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Comparative analysis of code-compliant seismic assessment methods through nonlinear static analyses and demand spectrum: N2 Method vs. Capacity Spectrum Method

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Abstract

This paper investigates the main differences in evaluating the seismic performance of buildings through nonlinear static procedures according to different code-compliant approaches, with a specific focus on the two alternative methods reported in the Italian Building Code, namely "Method A" and "Method B", referring to the N2 Method and the Capacity Spectrum Method, respectively. An extensive parametric analysis is carried out by performing several nonlinear static analyses on Multi-Degree-of-Freedom (MDoF) models of different Reinforced Concrete (RC) frame structures. Seismic assessment is then performed by applying the two spectrum-based methods, and results are compared in terms of safety evaluation and loss assessment. Results of the comparison highlight that the ductility capacity of the structure strongly affects the seismic assessment, leading to larger differences when more ductile structures are considered. This work could be considered as a preliminary step toward the development of specific guidelines including provisions on the recommended simplified approach to be adopted for seismic assessment of buildings (also based on the observed/expected seismic behavior) in practical applications.

Keywords: Nonlinear static analysis; Capacity Spectrum Method; Seismic assessment; Reinforced concrete frames

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1. Introduction and motivation

In seismic vulnerability and risk assessment applications, nonlinear static (pushover) analyses are often deemed as arguably the best compromise between accuracy and simplicity, as well as more suitable for engineering practice when compared to possibly more accurate, but certainly more complex and time-consuming, nonlinear dynamic (time-history) analyses. In these approaches, the seismic analysis of the structure is generally performed by comparing the lateral load capacity (i.e., the force-displacement capacity curve) and the seismic demand within an Acceleration Displacement Response Spectrum (ADRS) domain, based on the main assumption and simplification that the response of a Multi-Degree-of-Freedom (MDoF) system can be evaluated considering the response of an equivalent Single-Degree-of-Freedom (SDoF) system. In the last decades, different methods were proposed and adopted in the main international seismic codes/guidelines. Among others, the state-of-the-art methodologies include the Capacity Spectrum Method (CSM), adopted and described in the ATC-40 (1996) report, and the N2 method (Fajfar 2000), adopted in the Eurocode 8, EC8 (2005). The former (i.e., CSM) is conceptually based on a Capacity/Demand comparison in the ADRS domain considering an overdamped spectrum, evaluated through an equivalent viscous damping coefficient as a function of the ductility demand and the expected hysteretic behavior. On the other hand, the N2 method involves an elastic-perfectly plastic idealization of the force-displacement pushover curve and the use of a ductility-based reduction/modification factor (i.e., R_μ or the behavior factor q in the EC8) for the evaluation of the (pseudo-)inelastic spectrum. Depending on the initial stiffness of the structure (i.e., the fundamental period T_1), the reduction/modification factor (R_μ or q) is evaluated through the equal displacement rule or the equal energy rule. In both cases, the performance point can be evaluated through either closed-form expressions or visual comparison in the ADRS domain.

In line with the research developments at international level previously described, in the recent Italian Building Code (NTC 2018) two alternative code-compliant spectrum-based methods are presented as alternative options for the seismic assessment of an existing building, namely "Method A" and "Method B". More specifically, the two methods refer to the N2 Method and the CSM, respectively. The same procedure is applied for the evaluation of the equivalent SDoF response in both methods (i.e, the evaluation of the effective height displacement and the effective mass). However, the Italian Building Code does not include any provision on the recommended method to adopt in practical applications. This aspect can potentially lead to different assessment (both in terms of Safety Index and Economic Losses) results even when considering the same (pushover) capacity curve and seismic hazard. Although some comparative studies assessing the effectiveness of the two methods (i.e., CSM and N2 Method) are available in literature (e.g., Chopra and Goel 2000, Lin et al. 2004, Cardone 2007, Lagaros and Fragiadakis 2011, Nettis et al. 2021), specific provisions/guidelines discussing the recommended approach to be adopted for seismic response analyses in case of different expected seismic behavior (in terms of ductility or others relevant characteristic of the global behavior) are still missing.

Therefore, this paper investigates the main differences in terms of seismic performance and safety evaluation of buildings according to different code-compliant approaches, with a specific focus on the two methods reported in the Italian Building Code. An extensive parametric analysis is carried out by performing non-linear static analyses on Multi-Degree of Freedom (MDoF) models of different Reinforced Concrete (RC) frame structures. A two-dimensional (2D) lumped plasticity model is implemented in the structural software Ruaumoko (Carr, 2016) for each analyzed structure. Seismic assessment is thus performed by applying the two spectrum-based methods, following the code provisions. Results are finally compared in terms of building life safety performance and economic losses. The latter are evaluated following the Italian approach for seismic risk classification of buildings (DM 65 2017, Cosenza et al. 2018), based on the evaluation of a safety index, IS-V (equivalent to the %New Building Standard, %NBS, adopted in the New Zealand seismic assessment guidelines NZSEE 2017) and an economic index EAL (Expected annual losses, or PAM, Perdita Annua Media, in Italian). It is worth underlining that, at this stage of the research, the purpose of this work is limited to investigate whether the two alternative code-compliant assessment methods lead to different results and, if so, which parameters mostly affect the evaluation of the seismic performance of the structure.

The paper is structured as follows: in Section 2 the adopted research methodology is presented; the parametric analysis is reported in Section 3, including a description of the case-study structures and the adopted modelling approach, as well as a discussion of the comparative analysis; finally, conclusions are given in Section 4.

2. Research methodology

The adopted research methodology for the comparative analysis of the two alternative code-compliant pushover-based assessment methods reported in the NTC2018 is illustrated in Figure 1. Each step is discussed in detail below.

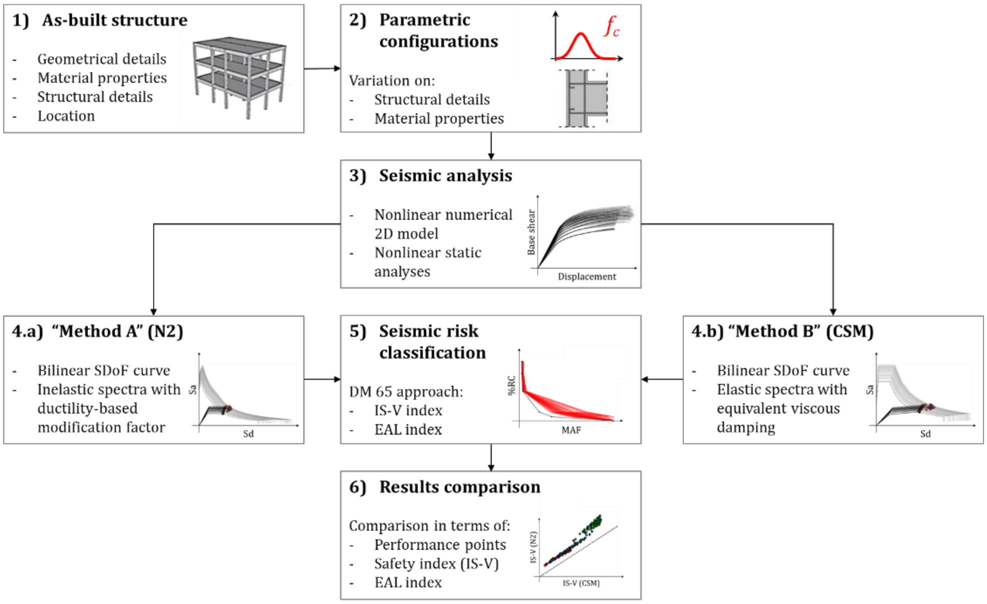

Figure 1. Flowchart of the adopted research methodology.

Firstly, a case study RC frame structure representative of an archetype pre-1970s building in the Italian region is selected and geometrical details, material properties, structural details, and site location are defined (*step 1*). Starting from this information, parametric configurations are defined by varying the material properties and the construction details of the beam-column panel joints (*step 2*), in order to analyze a wide range of different frame structures featured by construction details typical of pre-1970 buildings. More information on the considered construction details of the joints is given in Section 3.1. For each configuration, a 2D lumped-plasticity model is implemented in the structural software Ruaumoko (Carr, 2016) and nonlinear static (pushover) analyses are performed (*step 3*). More details about the adopted modelling approach are given in the following section. The results of the nonlinear static analyses (i.e., the pushover force-displacement capacity curves) are then used to evaluate the seismic performance of the structures through the two code-compliant approaches, i.e., "Method A"/N2 (*step 4.a*) and "Method B"/CSM (*step 4.b*). It is worth noting that both methods require a bilinearization of the pushover curve, and two different approaches are reported in the NTC2018 for "Method A" and "Method B". Therefore, differently from other comparative studies in which a database of SDoF capacity curves is considered (e.g., Nettis et al. 2021), in this work, in order to obtain an effective comparison of the two approaches, it is deemed necessary to perform nonlinear static analyses on MDoF numerical models and using the alternative bilinearization method. In other words, the comparison is made considering the same force-displacement capacity curve (output of the nonlinear static analysis) and the same demand spectrum. For each analyzed structure, the seismic risk class is evaluated according to the Italian "Guidelines for the seismic risk classification of buildings" (DM 65 2017, Cosenza et al. 2018) (step *5*). To implement the methodology, the capacity/demand ratios at different limit states are evaluated by applying both code-compliant spectrum-based methods. This allows estimating the safety index (IS-V) and the economic index (EAL, PAM) for both methodologies. Finally, results are compared in terms of IS-V index and PAM index.

The details and results of the performed parametric analysis are reported in the following section.

3. Parametric analysis

In this section, firstly the case-study buildings are presented. Then, the adopted modelling approach to perform non-linear static (pushover) analyses is briefly discussed. Finally, for each case-study building, results in terms of pushover curves are reported and the safety index (IS-V) and the economic index (PAM) are evaluated following the two code-compliant approaches and according to the Italian guidelines, DM 65 (2017).

3.1. Description of the case-study buildings

The case-study buildings have been derived starting from a RC frame located in a high-seismicity zone in Italy (L'Aquila). The structure is a three-bay, three-story frame designed for gravity loads only. Fig.2a and Fig.2b show the geometric characteristics and reinforcement details for beams and columns, respectively. Considering this frame, several case study buildings have been derived using different structural details for external beam-column joints as well as material properties as in Gentile et al. (2021). Specifically, three different details for beam-column joints, with no stirrups in the panel zone (reflecting the pre-1976's design practice in Italy) and alternative anchorage details for the beam rebars have been considered: plain round bars with hooked end anchorages (case 1), beam bars bent away from the joint (case 2), and beam bars bent into the joint (case 3), (Fig. 2c). Depending on the beam anchorage details, different damage mechanisms are expected and therefore different inelastic global behavior (different pushover curves) of the structures are expected. As reported in Pampanin et al. (2003), the solution leading to the highest strength capacity is the bent-in configuration followed by the bent out and hooked ones. In this study, variation in material properties is considered in terms of concrete cylindrical compressive strength (f_c) and steel yield stress (f_y). These variables are described with a normal distribution depending on the mean value (μ) and the standard deviation (σ). The mean values and the coefficients of variation (CoV) for both f_c and f_y are selected from Verderame et al. (2001, 2011). Specifically, for concrete, a mean value of f_{cm}=25.7MPa and a CoV=33.7% are selected, while for steel f_{ym}=322.3MPa and CoV=8.2%. Nine values of f_c and f_y are sampled from the normal distributions considering equally spaced points in the range [-2σ; $+2\sigma$] (as in Gentile et al. 2021). The maximum and minimum values for concrete are 8.38 MPa and 43.02 MPa respectively, while for steel are 269.44 MPa and 375.16 MPa. This leads to 81 samples for each of the three cases considered for the beam-column joints details. Using different values for f_c and f_y has a direct effect on the characterization of the behavior of beams, columns, and beam-column joints, leading to different pushover curves of the frame under investigation. Combining 81 samples for material properties, and 3 cases for the beam-column joints details, a total of 243 case study building configurations have been derived.

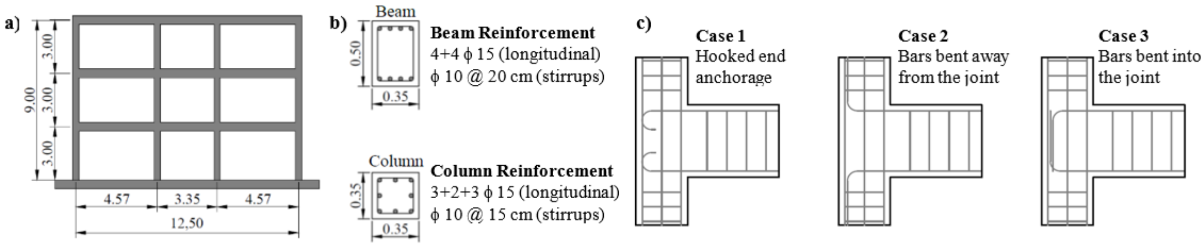

Fig. 2. (a) geometric characteristic of the frame; (b) reinforcement details of beams and columns; (c) alternative cases for beam-column joints.

3.2. Modelling approach

In order to perform nonlinear static pushover analyses, a refined two-dimensional (2D) lumped plasticity model is implemented in the finite-element software Ruaumoko, Carr (2016). As simplified assumptions, the soil-structure interaction contribution is neglected (i.e., fixed base joints are considered), while the floor diaphragms are assumed rigid in their plane. The RC frame members are modelled by Giberson elements, i.e., mono-dimensional elastic elements with plastic hinges at the connection interfaces. The beam plastic hinges are characterized by bi-linear moment-curvature relationships, while an axial load-moment (M-N) interaction diagram characterizes column plastic hinges. The shear failure mechanism is also evaluated, as well as the flexural/shear interaction mechanism (NZSEE

2017). Potential failure mechanisms in the beam-column joints (Pampanin et al. 2002) are also explicitly accounted by modelling the panel zones using rigid arms with dedicated nonlinear rotational springs to represent the capacity of the joints, as suggested in Pampanin et al. 2003. Specifically, the springs are characterized by equivalent column moment versus drift relationships, derived by principal tensile/compression stresses considerations; an axial load-moment interaction diagram is also implemented to consider the influence of the axial load on the beam-column joint capacity. A linear distribution of the lateral force profile is adopted for the seismic loads.

3.3. Results of nonlinear static analyses

Using the modelling approach outlined in the previous section, 81 pushover curves have been derived for each case of beam-column joint detail. As an example, Fig.3a shows the pushover curves for case 2 (i.e., beam bars bent away from the joint). Results highlight that the material properties have a direct effect on the RC members capacity and therefore on the hierarchy of strength of beam-column joint subassemblies, leading to different global behavior. Fig.3b shows the plastic mechanism in case of the red (lowest strength capacity) and the blue (highest strength capacity) pushover curves in Fig.3a. The red curve has been derived considering f_c=8.38MPa and f_y=269.44MPa, while the blue one considering f_c=43.02MPa and f_y=361.04MPa. The pushover curves obtained from the MDoF numerical models are then used to evaluate the equivalent SDoF systems following the procedure outlined in the Italian Building Code (NTC 2018), and finally, using the two alternative code-compliant approaches (i.e., "Method A" or "Method B") the bilinear curves (base shear vs. effective height displacement of the equivalent SDoF system) are derived, Fig.3c.

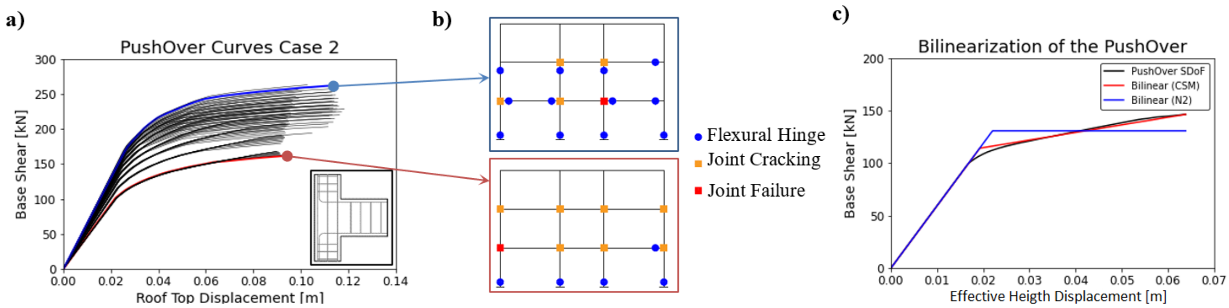

Fig. 3. (a) Pushover curves varying the material properties for case 2; (b) plastic mechanisms for two specific cases of material properties; (c) alternative bilinearization methods for the numerical pushover curve.

3.4. Results of the seismic risk classification

The seismic risk classification of the case-study buildings has been carried out according to the Italian "Guidelines for seismic risk classification of buildings" (SismaBonus, DM 65 2017). The seismic Risk Class of the building is defined as the minimum between the two classes associated to the IS-V index and the PAM index. Both indexes have been defined for each case study building following the two methods compliant with the Italian Building Code (NTC, 2018), i.e., the "Method A"/N2 and the "Method B"/CMS. Both methods allow the evaluation of the performance point of the structure through a Capacity/Demand comparison in the ADRS domain. However, the former (N2 method) uses (pseudo-)inelastic spectra modified by a reduction factor function of the ductility (demand at the intersection/performance point) of the structure, while in the latter (CSM), overdamped elastic spectra with equivalent viscous damping are considered. It is worth mentioning that the equivalent viscous damping value depends on the ductility demand and the hysteretic behavior of the structure, accounted by a specific coefficient. In this study, the coefficient related to structures characterized by reduced dissipative capabilities (typical of pre-1970's buildings) is considered (NTC, 2018). Fig.4 shows the performance points defined for the 81 pushovers for case 2 (i.e., beam-column joints reinforcement details featured by beam bars bent away from the panel zone region) using the "Method A" (N2 method) (a), as well as "Method B" (Capacity Spectrum Method) (b).

The Life Safety index (IS-V) is defined as PGA_C/PGA_D, where PGA_C is the capacity in terms of PGA (Peak Ground Acceleration) causing the attainment of the Life Safety Limit State (LSLS) of the structure, and PGA_D is the demand of an equivalent newly designed building at the same site.

The economic index (PAM or EAL) estimates the behavior of the building in terms of expected economic annual losses. In order to calculate the PAM index, the seismic performance of the structure is evaluated at different earthquake intensity levels associated to a certain return period T_R. Using the return period, it is possible to define the mean annual frequency of exceedance (MAF, λ=1/T_R) which is related with a repair cost (expressed as a fraction of the Reconstruction Cost, *%RC*) depending on the considered limit state. Connecting the points (λ, *%RC*) it is possible to define the PAM curve and to obtain the associated value of expected annual losses as the area above the curve itself. Fig.4c shows the PAM curves for the 81 pushover curves of case 2 defined using the CSM approach.

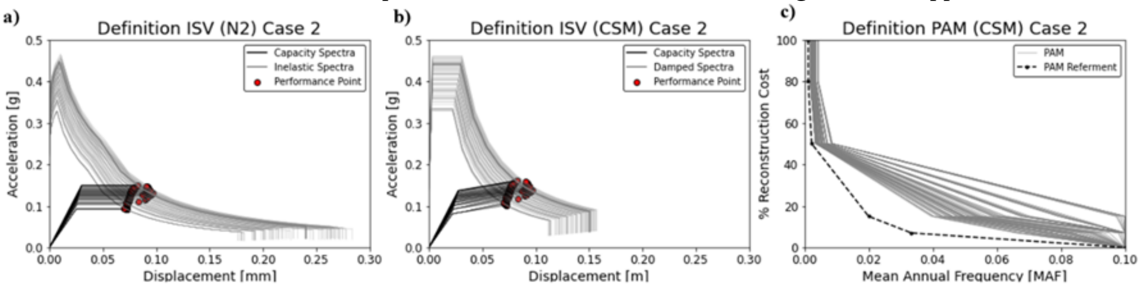

Fig. 4. (a) Performance points for case 2 defined using N2 method; (b) Performance points for case 2 defined using CSM method; (c) PAM curves for case 2 defined using CSM method.

The IS-V, as well as PAM values, have been determined for each of the 243 case study buildings. Fig.5a and Fig.5b compare the values derived from the two approaches (N2 and CSM) in the form of scattergrams. It is worth noting that the CSM results appear to be more conservative with respect to N2 method (in obvious turn, this would mean that the N2 method would appear to be less conservative than the CSM method) in all cases, for both the IS-V index (lower values defined with the CSM) as well as for PAM values (higher values defined with the CSM). Table 1 shows the results of IS-V and PAM in terms of both mean μ and standard deviation σ for each case of beam-column joints reinforcement detail (i.e., case 1, 2, and 3) varying the material properties.

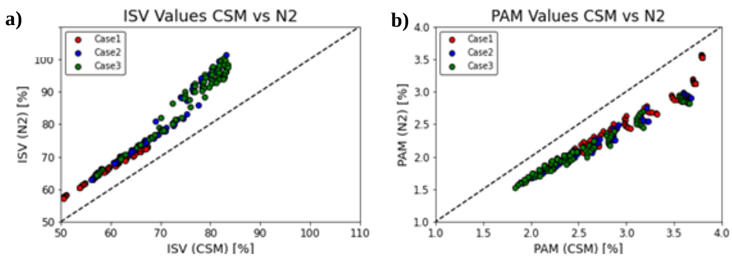

Fig. 5. (a) Scattergram IS-V values defined according to N2 and CSM method; (b) Scattergram PAM values defined according to N2 and CSM method.

Table 1. Mean μ and standard deviation σ values for every case of beam-column joint reinforcement details.

Case	$\mu_{IS-V,CSM}$	$\sigma_{IS-V,CSM}$	$\mu_{IS-V,N2}$	$\sigma_{IS-V,N2}$	$\mu_{PAM,CSM}$	$\sigma_{PAM,CSM}$	$\mu_{PAM,N2}$	$\sigma_{PAM,N2}$
1	60.30%	5.17%	66.82%	4.84%	3.10%	0.47%	2.63%	0.46%
2	70.97%	8.69%	81.16%	11.71%	2.60%	0.52%	2.15%	0.40%
3	72.37%	8.46%	83.28%	11.30%	2.56%	0.51%	2.11%	0.39%

The minimum mean value of IS-V is evaluated through the CSM for case 1 (μ = 60.30%), while the maximum is obtained through the N2 method for case 3 (μ = 83.28%). Further, considering the case 3, the maximum difference among the mean values of IS-V (10.91%) is obtained. The minimum value of standard deviation is derived for case 1 using the N2 method (σ=4.84%), while the highest value is expected for case 2 using the N2 method (σ=11.71%). Further, it is possible to note that the highest values of standard deviation are obtained for case 2 and 3. This is mainly due to the different inelastic mechanisms observed. Specifically, all the configurations of case 1 are characterized by local brittle mechanisms of external beam-column joints, whilst in some samples of case 2 and 3, more desirable mechanisms featured by the flexural hinging of members are observed. This leads to a more ductile behavior; thus, higher values of IS-V are obtained with respect to the cases characterized by brittle failure of beam-column joint.

Considering the results in terms of PAM, the maximum value is observed for case 1 (3.10%) using the CSM, while the lower is obtained through the N2 method for case 3 (2.11%). The maximum difference of the mean values in terms of PAM is determined for case 1 (0.45%). In this case, the standard deviation is roughly constant (ranging from 0.39% to 0.52%). The lower dispersion of these values with respect to the IS-V ones is justified as the PAM index is mainly governed by the performance of the structure for frequent events (Operational and Damage-Control Limit States), therefore a better performance at Life Safety Limit State (strictly correlated to the IS-V), when a more desirable plastic mechanism is predicted, has a reduced effect on the PAM values.

Further, following the approach presented in the Italian "Guidelines for seismic risk classification of buildings" (DM 65, 2017) the seismic risk class of each case study building has been defined. Using the CSM, 95 case studies have been ranked as class "C", 106 as class "D" and 42 as class "E", while considering the N2 method, 169 as class "C", 65 as class "D" and 9 as class "E". Specifically, even considering the same pushover curves and hazard, the two alternative code-compliant methods leads to a different seismic ranking in the 44% of all cases. Clearly, these differences could have significant repercussions in terms of actual safety assessment, decision-making on the extent of the retrofit intervention, access to financial incentives etc.

Finally, the influence of the ductility capacity of the structure in the seismic assessment through both N2 Method and CSM is investigated. Fig.6 shows the correlation between IS-V (a) and PAM (b) values with the ductility capacity.

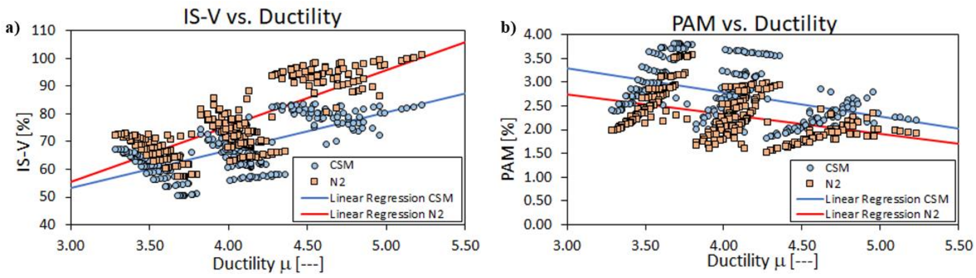

Fig. 6. (a) Comparison between IS-V values and ductility of the structure; (b) Comparison between PAM values and ductility of the structure.

The linear regressions of the data clearly show how the difference between the IS-V values, defined through the N2 and the CSM, increases when higher values of ductility are attained by the structure under investigation. On the contrary, considering the PAM values, even if the Capacity Spectrum Method appear to be more conservative with respect to the N2 method, the linear regressions are quite parallel. This confirms that the ductility level does not seem to play a significant role when computing the PAM loss index, but rather the IS-V Life-Safety Index, as stated before.

4. Conclusions

In this paper, a comparative analysis of different code-compliant approaches for the seismic assessment of buildings through nonlinear static analyses (pushover) was presented, with a specific focus on the two methods reported in the Italian Building Code, namely "Method A" and "Method B" (referring to N2 method and the Capacity Spectrum Method, respectively). The adopted research methodology involved a parametric analysis of different RC frame structures, in order to consider a wide range of pushover capacity curves. Specifically, the nonlinear static analyses were performed by implementing a MDoF numerical model in the finite-element software Ruaumoko. The seismic performance of the structures was then assessed by applying the two spectrum-based methods, following the code provisions. Results were finally compared in terms of safety index (IS-V or %NBS) as well as Expected Annual Losses

(EAL or PAM), following the approach described in the so-called Italian "SismaBonus" guidelines (DM 65, Cosenza e al. 2018). Following the two code-compliant approaches, the comparison highlights that the ductility capacity of the structure strongly affects the results obtained, specifically in terms of IS-V, leading to higher differences when more ductile structures are considered. Moreover, this study pointed out that in 44% of the analyzed configuration, the two code-compliant assessment methodologies led to different seismic risk classifications of the structure. This is deemed a critical aspect since the "SismaBonus" guidelines define the assessment of the seismic risk class of buildings in Italy, therefore the alternative code-compliant methods should at least lead to the same evaluation of the building itself. Therefore, specific code provisions/guidelines are needed to overcome this issue.

This work could be considered as a preliminary step towards the development of specific guidelines including provisions on the recommended simplified approach for seismic assessment of buildings in practical applications. These provisions should be based on the observed/expected seismic behavior of the analyzed structure. Nevertheless, future investigations are needed to better understand the differences between the two approaches, for instance by considering a wider range of structures including also modern (code-conforming) or retrofitted existing buildings. Moreover, a future comparison with the results of more refined analysis methods (e.g., nonlinear dynamic time-history analysis) is needed to assess the effectiveness of the methodologies and provide suggestions for the recommended simplified approach in practical applications.

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