Residual capacity of earthquake damaged buildings

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SUMMARY:
Seismic behavior of damaged buildings may be expressed as a function of their REsidual Capacity (REC), that is a measure of seismic capacity reduced due to damage; the diminishing of REC after an earthquake is representative of Performance Loss PL.
This paper deals with the problem of assessing building's REC for existing under-designed Reinforced Concrete (R.C.) buildings that are typically found in European Mediterranean regions. REC is evaluated based on pushover curves obtained for the structure in different damage state configurations, where building's behavior is simulated with a suitable modification of plastic hinges for damaged elements. Moreover, a simplified approach for REC assessment, that may be useful for a preliminary evaluation of possible damage dependent PL, is proposed.

Keywords: damage, residual capacity, pushover, reparability, mechanism based evaluation

1. INTRODUCTION

Generally, the seismic safety in the post-earthquake is evaluated based on a visual inspection of the buildings, with an "expert" assessment of the damage level and extension and of the relative building usability, that is performed by a team of experienced practitioners (Baggio et al. 2007, ATC, 1989). On the other hand, a decision on the needed structural interventions (repair/retrofit/upgrade) can only be demanded upon more detailed engineering analyses. Evidently, due to damaging of structural elements, a reduction of seismic capacity can be expected; the latter may be realistically evaluated only if suitable mechanical models are adopted.

In ATC-43 project (FEMA, 1998a) the available instruments and methods for seismic analyses of damaged buildings are analyzed. Adopting pushover as nonlinear analysis tool, damaged building's behavior may be simulated with a suitable modification of plastic hinges for damaged elements. Such a modification is based on stiffness, strength and displacement reduction factors accounting for the achieved damage states for the structural elements, as could be detected by visual inspection of post-earthquake damage.
In the guidelines for seismic assessment of damaged buildings (Bazzurro et al., 2004), the building tagging is based on the likelihood that an aftershock will exceed a specific (reduced) capacity associated with each damage state representing the quantitative measure of degradation; a detailed application of the procedure, which relies on the execution of pushover analyses of the buildings in various damage states as suggested in (FEMA, 1998a), may be found in (Maffei et al., 2006) for some steel buildings.
As a matter of fact, the seismic behavior of damaged buildings, and the relative seismic safety, may be adequately represented by its seismic capacity modified due to damage, the so called REsidual Capacity REC. In the framework of a mechanical based assessment of seismic vulnerability, REC may be evaluated based on pushover curves obtained for the structure in different (initial) damage state configurations, where damaged building's behavior is simulated with a modification of plastic hinges for damaged elements.

Although some indications exist for suitable values of the damage dependent modification factors to be adopted for masonry and Reinforced Concrete (R.C.) buildings (FEMA 1998b, Maeda et al., 2004), they refer to type of elements that may be very different from sub-standard ones that are typically found in existing R.C. buildings of the European-Mediterranean regions. Hence, there is a need for proper calibration of modification factors in order to be representative of under-designed construction typologies. In Di Ludovico et al. (2012) such calibration in order to characterize damaged plastic hinges has been carried out considering a number of experimental tests performed on nonconforming columns.

In this paper, the seismic performance of existing under-designed R.C. buildings that have been damaged by a main-shock event is investigated by means of a case study. Suitable modification factors for damaged columns are considered and the variation of building's REC depending on the damage states caused by a potential main-shock is studied.
Moreover, with the aim of allowing fast analysis for a building, that may be useful for a preliminary evaluation of possible damage dependent performance loss, a mechanism based simplified approach is proposed.

In the following sections, the post-earthquake evaluation procedures introduced in (Bazzurro et al., 2004) and (Nakano et al. 2004) are briefly described. Next, an approach in the framework of the Capacity Spectrum Method (CSM, Fajfar 1999) is proposed and applied with a case study to an existing R.C. building. Finally, the mechanism based simplified method for evaluation of REC in the intact and damaged states is introduced.

2. DAMAGE DEPENDENT RESIDUAL CAPACITY

The Japanese guidelines for building tagging (JBDPA, 1991; Nakano et al. 2004) systematize the post-earthquake screening methodologies, verifying their effectiveness after real surveys and calibrating the assessment procedure as a function of experimental results (Maeda et al. 2004). As explained in (Otani, 2000), the Japanese standard for the assessment of existing buildings (JBDPA, 1977) judges vulnerability based on a seismic index I_s, that is given by the product of a structural index E_0, representing seismic capacity, a configuration index S_D, representing possible irregularities in plan or elevation, and an age index T. E_0 may be evaluated as the product of a strength index C (expressed in terms of the base shear coefficient) and a ductility index F (representing the deformation capacity for the building). Otani (2003) observes that $C \cdot F$ is proportional to the maximum seismic intensity that a Single Degree Of Freedom SDOF system, equivalent to the real structure, can sustain before exhausting its seismic capacity. The variation of I_s due to earthquake induced structural damage, that may be computed based on damage dependent modification factors defined for the vertical resisting elements (columns, walls), is considered as an effective parameter for building tagging.

The assessment method proposed in (Bazzurro et al., 2004) for the building tagging is based on the quantitative evaluation of building's residual capacity and of the aftershock hazard for the site. In order to evaluate residual capacity, pushover analyses for building models representing the building in different damage states are performed. The residual capacity is defined as the minimum spectral acceleration (at the elastic period T_1 of the system and for 5% damping) that corresponds to local or global collapse during an aftershock. In particular, it is proposed to determine the residual capacity adopting the SPO2IDA tool (Vamvatsikos e Cornell 2006), that allows finding the median IDA curve and the relative percentiles (16% e 84%), normally calculated with incremental dynamic analyses, with the sole knowledge of pushover curve. Note that the residual capacity defined in (Bazzurro et al. 2004) is conceptually analogous to $C \cdot F$ introduced above.

In this paper a similar approach of the one proposed in (Bazzurro et al., 2004) is adopted, with the main difference that the relationship between the seismic demand and the seismic intensity is determined with an approach in line with the N2 method (Fajfar, 2000), the IN2 (Dolšek and Fajfar, 2004), that is also adopted within European codes for the determination of seismic demand starting from nonlinear static analysis. Moreover, specific application for existing under-designed R.C. buildings that are typically found in European Mediterranean regions is performed, for which suitable calibration of damage-dependent element's hinge modification factors is needed.

The basic steps for determining building REsidual Capacity REC for the intact and damaged states configurations of a building are illustrated in Fig. 1.

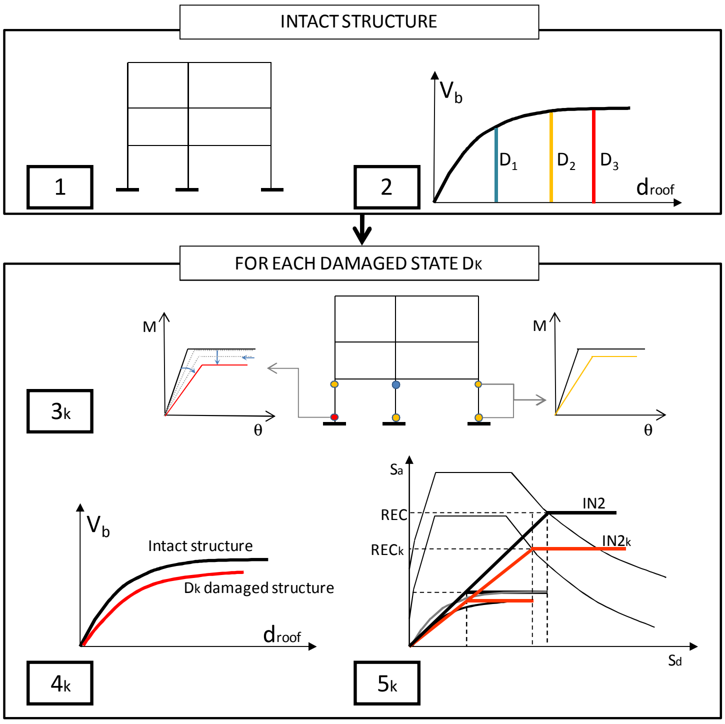
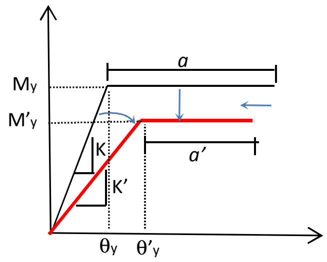


Figure 1. Flowchart illustrating the basic steps of the method for framed structures

Global capacity parameters may be determined with pushover analysis performed on a lumped plasticity model (Step 1). Element's flexural behavior is conveniently characterized by a bilinear moment-rotation plastic hinge, described by yielding (M_y and θ_y) and ultimate (M_u and θ_u) moment and rotation. The moments M_y and M_u can be determined by moment-curvature analyses for the element's end sections, while yielding and ultimate rotations can be determined with one of the existing literature approaches (e.g. ASCE-SEI41 (2007), with updated limit values as suggested in ACI 369R-11 (2011)). It is hypothesized that shear failures, due to local shear effects in the elements, and unconfined joint failures are prevented; hence, material nonlinearity is modeled only with the moment-rotation relationship in the plastic hinges of the columns and beams and, depending on ductility demand for the damaged elements, suitable modification of flexural hinges are introduced, as it will be explained next.

For the forecasting of damage-dependant performance loss in "peace-time", when a damaging earthquake has not occurred, the modified behavior for significant global damage levels defined on the pushover curve has to be studied. With the purpose of describing the progression of damage due to an hypothetic main-shock, three global damage states may be considered (Step 2) as reference for the assessment of REC variation: D1 (limited damage), D2 (moderate damage) and D3 (high damage). Ideally, global damage states should correspond to certain thresholds of reconstruction costs. However, defining such a criteria in terms of mechanical parameters is very complicated, and therefore in the current codes there is a lack of definition about the limit states at the level of the structure and the assessment is often based on the assumption the most critical element controls the state of the structure. In this study, it is assumed that D3 corresponds to the first attainment of Collapse Prevention CP limit state for an element (ACI 369R-11 (2011)); D2 to the first attainment of 0.5 CP and D1 state is attained at the Yield Displacement of the Idealized (YDI) pushover curve.

For each of the global damage levels considered (D_i, with i=1,2,3), a modified nonlinear model is built. In particular, considering the local damage level attained by the structural elements in the deformed configuration at D_i, the corresponding plastic hinges are modified (Step 3) with a suitable variation of the relative stiffness ($K'=\lambda_k K$), strength ($M_y'=\lambda_Q M_y$) and plastic rotation capacity ($a'= a-a_d = a-(\theta'_y-\theta_y)-RD = a-(\theta_y(\lambda_Q/\lambda_k-1)-RD$) (see Fig. 2), with λ stiffness or strength modification factors and RD element's residual drift.



	stiffness	strength	Plastic rotation
intact	K	M_y	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 (adapted from (FEMA, 1998a))

Nonlinear static analyses of the modified damaged models yield pushover curves that, depending on the number of elements involved in the damaged mechanism and on their damage level, may differ significantly with respect to original ones (Step 4).

Pushover curve (intact or modified one for each of the considered damage states) is an essential tool for the application of the Capacity Spectrum Method (CSM) (Fajfar, 1999), that allows determining the building response for earthquakes of a given spectral shape. In fact, given a suitably defined ultimate displacement on the intact or damaged pushover curve, the transformation of the pushover curve in bilinear form allows estimating significant parameters for the equivalent Single Degree Of Fredoom (SDOF) structure, i.e. yield and ultimate displacement d^*_y and d^*_u and the displacement ductility capacity μ_{cap} ($=d^*_u/d^*_y$), the base shear coefficient C_b and the period T_{eq}:

$$C_b = \frac{F^*_y}{m^* g} \tag{2.1}$$

$$T_{eq} = 2\pi\sqrt{\frac{m^* d^*_y}{F^*_y}} \tag{2.2}$$

with ($d_u^*=d_{roof}/\Gamma$; $F_y^*=V_b/\Gamma$) SDOF displacements and forces related to Multi-Degree Of Fredoom (MDOF) representative displacements and forces (d_{roof}, V_b) through the transformation factor Γ; m* the mass of the equivalent SDOF system and g the gravity acceleration.

Coherently with this approach, the residual capacity REC_{Sa} is defined, for each global damage state D_i (i=0,1,2,3 with 0 corresponding to the intact structure), as the minimum spectral acceleration (at the period T_{eq} of the equivalent SDOF) corresponding to building collapse. Moreover, considering the convenience of direct estimation of peak ground acceleration a_g as damaging 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}.

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; demand-capacity comparison has to be performed for increasing values of seismic intensity until the collapse intensity is found.

Adopting the Incremental N2 method (IN2) it is possible, with reference to an equivalent SDOF, to build the curve approximately relating the seismic demand with the seismic intensity, by way of a repetitive application of N2 method for increasing intensities up to collapse (Step 5). Generally, the

shape of IN2 curve depends on the shape of the relationship between reduction factor, *R,* and the period, *T*. In the simpler, but very common, case of applicability of the principle of equal displacement rule ($T_{eq} \geq T_c$ with T_c equal to the corner period at the upper limit of the constant acceleration region of the elastic spectrum) the IN2 curve is a straight line from the origin up to collapse point, the only point that is necessary to determine (see Fig. 1, panel 5).
It is easily verifiable that, in the hypothesis of 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}:

$$REC_{Sa} = C_b \cdot \mu_{cap} \qquad \text{for } T_{eq} \geq T_c \tag{2.3}$$

Analogously, it can be verified that, for $T_{eq}<T_c$, the residual capacity may be still put in relation with C_b and μ_{cap}; in fact, adopting the *R-T* relation introduced in (Vidic et al. 1994), and considering that for a seismic intensity bringing the structure to collapse *R* equals the ratio of REC_{Sa} versus C_b:

$$REC_{Sa} = C_b \cdot (\mu_{cap} - 1) \cdot \frac{T_{eq}}{T_c} + 1 \qquad \text{for } T_{eq}<T_c \tag{2.4}$$

In any case, the REC_{Sa} (and analogously REC_{ag}) depends on C_b and μ_{cap} of the intact (i=0) or damaged (i=1,2,3) equivalent system. Hence, the estimate of these two factors for different structural systems and mechanism type for varying damage levels, becomes crucial in the estimate of pre and post-earthquake safety level.

3. CASE STUDY FOR AN EXISTING R.C. BUILDING

The procedure described in the previous section was applied for the assessment of damage dependent REC_{Sa} (and REC_{ag}) of an existing R.C. moment frame structure, designed in 1979 in 2nd seismicity class with old Italian seismic standards, not applying principles of capacity design and proper reinforcement detailing.
The building has rectangular base with dimensions 12.40x24.25m. In elevation it has three storey and an attic; inter-storey height is 2.85 m for the first level and 3.05 m for the above levels.
Columns at the first level have dimension 250x500mm and 300x300mm (longitudinal geometrical reinforcement ratio, ρ= 0.6% and 0.9%); at upper stories only the dimensions of the square columns are reduced, becoming 250x250mm at the top storey. Either embedded and emergent beams are present at each storey. Emergent beams have a dimension of 250x500mm, while embedded ones are 400x200mm and 600x200mm at the first two levels, 1200x200mm at upper stories.

The lumped plasticity model was constructed with the modeling features described above; for the steel yield stress f_{ym} = 380MPa was assumed, compatible with common steel type Feb 38k used in Italy at the time of building construction, while for the concrete strength a mean value of f_{cm}=15.6 MPa, resulting from compression tests on cylindrical specimens extracted from the building, was considered.
Pushover analyses for the “intact” building were performed in both longitudinal (X) and transversal (Y) directions, applying two different horizontal force distributions (proportional to main vibration mode in each considered direction and proportional to masses). The building is approximately symmetric, hence only one analysis per direction and per force type was performed; Fig. 3 shows the resulting pushover curves as dashed grey lines, labeled as MAX(Y) analysis in X(Y) direction with forces proportional to masses, and MOX(Y) for forces proportional to mode shapes; also the relative bilinear curves are shown.

The collapse mechanism type is soft storey type at first and third storey, respectively, for MAY and MOX cases, while a two storey mechanism, involving mainly the columns of the 2nd and 3rd storey, and a three storey mechanism, involving mainly the columns of the first three levels, is formed for MOY and MAX cases, respectively. In any case, the percentage of beams involved in each of the mechanism is very low, being generally lower than 20% and with a maximum of 22% of the yielding hinges for MAX case.

The points evidenced on the dashed grey curves in Fig. 3, i.e. CP, 0.5CP and the one corresponding to the yield displacement of the idealized pushover curve, represent the global damage states D3, D2 and D1 that will be considered for further analyses of the “damaged” structure.

The modifications factors, λ, and *RD* element’s residual drift accounting for the local damage level attained by the structural elements in the deformed configuration at $D_{i,}$ have been computed based on experimental cyclic tests performed by authors on R.C. full scale columns (Di Ludovico et al., 2010). As it could be expected, for increasing ductility demand the λ modification factors decrease (e.g. λ_k becoming lower than 20% for μ=6), while *RD* increases (becoming larger than $2 \cdot \theta_y$ for μ=6). Details on the adopted calibration procedure may be found in (Di Ludovico et al., 2012), where a wider database of cyclic tests on nonconforming columns is considered.

For each of the global damage states a separate analysis of the “damaged” structure has been performed. In particular, each pushover analysis is stopped in the deformed configuration at D_i (for i=1,2,3) and the plastic hinges state (ductility demand) is registered. Next, the plastic hinges of the elements that have entered the plastic range are modified as a function of their ductility demand.

As a general observation, the cases where a local type mechanism develops (MAY and MOX) are characterized by high ductility demand for the most part of the elements involved in the mechanism; on the other hand, when the mechanism involves a larger number of elements, such as for the cases of MAX and MOY, the mean ductility demand is lower.

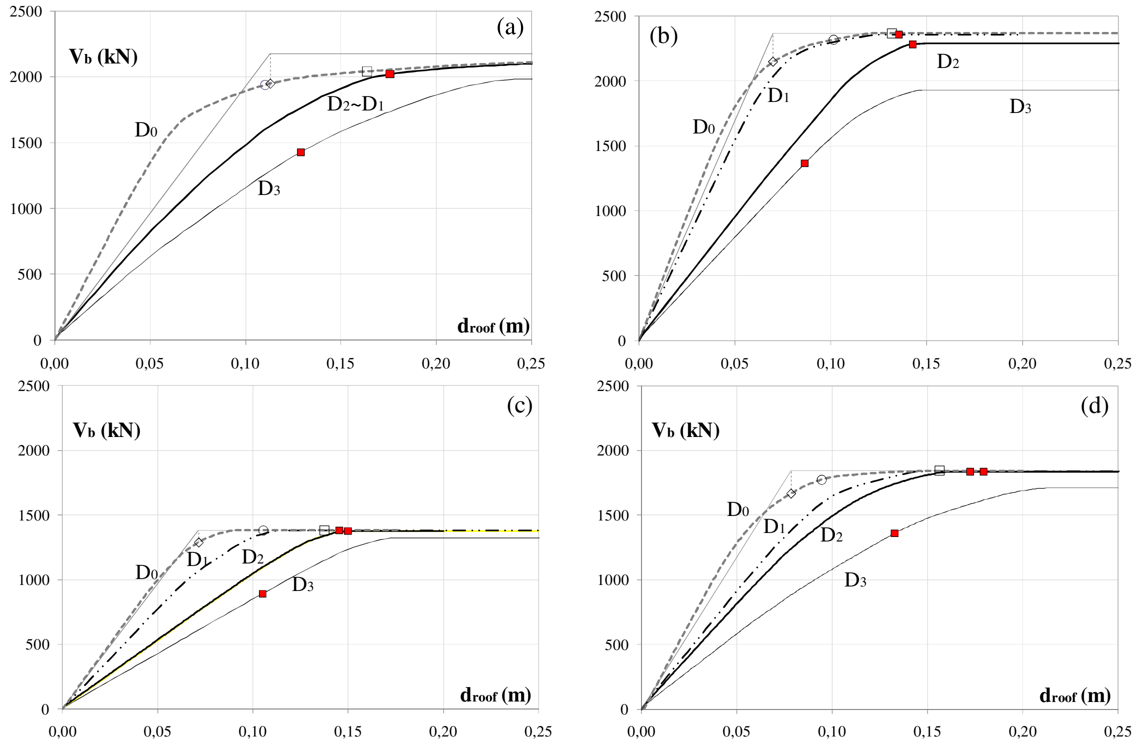


Figure 3. Pushover curves for the building in the three considered damage configurations ($D_{1,2,3}$) and for the intact structure (D_0): cases MAX (a), MAY (b), MOX (c) and MOY (d).

Pushover curves obtained for each of the considered damaged models are shown in Fig. 3 as continuous or dash-dot lines. The pushover curves for the intact structure are indicated as D_0. Because the deformed configuration at D1 for MAX is approximately the same that as for D2, for the case MAX only D2 and D3 states are considered in the study of the damaged building. On each of the curves corresponding to the analysis of the damaged building also the points corresponding to the first attainment of the (reduced) CP for an element are shown as small red squares.

Applying the above described methodology, the building residual capacity for the intact and damaged states were computed; Eurocode 8, soil type B spectral shape (CEN, 2005), was considered for derivation of REC_{ag} from REC_{Sa}.
For the case study, the condition $T_{eq}>T_c$ (=0.5 s) is always verified; hence the residual capacity in terms of spectral acceleration, for the different considered damage states, varies proportionally to the simple product of C_b per μ_{cap} (Eq. (2.3)). With reference to an EC8 spectral shape and considering a system with $T_C<T_{eq}<T_D$, the following relation applies: $REC_{ag}=REC_{Sa}/(S\cdot\eta\cdot 2.5)\ (T_{eq}/T_C)$. Hence, in the considered range of periods, REC_{ag} varies proportionally to the product of $C_b\ \mu_{cap}\ T_{eq}$.

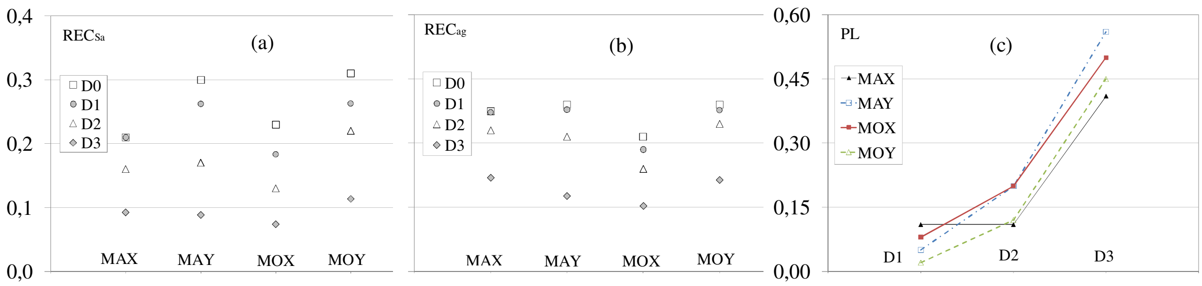


Figure 4. Values of REC_{Sa} (a) and REC_{ag} (b) for increasing damage level and *PL* variation (c)

Figure 4 shows the resulting values of REC_{Sa} (left panel) and REC_{ag} (central panel) for each of the performed analyses and for increasing damage states. It is evident the diminishing of REC for increasing damage.
In order to have a measure of the loss of lateral capacity the Performance Loss PL may be defined:

$$PL = 1 - \frac{REC_{ag,k}}{REC_{ag,0}} \tag{3.1}$$

with $REC_{ag,k}$ residual capacity in terms of peak ground acceleration of the structure in the D_k damage configuration and $REC_{ag,0}$ for the intact structure. It is interesting to observe that for an intermediate damage state such as D_2, the analysis configurations exhibiting a local type mechanism (MAY and MOX) are those affected by a higher *PL* (=20%), nearly double of the *PL* for the configurations characterized by a more global mechanism type (MAX and MOY) (see Fig. 4, right panel).

4. SIMPLIFIED APPROACH

As observed in § 2 the REC_{Sa} depends on C_b and μ_{cap} of the intact (i=0) or damaged (i=1,2,3) equivalent system, while, considering a system with $T_C<T_{eq}<T_D$, that is often the case for mid-rise existing R.C. buildings, REC_{ag} varies proportionally to the product $C_b\ \mu_{cap}\ T_{eq}$.
Hence, in order to investigate on possible values of REC_{Sa} and REC_{ag} that may be expected for assigned building typologies and on their range of variation due to damaging earthquakes, the evaluation of C_b, μ_{cap} and T_{eq} is needed.
For a preliminary assessment a simplified method may be adopted. Following the approach suggested in (Cosenza et al., 2005) building capacity is determined in the hypothesis that plastic mechanism forms for the structure. In particular, given the collapse mechanism, the base shear V_b is evaluated by equilibrium relations. In (Cosenza et al., 2005) a linear distribution of horizontal seismic forces, that stands for forces proportional to first mode shape, is considered; however, this approach is easily adjustable for a constant distribution of horizontal forces, that represents the case of forces proportional to seismic masses. By way of example, Figure 5 depicts the system of external and internal forces that should satisfy equilibrium for two hypothesized mechanisms; constant forces (proportional to masses) are considered and the corresponding base shear is calculated with Eq. (4.1) for the mechanism represented on the left hand and Eq. (4.2) for the one on the right.

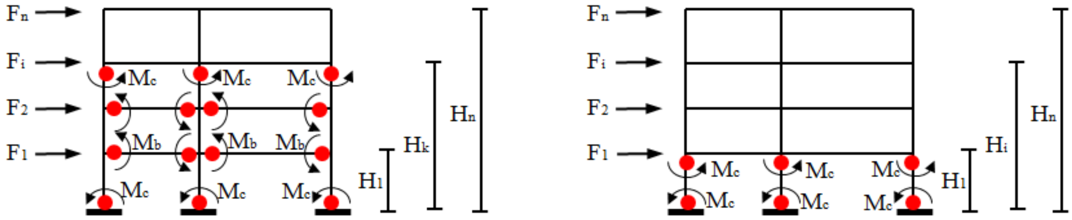


Figure 5. Example of collapse mechanism types.

$$V_b = \frac{\sum M_c^1 + \sum M_c^k + \sum_{i=1}^{k-1} \sum M_b}{\sum_{i=1}^{k} H_i + (n-k)\cdot H_k} \tag{4.1}$$

$$V_b = \frac{2 \cdot \sum M_c^1}{H_1} \tag{4.2}$$

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), M_b ($=M_{b,y}$) is the generic yielding moment for beam's ends 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.

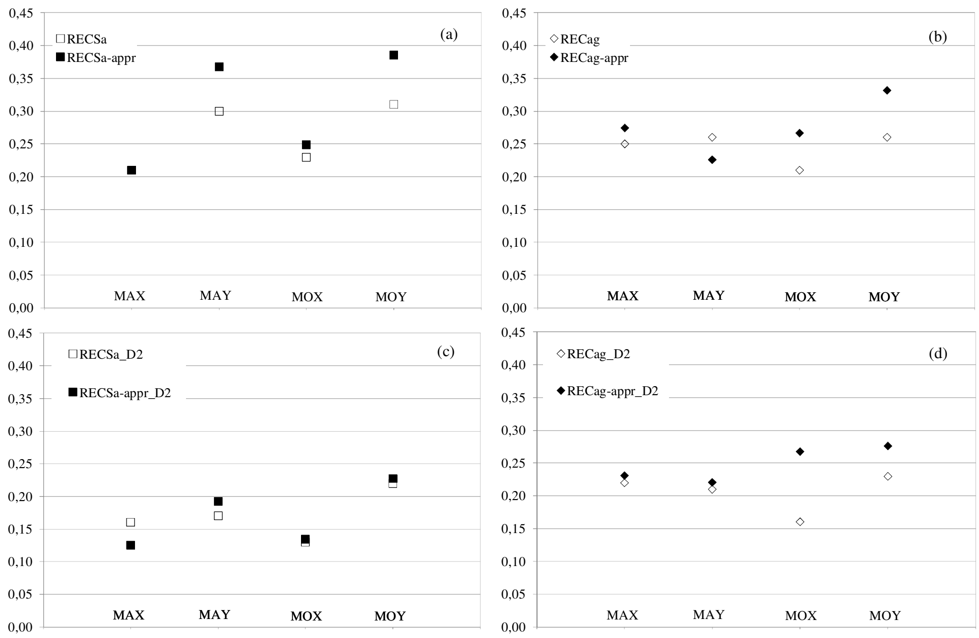


Figure 6. Comparison of *REC* obtained for the case study with those computed with the simplified approach. REC_{Sa} (a) and REC_{ag} (b) for the intact state, and $REC_{Sa\text{-}D2}$ (c) and $REC_{ag\text{-}D2}$ (d) for D_2 damage state

In order to evaluate system's ductility capacity μ_{cap} it is necessary the estimate of yielding and ultimate displacements of SDOF. In this paper, the approach suggested in (Borzi et al., 2008) is adopted, that allows to find limit state displacements for a SDOF given limit chord rotations for the columns and considering different possible mechanism types (global or local type); this approach allows also to account for different possible height of activation of the mechanism. Based on the rotation capacity of the elements at yielding θ_y and ultimate θ_u limit states, the yield d_y and ultimate d_u displacement for the equivalent SDOF (having a suitable equivalent height) are determined, as well as the ductility capacity $\mu_{cap}= d_u/ d_y$. Knowing the stiffness ($K_y=V_b/d_y$) and total mass M of the SDOF system also the effective period may be computed $T_* = 2\pi\sqrt{M/K_y}$. Then, REC_{Sa} is given by the product of C_b and μ_{cap}, and REC_{ag} can be computed as explained in §3.

In order to preliminary test the simplified procedure, it was applied to compute REC_{Sa} and REC_{ag} for the case study building; plastic mechanisms resembling the one that developed in pushover analyses were considered. In particular, the mechanisms depicted in Fig. 5 were adopted to simulate MAX and MAY, respectively, while for MOX and MOY a local type mechanism with activation of a soft storey at the third level and a mixed one, involving the 2nd and 3rd storeys, respectively, were considered; the equilibrium conditions for the two latter cases were written considering a linear distribution of horizontal seismic forces, as may be found in (Cosenza et al., 2005).

Figure 6 confronts the residual capacities that are based on pushover analyses with those derived using the simplified procedure. As it can be seen in the top panels (a and b), having imposed the mechanism types (coherently to those observed in pushover analyses) a relatively low scatter (max 20%) between the two approaches can be obtained.
The mechanism based approach is attractive also because the easy implementation of modification factors for the M-θ hinges of the elements involved in the plastic mechanism. In the real deformed configuration the ductility demand for the different elements involved in the plastic mechanism is variable. However, in order to perform preliminary calculations, the same value of ductility demand for the elements involved in each of the adopted mechanism may be considered. With this simplification, the mechanism based procedure was applied also for computing REC_{Sa} and REC_{ag} for the structure in a damaged state. For the two mechanism having a soft storey development a larger value of ductility demand was considered (μ=3), while for the remaining ones a lower value was tentatively adopted (μ=1.5). The bottom panels (c and d) in Figure 6 shows comparison between REC_{Sa} and REC_{ag} that are obtained for D2 global damage state with the pushover based and the mechanism based evaluation methods; apart few exceptions, a reasonable approximation is generally observed.

5. CONCLUSIONS

An effective parameter to represent the seismic behavior of damaged buildings is the REsidual Capacity REC_{Sa} (REC_{ag}) defined as the minimum spectral acceleration at the equivalent period T_{eq} (the minimum anchoring peak ground acceleration) corresponding to building collapse.
REC_{Sa} (REC_{ag}) can be calculated starting from pushover analysis, where the building model is suitably modified to account for element's damage. By application of the IN2 method, it is seen that REC_{Sa} depends on the product of C_b and μ_{cap} of equivalent system (in the intact or damaged state), while, considering a system with $T_C<T_{eq}<T_D$, that is often the case for mid-rise existing R.C. buildings, REC_{ag} varies proportionally to the product $C_b\ \mu_{cap}\ T_{eq}$.
The diminishing of REC_{ag} from the intact to damaged states gives an useful indication with respect to building safety variation, and a related Performance Loss index *PL* is introduced.
For a preliminary assessment of building's REC and PL a simplified mechanism based approach is presented; the comparison of the results of the simplified method with those obtained in the described case study shows an acceptable level of approximation.

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