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MRFs DESIGNED ACCORDING TO THE THEORY OF PLASTIC MECHANISM CONTROL VS CODE RULES

Roberta Muscati[1]

[1]Ph. D. student, Department of Civil Engineering, University of Salerno – Italy

E-mail: rmuscati@unisa.it

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Abstract. *In this paper the aim is to show a comparison between two design methodologies.*
In the following is reported the design of a reinforced concrete frame according code rules i.e. adopting the hierarchy criteria and the same frame designed by the Theory of Plastic Mechanism Control (TPMC).
The TPMC, based on the application of the kinematic theorem of plastic collapse, is a more sophisticated design procedure because it works in full compliance with codes recommendations considering that it respects the hierarchy criteria. These last, in fact, are fundamental to avoid dangerous collapse mechanisms such as "soft-storey" mechanism but they are not sufficient to guarantee the exploitation of the maximum dissipation capacity of the frame.
In addition push-over analyses have been made to investigate the actual collapse mechanism of the designed structure. All the obtained results confirm the ability of the design procedure to obtain a collapse mechanism of global type.

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1 INTRODUCTION

The optimization of the seismic structural response represents the primary purpose of every designer. This aim consists in the possibility to avoid dangerous and unsatisfactory collapse mechanism, in terms of energy dissipation capacity i.e. in the possibility to control the typology of collapse mechanism.

A perfect design should lead to the collapse of the structure according to the global mechanism, characterized by the development of plastic hinges in all the beam ends and at the base sections of the first storey columns.

To this scope in 1970s, in New Zealand [1-4], born the concept of capacity design i.e. the awareness that to reach the maximum dissipation capacity of the structures is necessary to design the different elements with several resistances. In fact, the basic principles of capacity design approach impose to differentiate on one hand, dissipative zones, designed according to the internal actions arising from the seismic forces provided by the codes, and on the other hand non-dissipative zones, proportioned on the maximum internal actions transmitted by the dissipative zones.

In a seismic resistant concrete frame this means designing structures with strong columns and weak beams. These design criteria are nowadays present in all the modern seismic codes, included the Eurocode 8 [5], under the name of the so called beam-column hierarchy criterion.

Unfortunately this approach is only able to prevent the development of a partial collapse mechanism but does not assure a collapse mechanism of global type because it isn't a rigorous application of the capacity design's criteria. Even if different strategies have been developed in order to improve the seismic behaviour of concrete moment resisting frames [6-18], considering also the influence of the soil conditions [19], a more sophisticated design procedure has been developed starting form an idea presented in 1997 [20] with reference to steel moment resisting frames. Starting from this first work, the "Theory of Plastic Mechanism Control" (TPMC) has been obtained as a powerful tool for the seismic design. In particular, it consists on the extension of the kinematic theorem of plastic collapse to the concept of mechanism equilibrium curve. This design approach was successively extended to a large variety of structural typologies [21-37]. In 2015 TPMC was developed also with reference to the reinforced concrete frames [38-40].

The strength of this method is its simplicity that will be emphasized by means of a worked example. The aim of this paper is to show the comparison of this methodology with that recommended by the codes in term of dissipation capacity. To this purpose a static inelastic analysis has been carried out for both design procedures.

2 DESIGN ACCORDING MODERN SEISMIC CODES

The Eurocode 8 suggest to design according to the hierarchy of strengths of the various structural components, in particular according to the beam to column hierarchy criterion.

In multi-storey buildings, formation of a soft storey plastic mechanism shall be prevented, as such a mechanism might entail excessive local ductility demands in the columns of the soft storey. To satisfy that requirement, in frame buildings with two or more storeys, the sum of the design values of resistance moments of the column framing the joint must be greater than the sum of the design values of the moments of resistance of the beams framing the same joint, multiplied by an over strength factor (γ_{Rd} = 1,3).

$$\sum M_{Rc} \geq \gamma_{Rd} \sum M_{Rb} \tag{1}$$

Exception is made for the roof level.

Unfortunately this method, based on a simple joint equilibrium, in most cases cannot achieve the design goal i.e. the development of a global type mechanism.

3 THEORY OF PLASTIC MECHANISM CONTROL FOR MRC FRAMES

To guarantee an accurate design method able to assure the development of a collapse mechanism of global type, in 1997, has been presented a new rigorous methodology "Theory of Plastic Mechanism Control" [20], indicated in the following with the acronym TPMC.

It is based on the upper bound theorem of plastic collapse extended to the concept of mechanism equilibrium curve. According to the theory of limit analysis, the assumption of a rigid-plastic behavior of the structure until the development of a collapse mechanism is made.

Given the above, in general, the structures can exhibit three main collapse mechanism typologies: these mechanisms, depicted in Figure 1, are to be considered undesired because they do not involve all the dissipative zones.

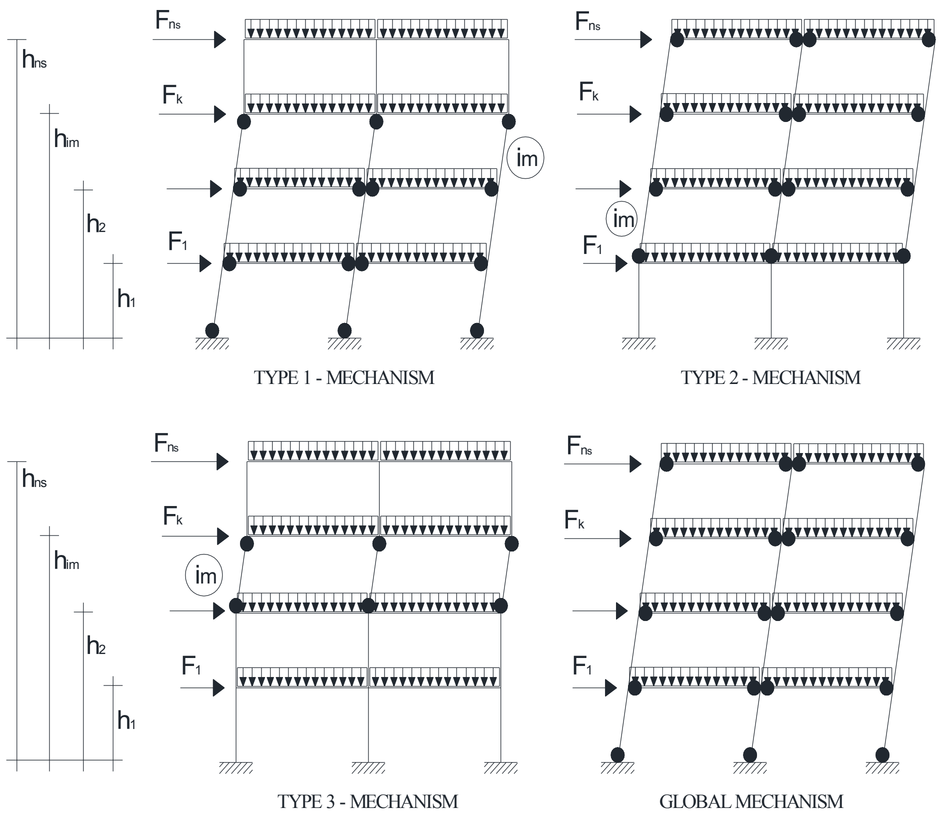

Figure 1: Collapse mechanism typologies

The TPMC is able to assure that plastic hinges develop only at beam ends while all the columns remain in elastic range with the only exception of the base sections at first storey columns. In this method is of paramount importance the introduction of the concept of linearized mechanism equilibrium curve for each considered mechanism.

The mathematical expression of this curve is represented by the Eq. (2).

$$\alpha = \alpha_0 - \gamma\delta \tag{2}$$

where α_0 is the kinematically admissible multiplier of horizontal forces and γ is the slope of the mechanism equilibrium curve. For any given collapse mechanism, according to a kinematic approach, the mechanism equilibrium curve can be easily derived by equating the external to the internal work.

However, the simple application of the upper bound theorem of plastic collapse is not sufficient to assure the desired collapse mechanism because the high horizontal displacements occur before the complete development of the kinematic mechanism. These displacements give rise to significant second order effects which cannot be neglected in the seismic design of structures. In order to account for second-order effects, the external second-order work due to vertical load is also evaluated.

For the better comprehension of the adopted notation, reference is made to Table 1.

n_c	number of columns	$M_{c,i,im}$	plastic moment of the i-th column at i_m-th storey
n_b	number of bays	$M_{c,im} = \sum_{i=1}^{n_c} M_{c,i,im}$	sum of plastic moments of columns at i_m-th storey
n_s	number of storeys	$M_V = \sum_{k=1}^{n_s} V_k h_k$	second-order work due to vertical loads in global mechanism
i_m	index of mechanism	$M_F = \sum_{k=1}^{n_s} F_k h_k$	external work due to horizontal forces in the global mechanism
H_o	sum of the interstorey heights of the storeys involved by the generic mechanism	$F = \sum_{k=1}^{n_s} F_k$	sum of the horizontal forces
h_k	height of the k-th storey (with k=1, 2, .., n_s)	$M_{b.jk}$	plastic design resistance of beam at j-th bay of the k-th storey
e	index of beam ends (e=L=left end, e=R=right end)	$M_{b,Rd,e} = \sum_{k=1}^{n_s} \sum_{j=1}^{n_b} M_{b.jk}$	sum of the plastic design resistances of beam ends (for e end) in the global mechanism

Table 1: Notation

In the following will be reported the main relations; for complete equations of this method is possible to refer to previously, specific paper [39].

Therefore, the application of the virtual work principle provides the kinematically admissible multiplier that, for LR (from Left to Right) and RL (from Right to Left) earthquake, can be written as reported in Eqs. (3) and (4), respectively.

$$\alpha_{0,LR}^{(g)} = \frac{\left[M_{c,1} + M_{b,Rd,L}^{+} + M_{b,Rd,R}^{-}\right]}{M_F} \tag{3}$$

$$\alpha_{0,RL}^{(g)} = \frac{\left[M_{c,1} + M_{b,Rd,L}^{-} + M_{b,Rd,R}^{+}\right]}{M_F} \tag{4}$$

The sign "+" is used to indicate that the bending moment produces tension in lower fibers of the beam section and compression in the upper fibers, while the sign "–" is used to indicate the opposite case.

In order to compute the slope of the mechanism equilibrium curve γ, it is necessary to evaluate the second-order work due to vertical loads.

In the case of global mechanism it is given by Eq. (5).

$$\gamma^{(g)} = \frac{\frac{1}{H_o} M_V}{M_F} = \frac{\frac{1}{h_{n_s}} M_V}{M_F} \tag{5}$$

Is possible to obtain a mechanism equilibrium curve for each considered mechanism.

In particular, for the $\mathrm{i_m}$-th mechanism $(i_m = 1,2,\ldots,n_s)$ of the t-th mechanism typology $(t = 1,2,3)$, the application of kinematic theorem of plastic collapse provides through the Eq. (6) for LR earthquake and the Eq. (7) for RL earthquake, the evaluation of the mechanism equilibrium curves.

$$\alpha_{im,LR}^{(t)} = \alpha_{0,im,LR}^{(t)} - \gamma_{im}^{(t)} \tag{6}$$

$$\alpha_{im,RL}^{(t)} = \alpha_{0,im,RL}^{(t)} - \gamma_{im}^{(t)} \tag{7}$$

$\alpha_{0,im}^{(t)}$ and $\gamma_{im}^{(t)}$ represent, respectively, the kinematically admissible multiplier and the slope of mechanism equilibrium curve of the i_m-th mechanism of the t-th mechanism typology.

Beam sections are designed to resist vertical loads so, the unknowns of the design problem, are the column sections. They could be determined by means of design conditions i.e. the equilibrium curve corresponding to the global mechanism has to lie below those corresponding to all other mechanisms for each value of the displacement δ, up to a selected ultimate displacement δ_u (Figure 2).

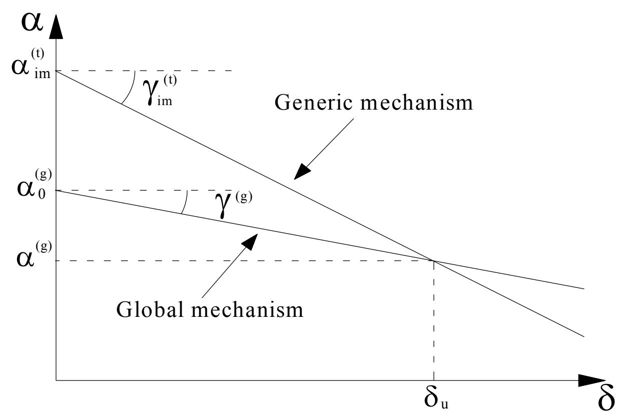

Figure 2: Design condition

Therefore, there are $6n_s$ design conditions to be satisfied for a structural scheme having n_s storeys. With reference to i_m-th mechanism of type-1, the kinematically admissible multiplier of seismic horizontal forces is given, for LR earthquake, by:

$$\alpha^{(1)}_{0,i_{m,LR}} = \frac{M_{c,1} + \sum_{k=1}^{i_m-1}\sum_{j=1}^{n_b} M^{+}_{b.jk,L} + \sum_{k=1}^{i_m-1}\sum_{j=1}^{n_b} M^{-}_{b.jk,R} + M_{c,im}}{\sum_{k=1}^{i_m} F_k h_k + h_{i_m}\sum_{k=i_m+1}^{n_s} F_k} \tag{8}$$

and for RL earthquake by:

$$\alpha^{(1)}_{0,i_{m,RL}} = \frac{M_{c,1} + \sum_{k=1}^{i_m-1}\sum_{j=1}^{n_b} M^{-}_{b.jk,L} + \sum_{k=1}^{i_m-1}\sum_{j=1}^{n_b} M^{+}_{b.jk,R} + M_{c,im}}{\sum_{k=1}^{i_m} F_k h_k + h_{i_m}\sum_{k=i_m+1}^{n_s} F_k} \tag{9}$$

while the slope of the mechanism equilibrium curve is the same for both directions:

$$\gamma^{(1)}_{i_m} = \frac{1}{h_{i_m}} \frac{\sum_{k=1}^{i_m} V_k h_k + h_{i_m}\sum_{k=i_m+1}^{n_s} V_k}{\sum_{k=1}^{i_m} F_k h_k + h_{i_m}\sum_{k=i_m+1}^{n_s} F_k} \tag{10}$$

With reference to i_m-th mechanism of type-2 the kinematically admissible multiplier of seismic horizontal forces is given, for LR earthquake, by:

$$\alpha^{(2)}_{0,i_{m,LR}} = \frac{M_{c,im} + \sum_{k=i_m}^{n_s}\sum_{j=1}^{n_b} M^{+}_{b.jk,L} + \sum_{k=i_m}^{n_s}\sum_{j=1}^{n_b} M^{-}_{b.jk,R}}{\sum_{k=i_m}^{n_s} F_k\left(h_k - h_{i_m-1}\right)} \tag{11}$$

and for RL earthquake by:

$$\alpha^{(2)}_{0,i_{m,RL}} = \frac{M_{c,im} + \sum_{k=i_m}^{n_s}\sum_{j=1}^{n_b} M^{-}_{b.jk,L} + \sum_{k=i_m}^{n_s}\sum_{j=1}^{n_b} M^{+}_{b.jk,R}}{\sum_{k=i_m}^{n_s} F_k\left(h_k - h_{i_m-1}\right)} \tag{12}$$

while the slope of the mechanism equilibrium curve is:

$$\gamma^{(2)}_{i_m} = \frac{1}{h_{n_s} - h_{i_m-1}} \frac{\sum_{k=i_m}^{n_s} V_k\left(h_k - h_{i_m-1}\right)}{\sum_{k=i_m}^{n_s} F_k\left(h_k - h_{i_m-1}\right)} \tag{13}$$

It is useful to note that, for $i_m = 1$ Eqs. (11), (12) and (13) are coincident with Eq. (3), (4) and (5) respectively, because in such case the mechanism is coincident with the global one. In addition, these relations for $i_m = 1$ include the term $h_{i_m-1} = h_0$ which is to be assumed equal to zero. Finally, with reference to i_m-th mechanism of type-3, the kinematically admissible multiplier of horizontal forces, is given by:

$$\alpha^{(3)}_{0,i_m} = \frac{2M_{c,im}}{\left(h_{i_m} - h_{i_m-1}\right)\sum_{k=i_m}^{n_s} F_k} \tag{14}$$

In this case the expression is the same for both directions of earthquake because the beams are not involved in this collapse mechanism.
In addition, the corresponding slope of the mechanism equilibrium curve is given by:

$$\gamma_{i_m}^{(3)} = \frac{\sum_{k=i_m}^{n_s} V_k}{\left(h_{i_m} - h_{i_m-1}\right)\sum_{k=i_m}^{n_s} F_k} \tag{15}$$

It is important to underline that, for any given geometry of the structural system, the slope of mechanism equilibrium curve attains its minimum value when the global type mechanism is developed. In fact, it is easy to check that $\gamma^{(g)}$, which is equal to $\gamma_1^{(2)}$, is always the minimum value among all the $\gamma_{i_m}^{(t)}$.

4 WORKED EXAMPLE

The structural scheme in Figure 3 consists of five-bay six-storey moment resisting frame.

The interstorey height is equal to 3m. The characteristic values of the vertical loads acting on the beams are equal to 10.7 kN/m and 4 kN/m for permanent (G_k) and live (Q_k) actions, respectively. The structural materials adopted are concrete C25/30 and reinforcement of steel grade B450C.

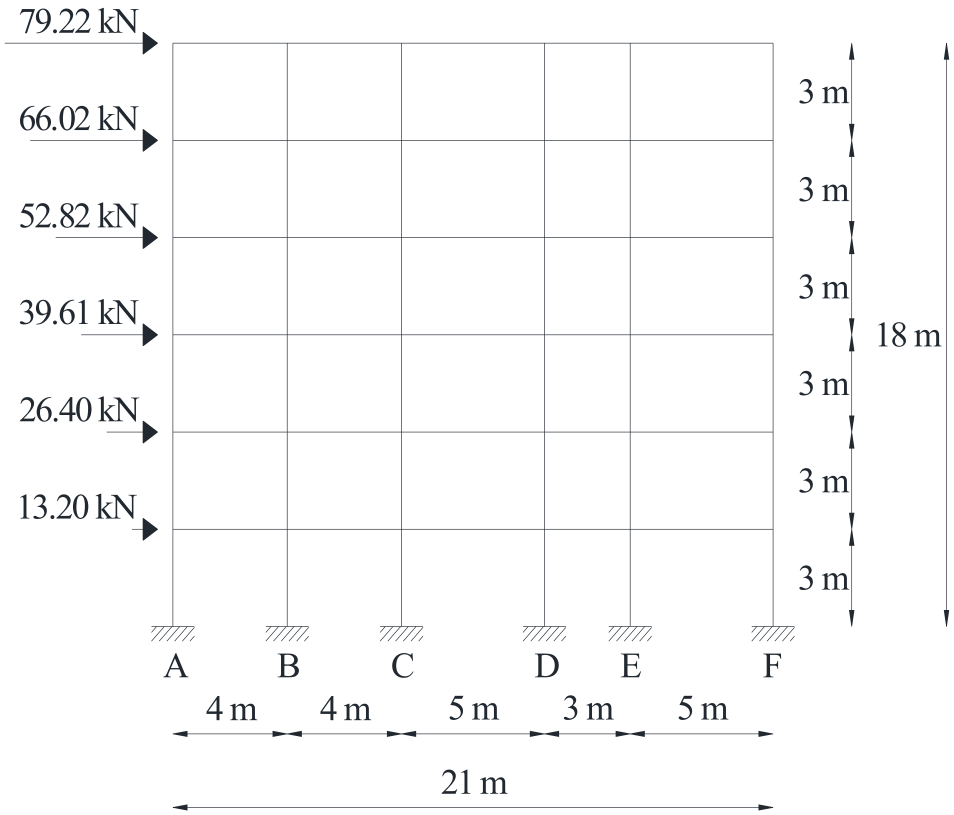

Figure 3: Structural scheme of the designed frame

According to Eurocode 8, the value of the period of vibration used for preliminary design is:

$$T = 0.075\, H^{3/4} = 0.075 \cdot 18^{3/4} \approx 0.65\, s \tag{16}$$

where H is the total height of the frame.

With reference to the design spectrum for stiff soil conditions (soil class A of Eurocode 8) and by assuming a behaviour factor q equal to 3.9, the horizontal seismic forces are those depicted in Figure 3. The design requirements are that the beams should remain elastic and the columns should provide the required ductility by plastic hinges. In this specific case, has been considered a DCM (medium ductility) building.

Regarding the structural modelling, the mechanical non-linearities have been concentrated at beam and column ends by means of plastic hinge elements. The constitutive law of such plastic hinge is provided by a rigid plastic moment-rotation curve. The type of hinge depends on the element considered i.e. by its internal action. In fact, for the beams and the columns M3 and P-M3 hinge type have been considered, respectively. In case of P-M3 hinge type, the interaction domain P-M has been evaluated for each column and used in SAP2000 computer program [41].

In order to evaluate the inelastic behaviour of the designed structure, a push-over static analysis, has been conducted for both design procedures.

4.1 Results of the TPMC design procedure

In the TPMC procedure the first step is the selection of the maximum top sway displacement up to which the global mechanism has to be assured.

$$\delta_u = \theta_u \cdot h_{ns} = 0.01 \cdot h_{ns} = 0.01 \cdot 18 = 0.18\, m \tag{17}$$

The reinforcement at the beam ends is the same at all the storeys; the values are reported in Table 2.

PART OF SECTION	L_{AB} = 4m		L_{BC} = 4m		L_{CD} = 5m		L_{DE} = 3m		L_{EF} = 5m	
	e = L	e = R	e = L	e = R	e = L	e = R	e = L	e = R	e = L	e = R
Top	4 ϕ16	4 ϕ16	4 ϕ16	4 ϕ16	4 ϕ16	4 ϕ16	4 ϕ16	4 ϕ16	4 ϕ16	4 ϕ16
Bottom	4 ϕ16	4 ϕ16	3 ϕ16	4 ϕ16	4 ϕ16	4 ϕ16	3 ϕ16	4 ϕ16	3 ϕ16	4 ϕ16

Table 2: Reinforcement at the beam ends (L = left and R = right).

In Table 3 the axial forces N_q, due to vertical loads, for both directions of earthquake, are reported for each storey and for each column.

Storey	Column A		Column B		Column C		Column D		Column E		Column F	
i_m	$N_{q,LR}$ [kN]	$N_{q,RL}$ [kN]	$N_{q,LR}$ [kN]	$N_{q,RL}$ [kN]	$N_{q,LR}$ [kN]	$N_{q,RL}$ [kN]	$N_{q,LR}$ [kN]	$N_{q,RL}$ [kN]	$N_{q,LR}$ [kN]	$N_{q,RL}$ [kN]	$N_{q,LR}$ [kN]	$N_{q,RL}$ [kN]
1	143.2	143.2	286.5	286.5	322.3	322.3	286.5	286.5	286.5	286.5	179.1	179.1
2	119.4	119.4	238.8	238.8	268.6	268.6	238.8	238.8	238.8	238.8	149.2	149.2

3	95.52	95.52	191.0	191.0	214.9	214.9	191.0	191.0	191.0	191.0	119.4	119.4
4	71.64	71.64	143.2	143.2	161.1	161.1	143.2	143.2	143.2	143.2	89.55	89.55
5	47.76	47.76	95.52	95.52	107.4	107.4	95.52	95.52	95.52	95.52	59.70	59.70
6	23.88	23.88	47.76	47.76	53.73	53.73	47.76	47.76	47.76	47.76	29.85	29.85

Table 3: Axial forces acting in the columns related to the vertical loads for both directions of earthquake

In Table 4 are reported the axial loads related to the shear actions due to the plastic hinges, developed at the beam ends $N_{M,LR}$ ($N_{M,RL}$), for LR earthquake (for RL earthquake).

Storey	Column A		Column B		Column C		Column D		Column E		Column F	
i_m	$N_{M,LR}$ [kN]	$N_{M,RL}$ [kN]	$N_{M,LR}$ [kN]	$N_{M,RL}$ [kN]	$N_{M,LR}$ [kN]	$N_{M,RL}$ [kN]	$N_{M,LR}$ [kN]	$N_{M,RL}$ [kN]	$N_{M,LR}$ [kN]	$N_{M,RL}$ [kN]	$N_{M,LR}$ [kN]	$N_{M,RL}$ [kN]
1	-326.62	326.62	40.08	-12.00	25.24	-65.21	-120.76	174.04	152.82	-174.14	229.23	-261.20
2	-272.18	272.18	33.40	-10.00	21.04	-54.34	-100.63	145.03	127.35	-145.11	191.03	-217.67
3	-217.75	217.75	26.72	-8.00	16.83	-43.47	-80.51	116.03	101.88	-116.09	152.82	-174.14
4	-163.31	163.31	20.04	-6.00	12.62	-32.60	-60.38	87.02	76.41	-87.07	114.62	-130.60
5	-108.87	108.87	13.36	-4.00	8.41	-21.74	-40.25	58.01	50.94	-58.05	76.41	-87.07
6	-54.44	54.44	6.68	-2.00	4.21	-10.87	-20.13	29.01	25.47	-29.02	38.21	-45.53

Table 4: Axial forces acting in the columns related to the shear actions for both directions of earthquake

Applied the design procedure, prevented the technological condition (checking that the column section does not increase along the height) and optimized the reinforcement in the column; the definitively values of required bending moment for each column $M_{c,i,1}$, the section, the upper and lower reinforcement and the axial force, for both directions of the earthquake are reported in Table 5.

Storey	Column	$M_{c,im,LR}$ [kNm]	$M_{c,im,RL}$ [kNm]	b x h	$A_s = A'_s$	N_{LR} [kN]	N_{RL} [kN]
1°	A	153.63	164.05	30x60	3 Φ 28	-183.34	469.90
	B			50x30	2 Φ 28	326.64	286.44
	C			50x30	2 Φ 28	347.62	257.17
	D			50x30	2 Φ 28	165.80	460.60
	E			50x30	2 Φ 28	439.38	112.42
	F			30x70	3 Φ 28	408.33	-82.10
2°	A	178.87	202.80	30x60	4 Φ 28	-152.79	391.58
	B			50x30	2 Φ 28	272.2	238.70
	C			50x30	2 Φ 28	289.69	214.31
	D			50x30	2 Φ 28	138.17	383.83
	E			50x30	2 Φ 28	366.15	93.69
	F			30x70	4 Φ 28	340.28	-68.42
3°	A	250.61	276.58	30x60	5 Φ 28	-122.23	313.27
	B			50x30	3 Φ 28	217.76	190.96
	C			50x30	3 Φ 28	231.75	171.45
	D			50x30	3 Φ 28	110.53	307.07
	E			50x30	3 Φ 28	292.92	74.95
	F			30x70	5 Φ 28	272.22	-54.74

4°	A	281.75	307.20	30x60	4 Φ 32	-91.67	234.95
	B			50x30	3 Φ 28	163.32	143.22
	C			50x30	3 Φ 28	173.81	128.59
	D			50x30	3 Φ 28	82.90	230.30
	E			50x30	3 Φ 28	219.69	56.21
	F			30x70	5 Φ 32	204.17	-41.05
5°	A	258.75	280.27	30x60	5 Φ 28	-61.11	156.63
	B			50x30	3 Φ 28	108.88	95.48
	C			50x30	3 Φ 28	115.87	85.72
	D			50x30	3 Φ 28	55.27	153.53
	E			50x30	3 Φ 28	146.46	37.47
	F			30x60	5 Φ 28	136.11	-27.37
6°	A	168.09	181.41	30x50	3 Φ 28	-30.56	78.32
	B			40x30	3 Φ 28	54.44	47.74
	C			40x30	3 Φ 28	57.94	42.86
	D			40x30	3 Φ 28	27.63	76.77
	E			40x30	3 Φ 28	73.23	18.74
	F			30x50	4 Φ 28	68.06	-13.68

Table 5: Design of column sections at each storey for earthquake

In order to validate the designed frame a pushover analysis has been carried out in SAP2000 computer program.

The results of the push-over analysis are mainly constituted by base shear – top sway displacement curve, for both direction of earthquake, that are depicted in Figure 4.

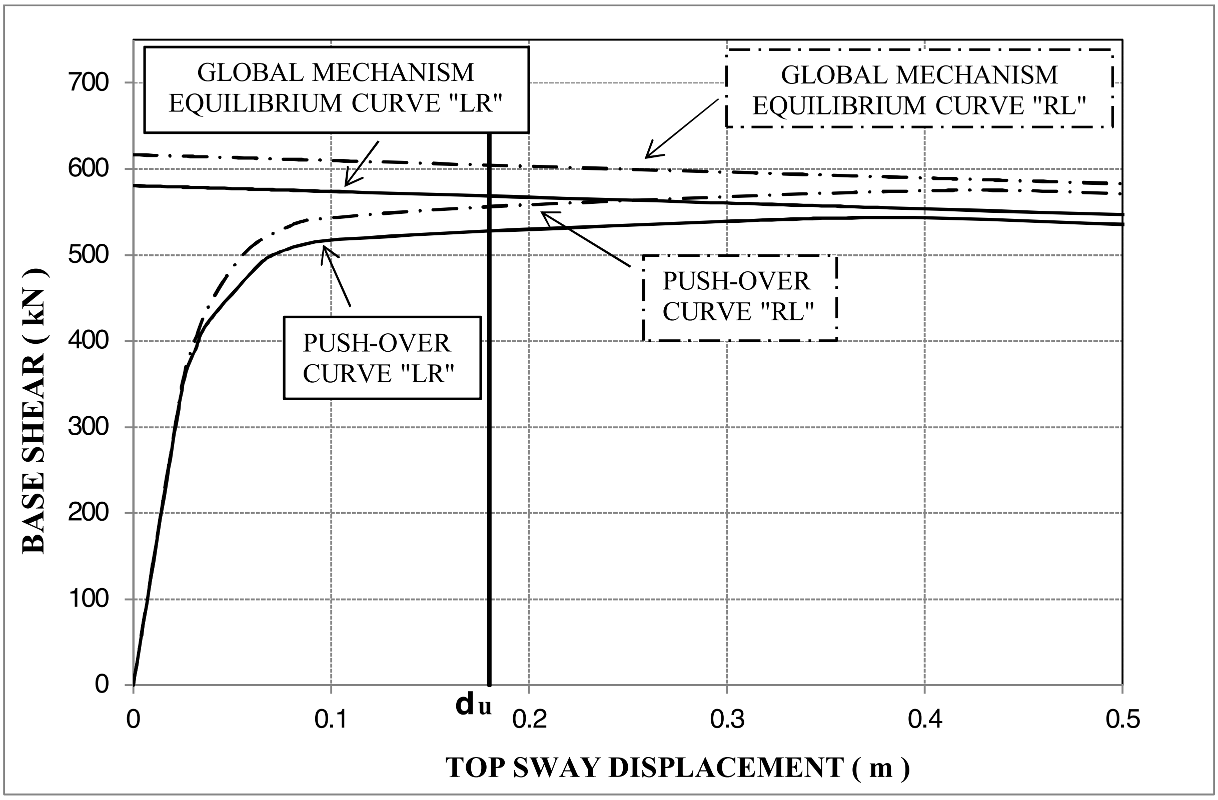

Figure 4: Overlap of the push-over curve with the global mechanism equilibrium curve.

As is possible to observe the seismic response of the designed frame confirms the development of the desired collapse mechanism typology.

In the same figure the mechanism equilibrium curve of global mechanism, both for LR and RL earthquake, is very close and parallel to the corresponding push-over curve. In the following has been reported the equations of these curves.

$$\alpha_{LR}^{(g)} = 2.0938 - 0.002435\,\delta \tag{18}$$

$$\alpha_{RL}^{(g)} = 2.2233 - 0.002435\,\delta \tag{19}$$

A further confirmation, of the fulfilment of the design objective, are reported in Figure 5.

The distribution of the pattern of yielding, developed at the occurrence of the design ultimate displacement, corresponding to both seismic direction is, in fact, in perfect agreement with the global mechanism.

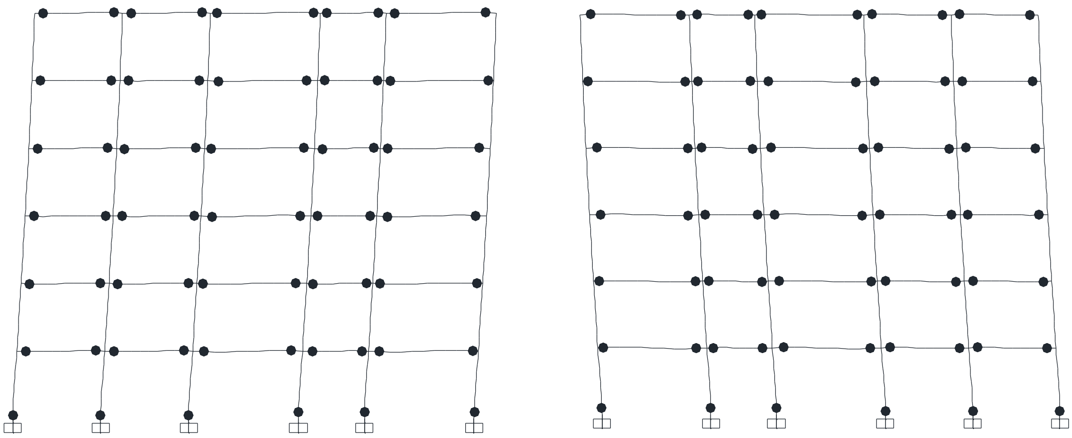

Figure 5: Pattern of yielding of the designed frame at $\delta = \delta_u$ for LR and RL earthquake direction.

An appropriate design will require to concentrate the attention on the control and limitation of displacements that could occur during the earthquake.

According to the limit reported in the Eurocode 8, the interstorey drift have been checked; in particular the considered limit refers to buildings having non structural elements of brittle materials attached to the structure:

$$d_r\,\nu \leq 0.005\,\mathrm{h} \tag{20}$$

In Table 7 the final results are reported.

STOREY	d_s [cm]	d_r [cm]	ν	$d_r\,\nu$	0.005 h
6°	9.4871	1.0253	0.5	0.5126	1.5
5°	8.4617	1.2506		0.6253	1.5
4°	7.2111	1.6513		0.8256	1.5
3°	5.5598	1.9617		0.9808	1.5
2°	3.5980	2.0957		1.0478	1.5
1°	1.5022	1.5022		0.7511	1.5

Table 6: Limitation of interstorey drift for TPMC frame.

If this serviceability requirement is not verified, the structural stiffness can be improved by increasing the beam sections or the ultimate design displacement. In fact, in both cases the final results will be a more rigid structure with respect to the one obtained in the worked example herein presented.

4.2 Frame designed in accordance with Eurocode 8

The same five-bay six-storey moment resisting frame, previously presented, has been designed even in accordance with Eurocode 8. It is clear that the formation of soft-storey, whereby plastic hinges can develop simultaneously at the top and bottom end of columns in a storey, is unacceptable in multi-storey framed buildings.

Therefore columns of storeys must be stronger than the beams that frame into them.

Designed the structure, applying the beam-column hierarchy criterion, the definitively values of the section, the upper and lower reinforcement are reported in Table 7.

STOREY	Column	b x h	$A_s = A'_s$
1°	A	30x30	4 Φ 16
	B	40x30	4 Φ 16
	C	40x30	4 Φ 16
	D	40x30	4 Φ 16
	E	40x30	4 Φ 16
	F	30x30	4 Φ 16
2°	A	30x30	5 Φ 16
	B	40x30	5 Φ 16
	C	40x30	4 Φ 16
	D	40x30	5 Φ 16
	E	40x30	5 Φ 16
	F	30x30	4 Φ 16
3°	A	30x30	5 Φ 16
	B	40x30	5 Φ 16
	C	40x30	5 Φ 16
	D	40x30	5 Φ 16
	E	40x30	5 Φ 16
	F	30x30	5 Φ 16
4°	A	30x30	5 Φ 16
	B	40x30	6 Φ 16
	C	40x30	6 Φ 16
	D	40x30	6 Φ 16
	E	40x30	6 Φ 16
	F	30x30	5 Φ 16
5°	A	30x30	5 Φ 20
	B	40x30	5 Φ 20
	C	40x30	5 Φ 20
	D	40x30	5 Φ 20
	E	40x30	5 Φ 20
	F	30x30	5 Φ 20

6°	A	30x30	5 Φ 20
	B	40x30	5 Φ 24
	C	40x30	5 Φ 24
	D	40x30	5 Φ 24
	E	40x30	5 Φ 24
	F	30x30	5 Φ 20

Table 7: Design of column sections at each storey for earthquake

In order to validate the designed frame, in Figure 6 are reported the pattern of yielding corresponding to both seismic direction, obtained through a pushover analysis.

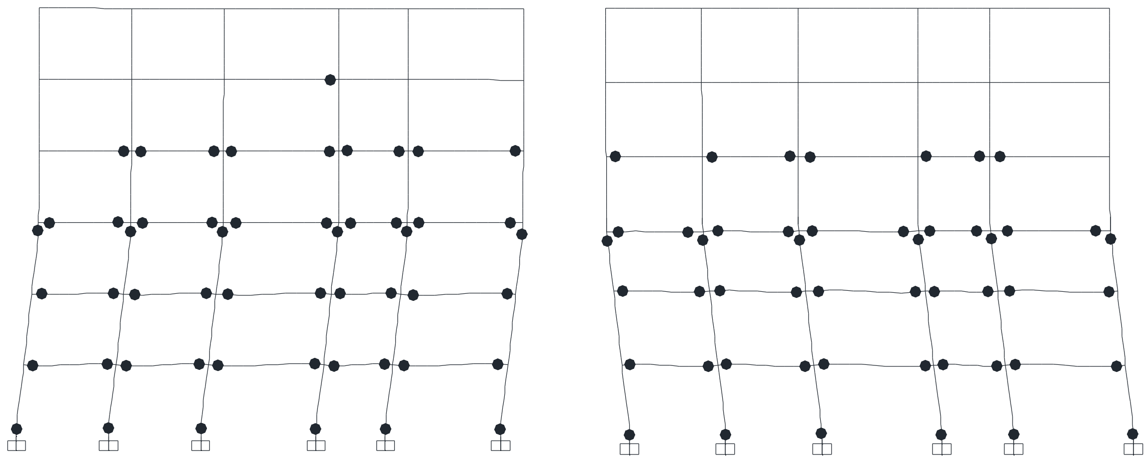

Figure 6: Pattern of yielding in the frame designed in according with Eurocode 8, for both earthquake directions.

As is possible to observe from Figure 6, the structure exhibits a Type 1 collapse mechanism, extended to the third storey of the frame.

Therefore, the fulfilment of the strong-column/weak-beam principle is fundamental to avoid soft-storey mechanism but it is no sufficient to guarantee the development of the maximum number of plastic hinges. Even in this case is very important to verify the serviceability requirements, expressed through the (20). In Table 8 the final results are reported.

STOREY	d_s [mm]	d_r [mm]	v	$d_r\, v$	0.005 h
6°	9.7710	0.7558	0.5	0.3779	1.5
5°	9.0151	1.3036		0.6518	1.5
4°	7.7114	1.7522		0.8761	1.5
3°	5.9591	2.0783		1.0391	1.5
2°	3.8808	2.2281		1.1140	1.5
1°	1.6526	1.6526		0.8263	1.5

Table 8: Limitation of interstorey drift for EC8 frame.

1. CONCLUSIONS

In this paper a comparison between two design procedure, for reinforced concrete moment resisting frames, has been presented.

The procedure called “Theory of Plastic Mechanism Control”, based on the extension of the kinematic theorem of plastic collapse to the concept of mechanism equilibrium curve, allows to evaluate the sum of plastic moments of the columns required at each storey in order to develop a collapse mechanism of global type.

Beams are designed in order to bear external loads, while columns are designed according to the maximum internal actions transmitted by the dissipative zones, in this way TPMC constitutes a rigorous application of the capacity design principles.

On the other hand has been presented the design carried out through the beam-column hierarchy criterion, commonly suggested by seismic codes.

Unfortunately the beam-column hierarchy criterion is not sufficient to assure the development of the flexural plastic hinges at the ends of all the beams, combined with the only base column.

In conclusion, the closed form solution of the design conditions that makes the TPMC procedure very easy to be applied even by means of hand calculations, it could also be suggested for code purpose to definitely solve the problem of collapse mechanism control whose importance in seismic design is universally recognised. Therefore, is important to underline that the TPMC method is in perfect agreement with that recommended by the code. Finally, it is important to underline that also the joists influence should be accounted for [42]. This is a possible development of future research.

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