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MECHANISM BASED ASSESSMENT OF DAMAGED BUILDING'S RESIDUAL CAPACITY

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Abstract. *Seismic behavior of damaged buildings may be expressed as a function of their REsidual Capacity (REC). The residual capacity* REC_{Sa} *is defined as the minimum spectral acceleration (at the period* T_{eq} *of the equivalent SDOF) corresponding to building collapse. When referring to peak ground acceleration* a_g *as damaging intensity parameter,* REC_{ag} *is defined as the minimum anchoring peak ground acceleration such as to determine building collapse. For a given spectral shape,* REC_{ag} *corresponds to* REC_{Sa} *scaled by the spectral amplification factor for* T_{eq}*.* REC_{Sa} *and* REC_{ag}*, generally indicated as REC, lower with increasing damage level in buildings; hence REC may be very useful in estimating the post-seismic building safety. In a recent work [1] it has been shown how it is possible to derive REC (*REC_{Sa} *and* REC_{ag}*) through Pushover Analyses (PA), where a suitable modification of plastic hinges for damaged elements is applied. The applicability of PA for damaged structures is verified in [2] by comparison of the PA results with those on nonlinear time-history analyses. On the other hand, it is unrealistic that in the aftermath of an earthquake, when the assessment of building safety has to be performed in an emergency situation, there would be time for the execution of detailed nonlinear analyses. Acknowledging the need for easier and faster evaluation tools, in [3] a simplified MEChanism based method (MEC) for evaluating the building REC was preliminary tested. The present work extends the comparison of the results (in terms of REC), that could be obtained by PA and MEC analyses, considering a number of Reinforced Concrete (RC) frames building typologies. Moreover, by adopting the MEC approach, the possible variation of REC as a function of seismic demand is investigated. The simplified method can be used to explore the possible ranges of REC variation for RC building classes considering the anticipated mechanism formation after an earthquake; in addition, it could be used in the post-seismic phase for estimating the REC of damaged buildings having undergone identifiable plastic mechanisms and for fast assessment of damage-dependent collapse fragility curves.*

1 INTRODUCTION

One of the most controversial problem in the aftermath of damaging earthquakes is the lack of agreed and transparent policies for acceptable levels of safety. With the purpose of facilitating the assessment process in the post-earthquake, it was evidenced the need of a method for measuring structural capacity for damaged buildings, to be compared with that related to the undamaged state [4]. In the ATC-43 project [5] the available instruments and methods for seismic analyses of damaged buildings were analyzed: by adopting pushover analysis as a nonlinear analysis tool, the behavior of damaged buildings may be simulated with suitable modification of plastic hinges for damaged elements. Taking into account the evaluation approach proposed in [5], and applied in [6, 7] for some steel buildings, the authors developed an assessment procedure that allows to express the seismic behavior of damaged RC buildings as a function of their REsidual Capacity (REC), that is a measure of seismic capacity, reduced due to damage [1]. As it will be explained in the paragraph 2.1, the REC may be evaluated via Pushover Analysis (PA) on a suitably modified lumped plasticity building model. On the other hand, acknowledging the need for easier and faster evaluation tools, in [3] a simplified method for evaluating the building REC was preliminary tested. This paper continues the study initiated in [3] extending the comparison of the results of pushover analyses with those of MEChanism based analyses (MEC) for a number of RC frames representative of existing building typologies in the Mediterranean area. In particular, paragraph 2.2 explains the main features of the MEC approach that is adopted, while section 3, after presentation of the studied RC frame buildings, compares the results of both type of analyses (PA and MEC) for the structures in their “intact” and “damaged” states. Next, in section 4 a possible application of MEC assessment for evaluation of variation of buildings residual capacity with increasing seismic demand (in terms of global ductility demand) is presented. As it will be shown (section 5) the proposed procedure may be applied for investigation of damage-dependent vulnerability and collapse fragility curves derivation for classes of buildings whose plastic mechanism is identified.

2 EVALUATION OF BUILDING’S RESIDUAL CAPACITY

The residual capacity REC is a parameter aimed at representing the building seismic capacity (up to collapse) in terms of a spectral quantity. In [1] REC_{Sa} is defined as the spectral acceleration (at period T_{eq} of the Single Degree Of Freedom SDOF system equivalent to the real structure) corresponding to suitably defined building collapse condition. Moreover, given the convenience of direct estimation of peak ground acceleration, a_g, as a damage-intensity parameter, the residual capacity is evaluated also in terms of a_g: given the spectral shape, REC_{ag} is the minimum anchoring peak ground acceleration such as to determine building collapse and corresponds to REC_{Sa} scaled by the spectral amplification factor for T_{eq}.

By way of example, with reference to an EC8 spectral shape and considering a system with $T_C<T_{eq}<T_D$, the following relation applies:

$$REC_{ag} = \frac{REC_{Sa}}{(S \cdot \eta \cdot 2.5)} \cdot \left(\frac{T_{eq}}{T_C} \right) \tag{1}$$

2.1 Pushover based approach

The flowchart in Figure 1, referring to framed structures, illustrates the basic steps needed in order to determine REC_{Sa} (or REC_{ag}) for the intact structure as well as considering different damaged states possibly caused by a main-shock. The method is briefly described next; for more detailed description the interested reader may refer to [1].

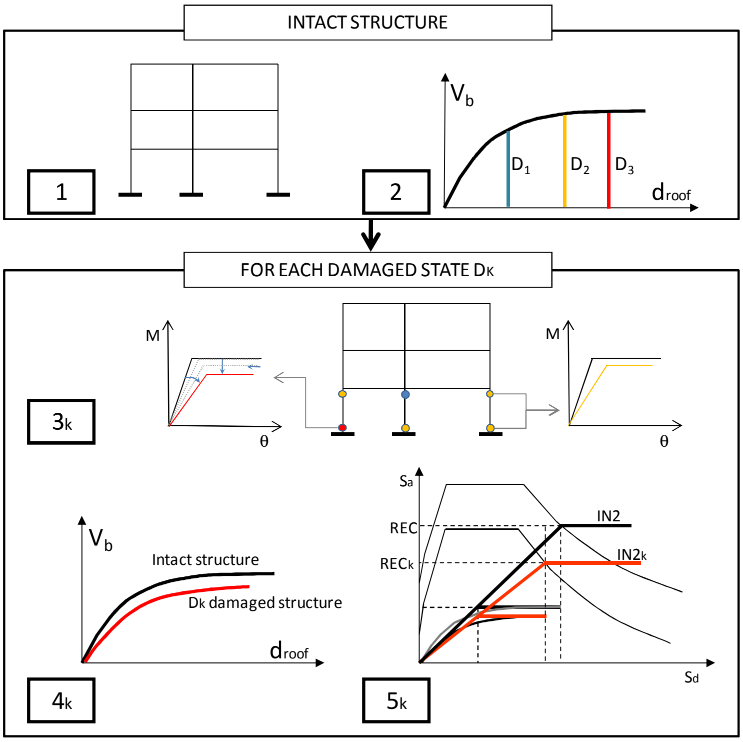


Figure 1: Flowchart illustrating the basic steps of the method for framed structures

In order to determine REC_{Sa} (or REC_{ag}) it is necessary to find the relationship between the seismic demand, expressed in terms of displacement, and the seismic intensity, that may be represented by the spectral acceleration $S_a(T_{eq})$ or by peak ground acceleration a_g. To this end, the incremental N2 method (IN2) [8] may be applied, that allows the construction, with reference to an equivalent SDOF obtained based on the Pushover Analysis (PA) of the building, of the curve approximately relating seismic demand to seismic intensity. In the simpler, but very common, case of applicability of the principle of equal displacement rule ($T_{eq} \gtrsim T_c$) the IN2 curve is a straight line from the origin up to collapse point, the only point that needs to be determined and that corresponds to REC_{Sa}. It is easily verifiable that, in the hypothesis of the equal displacement rule, the REC_{Sa} may be simply calculated as the product of the base shear coefficient C_b and the displacement capacity in terms of ductility μ_{cap}:

$$REC_{Sa} = C_b \cdot \mu_{cap} \qquad \text{for } T_{eq} \geq T_c \tag{2}$$

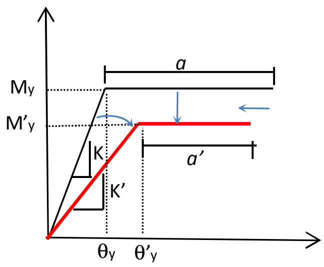
Analogously, it can be verified that, for $T_{eq}<T_c$, the residual capacity may still be associated to C_b and μ_{cap}. Indeed, adopting the R-μ-T relation introduced in [9], and given that for a seismic intensity bringing the structure to collapse R equals the ratio REC_{Sa}/C_b:

$$REC_{Sa} = C_b \cdot (\mu_{cap} - 1) \cdot \frac{T_{eq}}{T_c} + 1 \qquad \text{for } T_{eq}<T_c \tag{3}$$

When referring to the structure in its undamaged state the initial REC ($REC_{Sa,0}$ or $REC_{ag,0}$) is obtained. On the other hand, after a main-shock the structure may be damaged to a global damage state D_i and the structural elements in a RC frame may have been locally subjected to a ductility demand (also corresponding to local damage level). In this study, the near collapse damage state D_3 and the moderate damage state D_2 are defined based on the assumption that the most critical element controls the state of the structure; in particular, D_3 corresponds to the first attainment of Collapse Prevention CP limit state for an element [10] and D_2 to the first attainment of 0.5 CP. For the limited damage state (D_1) it is assumed that it is attained at the Yield Displacement of the Idealized (YDI) pushover curve.

It is here noted that that the evaluation approach is very simplified, since brittle shear failures in columns or beams, that may be expected in existing under-designed buildings [11, 12], or brittle behavior of beam-columns joints [13], are not considered in this study. Further studies will have to address the influence of brittle failures on the safety loss of existing buildings.

In the hypothesis of studying the structural behavior via pushover analysis both the global damage state and the local ductility demand may be determined through the analysis. Then, the behavior of damaged buildings may be studied with nonlinear static analyses performed on a suitably modified structural model. In particular, considering the ductility demand in each of the structural elements caused by an hypothetical main-shock, the relative force-deformation or moment-rotation relationships may be modified through ductility dependent modification factors [14], see Figure 2, and pushover curves for the damaged structure may be derived [1]. Next, applying the same method used for determining $REC_{Sa,0}$ (and $REC_{ag,0}$) also the REC at damage state D_i ($REC_{Sa,i}$ or $REC_{ag,i}$) may be computed.



	stiffness	strength	Plastic rotation
intact	K	My	a
damaged	$K'=\lambda_K K$	$M_y'=\lambda_Q M_y$	$a'=a-a_d$

Figure 2: Modeling criteria for the damaged plastic hinges: the bilinear moment-rotation plastic hinge is modified with a suitable variation in the relative stiffness ($K'=\lambda_k K$), strength ($M_y'=\lambda_Q M_y$) and plastic rotation capacity ($a' = a-a_d = a-(\theta'_y - \theta_y)$ -RD = $a-(\theta_y(\lambda_Q/\lambda_k -1)$-RD), with λ stiffness or strength modification factors and residual drift (RD) of the element (for further details see [1, 13]).

2.2 Mechanism based approach

The assessment of the residual capacity *(REC)* of the structure via PA can be too computing expensive if a population of building is to be analyzed. Therefore, an approximate method, which allows to assess *REC* by simpler estimation of parameters (C_b, μ_{cap} and T_{eq}), is proposed. Indeed, by observing Eqs. (1), (2) and (3) it may be noted that, considering a system with $T_C<T_{eq}<T_D$, that is often the case for mid-rise existing RC buildings, REC_{Sa} depends on C_b and μ_{cap} of the equivalent system, while REC_{ag} varies proportionally to the product C_b μ_{cap} T_{eq}. These relationships apply either for the intact building either for the building in its generic damaged state, provided that in the latter case C_b, μ_{cap} and T_{eq} are computed accounting for the state of damage of the structural elements. Hence, estimation of these factors (C_b, μ_{cap} and T_{eq}) for different structural systems and mechanism types and for varying damage levels becomes crucial in the estimate of pre- and post-earthquake safety levels.

C_b, μ_{cap} and T_{eq} may be easily computed adopting a MEChanism based (MEC) procedure. In particular, the proposed steps of assessment for a given moment resisting frame in its "intact" state are:

1. Evaluation of probable plastic mechanism for the given frame

2. Calculation of base shear V_b and base shear coefficient C_b via simplified mechanism based formulations (see NOTATION section)
3. Calculation of yield and ultimate displacements, d^*_y and d^*_u for the equivalent SDOF system and evaluation of $\mu_{cap}= d^*_u/ d^*_y$
4. Calculation of the equivalent period T_{eq}

In order to evaluate the probable mechanism that could be expected (local soft storey mechanism, involving mainly the columns at a single storey level, or a more global one, involving also the beams) a method that is based on the assessment of the sway potential index S_i [15, 16] and the sway-demand index *SDi* [17] may be applied.

In particular, the S_i is computed based on the relative strengths of beams and columns at each storey; in case of identification of a probable local mechanism type (S_i≥0.85), the assessment of the sole S_i index gives no indication at which floor the mechanism will occur. For this reason, the concept of a sway-demand index, SD_i, is introduced in [17], comparing the amount of the shear demand at each storey (i.e. storey shear demand vs base shear demand) to the relative storey shear strength (i.e. storey shear strength vs shear strength of the first storey). The higher the SD_i, the higher the likelihood of a column-sway forming at the i^{th} storey.

Once the probable mechanism type is determined, the corresponding base shear V_b may evaluated by equilibrium relations, as suggested in [18]. For what concerns the external force distribution, two load paths are assumed, namely a linear distribution (identified as MO resembling the modal shape) and a uniform one (identified as MA resembling horizontal forces proportional to seismic masses). For example, Figure 3 depicts the system of external and internal forces that should satisfy equilibrium for two hypothesized mechanism types (k^{th} storey sway local mechanism and global mechanism) and two distributions of horizontal forces (MO and MA).

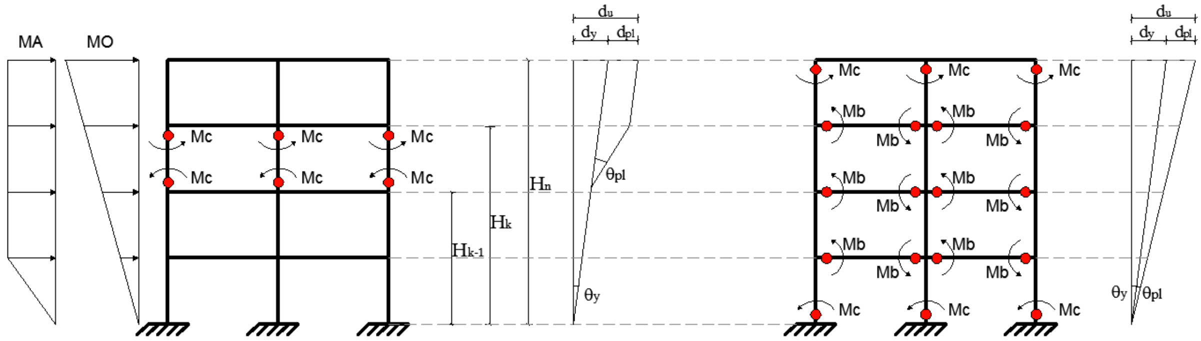


Figure 3: Example of collapse mechanism type and horizontal forces distribution. Left: local soft storey mechanism type; right: global mechanism type.

Accordingly the base shear corresponding to equilibrium of internal and external forces may be computed with Equation (4)-(5) for a local type mechanism with MO or MA horizontal forces distribution, while Eq. (6)-(7) may be adopted for a global type mechanism under MO or MA external forces distribution.

$$V_b = \frac{2 \cdot \sum M_C^k}{\sum_{i=k}^{n} H_i \cdot (H_k - H_{k-1})} \sum_{i=1}^{n} H_i \tag{4}$$

$$V_b = \frac{2 \cdot \sum M_C^k}{(H_k - H_{k-1})} \cdot \frac{n}{n-k+1} \tag{5}$$

$$V_b = \frac{\sum M_C^1 + \sum M_C^n + \sum_{i=2}^{n-1} \sum M_b}{\sum_{i=1}^{n} H_i^2} \cdot \sum_{i=1}^{n} H_i \quad (6)$$

$$V_b = \frac{\sum M_C^1 + \sum M_C^n + \sum_{i=2}^{n-1} \sum M_b}{\sum_{i=1}^{n} H_i} \cdot n \quad (7)$$

In the above equations $M_c^k(=M_{c,y}^k)$ represents the generic yielding moment at the base or top section of the k[th] floor columns (it is hypothesized that $M_{y,base}=M_{y,top}$ for the columns), ΣM_b $(=\Sigma M_{b,y})$ represents the sum of beam end yielding moments at a storey (selecting positive or negative ones depending on sway mechanism) and H_i is the i[th] storey height to foundation level. Once the base shear is calculated, the corresponding base shear coefficient C_b is easily determined.

For what concerns calculation of yield and ultimate displacements, a similar approach to the one proposed in [19] is adopted. In particular, the yield displacement d_y at the roof level of the MDOF system is calculated assuming a linear deformed shape within the elastic range (see Figure 3 and Eq. (8)), while the ultimate displacement at the same level is given by the sum of d_y plus the plastic contribution that is developed according to the hypothesized plastic mechanism (see Figure 3 left panel and Eq. (9) for the case of soft storey and Figure 3 right panel and Eq. (10) for the case of a global mechanism):

$$d_y = \vartheta_y \cdot H_n \quad (8)$$

$$d_u = d_y + \vartheta_{pl} \cdot (H_k - H_{k-1}) \quad (9)$$

$$d_u = d_y + \vartheta_{pl} \cdot H_n \quad (10)$$

Adopting a MEC approach it is very difficult to capture the roof displacements that may be determined via nonlinear static pushover analysis, where the mechanism is developed upon gradual loading of the structural system and the involved hinges have different local ductility demand. In order to minimize the scatter of yield and ultimate roof displacements that are obtained with the Equations (8)-(10) with the corresponding displacement values derived by the equivalent bi-linearization of the pushover curve, appropriate value for yield and plastic rotation θ_y and θ_{pl} should be chosen. In this study it is adopted a θ_y corresponding to the maximum yield rotation of the base columns in the (8), while θ_{pl} in the (9)-(10) is assumed as the minimum value of θ_u-θ_y among the hinges involved in the plastic mechanism, where θ_u is the rotation corresponding to Collapse Prevention CP limit state according to ACI 369R-11 [10] for the generic considered hinge.

Once the roof displacements of the MDOF system are known, the equivalent d^*_u and d^*_y for the SDOF and the relative ductility capacity are straightforwardly determined, as well as the equivalent period T_{eq} (see NOTATION section).

For what concern the study of the "damaged" structure it is hypothesized that the plastic mechanism is the same as the one that forms for the "intact" structure. Hence, the assessment of the behaviour of the damaged system comprises only the steps 2 to 4 of the previous list.

In particular, given the generic global roof displacement d_{roof} (e.g. due to an hypothetical mainshock), the corresponding demanded plastic rotation $\theta_{pl,d}$ and rotation demand θ are straightforwardly determined by inversion of the (9) or (10) for local or global sway mechanisms, respectively:

$$\vartheta_{pl,d} = \vartheta - \vartheta_y = \frac{(d_{roof} - d_y)}{(H_k - H_{k-1})} \tag{11}$$

$$\vartheta_{pl,d} = \vartheta - \vartheta_y = \frac{(d_{roof} - d_y)}{H_n} \tag{12}$$

The knowledge of the rotation demand needed to attain d_{roof} with a plastic mechanism allows to determine the local ductility demand μ_j for the generic jth element involved:

$$\mu_j = \frac{\theta}{\theta_{y,j}} \tag{13}$$

where $\theta_{y,j}$ is the yield rotation for the jth element.

Then, adopting the formulation proposed in [14] based on local ductility demand μ_j, the plastic hinges of the elements involved in the plastic mechanism may be suitably modified, i.e. yield moment and stiffness are reduced as well as the plastic post-yield branch (see Figure 2). Once the plastic hinges are modified, the C_b for the damaged structure may be computed with the same methodology as described for the intact structure. For what concerns the ductility capacity and equivalent period it is here noted that the assessment of μ_{cap} and T_{eq} via MEC approach is strongly influenced by the evaluation of d^*_y (see NOTATION section); the latter parameter should be calculated based on Eq.(8) where the θ_y is modified due to damage in the plastic hinges. However, such a modification determines an unrealistic increase of the yield displacement of the equivalent SDOF, and consequently an abnormal decrease of the relative μ_{cap} and increase of T_{eq}. For this reason, when applying the MEC approach, it is proposed to calculate the d^*_y for a system having attained a generic damage state by multiplying the d^*_y calculated of the intact system by a correcting factor. Such factor is proportional to the mean ratio of the d^*_y calculated with PA for the damaged structure versus the d^*_y calculated with PA for the intact one. Accordingly, considering the d^*_y calculated with MEC applying the correction factor for the damaged state, the μ_{cap} and T_{eq} for the damaged structure can be coherently determined.

3 COMPARISON OF PUSHOVER AND MECHANISM BASED RESULTS

In this paper we want to extend the comparison between PA and MEC approach, initiated in [3] for a single case study, by considering a number of Reinforced Concrete (RC) frames building typologies. In particular, the comparison of the results of PA with those of the MEC analysis is performed confronting the parameters that may be retrieved by the equivalent bi-linearization of equivalent SDOF capacity curve obtained with PA, i.e. C_b, μ_{cap}, T_{eq}, as well as REC_{Sa} (or REC_{ag}), with the homologous parameters that are computed as explained in § 2.2 with the MEC approach.

By way of example, Figure 4 (a) shows the pushover curves that are obtained for one of the studied moment resisting Reinforced Concrete Frames RCF (described in § 3.1) for the case of the intact building and building (D_0) that has attained a D_2 damaged state, together with the equivalent bi-linearization for both cases; analogously, Figure 4 (b) shows the bilinear curves that are obtained with MEC approach for the same structure in the same states (D_0 and D_2).

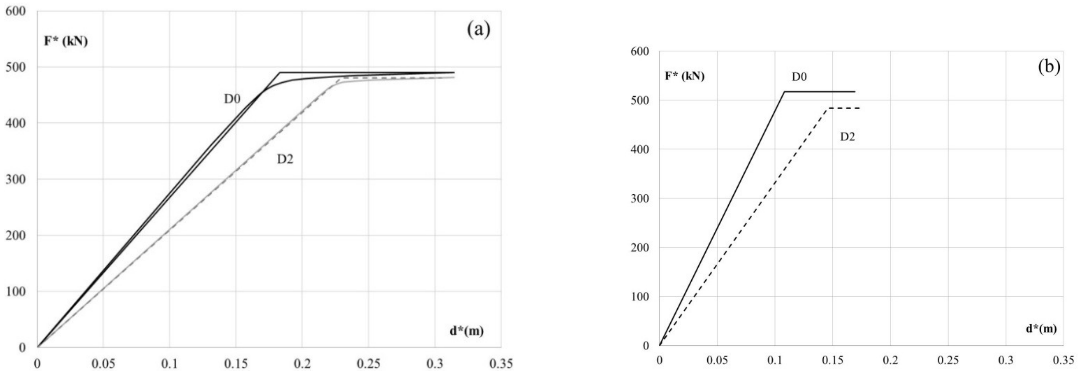


Figure 4: Pushover curve and bi-linearization for the 8_005_MA case, intact state D_0, and damaged state D_2 (a); bilinear curves obtained with the MEC approach for the 8_005_MA for intact state D_0, and damaged state D_2 (b)

3.1 Description of the RCF building models

The comparison of pushover with mechanism based analyses is performed considering nine Reinforced Concrete Frames RCF that are designed to be representative of Gravity Load Designed (GLD) buildings or buildings designed according to old Italian seismic standards in force at the beginning of age '60s [20], not applying principles of capacity design or proper reinforcement detailing and based on allowable stress method. In particular, the structural and mechanical characteristics of the frames belonging to 4, 6 or 8 storey buildings, are obtained with a simulated design approach as suggested in [21]. A common geometric planar configuration of a building having base dimensions 18mx10m, two bays in transversal direction and 4 bays in the longitudinal one, as well as 3 m inter-storey height (see Figure 5, referring to a 4 storey GLD building) is assumed. Then, the elements dimensions and reinforcement of the perimeter transversal frames (that are the ones chosen for PA-MEC comparison) are designed either for gravity loads either considering two different values of the seismic coefficient *C* 0.1, 0.05, i.e. in the first or second seismicity class according to [20]. Allowable stresses of σ_c=6 MPa for elements under pure axial load and 7.5 MPa for elements under combination of flexure and axial load are assumed, while the allowable stress for steel, that considering the design period is assumed to be a smooth type Aq50 [22], is σ_s=180 MPa [23]. The column area A_c is dimensioned based only on the axial load N and the concrete design stress σ_c where the axial load depends on the permanent and live loads on the area of influence of the generic column. The resulting columns dimensions for the perimeter frame are 30x30 (corner columns) and 40x30 (central column) at the base storey of the 4 storey GLD frame (see Figure 5). For the seismic design of the 4 storey building with C=0.1 and 0.05 the column section are equal to the gravitational load design. In 6 storeys and 8 storeys frames the columns size are proportionately greater up to the maximum size, for the 8 storey building designed with C0.1, of 45x35 for corner base columns and 55x35 for central ones. As a general rule, the column sections are gradually reduced in elevation, when possible according to design issues, to the minimum size 30x30.

The 4 storeys GLD beams have section 30x50 at all storeys; in the seismic design with C 0.1 and 0.05 beams section are 30x60 and 30x55, respectively for the first and second storeys. In 6 storeys and 8 storeys frames the beams size are proportionately greater up to the maximum size, for the 8 storey building designed with C 0.1, of 35x65 for first and second storeys.

The longitudinal bars percentage in the base columns vary from approximately 7.0‰ to 1.3%, consistently with low minima design requirements of old type codes. The lumped plasticity model for PA analysis is built considering bilinear flexural hinges, where yield and ultimate (at Collapse Prevention limit state, CP) moment and rotation are determined according to ASCE-SEI/41[24], adopting a mean concrete strength of f_c=26.7 MPa and a steel yield stress of f_y=370 MPa.

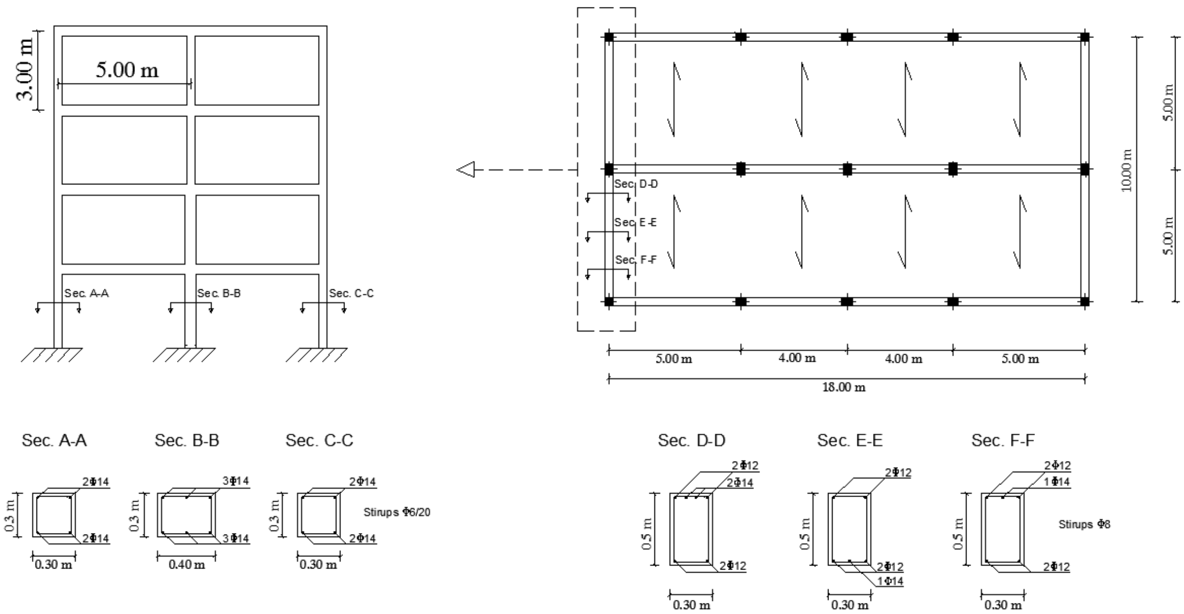


Figure 5: Building plan and elevation of perimeter frame for the 4 storey GLD building.

Each of the designed building have been analyzed with PA adopting two lateral load distributions, namely modal shape (MO) and constant shape (proportional to masses) MA. Hence, a total number of 18 PA analyses were performed. In the following, we identify the building and analysis type with notation N_D_FO where N, indicating the number of storeys, may be 4, 6 or 8, D indicating the design type, may be GLD, 01 or 005, and FO indicating the horizontal load path may be MO (for modal load) or MA (for constant load in elevation).

3.2 PA and MEC results and comparisons

Adopting the methodology described in § 2.1 and 2.2 the representative parameters C_b, μ_{cap} and T_{eq}, as well as REC_{Sa} and REC_{ag} are calculated for the 18 identified analyses cases with both the PA and MEC approaches and considering either the structures in their intact state (denoted as D_0) either structures that are supposed to have reached the D_2 damage state due to an hypothetical main-shock.

Table 1, referring to undamaged RCFs, and Table 2, referring to D_2 damaged RCFs, summarize the results for all the analyzed cases allowing for comparison of the PA and MEC approach. The subscripts m in Tables 1 and 2 indicate the results for the MEC analyses, while subscripts p the PA ones.

It is noted that, calculating the ratio of $d^*_{y,p}$ (D_2) versus $d^*_{y,p}$ (D_0) for all the considered cases, a mean value of 1.35 is obtained. Hence, as anticipated in § 2.2, the $\mu_{cap,\mathrm{m}}$ (D_2) and $T_{eq,m}$ (D_2) are determined after suitable calculation of $d^*_{y,m}$ (D_2) (i.e. by increasing by 30% the $d^*_{y,m}$ (D_0)).

As a general comment, it is evidenced that adopting the sway index S_i and the sway demand index SD_i it was generally possible to identify the soft storey formation and the relative storey where the plastic deformations are concentrated for the MEC approach, finding good agreement with PA. On the other hand, when a soft storey is not evidenced (Si lower than 0.85), it was hypothesized the formation of a global mechanism developed over the whole building height, involving also the beams of the RCF; this hypothesis is not always realistic according to PA results, where the global type mechanisms are generally developed only to a limited number of storeys, but it was chosen in order to remain at a more general level.

INTACT RCF (D_0)										
ID	T_{eq},m(s)	T_{eq},p(s)	μ_{cap},m	μ_{cap},p	C_b,m(g)	C_b,p(g)	REC_{Sa},m(g)	REC_{Sa},p(g)	REC_{ag},m(g)	REC_{ag},p(g)
4_01_MO	1.14	1.22	2.04	1.90	0.16	0.18	0.33	0.34	0.25	0.28
4_01_MA	1.37	1.40	1.92	1.80	0.15	0.15	0.29	0.27	0.26	0.25
4_005_MO	1.34	1.26	2.25	2.00	0.11	0.15	0.24	0.31	0.21	0.26
4_005_MA	1.38	1.45	2.03	1.85	0.13	0.13	0.27	0.25	0.25	0.24
4_G_MO	1.79	1.89	4.84	3.08	0.06	0.06	0.29	0.17	0.34	0.22
4_G_MA	2.04	2.15	4.84	2.81	0.06	0.05	0.28	0.15	0.39	0.23
6_01_MO	1.23	1.52	1.79	1.46	0.17	0.20	0.31	0.29	0.25	0.29
6_01_MA	1.37	1.78	1.69	1.45	0.19	0.19	0.32	0.27	0.29	0.32
6_005_MO	1.76	1.72	1.79	1.58	0.10	0.14	0.17	0.21	0.20	0.24
6_005_MA	1.61	2.00	1.65	1.77	0.15	0.13	0.25	0.24	0.26	0.32
6_G_MO	2.32	2.61	4.62	2.34	0.05	0.05	0.25	0.11	0.45	0.26
6_G_MA	2.72	2.98	4.62	2.45	0.05	0.04	0.24	0.11	0.59	0.32
8_01_MO	1.28	1.68	1.80	1.41	0.22	0.21	0.40	0.30	0.34	0.33
8_01_MA	1.36	1.97	1.60	1.52	0.20	0.19	0.31	0.29	0.28	0.39
8_005_MO	1.32	1.93	1.59	1.90	0.18	0.17	0.29	0.32	0.25	0.42
8_005_MA	1.67	2.23	1.56	1.72	0.16	0.15	0.24	0.25	0.27	0.42
8_G_MO	3.04	3.43	4.72	2.37	0.04	0.03	0.18	0.08	0.57	0.32
8_G_MA	3.63	4.01	4.72	2.24	0.04	0.03	0.18	0.07	0.78	0.40

Table 1: Representative parameters of the equivalent SDOF system for the structure in the intact (D_0) configuration and residual capacity in terms of spectral acceleration and anchoring (peak) ground acceleration; subscript p stays for results referring to PA, while subscript m refers to MEC.

DAMAGED RCF (D_2)										
ID	T_{eq},m(s)	T_{eq},p(s)	μ_{cap},m	μ_{cap},p	C_b,m(g)	C_b,p(g)	REC_{Sa},m(g)	REC_{Sa},p(g)	REC_{ag},m(g)	REC_{ag},p(g)
4_01_MO	1.36	1.49	1.36	1.31	0.15	0.17	0.21	0.22	0.19	0.22
4_01_MA	1.64	1.65	1.32	1.34	0.14	0.15	0.19	0.19	0.21	0.21
4_005_MO	1.61	1.55	1.45	1.36	0.10	0.15	0.14	0.20	0.15	0.21
4_005_MA	1.65	1.71	1.357	1.36	0.12	0.13	0.17	0.18	0.18	0.20
4_G_MO	2.20	2.42	2.27	1.95	0.05	0.05	0.12	0.10	0.19	0.20
4_G_MA	2.51	2.76	2.27	1.79	0.05	0.05	0.12	0.09	0.25	0.23
6_01_MO	1.46	1.69	1.28	1.19	0.17	0.19	0.21	0.23	0.21	0.26
6_01_MA	1.65	1.98	1.25	1.18	0.17	0.18	0.22	0.22	0.24	0.29
6_005_MO	2.11	1.95	1.28	1.26	0.09	0.13	0.11	0.16	0.17	0.21
6_005_MA	1.95	2.26	1.23	1.39	0.14	0.13	0.17	0.18	0.22	0.31
6_G_MO	2.83	3.18	2.15	1.56	0.05	0.05	0.11	0.07	0.28	0.25
6_G_MA	3.32	3.69	2.15	1.61	0.05	0.04	0.10	0.07	0.37	0.31
8_01_MO	1.53	1.88	1.28	1.15	0.15	0.21	0.19	0.24	0.19	0.30
8_01_MA	1.68	2.21	1.19	1.23	0.17	0.19	0.21	0.23	0.23	0.37
8_005_MO	1.58	2.25	1.21	1.34	0.17	0.17	0.21	0.22	0.22	0.38
8_005_MA	2.01	2.52	1.2	1.33	0.15	0.14	0.17	0.19	0.24	0.41
8_G_MO	3.78	4.21	2.31	1.50	0.03	0.03	0.08	0.05	0.38	0.29
8_G_MA	4.51	4.78	2.31	1.63	0.03	0.03	0.08	0.05	0.51	0.39

Table 2: Representative parameters of the equivalent SDOF system for the structure in the damaged (D_2) configuration and residual capacity in terms of spectral acceleration and anchoring (peak) ground acceleration; subscript p stays for results referring to PA, while subscript m refers to MEC.

Figures 6 to 8 show the comparison of PA and MEC for the single parameters.

In particular Figure 6 shows the comparison of PA and MEC in terms of C_b, for the intact structures (a) and D_2 damaged ones (b). As it may be observed, the parameter C_b can be satisfactorily represented with a MEC assessment for both the cases of intact D_2 damaged structures. The comparison of ductility capacity μ_{cap} is presented in Figure 7. Figure 7 (a), referring to intact structures, evidences that when the mechanism type is not adequately captured by MEC analysis the μ_{cap} may be considerably overestimated, while in case of good agreement of the hypothesized mechanism type the μ_{cap} are satisfactorily captured.

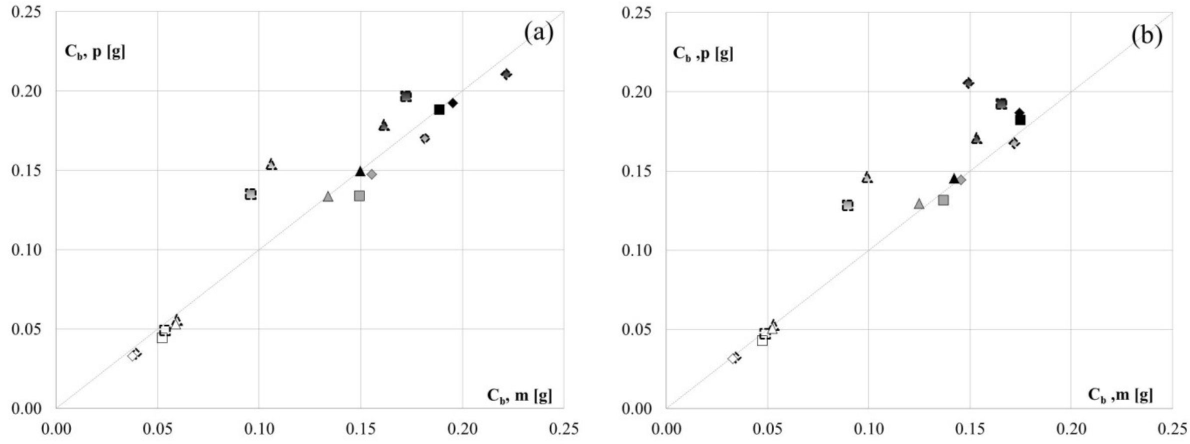


Figure 6: Comparison of C_b obtained with PA (denoted as $C_{b,p}$) with the one obtained with MEC approach (denoted as $C_{b,m}$) for intact structures (a) and structures that have reached D_2 damage state for an hypothetical main-shock (b)

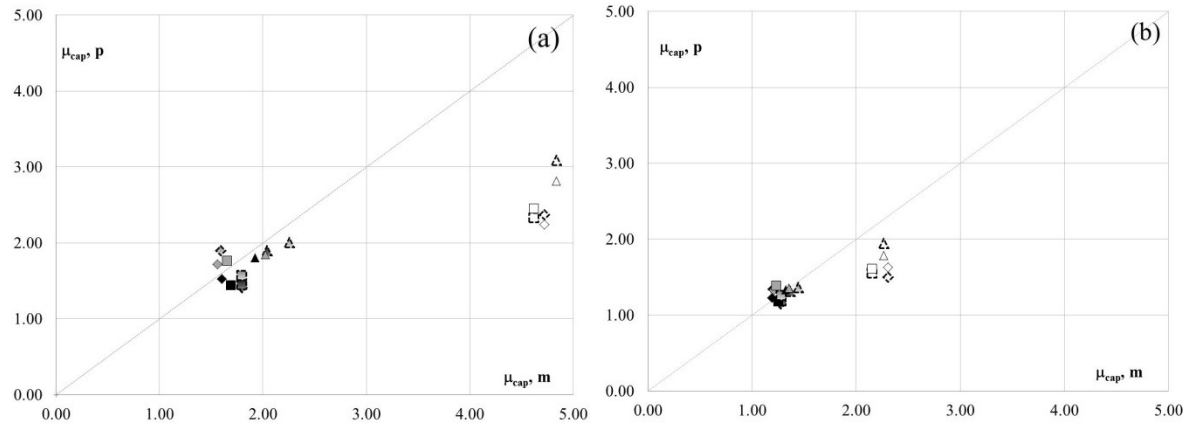


Figure 7: Comparison of μ_{cap} obtained with PA (denoted as $\mu_{cap,p}$) with the one obtained with MEC approach (denoted as $\mu_{cap,m}$) for intact structures (a) and structures that have reached D_2 damage state for an hypothetical main-shock (b)

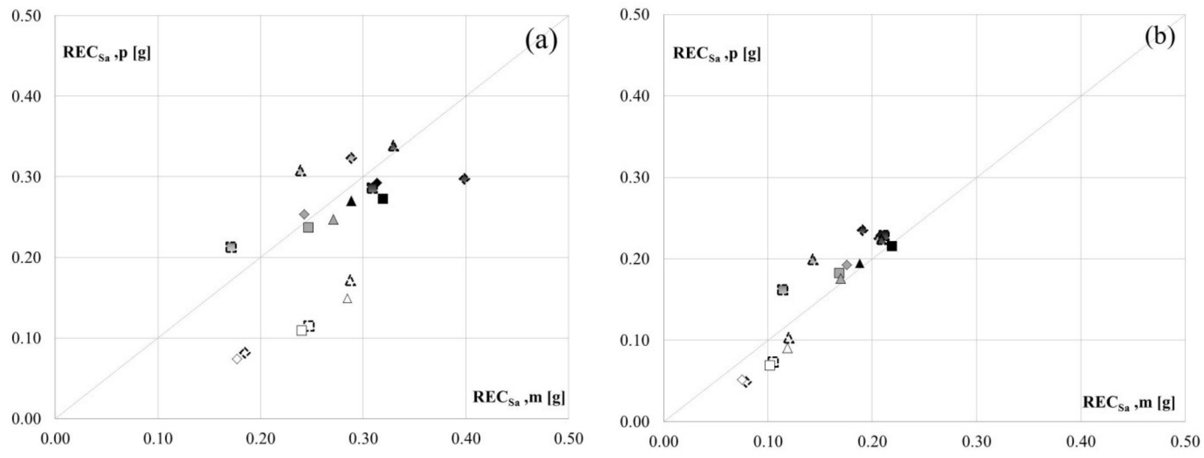


Figure 8: Comparison of REC_{Sa} obtained with PA (denoted as $REC_{Sa,p}$) with the one obtained with MEC approach (denoted as $REC_{Sa,m}$) for intact structures (a) and structures that have reached D_2 damage state for an hypothetical main-shock (b)

In fact, the white symbols in Figure 7 (a) represent the case of GLD buildings, where the global type mechanism that is hypothesized with MEC does not fully correspond to the mechanism that forms with PA. By multiplying the base shear coefficient C_b for the ductility capacity, the *REC* for the intact and damaged structures may be obtained, as explained in § 2.1.

Figure 8, representing the REC_{Sa} for the intact (Figure 8 (a)) and damaged structure (Figure 8 (b)), synthesizes those results evidencing the same discrepancies that were observed for μ_{cap}, as well as satisfactory agreement for the case of D_2 damaged structures.

4 REC VARIATION IN FUNCTION OF SEISMIC DEMAND

The comparison of the results of PA and MEC analyses shows that, when it is possible to realistically capture the plastic mechanism type that is forming in a RCF, the MEC allows to derive significant parameters with relatively small error and comparatively small computational effort. For this reason, MEC approach may be usefully adopted to investigate on the possible REC variation as a function of seismic demand. Indeed, once the bilinear representation of the equivalent SDOF capacity curve for the undamaged system is available, the seismic displacement demand, and ductility, may be determined by application of the Capacity Spectrum Method CSM [25]. Given the global ductility demand μ for the equivalent SDOF, it may be easily transformed in local ductility demand for the elements involved in the known plastic mechanism. By way of example, hypothesizing a first storey local mechanism with MEC approach, the elements rotation demand corresponding to the global ductility μ may be derived as:

$$\theta = \theta_y \cdot \left[1 + (\mu - 1) \frac{H_n}{H_1} \right] \tag{14}$$

On the other hand, if a global type mechanism, involving all storeys, is assumed, the rotation is simply μ times the yield rotation:

$$\theta = \theta_y \cdot H_n \tag{15}$$

In MEC approach, all the elements involved in the mechanism are subject to the same rotation; hence the element's ductility demand, depending on global μ, may be calculated based on Eq. (14) and (13) for a first storey mechanism or (15) and (13) for a global one. Given the local ductility demand μ_i for the generic plastic hinge, the relative M-θ relationship may be suitably modified to account for damage by introduction of the proper plastic hinges modification factors [14]. This way, apart from the REC_0 ($REC_{Sa,0}$ and/or $REC_{ag,0}$) that may be calculated for the structure in its initial undamaged state D_0, also the REC_μ ($REC_{Sa,\mu}$ and/or $REC_{ag,\mu}$) corresponding to given global ductility demand and the relative REC variation $REC_{ag,\mu}/REC_{ag,0}$, can be determined.

It has to be observed that the correction factor to be applied to $d^*_{y,m}$ of the damaged system, that with reference the studied structures was computed based on the ratio of $d^*_{y,p}$ (D_2) versus $d^*_{y,p}$ (D_0) for all the considered cases (see § 3.2), is significant for structures that have attained D_2 damage state; considering the analyzed structures, the latter corresponds to a mean ductility demand of 1.33. However, it may be expected that with increasing ductility demand for the structures, the ratio of $d^*_{y,p}$ (D_2) versus $d^*_{y,p}$ (D_0) shall increase. Accordingly, it is supposed that for the generic ductility demand:

$$\frac{d_{y,\mu}}{d_{y,0}} = \frac{1.35}{1.33} \cdot \mu \approx \mu \tag{16}$$

Hence, in the hypothesis that the variation of F^* with ductility demand may be neglected with respect to the variation of d^*_y, the ratio of $T_{eq,\mu}$ versus $T_{eq,0}$ corresponds to $\sqrt{\mu}$, and consequently Eq. (17) may be used to compute the ratio $REC_{ag,\mu}/REC_{ag,0}$:

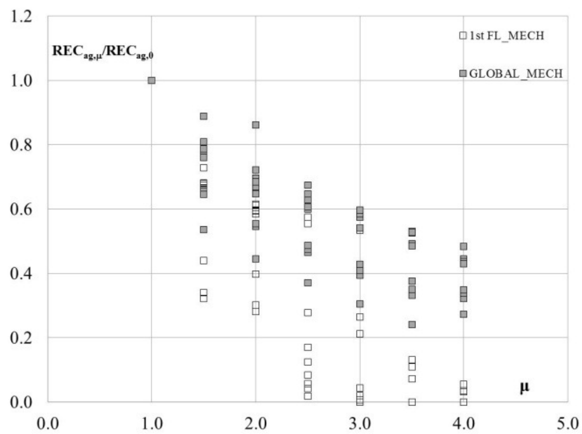
$$\frac{REC_{ag,\mu}}{REC_{ag,0}} \approx \frac{REC_{Sa,\mu}}{REC_{Sa,0}} \cdot \sqrt{\mu} \tag{17}$$

An important information that may be inferred, once the REC variation is known, is the Performance Loss, *PL*, for each seismic demand. *PL*, that represents a measure of the loss of lateral capacity, is defined as:

$$PL = 1 - \frac{REC_{ag,\mu}}{REC_{ag,0}} \tag{18}$$

with $REC_{ag,\mu}$ residual capacity in terms of peak ground acceleration of the structure that has been subject to the global ductility demand μdue to main-shock and $REC_{ag,0}$ the residual capacity for the intact structure.

Considering the 9 RCF typologies and two types of hypothesized plastic mechanism, namely a soft storey at the first floor and a global one involving all storeys, the $REC_{ag,\mu}/REC_{ag,0}$ and the relative *PL* have been calculated up to a ductility demand of 4.



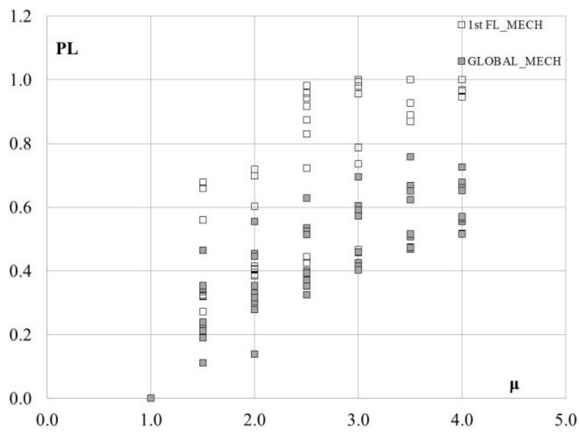


Figure9: $REC_{ag,\mu}/REC_{ag,0}$ variation for increasing global ductility demand for intact structure exhibiting local (first storey) or global plastic mechanism (left); Same variation for the PL (right)

Figure 9 shows the $REC_{ag,\mu}/REC_{ag,0}$ and *PL* variation with increasing ductility demand. As it can be noted, the *REC* ratio decreases more rapidly for structures exhibiting a local type mechanism with respect to those with a global one. Indeed, the local ductility demand for the elements involved in the mechanism, given the global μ, is higher in the former case and therefore a larger *REC* reduction is expected. Analogously, for local type mechanisms a larger *PL* with respect to global one is observed for each ductility demand. This trend confirms what was already noted in [1] for a detailed case study.

5 POSSIBLE APPLICATION IN THE CONTEXT OF DAMAGE DEPENDENT VULNERABILITY ASSESSMENT

As explained in [1], the REC_{ag} represents the peak ground acceleration corresponding to the 50% probability of attaining collapse damage state, hence it may be employed in a HAZUS like representation of collapse fragility curves:

$$P[col|a_g] = \Phi\left[\frac{1}{\beta}\cdot\ln\left(\frac{a_g}{\hat{a}_g}\right)\right] = \Phi\left[\frac{1}{\beta}\cdot\ln\left(\frac{a_g}{REC_{ag}}\right)\right] \quad (19)$$

where β represents a global value of dispersion, due to modeling uncertainties and inherent randomness associated to earthquake variability [1, 26]. Therefore, the knowledge of the *REC* variation due to a given ductility demand (see § 4) allows assessing the relative damage-dependent variation of collapse fragility curves (e.g. Figure 10) with a mechanical approach.

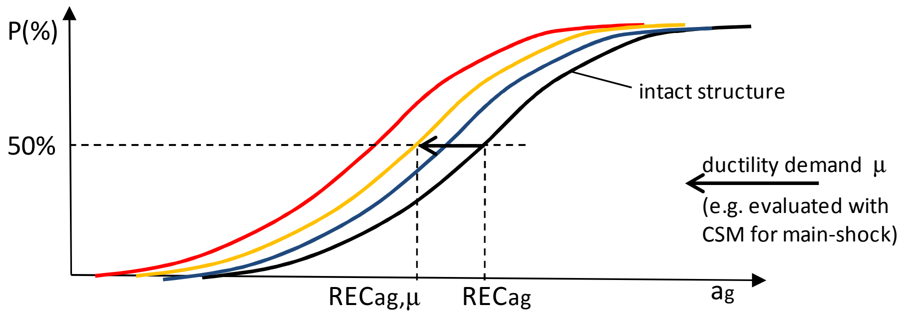


Figure 10: Example collapse fragility curves derived through Eq. (23) in function of REC_{ag}, and fragility curves variation depending on global ductility demand μ

The authors are aware that this simple collapse fragility representation, considering a collapse criterion defined assuming that only the most critical element controls the state of the structure, may be too restrictive for a realistic assessment of the ultimate load bearing capacities of existing RC frames. Indeed, some past studies [27, 28] point out the need for a proper calibration of mechanical based vulnerability assessment methodologies with empirical ones, that rely on observation of real damages from past earthquakes [29, 30, 31]. On the other hand, there is a lack of observational data allowing for empirical based derivation of damage-dependent fragility curves. Therefore, the proposed approach could be conveniently applied for assessment of damage (or ductility) dependent variation of fragility curves; the initial collapse fragility curves (referring to undamaged state) shall be preventively calibrated based on observational data.

6 CONCLUSIONS

This paper extends the comparison of PA and MEC analyses, initiated in [3], studying 9 RCF (4, 6 and 8 storey, 3 design typologies) under two different distributions of horizontal loads (linearly increasing with height MO, and proportional to storey masses MA); the parameters retrieved from 18 capacity curves obtained with PA are compared with those deriving from simplified MEC approach. Adopting the sway index *Si* and the sway demand index *SDi* it was generally possible to identify the soft storey formation and the relative storey where the plastic deformations are concentrated for the MEC approach, finding good agreement with PA. On the other hand, when a soft storey is not evidenced (Si lower than 0.85), it was hypothesized the formation of a global mechanism developed over the whole building height, involving also the beams of the RCF; this hypothesis is not always realistic according to PA results, where the global type mechanisms are generally developed only to a limited number of storeys, but it was chosen in order to remain at a more general level. Accordingly, the MEC approach yields results that are closer to PA for the cases where a soft storey mechanism forms, while for more global type mechanisms a higher scatter is observed.

Despite the discrepancy of results of MEC and PA in case of global type mechanism, the MEC approach can be usefully adopted to assess the *REC* variation for structures that had sustained damaging main-shock and for which the plastic mechanism that has formed can be recognized. In fact, given the mechanism type, and having the possibility to derive the essential parameters C_b, μ_{cap} and T_{eq}, as well as REC_{Sa} and REC_{ag} by a simplified, MEC approach, the building relative Performance Loss may be easily determined, as it is shown in Section 4.

The REC variation is the starting point to build damage-dependent collapse fragility curves. Further study is required to calibrate initial collapse fragility curves with observational data from past earthquakes. Moreover, more studied building typologies should be considered, in order to investigate on possible ranges of variation of REC depending on building class.

Finally, the effect of brittle type mechanisms (e.g. joint failure, shear failure for beams and columns) should be properly considered in future studies.

7 ACKNOWLEDGEMENTS

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NOTATION

$m^* = \sum m_i \cdot \Phi_i$	mass of the ESDOF system
$C_b = \frac{F_y^*}{m^* g}$	base shear coefficient
$T_{eq} = 2\pi\sqrt{\frac{m^* d_y^*}{F_y^*}}$	elastic period of the ESDOF
m_i	the seismic mass at the generic ith story
Φ	shape vector of the force distribution applied in pushover analysis (main vibration mode or linear distribution for the MO case, constant unit vector for the uniform load MA case)
$d^* = \frac{d_{roof}}{\Gamma}$	displacement for equivalent SDOF
$F^* = \frac{V_b}{\Gamma}$	force for equivalent SDOF
$\Gamma = \frac{\sum m_i \cdot \Phi_i}{\sum m_i \cdot \Phi_i^2}$	MDOF-ESDOF transformation factor

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