
Palm Springs	0.70 g	0.80 g	0.60 g	0.80 g
Santa Barbara	0.80 g	0.70 g	0.50 g	0.90 g

Spitak Armenia	1.30 g	1.20 g	1.00 g	1.20 g
Victoria Mexico	0.60 g	0.50 g	0.40 g	0.80 g
Mean value	**0.89 g**	**0.83 g**	**0.61 g**	**0.94 g**

Table 6 – Sa (T_1) values corresponding to the attainment of the collapse condition for the 6-storey structures

	6 st. K-scheme	**6 st. D-scheme**	**6 st. V-scheme**	**6 st. Inv. Y-scheme**
	Sa(T_1)	**Sa(T_1)**	**Sa(T_1)**	**Sa(T_1)**
Coalinga	0.70 g	0.70 g	0.50 g	0.80 g
Friuli, Italy	0.70 g	0.70 g	0.40 g	0.70 g
Imperial Valley	0.55 g	0.60 g	0.40 g	0.60 g
Irpinia, Italy	0.80 g	0.90 g	0.50 g	1.00 g
Kobe	0.65 g	0.60 g	0.40 g	0.70 g
Northridge	0.50 g	0.50 g	0.40 g	0.70 g
Palm Springs	0.40 g	0.40 g	0.30 g	0.50 g
Santa Barbara	1.00 g	0.90 g	0.60 g	0.90 g
Spitak Armenia	0.55 g	0.50 g	0.40 g	0.60 g
Victoria Mexico	0.55 g	0.60 g	0.50 g	0.60 g
Mean value	**0.64 g**	**0.64 g**	**0.44 g**	**0.71 g**

Table 7 – Sa (T_1) values corresponding to the attainment of the collapse condition for the 8-storey structures

	8 st. K-scheme	**8 st. D-scheme**	**8 st. V-scheme**	**8 st. Inv. Y-scheme**
	Sa(T_1)	**Sa(T_1)**	**Sa(T_1)**	**Sa(T_1)**
Coalinga	0.70 g	0.70 g	0.40 g	0.85 g
Friuli, Italy	0.90 g	0.80 g	0.50 g	1.10 g
Imperial Valley	0.50 g	0.50 g	0.50 g	0.65 g
Irpinia, Italy	0.70 g	0.70 g	0.40 g	0.80 g
Kobe	1.00 g	1.00 g	0.60 g	0.90 g
Northridge	0.50 g	0.30 g	0.30 g	0.55 g
Palm Springs	0.25 g	0.20 g	0.20 g	0.40 g
Santa Barbara	0.35 g	0.40 g	0.30 g	0.70 g
Spitak Armenia	0.45 g	0.40 g	0.50 g	1.55 g
Victoria Mexico	0.65 g	0.50 g	0.50 g	0.70 g
Mean value	**0.60 g**	**0.55 g**	**0.42 g**	**0.82 g**

5 Conclusions

The evaluation of the influence of brace geometry on the seismic performances of MRF-EBF dual systems designed by means of Theory of Plastic Mechanism Control (TPMC) has been investigated. The use of TPMC has assured a collapse mechanism of global type as confirmed by the results of push-over analyses.

IDA have been carried out to check the actual performances of the designed frames. The results obtained show that V-scheme structures always exhibit the worst performances. This result is more evident as the number of storey increase. On the contrary, structure whose as the same as of K-scheme and D-scheme. On the contrary, structures whose braces are arranged according to the inverted Y-scheme lead to the best seismic performances.

Despite of this paper investigates only 5 bays structures with 4, 6 and 8 storeys, a 30% reduction of the q-factor can be preliminarily suggested for the seismic design of eccentrically braced frames with V-scheme.

However, additional analyses on different structural schemes with different number of bays should be carried out in order to provide a more robust proposal.

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5. References

[1] CEN (2005): "EN 1998-1-1: Eurocode 8 - Design of Structures for Earthquake Resistance. Part 1: General Rules, Seismic Actions and Rules for Buildings", Comite Europeen de Normalisation, CEN/TC 250.

[2] Mazzolani F.M., Piluso V. (1997): "Plastic design of seismic resistant steel frames", Earthquake Engineering and Structural Dynamics, Vol. 26, pp. 167-191.

[3] Piluso V., Nastri E., Montuori R., "Advances in Theory of Plastic Mechanism Control: Closed Form Solution for MR-Frames", Earthquake Engineering and Structural Dynamics, Volume 44, Issue 7, 1 June 2015, Pages 1035-1054.

[4] Longo A., Nastri E., Piluso, V. "Theory of Plastic Mechanism Control: State of the Art", The Open Construction and Building Technology Journal, Volume 8, 2014, Pages 262-278.

[5] Conti M.A., Mastandrea L., Piluso V., "Plastic Design and Seismic Response of Knee Braced Frames", Advanced Steel Costructions, Vol. 5, n.3 September, pp. 343-363, 2009.

[6] Longo A., Montuori R., Piluso V., "Theory of Plastic Mechanism Control of Dissipative Truss Moment Frames", Engineering Strucutures, Vol. 37, pp. 63-75, 2012.

[7] Longo A., Montuori R., Piluso V., "Failure Mode Control and Seismic Response of Dissipative Truss Moment Frames", Journal of Structural Engineering, Vol. 138, pp.1388-1397, 2012.

[8] Longo A., Montuori R., and Piluso V. "Theory of plastic mechanism control for MRF–CBF dual systems and its validation." Bulletin of Earthquake Engineering, pp. 1-31, 2014.

[9] Montuori R., Muscati R., "A general design procedure for failure mechanism control of reinforced concrete frame", Engineering Structures, Volume 118, July 01, 2016, Pages 137-155, 2016.

[10] Montuori R., Nastri E., Piluso V. "Theory of Plastic Mechanism Control for MRF–EBF dual systems: Closed form Solution", Engineering Structures, Volume 118, 1 July 2016, Pages 287–306, 2016.

[11] Montuori R., Nastri E., Piluso V. (2013): "Rigid-Plastic Analysis and Moment-Shear Interaction for Hierarchy Criteria of EB-Frames with Inverted Y-Scheme", Journal of Constructional Steel Research, Volume 95, January 2014, pp. 71–80.

[12] Nastri E. "Theory of Plastic Mechanism Control for Eccentrically Braced Frames: Closed Form Solution", PhD Thesis, 2015.

[13] Mastrandrea L., Nastri E., Piluso, V., "Validation of a Design Procedure for Failure Mode Control of EB-Frames: Push-over and IDA analyses", The Open Construction and Building Technology Journal, 7, 193-207, 2013.