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PRELIMINARY ANALYSIS ON THE INFLUENCE OF THE LINK CONFIGURATION ON SEISMIC PERFORMANCES OF MRF-EBF DUAL SYSTEMS DESIGNED BY TPMC

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SUMMARY: *The work is devoted to the evaluation of the influence of link configuration on seismic performances of Moment Resisting Frames-Eccentrically Braced Frames dual systems (MRF-EBF dual systems) designed by means of Theory of Plastic Mechanism Control (TPMC). As it is known TPMC assures the development of a collapse mechanism of global type, therefore, the seismic performances evaluated by means of IDA analyses, are affected only by the structural scheme configuration. However, in this paper only a 5 bays structure with 4, 6 and 8 storeys are reported, so that, the work configures as a preliminary evaluation. Additional analyses on different structural schemes with different number of bays will be the natural development of the work herein presented.*

KEYWORDS: *EBFs, eccentrically braced frames, seismic, kinematic theorem, TPMC*

1. Introduction

The work herein presented is devoted to the evaluation of the seismic performance of Moment Resisting Frame-Eccentrically Braced Frame dual systems (MRF-EBF dual systems) considering all the possible configuration of braces and links as reported in the Eurocode 8 [CEN, 2015] (Fig. 1). As it is known, EBFs constitute a suitable compromise between MRF and CBF [Longo *et al.*, 2014; D'Aniello *et al.*, 2010; D'Aniello *et al.*, 2012; D'Aniello *et al.*, 2015], due both to their lateral stiffness given by the braces and adequate dissipation capacity given by links able to develop wide and stable hysteresis loops [Della Corte *et al.,* 2013; Della Corte *et al.,* 2007]. The number of the proposed link configuration is 4: three of which have a horizontal link configuration (K-scheme, D-scheme, V-scheme) while the last one has a vertical link configuration (Inverted Y-scheme). In this paper, MRF-EBF dual systems with 4, 6 and 8 storeys have been designed using the Theory of Plastic Mechanism Control for a total number of 12 study cases. The scope is to investigate the structural seismic response of the designed structure being assured, by the TPMC design approach, a collapse mechanism of global type. In this way, the seismic performances are only affected by the structural scheme, i.e. by the link configuration. For this reason both push-over and Incremental Dynamic Analyses (IDA) have been carried out on the designed structure with the scope to point out, on one hand, the accuracy of the TPMC design approach, and on the other hand, downstream the validation, the different seismic performances achieved by the differently configured structure. However, in this paper only a 5 bays structure is investigated, so that the work configures as a preliminary evaluation. For a higher accuracy, additional analyses on different structural schemes with different storey and bay numbers will be reported in further works.

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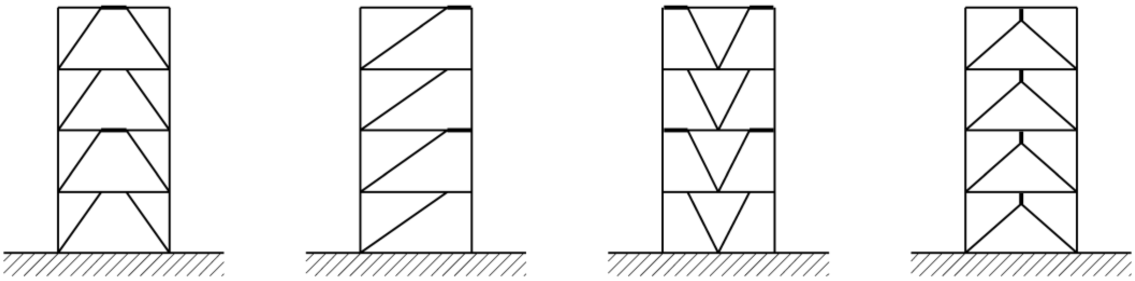

Figure 1. *EBFs typologies proposed by Eurocode 8*

2. DESIGN APPROACH

TPMC is based on a rigorous approach assuring a collapse mechanism of global type [Piluso *et al.*, 2015; Montuori and Muscati, 2015; Longo *et al.,* 2014] which exploits the kinemathic theorem of plastic collapse extended to the concept of collapse mechanism equilibrium curve:

$$\alpha = \alpha_0 - \gamma\delta \qquad (1)$$

where α_0 is the first order collapse multiplier of horizontal forces, γ is the slope of the linearized mechanism 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 (Figure 2). Column sections at each storey needed to assure a collapse mechanism of global type are unknown of the design problem, while link, beam and diagonal members are preliminarily determined as known quantities of the design problem. More details about this design procedure devoted to EBFs are reported in previous works [Longo *et al.,* 2014; Montuori *et al.*, 2013; Montuori *et al.*, 2014a; Montuori *et al.*, 2014b; Montuori *et al.*, 2016]

3. STUDY CASES

The study cases herein investigated are referred to a building whose plan configuration is depicted in Fig. 3. In particular, considering only the perimeter frames whose secondary floor warping is orthogonal to the beam of the frame, four structures have been designed for each selected number of storey (4, 6 and 8) according to TPMC for a total number of 12 frames.
The corresponding seismic resistant schemes are depicted in Figure 3 with reference to the 6-storey building where also the leaning column adopted in structural modelling to account for second order effects due to the internal gravity load resisting system is reported. Regarding the length of the links, at each storey, it is equal to 1.20 m for K-scheme and D-scheme EBFs while is equal to 0.60 m for V-scheme EBFs. As regards the inverted Y-scheme, link length has been assumed equal to 0.70 m at each storey because the link length has been defined in order to assure that the ultimate design displacement, δ_u, for the application of TPMC is the same both for the all the structural configuration, according to the following relations:

$$\delta_u = \gamma_u\left(e/L_j\right)h_{n_s} = 0.08(1.2/6)h_{n_s} = 0.016h_{n_s} \quad \text{for K-scheme and D-scheme} \qquad (2)$$

$$\delta_u = \gamma_u\left(2\,e/L_j\right)h_{n_s} = 0.08(2\ 0.6/3.5)h_{n_s} = 0.016h_{n_s} \quad \text{for inverted V-scheme} \qquad (3)$$

$$\delta_u = \gamma_u(e/h_i)h_{n_s} = 0.08(0.7/3.5)h_{n_s} = 0.016h_{n_s} \quad \text{for inverted Y-scheme} \qquad (4)$$

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.

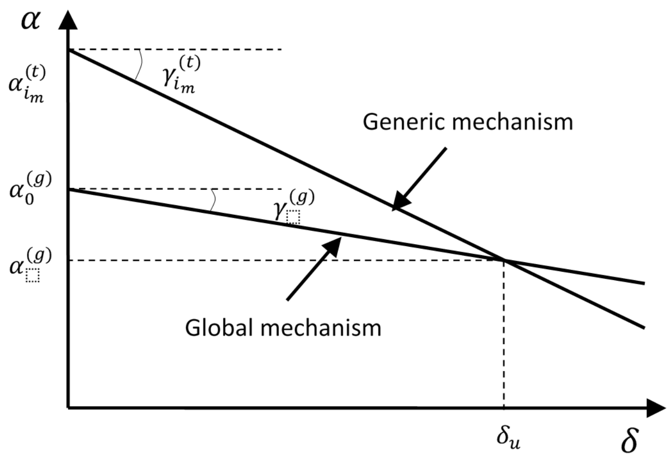

Figure 2. *TPMC statement*

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 EC8, $G_k + \psi_2 Q_k + E_d$ (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 have been designed to withstand vertical loads accounting also for serviceability requirements and they are delivered in Figure 3. The design horizontal forces have been determined according to EC8, assuming a peak ground acceleration equal to 0.35g, a seismic response factor equal to 2.5 and a behaviour factor equal to 6. It is important to observe that EC8 does not make any difference in term of behaviour factor between the EBFs configuration. 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 addition, for each structure the link length combined with the resistance belonging to the selection of the section from the standard shape assure a short-link behaviour, according to the classification given in EC8. Short links show many advantages if compared with long and intermediate links due to their performance in terms of both stiffness and ductility. In fact, the cyclic behaviour of short links is characterised by wide and stable cycles allowing the development of high energy dissipation capacity and highest rotational capacity if compared with long and intermediate ones [Bosco *et al.*, 2015; Mastrandrea *et al.*, 2013; Ohsaki *et al.*, 2012; Gulec *et al.*, 2011]. Table 1 the link and diagonal sections for the designed structures are reported. In addition, in the Table 2 column sections of each structure are delivered. For sake of shortness, the numerical details for the application of TPMC procedure are given elsewhere [Longo *et al.,* 2014; Montuori *et al.*, 2014a; Montuori *et al.*, 2014b] so only the results obtained are herein presented.

4. PUSH-OVER RESULTS

With reference to the seismic resistant system depicted in Figure 3, 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 midspan of 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 with a shear force versus shear displacement rigid-hardening constitutive model (1.50 of overstrength).

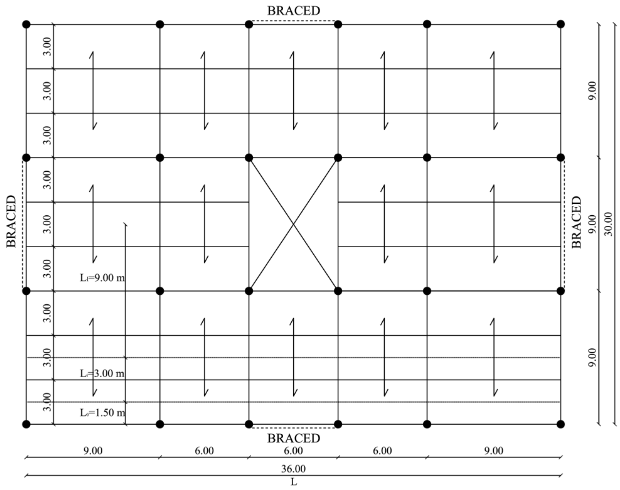

Figure 3. *Plan configuration of the analysed structures*

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

			K-scheme		**D-scheme**		**V-scheme**		**Inverted Y-scheme**	
4-STOREY	n_s	F [kN]	LINKS	DIAGONALS	LINKS	DIAGONALS	LINKS	DIAGONALS	LINKS	DIAGONALS
	1	96.097	HEB 240	CHS 323.9x20	HEB 240	CHS 323.9x20	HEB 160	CHS 323.9x20	HEB 300	CHS 244.5x20
	2	192.195	HEB 200	CHS 323.9x20	HEB 200	CHS 323.9x20	HEB 140	CHS 323.9x20	HEB 300	CHS 244.5x20
	3	288.293	HEB 180	CHS 323.9x20	HEB 180	CHS 323.9x20	HEB 140	CHS 323.9x20	HEB 240	CHS 244.5x20
	4	384.391	HEB 140	CHS 323.9x20	HEB 140	CHS 323.9x20	HEB 140	CHS 323.9x20	HEB 180	CHS 244.5x20
6-STOREY			**K-scheme**		**D-scheme**		**V-scheme**		**Inverted Y-scheme**	
	n_s	F [kN]	LINKS	DIAGONALS	LINKS	DIAGONALS	LINKS	DIAGONALS	LINKS	DIAGONALS
	1	50.643	HEB 240	CHS 355.6x16	HEB 200	CHS 355.6x16	HEB 180	CHS 355.6x16	HEB 200	CHS 244.5x20
	2	101.285	HEB 240	CHS 355.6x16	HEB 200	CHS 355.6x16	HEB 180	CHS 355.6x16	HEB 200	CHS 244.5x20
	3	151.928	HEB 220	CHS 355.6x16	HEB 200	CHS 355.6x16	HEB 180	CHS 355.6x16	HEB 200	CHS 244.5x20
	4	202.571	HEB 200	CHS 355.6x16	HEB 180	CHS 355.6x16	HEB 160	CHS 355.6x16	HEB 180	CHS 244.5x20
	5	253.214	HEB 160	CHS 355.6x16	HEB 160	CHS 355.6x16	HEB 140	CHS 355.6x16	HEB 160	CHS 244.5x20
	6	303.856	HEB 140	CHS 355.6x16	HEB 160	CHS 355.6x16	HEB 140	CHS 355.6x16	HEB 160	CHS 244.5x20
8-STOREY			**K-scheme**		**D-scheme**		**V-scheme**		**Inverted Y-scheme**	
	n_s	F [kN]	LINKS	DIAGONALS	LINKS	DIAGONALS	LINKS	DIAGONALS	LINKS	DIAGONALS
	1	31.745	HE 240 B	CHS 406.4x32	HEB 340	CHS 406.4x32	HE 180 B	CHS 406.4x12.5	HEB 340	CHS 406.4x32
	2	63.489	HE 240 B	CHS 406.4x32	HEB 340	CHS 406.4x32	HE 180 B	CHS 406.4x12.5	HEB 340	CHS 406.4x32
	3	95.234	HE 240 B	CHS 406.4x32	HEB 320	CHS 406.4x32	HE 180 B	CHS 406.4x12.5	HEB 320	CHS 406.4x32
	4	126.978	HE 220 B	CHS 406.4x32	HEB 300	CHS 406.4x32	HE 180 B	CHS 406.4x12.5	HEB 300	CHS 406.4x32
	5	158.723	HE 200 B	CHS 406.4x32	HEB 280	CHS 406.4x32	HE 160 B	CHS 406.4x12.5	HEB 280	CHS 406.4x32
	6	190.467	HE 180 B	CHS 406.4x32	HEB 240	CHS 406.4x32	HE 160 B	CHS 406.4x12.5	HEB 240	CHS 406.4x32
	7	222.212	HE 160 B	CHS 406.4x32	HEB 200	CHS 406.4x32	HE 140 B	CHS 406.4x12.5	HEB 200	CHS 406.4x32
	8	253.956	HE 140 B	CHS 406.4x32	HEB 140	CHS 406.4x32	HE 140 B	CHS 406.4x12.5	HEB 140	CHS 406.4x32

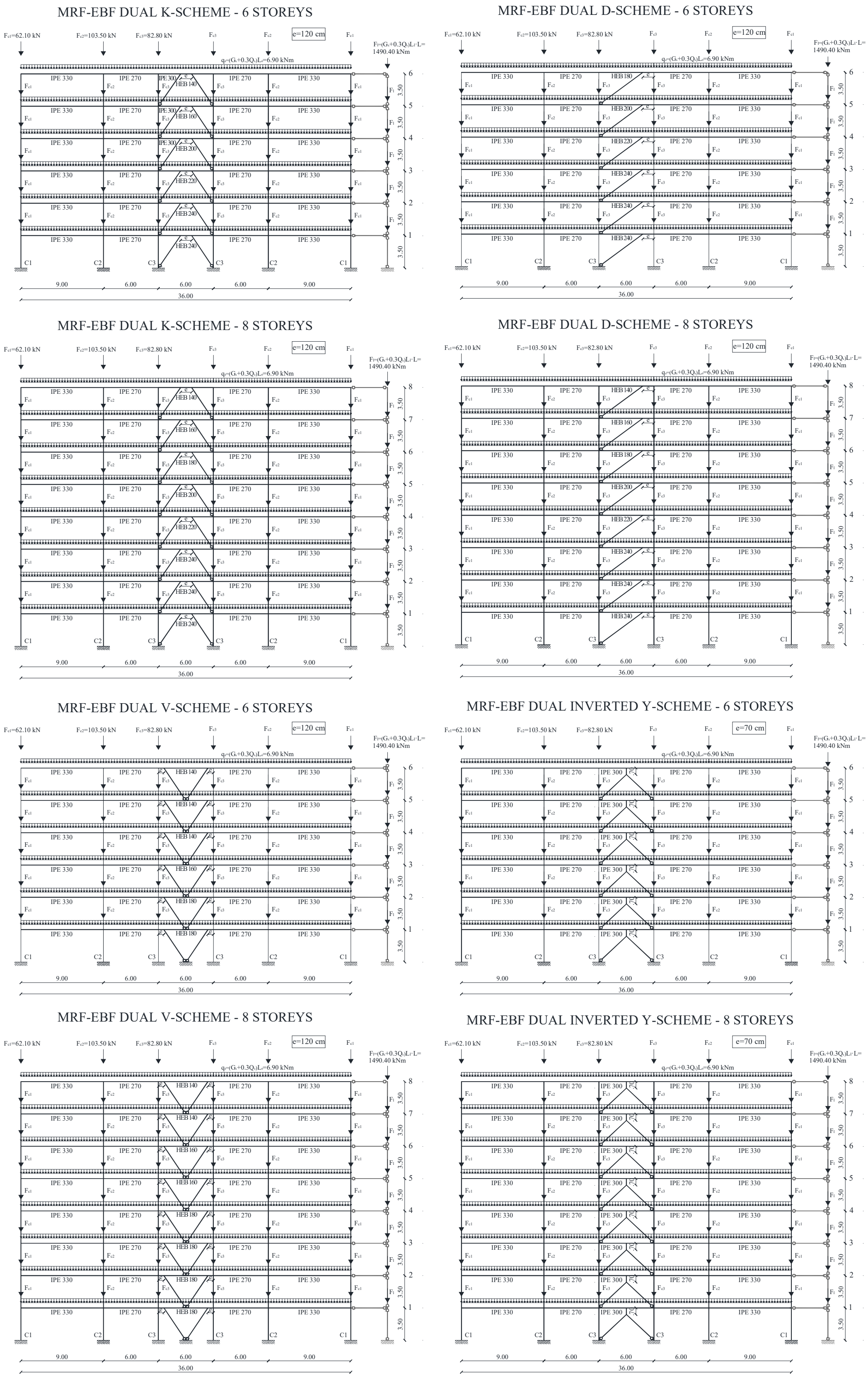

Figure 4. *Structral schemes of the longitudinal seismic resistant system*

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 pushover analyses are reported in Fig. 4, Fig. 5 and Fig. 6 for the 4-storey, 6-storey and 8-storey buildings, respectively. In particular, the results provided by the analyses show that the softening branch of the push-over curve corresponding to the structure 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 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 BUILDING	**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 BUILDING	**1**	HEB 320	HEB 340	HEB 650	HEB 300	HEB 320	R=HEB 700 L=HEB 700	HEB 340	HEB 340	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 340	HEB 550	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 340	HEB 550	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 340	HEB 550	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 340	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 340	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 320	HEB 320	HEB 500	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

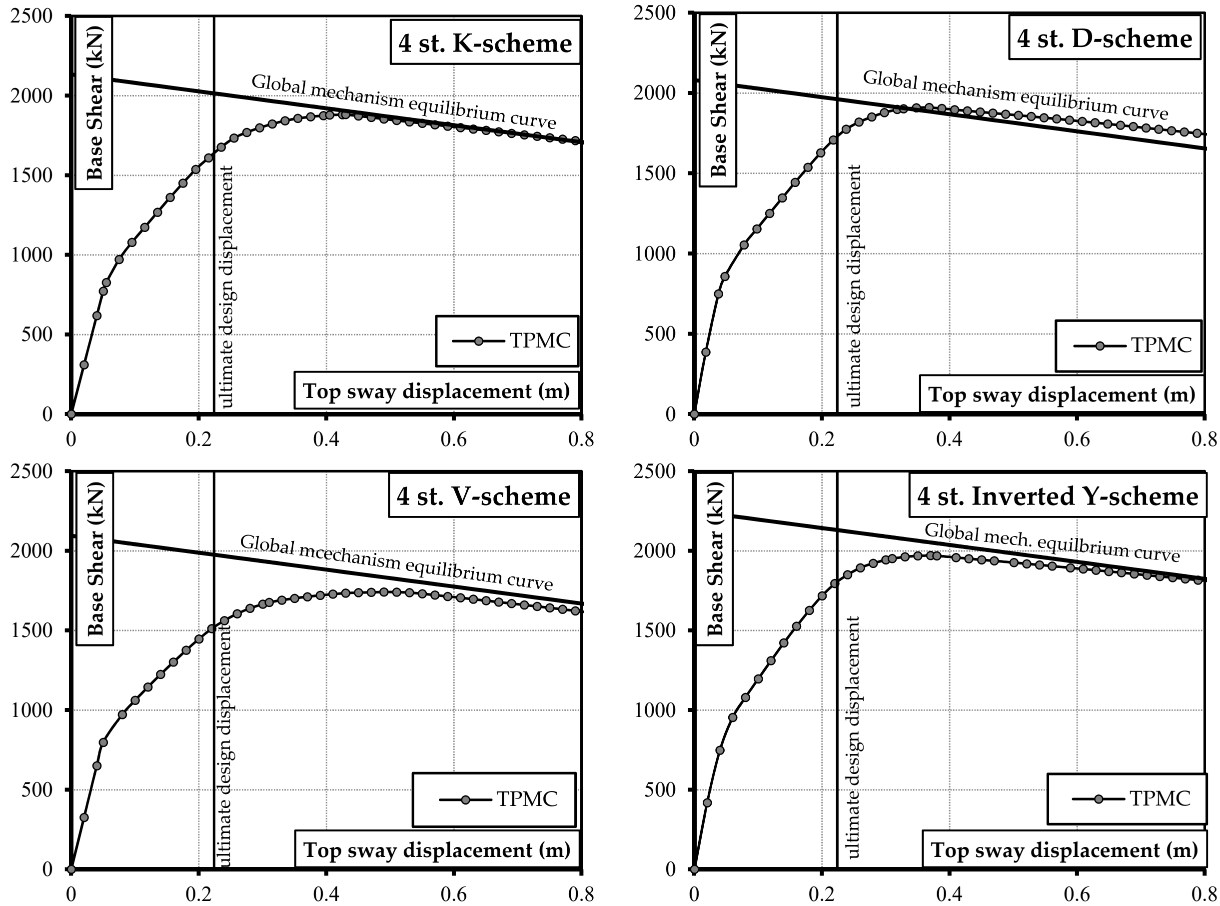

Figure 5. *Push-over curves for 4-storey structures*

5. IDA Analyses Results

While previous works have investigated the different performances achieved by the structure designed by means of TPMC and EC8, pointing out the higher performances of TPMC designed strucutures [Montuori *et al.*, 2015a; Montuori *et al.,* 2015b] the scope of the IDA analyses herein carried out is the evaluation of the seismic performances of the structures considering the four link configuration proposed by EC8 given the number of storeys given the same design approach. To this purpose IDA analyses have been carried out on the same structural model already adopted for push-overs by assuming a 5% damping according to Rayleigh with the proportional factors computed with reference to the first and third mode of vibration (Table 3). Record-to-record variability has been accounted for by considering 10 recorded accelerograms selected from PEER data base [Pacific Earthquake Engineering Research Center]. These recorded accelerograms 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; 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 up to the attainment of the limit value of the chord rotation of structural members that is assumed equal to 0.04 rad in the case of beams, columns and diagonals and equal to 0.08 rad in the case of links [CEN, 2005].

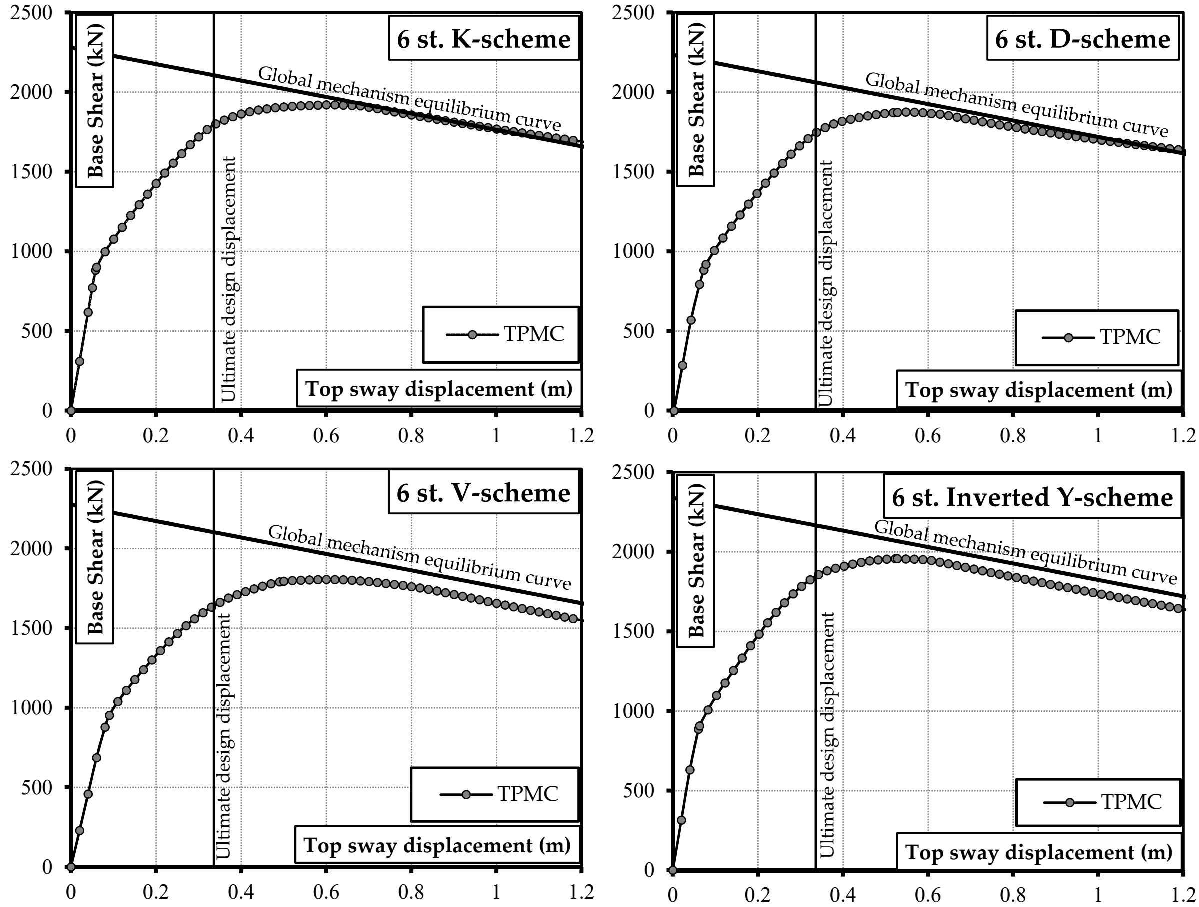

Figure 6. *Push-over curves for 6-storey structures*

In Table 5, Table 6 and Table 7 the $S_a(T_1)/g$ values corresponding to the structural collapse for each ground motion are reported. In particular, by comparing the average value of the collapse $S_a(T_1)/g$ 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. Inverted Y-scheme structures always confirm the best performances which becomes more relevant at the increasing of the structural height compared with the second classified of the list, i.e. K-scheme. It is also possible to observe that the seismic performances of D-scheme and V-scheme decrease as far as the structural height increase, therefore it is preferable not to use D-scheme and V-scheme EBFs for tall buildings. Conversely, inverted Y-scheme increase its performances as the number of storey increase, that makes it more suitable to tall building application. Other benefits given by inverted Y-scheme EBFs, not belonging to the beam member, are the chance to be easily substituted after destructive seismic events and to conceive the scheme within the framework of supplementary energy dissipation strategy [Castaldo, 2014; De Iuliis and Castaldo, 2012; Castaldo and De Iuliis, 2012; Palazzo *et al.*, 2015; Castaldo *et al.*, 2016], by substituting the vertical link member with dissipative device, such as a friction damper or hysteretic damper, which is able to exhibit a highly dissipative behaviour if compared with traditional link members[Montuori *et al.*, 2014b; Nastri, 2016].

In addition, such link configuration can be suitable also for the seismic retrofitting of existing buildings [D'Aniello *et al.*, 2006; Mazzolani et al., 2009].

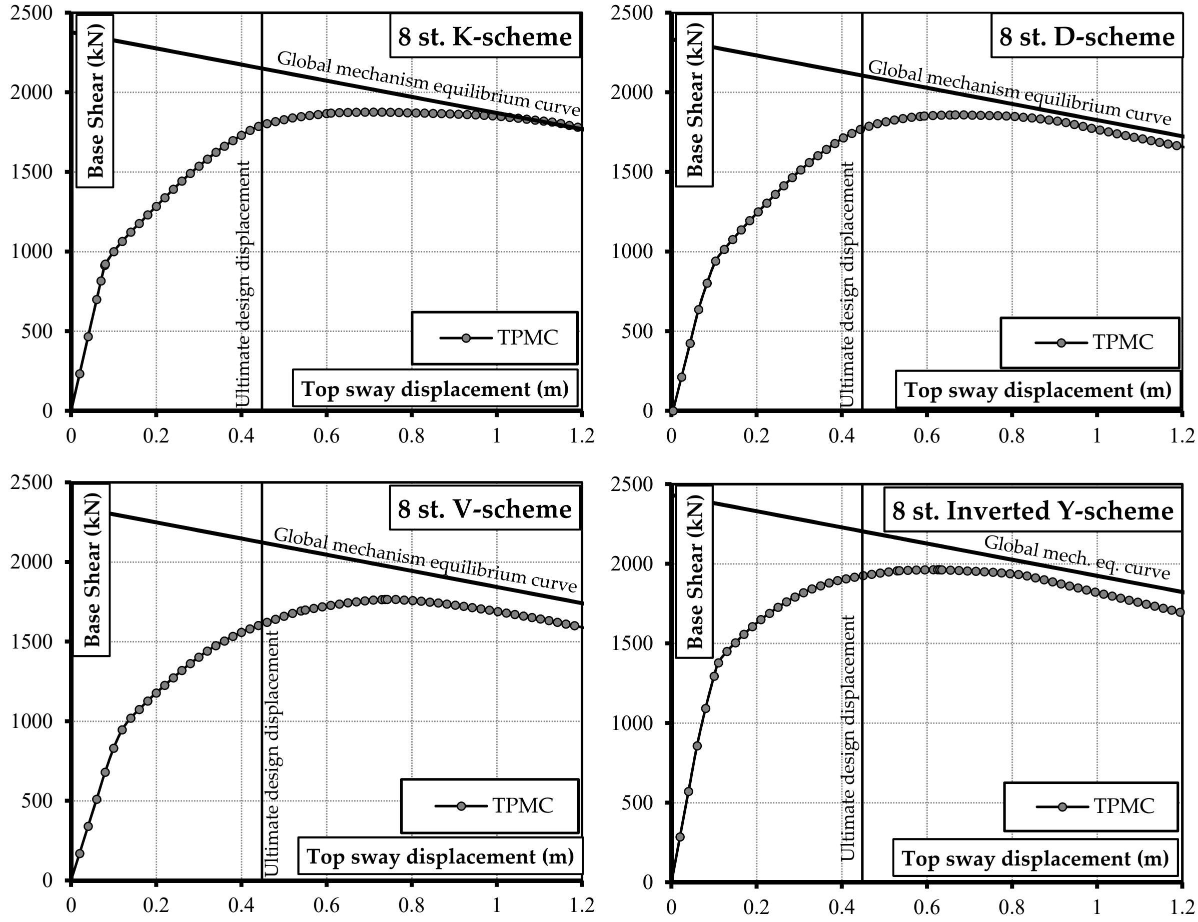

Figure 7. *Push-over curves for 8-storey structures*

Given the above, it is possible to conclude that all the EBF configuration show quite the same performances for a lower 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 EBF configuration should be assumed. However, in this paper only a 5 bays structure 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 a more exhaustive research.

Table 3. *First and third vibration mode period of buildings designed*

	4 STOREYS				6 STOREYS				8 STOREYS			
	TPMC		EC8		TPMC		EC8		TPMC		EC8	
	T_1 (s)	T_3 (s)	T_1 (s)	T_3 (s)	T_1 (s)	T_3 (s)	T_1 (s)	T_3 (s)	T_1 (s)	T_3 (s)	T_1 (s)	T_3 (s)
K-scheme	1.00	0.45	1.02	0.46	1.38	0.56	1.46	0.60	1.80	0.67	1.92	0.73
D-scheme	1.01	0.44	1.04	0.46	1.42	0.56	1.40	0.52	1.87	0.67	2.00	0.73
V-scheme	1.00	0.42	1.00	0.43	1.40	0.51	1.44	0.53	1.74	0.59	1.88	0.65
Inverted Y-scheme	1.01	0.44	1.08	0.47	1.39	0.54	1.47	0.58	1.62	0.60	1.70	0.63

Table 4. *Analyzed 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
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. *S_a (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. *S_a (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. *S_a (T_1) values corresponding to the attainment of the collapse condition for the 8 st. 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**

6. CONCLUSIONS

In this paper, the evaluation of the influence of link configuration on seismic performances of Moment Resisting Frames-Eccentrically Braced Frames dual systems (MRF-EBF dual systems) designed by means of Theory of Plastic Mechanism Control (TPMC) is reported. TPMC is based on the kinematic theorem of plastic collapse and its extension to the concept of mechanism equilibrium curve and assure to design structure showing at the collapse a global mechanism. For this reason, being assured and verified the collapse mechanism of global type through push-over analyses, IDA have been carried out to check the actual performances of the designed frames.

Results show that V-scheme structures always show the worse performances which even decrease as far as the number of storey increase. On the contrary inverted Y-scheme confirms the best performances. Notwithstanding, inverted Y-scheme increases its performances as the number of storey increases, that makes this structural scheme more suitable for tall buildings, while all the horizontal link EBFs decrease or at least keep constant their performances being an increase of the number of storeys.

However, in this paper only a 5 bays structure 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 a more exhaustive research.

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ANALISI PRELIMINARE DELL'INFLUENZA DELLA CONFIGURAZIONE NELLE PERFORMANCE SISMICHE DEI SISTEMI ACCOPPIATI CON CONTROVENTI ECCENTRICI PROGETTATI MEDIANTE LA TPMC

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SOMMARIO: *Il lavoro riguarda l'influenza della configurazione del link nella valutazione delle prestazioni sismiche di telai accoppiati con controventi eccentrici (MRF-EBF dual systems) progettati mediante la Teoria del Controllo del Meccanismo di Collasso (TPMC). Come è noto, tale teoria assicura che i telai progettati in accordo con essa manifestino al collasso un meccanismo globale. Per tale motivo, con riferimento ai telai riportati nel presente lavoro, l'unico elemento che influenza le prestazioni sismiche è la disposizione dei link. In ogni caso, in questa memoria sono state riportate le analisi push-over e IDA per una struttura a 5 campate facendo variare il numero di piani (4, 6 e 8). Essendo essa solo un'analisi preliminare risulta superfluo affermare che una campagna di analisi più approfondita, riguardante strutture con diverso numero di piani e di campate sarà oggetto di un futuro approfondimento.*

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