Spectral Shape Proxies and Simplified Fragility Analysis of Mid-Rise Reinforced Concrete Buildings

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ABSTRACT: The objective of this study is to identify an optimal intensity measure (IM) for conditioning probabilistic seismic demands of case-study reinforced concrete (RC) frame buildings, representative of mid-rise RC building classes in the Mediterranean region. The prediction is performed via statistical relationship between multiple IMs (particularly advanced scalar parameters accounting for spectral shape over a range of periods) and various displacement-based engineering demand parameters (EDPs). Such statistical relationships are built on data obtained from analysis of the frames subjected to over nine hundred ground motion records by employing an innovative capacity spectrum method, introduced in the paper, which uses inelastic response spectra derived from actual earthquake accelerograms to estimate seismic demand and derive fragility curves. The outcomes of the present work are in a good agreement with previous investigations conducted by other researchers on selecting optimal IMs for predicting structural response by using full nonlinear dynamic analyses for different structural typologies.

1. INTRODUCTION

Recent earthquakes in Maule, Chile (2010), Tohoku, Japan (2011) and Christchurch, New Zealand (2011) have resulted in extensive concentration of damage and significant losses in existing, low seismic designed, reinforced concrete (RC) structure and particularly mid-rise buildings for both residential and commercial occupancy. The limited availability of historical damage data associated with most seismic prone areas makes the derivation of analytical fragility functions (D'Ayala *et al.* 2014) an essential component of seismic risk assessment. In particular, nonlinear dynamic analysis (NLDA) represents the tool for assessing inelastic structural response with relatively low uncertainty, accurately capturing failure modes. Apart from the undoubted advantages of using NLDA, the required computational resources and high cost (in terms of time consumption), precludes this approach when analyzing large populations or portfolios of buildings, for example for catastrophe modeling purposes. In contrast, several variants of capacity spectrum methods based on incremental dynamic analysis (IDA) and static push-over analyses have been proposed. These capacity assessment methods, such as the N2 method (Fajfar, 2000), and the recently proposed FRACAS (introduced in *Section 2*) among others, often rely on simplifying assumptions in assessing both the structural capacity and the seismic demand. In particular, FRACAS uses suites of scaled and/or unscaled ground motion records (simply GMs hereinafter) and delivers immediately the fragility function of the considered structure.

Nonetheless, the effect of selecting and implementing different combinations of intensity measure (IMs) and engineering demand parameters (EDPs) in simplified fragility analysis has not been appropriately investigated. Thus, one faces the question of how suitable the adopted IM is for representing GM uncertainty. To this aim, the development of fragility functions requires the choice of an IM which is suitable to predict the response of the system with the smallest scatter ("efficiency") and providing a significant amount of information ("sufficiency") to predict the responses quantities involved in the performance objectives (e.g., Jalayer *et al*., 2012). In addition, many researchers have investigated other IM selection criteria, related for example to "hazard computability", "proficiency", and "practicality".

This paper aims to 1) introduce FRACAS, an effective tool for simplified seismic fragility analysis and, 2) shed light in comparing different IM/EDP combinations for the fragility analysis of mid-rise RC buildings by FRACAS.

2. FRACAS

In the current study, the simplified capacity assessment methodology, and related computer codes, known as FRACAS (FRAgility through CApacity Spectrum assessment) is implemented in order to determine the performance points (PPs) of case-study structures for different GM inputs. FRACAS is based on the displacement-based procedure, originally proposed by Rossetto and Elnashai (2005). The step by step procedure followed by the methodology is summarized below (Figure 1):

1. Conversion of a pushover curve (force-displacement space) for the considered structure to an equivalent single degree of freedom (SDoF) -based capacity curve (acceleration-displacement response spectrum, ADRS format) taking into consideration the floor masses and the inter-story displacements (Figure 1a).
2. Idealization of the capacity curve. The user can choose different idealized models, yielding point, ultimate point and hardening options (Rossetto *et al*., 2014 and Figure 1a).
3. Discretization of the capacity curve to a series of checking points associated with various pre- and post-yield periods. The number of pre- and post-periods can be selected by the user (Figure 1b).
4. Computation of elastic response spectrum from the inputted GMs. The elastic demand is calculated for periods up to the yielding period T_y (Figure 1c).
5. Calculation of the inelastic demand of the equivalent SDoF for the selected post-yield periods (Figure 1d).
6. Determination of PP at the intersection of the capacity with the demand curve (Figure 1d). The corresponding EDP values are then obtained from the back-calculation of PP to the force-displacement format.

It is noteworthy to mention that unlike other capacity spectrum methods, FRACAS does not rely on reduction factors or indices to determine the inelastic spectrum from the elastic one. Instead, it carries out, for each target ductility and period, a simplified dynamic analysis on the idealized nonlinear SDoF model corresponding to the capacity curve. This feature also has the advantage of permitting the use of various GM records that generate unsmoothed spectra as opposed to standardized design spectra. Therefore, the record-to-record variability can be directly introduced and the resulting cloud of performance points leads to fragility curves that account for the natural variability in the seismic demand. In particular, the computed EDPs corresponding to different scaled/unscaled seismic demand inputs, in conjunction with user defined damage states are used for the generation of analytical fragility curves. This method is recommended in the recently published GEM Guidelines for Analytical Vulnerability Estimation, (D'Ayala *et al.* 2014), where further details are also provided.

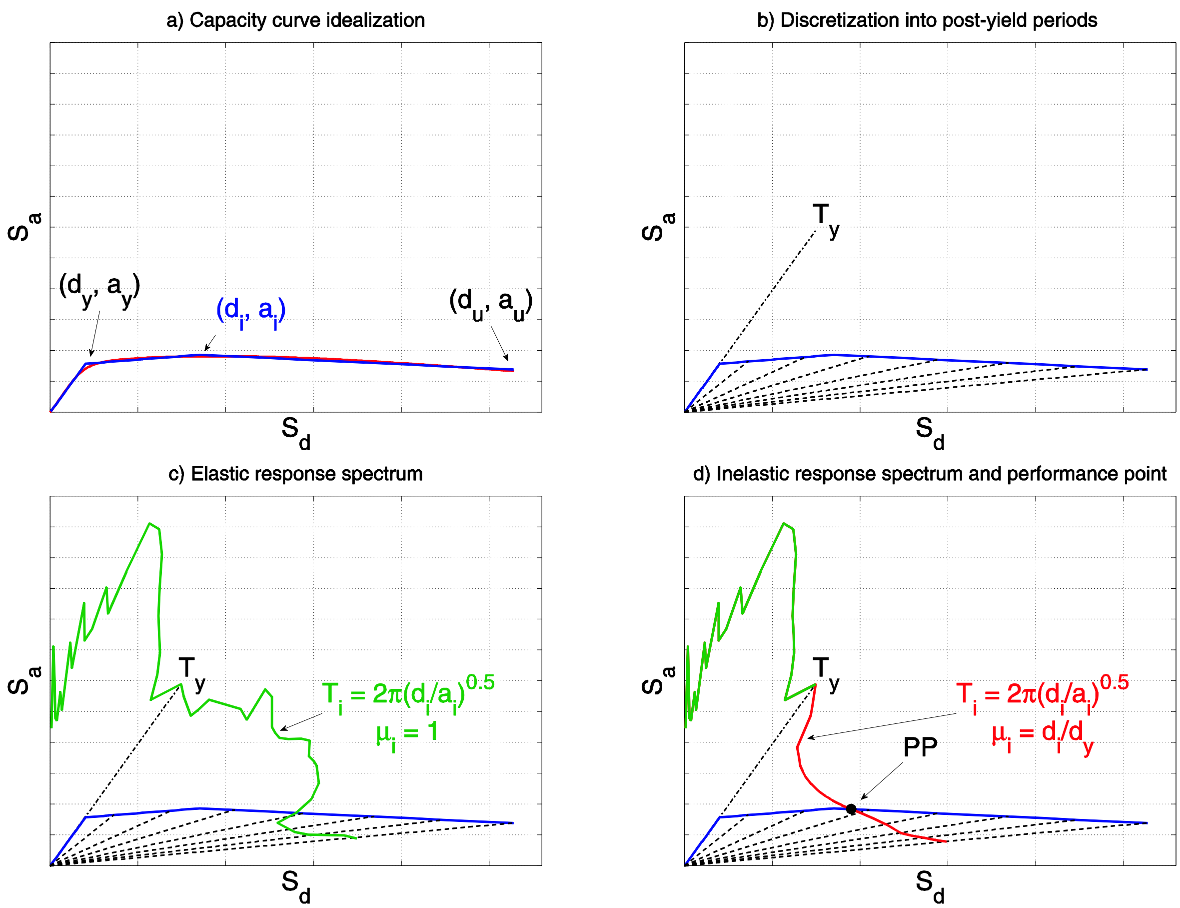

Figure 1. Main steps of FRACAS for the derivation of the Performance Point using trilinear idealization model.

3. CONSIDERED INTENSITY MEASURES

In order to quantify the GM features that influence the nonlinear response of the structures of interest, several types of IMs are tested. Conventional IMs namely peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), and spectral (pseudo-) acceleration at the initial fundamental period (for a damping ratio of 5%), are the most commonly used IMs and are considered here. In general, PGA and $S_a(T_1)$ poorly predict the structural response of mid- to high-rise moment resisting frames (MRFs), although the latter IM sufficiently captures the elastic behavior of first-mode dominated SDoF systems, especially in the case of low to moderate fundamental periods. However, the behavior of highly nonlinear structures or structures dominated by higher-mode periods (less than T_1) are not very well represented by utilizing $S_a(T_1)$ due to the lack of information on the spectral shape provided by this IM. Therefore, it is becoming essential to implement advanced IMs that account for the elongated periods and/or consider nonlinear demand dependent structural parameters. More specifically, the first advanced scalar IM considered is S_a^c (proposed by Cordova *et al.*, 2000), which utilizes spectral shape information (period elongation), and is expressed as:

$$S_a^c = S_a(T_1)\left[\frac{S_a(cT_1)}{S_a(T_1)}\right]^{\alpha} \tag{1}$$

where c and α are coefficients assumed to be c = 2 and α = 0.5 respectively, based on the calibration carried out by the authors in the original study.

Bojórquez and Iervolino (2011) also proposed the advanced scalar IM, I_{N_p}, which is based on $S_a(T_1)$ and the parameter N_p, defined as:

$$I_{N_p} = S_a(T_1) N_p^{\alpha} \tag{2}$$

where α is a parameter to be calibrated and N_p is defined as:

$$N_p = \frac{S_{a,avg}(T_1,\ldots,T_N)}{S_a(T_1)} = \frac{\left[\prod_i^N S_a(T_i)\right]^{1/N}}{S_a(T_1)} \tag{3}$$

where T_N corresponds to the maximum period of interest and lays within a range of 2 and 2.5T_1, as suggested by the authors. In this study T_N value is obtained directly from the FRACAS analysis (*Section 4*). Ten different values, from 0.1 to 1, for the α- parameter are considered here in order to identify the optimal value for α, to follow.

4. CASE STUDY STRUCTURES

Two regular RC 4-storey, 4-bay bare frames, representing different vulnerability classes based on the design codes used for their construction, are selected to illustrate the evaluation of the studied IMs. Specifically, the two selected case-study structures share the same geometry (bay widths and story heights) but characterized by different material properties, elements geometry and reinforcement detailing. The first frame is designed to only sustain gravity loads following the Royal Decree n. 2239 of 1939 that regulated the design of RC buildings in Italy up to 1971, hereafter Pre-Code building; the second frame is designed according to the latest Italian seismic code (or NIBC08; CS.LL.PP. 2008), following the High Ductility Class (DCH) rules, hereafter Special-Code building. Further information regarding the design of those two buildings is available in De Luca *et al.* (2009). Inter-story heights, span of each bay and cross-sections dimensions for the two case-study building are reported in Figure 2. The considered frames are regular (both in plan and in elevation); the dimensions in brackets refer to the Pre-Code building (all beams have the same cross-sections in both cases).

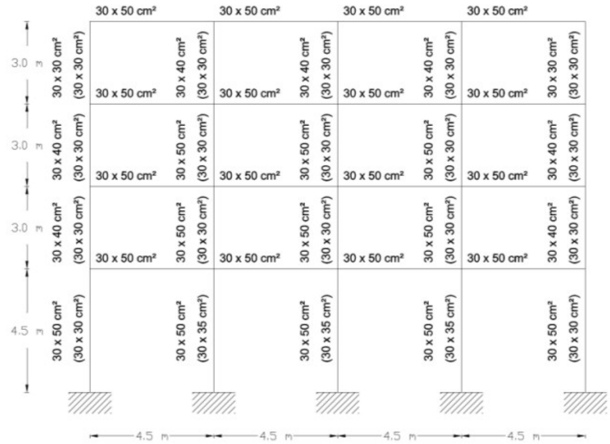

Figure 2. Elevation dimensions and members cross-sections of the case-study buildings.

The two case-study frames are modeled using the SeismoStruct finite element software (http://wwwseismosoftcom); two separate sets of conventional static PO methods are selected for the analysis of the abovementioned frames. Incremental lateral loads are applied in different load phases at the side nodes at each floor level. The lateral load increments are distributed uniformly or following an inverse triangular pattern (uniform PO and triangular PO), corresponding to floor masses and story heights respectively. Table 1 summarizes the dynamic information associated with each of the tested buildings required to compute different IMs, namely structural analysis method, fundamental period T_1 (based on Seismostruct and FRACAS estimations, denoted as T_1*) as well as elongated period T_N. T_1* is derived from the stiffness of the idealized capacity curve used in FRACAS, while the elongated period T_N corresponds to the ultimate point of the capacity curve associated to each building-PO analysis method.

Figures 3 (top panel) presents the static PO curves (triangular PO for illustrative purposes) for the case-study buildings. The curves are reported in terms of top center of mass displacement divided by the total height of the structure (i.e., the roof drift ratio, RDR) along the horizontal axis of the diagram and base shear divided by the building seismic weight (i.e., the base shear coefficient) along the vertical axis. It is noted that a highly nonlinear behavior is observed over certain RDR thresholds for the studied structures.

Table 1. Dynamic information for each case-study structure.

Building	PO	T_1 (s)	T_1* (s)	T_N (s)
Pre-Code	TRI	0.889	1.106	2.408
	UNI	0.889	1.058	2.393
Special-Code	TRI	0.498	0.717	1.885
	UNI	0.498	0.681	1.903

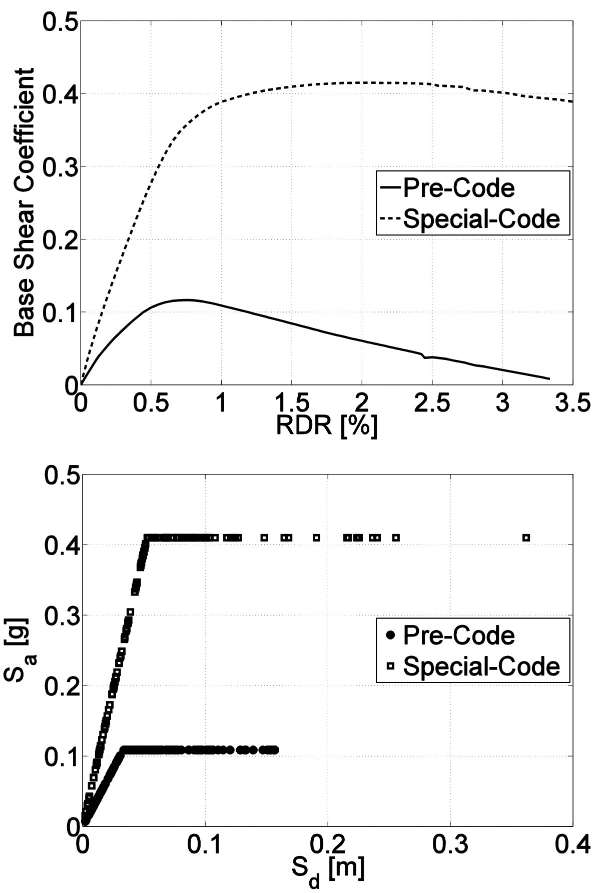

Figure 3. Static PO curves for the case-study buildings (top) and performance points generated by FRACAS (bottom).

Figure 3 (bottom panel) shows the performance points in the ADRS space computed by FRACAS (an Elastic Perfectly Plastic idealization model is employed) by using the GMs records described in *Section 5*.

5. GROUND MOTION DATABASE

The SIMBAD database (*Selected Input Motions for displacement-Based Assessment and Design*; Smerzini et al., 2014), used here, consists of 467 records, each including the two horizontal (X-Y) and one vertical (Z) components (1401 recordings), generated by 130 seismic events (including mainshocks and aftershocks) that occurred worldwide. These accelerograms are assembled from various ground motion databases derived for different regions of the world following the selection criteria addressed below:

1. Shallow crustal earthquakes worldwide with moment magnitude (M) ranging from 5 to 7.3 and epicentral distance R $\leq$ 35 km. This ensures to provide strong ground motion records of engineering relevance for most of the design conditions of interest that can be used without introducing large scaling factors.
2. Good quality at long periods, so that only records for which the high-pass cut-off frequency used by the data provider is below 0.15 Hz are considered. Therefore, most records are from digital instruments (about 80%), while from analog instruments only those records with a good signal to noise ratios at long periods, typically from large magnitude earthquakes, are retained.
3. Availability of site class information based on quantitative criteria.

6. METHODOLOGY

In the present study, statistical regression techniques are implemented to determine the IM that better predicts each considered EDPs. Hence, to determine the statistical properties of the *cloud* response, the linear least squares is applied on EDPs versus IMs pairs for the suite of GMs (unscaled) in order to estimate the conditional mean and standard deviation of

EDP given IM. The simple power-law model in Eq. (4) is used here:

$$EDP = aIM^{b} \tag{4}$$

where a and b are the parameters of the regression. The regression's standard deviation (s) is assumed to be constant with respect to IM over the range of IMs in the cloud. The power-law model illustrate in Eq. (4) can be simply re-written as shown below in Eq. (5), as a linear expression of the natural logarithm of the EDP and the natural logarithm of the IM:

$$\ln(EDP) = \ln(a) + b\ln(IM) \tag{5}$$

The use of logarithmic transformation indicates that the EDPs are assumed to be conditionally lognormally distributed (conditional upon the values of the IMs); this is a common assumption that has been confirmed as reasonable in many past studies. In the current study, the focus is laid on deformation-based EDPs, which are listed below:

1. peak (over time) inter-story drift ratio, as the largest difference between the lateral displacements of two adjacent floors, divided by the height of the story (denoted as IDR_i for story i-th);
2. maximum (over all stories) peak interstorey drift ratio (denoted as MIDR);
3. ratio of the peak lateral roof displacement to the building height (i.e., RDR).

The abovementioned have demonstrated to be well correlated to both structural and non-structural damage. Thus, they can be used to compute local or global instability of RC MRFs.

7. OPTIMAL IM SELECTION CRITERIA

As discussed in *Section* 1, the selected IM has a significant effect on the uncertainty associated with the resultant fragility curves. Therefore, the selection of optimal IMs is of high importance within the entire risk assessment process and consequently, raised the need for defining quantitative and qualitative selection criteria in order to facilitate this selection. The most commonly used criteria for the determination of an optimal IM used in this study are briefly discussed in the following subsections:

7.1. Efficiency

Efficiency is the most commonly used quantitative criterion for the determination of optimal IMs, and is related to the variation of demand estimates for different values of the considered IM (e.g., Giovenale *et al*., 2004). Specifically, more efficient IMs result in reduced dispersion of the median EDP estimates conditional to a given IM. As a result, less analysis runs are required to narrow down the confidence intervals. An efficient IM is the one that provides the smallest value of the standard deviation s from the regression analysis.

7.2. Sufficiency

An innovative definition of sufficiency, in particular relative sufficiency, was recently proposed by Jalayer *et al*. (2012). In particular, to investigate the relative sufficiency of a second IM, i.e. IM_2, with respect to a first one, i.e. IM_1, a quantitative measure may be employed. This measure is derived on the basis of information theory concepts and quantifies the suitability of one IM relative to another. Specifically, the relative sufficiency measure, denoted herein as $I(EDP|IM_2|IM_1)$, is equal to the average difference between the information gained about the performance variable EDP given IM_1 and IM_2 and that gained given IM_1 only. Therefore, for each cloud analysis performed, one can estimate this measure using the equations provided in Jalayer *et al*. (2012). The relative sufficiency measure is expressed in units of bits of information. If the relative sufficiency measure, $I(EDP|IM_2|IM_1)$, is zero, this indicates that on average the two IMs provide the same amount of information about the EDP. In other words, they are equally sufficient. If the relative sufficiency measure is positive, this means that on average IM_2 provides more information than IM_1 about the EDP, so IM_2 is more sufficient than IM_1. Similarly, if the relative sufficiency measure is negative, IM_2 provides on average less information than IM_1 and so IM_2 is less sufficient than IM_1.

7.3. Hazard computability

According to the definition given by Giovenale *et al*. (2004), hazard computability describes the process to obtain the earthquake hazard for a given IM. Numerous hazard maps and Ground Motion Prediction Equations (GMPE) exist for more commonly used IMs, namely PGA and spectral ordinates at given periods (representing sometimes a restricted range of possible discrete periods), making these IMs more favorable from the hazard computability perspective; whereas, other IMs may require interpolation or supplementary structural or dynamic information, making the computation of the hazard a more time-consuming process.

8. RESULTS AND DISCUSSION

For sake of brevity, only the results for the case of triangular PO loads and MIDR are presented; the aim is to show the process to determine the optimal IM for the fragility analysis of the particular building class. However, the same methodology is applied to all the case-study buildings and results of the analysis, essentially consistent across all the case-study buildings and EDPs, are reported in Minas (2014). As shown in Figure 3 (top panel) the selected structure behaves highly nonlinearly over certain RDR thresholds. As a consequence, the actual number of GM that pushed the frame into the nonlinear range is relatively small but still statistically significant. Therefore, the regression parameters *a, b, s* and R^2 for each EDP and each IM are estimated only considering the GM records resulting in actual nonlinear response.

Figure 4 shows the obtained *s* values corresponding to MIDR vs IMs regression for both case-study building. With regard to efficiency, the visual inspection of Figure 4 confirm that deformation-based EDPs appear to be better correlated with the spectral shape parameter I_{N_p} (the optimal α-value can be identified from Figure 4); while $S_a(T_1)$ performs better than the other conventional IMs and closely matches the S_a^c estimations. It is also confirmed that PGA, as well as PGD, are poor predictors of the nonlinear structural response of mid- to high-rise moment resisting frames (highest values of *s*). For the Special-Code building, the spectral shape parameter S_a^c provide the highest values of *s* comparing to the other advanced IMs, but still outperforms all conventional scalar IMs. A potential improvement may be obtained by calibrating *c* and α (in the case of S_a^c) for the specific case-study structures rather than using the values suggested by other researchers for different case-study structures.

The relative sufficiency measure for MIDR and the candidate IMs is shown in Figure 5 for both buildings. The considered IM_2 is the one corresponding to the lowest *s* value from the regression (Figure 4). The results in Figure 5 confirm the results in terms of efficiency (Figure 4). The IMs resulting in the highest efficiency are also characterized by the highest relative sufficiency.

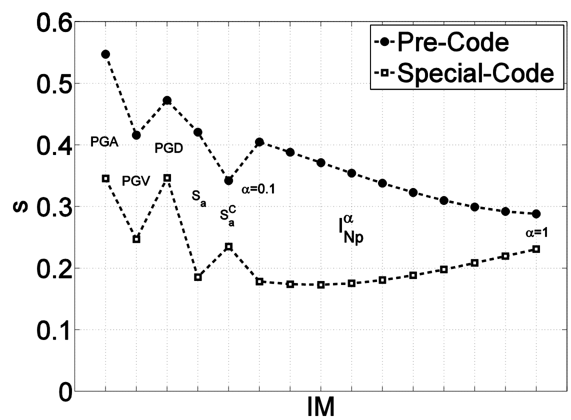

Figure 4. Standard deviation (dispersion) of residuals of MIRD for the considered IMs and each case-study building.

Last criterion for the determination of an optimal intensity measure is the hazard computability. For the current criterion, conventional IMs have a significant advantage over the advanced ones, as numerous GMPEs and hazard maps exist particularly for PGA, PGV and PGD, and some spectral ordinates for specific ranges of periods. On the other hand, it is still possible to derive GMPE for spectral acceleration–based advanced IMs, as shown in Cordova *et al*. (2000) and Bojórquez and Iervolino (2011).

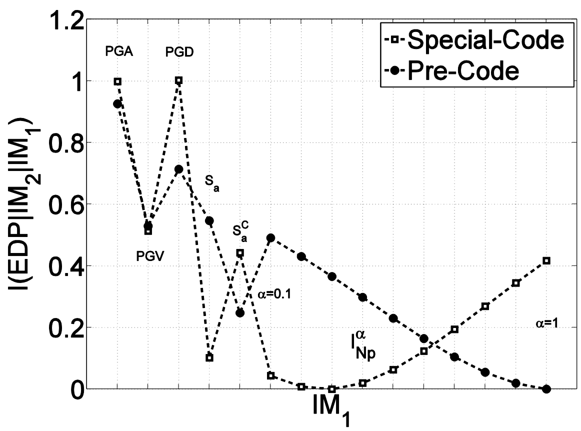

Figure 5. Relative sufficiency measure for alternative IMs with respect to the IM with the lowest dispersion (Figure 4) for each case-study building.

9. CONCLUSIONS

This paper summarizes the results of an investigation aiming at identifying the GM parameters that are better correlated with displacement-based response parameters for simplified fragility analysis of mid-rise RC buildings. The outcomes of the present work are consistent with previous investigations conducted by the authors and other researchers on selecting optimal IMs (scalar or vector-valued) for predicting structural response by using NLDA. In general, the advanced IMs, properly calibrated for the specific building typology, that account for the period elongation and demand dependent structural parameters, comfortably satisfy all the selection criteria, and represent then optimal IMs for simplified fragility analysis of mid-rise RC buildings.

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