TIME-DOMAIN-BASED INTENSITY MEASURE FOR THE COLLAPSE ESTIMATION OF MOMENT FRAME BUILDINGS

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Abstract

Selecting a ground motion intensity measure (IM) that is strongly correlated with nonlinear structural response not only reduces a possible bias and the number of analyses required to estimate an engineering demand parameter with a certain level of confidence, but it also helps in the ground motion selection process required for a structural performance assessment. Among many IMs, the most commonly used is the 5%-damped pseudo-acceleration spectral ordinate of a SDOF system with a period of vibration equal to the fundamental period of the structure, $S_a(T_1)$. This study presents the development of an alternative IM aimed specifically at seismic collapse estimation, which is based on time domain features of the acceleration time series. This new IM is termed *FIV3* and it consists on the sum of the areas of the three largest acceleration pulses obtained from a period-dependent filtered acceleration time series. The efficiency, in other words, the level of dispersion on collapse capacities, and the sufficiency of this IM with respect to several ground motion parameters are evaluated and compared against the scalar IM $S_a(T_1)$, its adjustment using the spectral shape proxy ε, and two other recently proposed IMs: IM_{comb} and Sa_{avg}. Results from six moment frame buildings ranging from 1 to 20 stories suggest that *FIV3*, IM_{comb}, and Sa_{avg} are much more efficient and sufficient intensity measures than $S_a(T_1)$ or its adjustment using ε. Moreover, they suggest that *FIV3* is a promising candidate IM for seismic collapse estimation as it outperforms all the IMs evaluated in the four buildings with periods of vibration lower than 1s and is comparable to Sa_{avg} and IM_{comb} in the longer period buildings.

Keywords: Intensity measure, Incremental Velocity, Collapse estimation, Efficiency, Sufficiency.

1. Introduction

A ground motion intensity measure (IM) should serve to main purposes: (1) characterize the damage potential that the ground motion has over a structure; and (2) provide a link between the probabilistic seismic hazard analysis and the structural response analysis in what is commonly known as probabilistic seismic demand analysis. In the case of structural collapse assessments, the IM should provide relatively unbiased collapse intensities and minimize the computational effort involved in the required nonlinear response history analyses (NRHA) by providing collapse intensities with low dispersion.

In the Performance-Based earthquake engineering framework of the Pacific Earthquake Engineering Research (PEER) Center [1-3], the most commonly used IM for seismic collapse estimations is the 5%-damped pseudo-acceleration spectral ordinate at the fundamental period of vibration (T_1 or T_N) of the structure, S_a. Thus, many recent seismic design guidelines and studies adopt this IM to assess the seismic performance of buildings, including collapse risk estimation (e.g., [4, 5]) even when several researchers have pointed out several important shortcomings in its use (e.g., [6-8]). As a result, several methods for reducing some of its shortcomings (e.g., [9, 10)]) or alternative IMs that are specifically aimed at dealing with structural collapse assessment (e.g., [11-14]) have been proposed.

Recognizing the bias that could be introduced in the estimation of collapse capacities from structural models, a method to adjust the S_a-based collapse fragility curve using the spectral shape proxy ε has been proposed [4, 9]. The parameter ε is defined as the number of standard deviations by which the logarithmic spectral acceleration ordinate of the ground motion record differs from the mean logarithmic spectral acceleration ordinate estimated by ground motion prediction model. By conducting this adjustment, the bias in collapse risk resulting from using records with spectral shapes (measured through its proxy ε) significantly different from the target spectral shape is reduced. Nonetheless, this method does not provide a significant reduction in the dispersion of the collapse capacities [9, 13], which means that the efficiency of combining S_a and ε, which consists in correcting S_a by ε (hereafter referred to as $S_a+\varepsilon$) is similar to the one using only S_a.

Sa_{avg} and IM_{comb} are two advanced IMs that have been recently proposed by Eads et al. [13] and Marafi et al. [15], respectively, which can outperform S_a and $S_a+\varepsilon$ both in terms of efficiency and sufficiency [8]. Eads et al. [13] proposed Sa_{avg}, which is defined as the geometric mean of pseudo-acceleration spectral ordinates in a range of periods between $0.2{\cdot}T_1$ and $3{\cdot}T_1$. Based on results from nearly 700 moment-resisting frame and shear wall structures they concluded that Sa_{avg} is in most cases more efficient, more sufficient, and provides more stable collapse risk estimates when using different ground motion sets than when using S_a or $S_a+\varepsilon$ as IMs. They demonstrated that one of the reasons behind the superior performance of Sa_{avg} is that it includes information about the spectral ordinate at T_1 relative to a wide range of spectral ordinates at periods shorter and longer than T_1, which makes it an improved measure of spectral shape. Marafi et al. [15] proposed IM_{comb}, a ductility-dependent IM which combines S_a with a measure of spectral shape very similar to the one proposed in [13] and the duration of the record as measured by the Trifunac-Brady significant duration [16]. Its adequate efficiency and sufficiency was demonstrated using a wide range of elasto-plastic SDOFs and collapse capacity results from 30 reinforced concrete (RC) special moment-frame buildings.

Recently, Dávalos and Miranda [17] used collapse results from a 4-story RC building to propos a new IM referred to as *FIV3* that is based directly on time-domain features of the ground motion record, namely, characteristics of a small number of long duration acceleration pulses. This novel IM is based on a period-dependent version of the incremental velocity (*IV*) parameter proposed by Bertero and his collaborators in the 70s (i.e., [18, 19])) but computed from a low-pass filtered acceleration time series. Furthermore, rather than only focusing on the pulse with the largest ground incremental velocity, *FIV3* considers the three pulse segments with the largest area under the acceleration time history accumulated over a period-dependent time interval acting on the same direction. The consideration of pulse segments with the same sign is aimed at improving the IM's correlation with large inelastic excursions and minimizing the variability in structural collapse capacities by capturing a possible ratcheting behavior in the structure leading to collapse. In that study the authors conducted a first evaluation of the efficiency in collapse capacity estimates, sufficiency, and scale factor robustness of *FIV3*. They concluded that *FIV3* appeared to be a very promising IM as it was highly efficient and was adequately sufficient with respect to several ground motion characteristics such as magnitude, spectral shape, significant duration, etc. However, they mentioned that further research was necessary in order to extend those conclusions to a wider range of structures.

The main objective of this paper is to assess the performance of *FIV3* as an IM for seismic collapse estimation using several moment-resisting-frame buildings ranging from one to twenty stories and with fundamental periods of vibration ranging from 0.42s to 2.36s. The buildings used in this investigation are steel and concrete structures that have been studied previously and that are well documented in the literature [20, 21]. The performance of *FIV3* as an IM is evaluated by quantifying its efficiency and sufficiency with respect to moment magnitude, spectral shape, and strong motion duration. Moreover, it is compared with traditional intensity measures (S_a and $S_a+\varepsilon$), as well as with recently proposed improved IMs that either inherently or explicitly account for improved measures of spectral shape (Sa_{avg} and IM_{comb}).

2. Structures and set of ground motions

This study evaluates the performance of *FIV3* using six different models of moment-resisting frame structures. The first structure is a four-story steel special moment-resisting frame structure designed by Lignos and Krawinkler [22] and modeled in the OpenSees structural platform [23] by Eads and Miranda [20]. The nonlinear modeling of the model consists of concentrated plasticity elements at the ends of beams of columns that are characterized by a modified version of the IMK deterioration model. Additional information regarding this model is provided in [20, 22]. The remaining five models are reinforced concrete (RC) special moment frames designed and modeled by Haselton and Deierlein [21] in OpenSees using two-dimensional frames with the objective of evaluating the collapse risk of RC moment frame buildings designed with recent codes in California. The number of stories of these models ranges between 1 and 20 stories. The nonlinear modeling of these structures consists of concentrated plasticity springs at the ends of beams and columns and beam-column elements characterized by a hysteretic model based on the modified Ibarra-Medina-Krawinkler (IMK) but with parameters specifically calibrated for RC structures. Each model captures both stiffness and strength deterioration, including in-cycle degradation in the behavior of the nonlinear elements. P-Δ effects are incorporated using a leaning column connected to the main frame using axially rigid beams pinned at their ends, which carries gravity loads not acting directly on the modeled lateral resisting system. Table 1 presents information on the number of stories, the fundamental period of vibration, and the material of the six structural models.

All of these models were subjected to Incremental dynamic analyses (IDA) [24] in order to estimate the collapse intensity using a set of 269 acceleration records. These were selected from the the PEER NGA-West ground motion database [25]. All the records are from free-field stations located on sites with NEHRP site classes C and D. They correspond to 11 earthquake events from active crustal regions having moment magnitudes, M_w, ranging between 6.9 and 7.6 and Joyner-Boore distances, R_{jb}, between 0 and 27 km.

For each structure, ground motion records are only considered if the scale factor required to trigger collapse does not exceed 20.

Table 1. General information of the structural models.

Structure	ID	# of stories	T_1 [s]	Material
1	2061	1	0.42	RC
2	1001	2	0.63	RC
3	5000	4	1.33	Steel
4	1008	8	1.80	RC
5	1012	12	2.14	RC
6	1021	20	2.36	RC

3. *FIV3* motivation and development

Anderson and Bertero [18] conducted a study of the structural response of a steel moment frame subjected to ground motions recorded during the 1978 Imperial Valley earthquake and concluded that the maximum incremental velocity, *IV*, defined as the largest area under an acceleration pulse between two consecutive zero acceleration crossings, is a parameter closely related to structural damage potential of a ground motion.

An important shortcoming of the original definition of *IV* when using it as an IM is that it is independent on the period of vibration of the structure, T_n. In other words, it indicates the same intensity level for structures with different periods of vibration. Nonetheless, it is well known that the same ground motion may affect very differently structures with different periods of vibration. In order to demonstrate the close relation between structural damage and acceleration pulses but also the influence of the ratio between the period of vibration of the structure and the pulse period, Figure 1 presents segments of ground acceleration and ground velocity time series along with the lateral displacement response of two SDOF systems subjected to the EW component of the 1999 Chi-Chi earthquake recorded at the TCU068 station. In both cases, the SDOFs have a bilinear hysteretic behavior with a negative post-elastic stiffness (γ) of 5% of the initial stiffness and a 5% viscous damping ratio (ζ). The lateral strength of the systems is defined using the normalized yield strength coefficient, C_y, defined in Eq. (1) as

$$C_y = \frac{F_y}{m \cdot g} \tag{1}$$

where F_y is the yield lateral strength of the SDOF and the product $m \cdot g$ corresponds to its weight. This and all subsequent SDOF cases use γ = -5% and ζ = 5%. The left subpanels correspond to a system with T_n = 1.5s and a C_y = 0.1 while the right subpanels correspond to a system with T_n = 0.5s and C_y = 0.4. The two upper figures present segments of their corresponding acceleration time series. The red vertical dashed lines denote the beginning and end of the acceleration pulses with the largest *IV* (denoted as *IV* pulse hereafter for the sake of brevity) which are shown as red pulses with a light shading between the pulse ordinates and the zero line. The middle subfigures present the ground velocity where it is clear that severe long-duration acceleration pulses translate into large changes in ground velocity. The lower figures present the lateral displacement response of the SDOFs. It is seen that for the system with T_n = 1.5s, the *IV* pulse is the one that triggers the collapse of the system. Conversely, for the T_n = 0.5s system, the large displacement demand occurring around 36s is caused by a short-duration pulse occurring before the *IV* pulse. These two examples illustrate that an improved definition of *IV* should take into account the period of vibration of the system because structures with different fundamental periods of vibration respond differently to a given acceleration pulse. In particular, the ratio of the period of vibration to the duration of the pulse has an important influence on the peak displacement demand. For example, short-duration pulses will most likely be benign in a first-mode dominated structure with a relatively long fundamental period of vibration, whereas a long duration pulse can cause an important inelastic displacement demand.

A second shortcoming in the original definition of *IV* occurs in situations where it fails to reflect the damaging potential of a ground motion due to a capping in the accumulation of *IV* caused by a zero-crossing. One of these situations occurs when two large acceleration pulses are separated by a small pulse with opposite polarity. A second situation occurs in the presence of high-frequency content in the acceleration time series in combination with an otherwise long-duration acceleration pulse with considerable damaging potential.

The left subpanel of Figure 2 shows an example of the first situation by presenting the response of an SDOF with T_n = 2s and C_y = 0.06 subjected to the NS component of the Chi-Chi 1999 earthquake recorded at the CHY029 station. In this case, the *IV* pulse does not produce a large displacement demand. Rather, it is two almost consecutive pulses with positive acceleration ordinates, shown in blue, those that produce a very large displacement increment of almost 29.6 cm. However, in this case, the accumulation of *IV* is interrupted by the

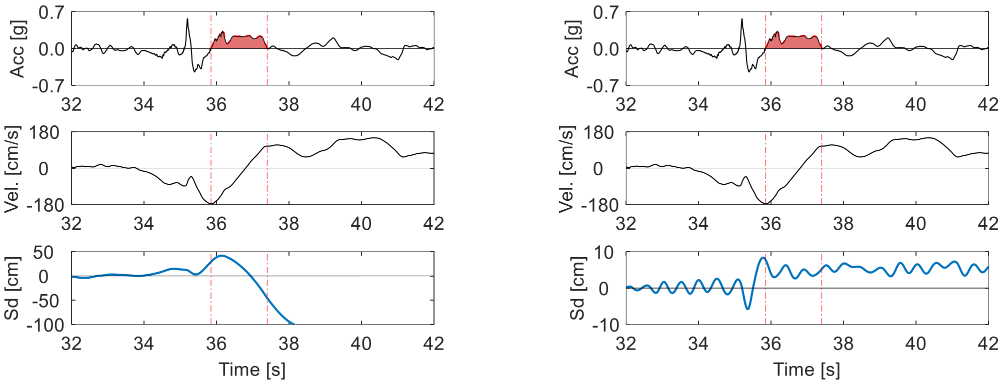

Fig 1. Effect of the IV pulse on two SDOFs subjected to the EW component of the TCU068 station recording during the 1999 Chi-Chi earthquake. Left: T_n = 1.5s. Right: T_n = 0.5s. Ground acceleration (in the top), ground velocity (in the middle), and lateral displacement demand (in the bottom).

tiny pulse with negative acceleration that occurs between the two larger pulses with positive acceleration. If one ignores the zero-crossings from this pulse, an IV of 70.1 cm/s is computed. This modified IV is 33.8% larger than the IV of the shaded pulse. It is this larger IV associated with a duration of t_d =1.82s, which is close to the period of vibration of the SDOF, what causes the large displacement demand. This suggests that an improved definition of IV should be period-dependent. For example, by accumulating area under the acceleration time series for a time equal to 90% of the period of vibration of the SDOF (which corresponds to the area between the blue "pulse" and the zero line), one is able to adequately capture the most damaging portion of the ground motion for this SDOF system.

An example of the presence of high-frequency acceleration on otherwise long-duration acceleration pulses, is shown on the right panel where a SDOF with T_n = 1.0s and C_y = 0.15 is subjected to the EW component of the 1989 Loma Prieta earthquake recorded at the Gilroy Array #2 station. In this case it is seen that the most important displacement demand occurs early in the record between times t = 3.68s and t = 4.58s. In this case, the *effective acceleration pulse* consists of three pulses with negative acceleration and two smaller pulses with positive acceleration having very short durations. These pulses with positive acceleration and very short durations are caused by the presence of high frequency spikes in this portion of the record, which lead to interruption of the accumulation in ground velocity. If one ignores the zero-crossings and computes the area between the acceleration ordinates and the zero line in a range of time equal to 85% of the period of vibration of the SDOF, which is represented by the blue "pulse", an updated effective IV of 75cm/s is obtained. This

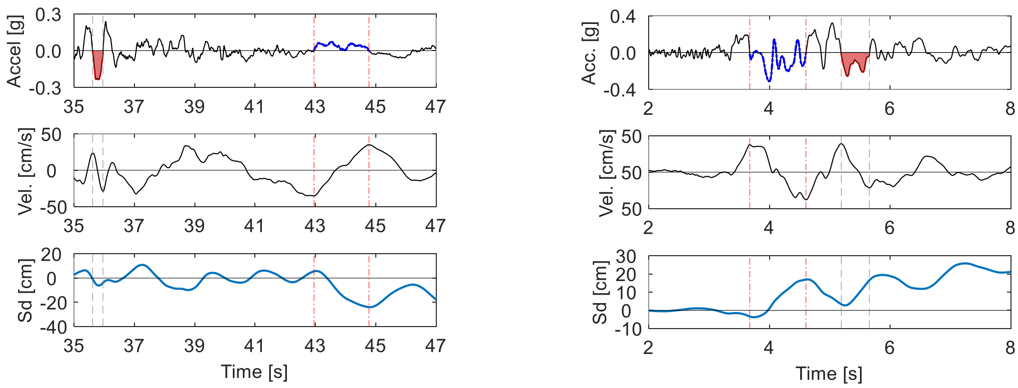

Fig 2. Illustrations of zero-crossings in acceleration records that interrupt the accumulation of ground acceleration in the original definition of IV. Left: Tn = 2.0s SDOF subjected to the NS component of the CHY029 recording from the 1999 Chi-Chi earthquake. Right: Tn = 1.0s SDOF subjected to the EW component of the Gilroy Array #2 recording from the 1989 Loma Prieta earthquake. Ground acceleration (on top), ground velocity (in the middle), and lateral displacement demands (on the bottom).

effective *IV* is 24% larger than the largest *IV* computed considering the zero-crossings and properly reflects an important change in displacement demand.

Both examples presented in Figure 2 suggest that effective acceleration pulses with long durations relative to the period of vibration of the structure are those that, in general, produce large displacement excursions. Thus, it is argued that a period-dependent accumulation of acceleration and the removal of the high-frequency content in the record could lead to an improved identification of damaging acceleration pulses in a ground motion. These two ideas form the basis of *FIV3* developed in [17].

A thorough evaluation of the dispersion of collapse capacities from the six structures subjected the 269 ground motions were used to find the optimum parameters, that is, the cutoff frequency to remove high-frequency content and the time accumulation factor, α, (as a percentage of T_n) that helped to re-calibrate the original definition of *FIV3* proposed in [17]. Therefore, for this study the *FIV3* definition is presented in Eq. (2) and Eq. (3)

$$FIV3 = max\{V_{s,max1} + V_{s,max2} + V_{s,max3}, |V_{s,min1} + V_{s,min2} + V_{s,min3}|\} \tag{2}$$

$$V_s(t) = \left\{\int_t^{t+0.7\cdot T_n} \ddot{u}_{gf}(\tau)d\tau, \ \ \forall\, t < t_{end} - 0.7\cdot T_n\right\} \tag{3}$$

where $V_{S,max1}$, $V_{S,max2}$, and $V_{S,max3}$, are the first, second, and third local maximum incremental velocities computed by accumulating ground accelerations in a time segment with duration $0.7\cdot T_n$ starting at time t, respectively, and similarly $V_{S,min1}$, $V_{S,min2}$, and $V_{S,min3}$, are the first, second, and third local minimum incremental velocities computed accumulating ground accelerations over durations of $0.7\cdot T_n$, respectively. T_n corresponds to the fundamental period of vibration of the structure, and $\ddot{u}_{gf}$ to the ground acceleration time series filtered using a 2nd order Butterworth low-pass filter with a cut-off frequency of 1.0 Hz.

4. Efficiency evaluation

Efficiency is defined by Shome et al. [6] as the ability of an IM to estimate structural responses with a small variability. In our case, these structural responses correspond to collapse intensities. The efficiency of *FIV3* was compared with that achieved by two commonly-used ground motion intensity measures for seismic collapse risk estimation (S_a and S_a+ε), and with that achieved with two recently proposed improved *IMs* (Sa_{avg} and IM_{comb}). The computation of the ε values was conducted using the 2008 Boore and Atkinson GMPM [26]. These values were then used to adjust the S_a-based collapse capacities following the procedure recommended in [4, 9].

A comparison of the efficiency of the five IMs, measured using the logarithmic standard deviation of collapse capacities (σ_{lnIM}), and for the six structures considered in this study is shown in Figure 3 for the six structures. The upper right corner of each subfigure indicates the fundamental period of vibration along with the number of stories of the structure. From these figures, it is clear that S_a is the IM that leads to the largest dispersion on collapse capacities in all six structures. Adjusting collapse intensities by considering the ε of each record as recommended by Haselton et al. [9] causes a reduction of approximately 20% in the dispersion of collapse intensities when compared to σ_{lnSa} for the two shorter period structures. However, much smaller reductions (on the order of 7%) are obtained in the remaining four structures. Results from Figure 3 correspond to reductions in the dispersion of collapse intensities measured using Sa_{avg} and IM_{comb} of 50% and 52%, respectively, with respect to σ_{lnSa}. The largest average reduction in dispersion corresponds to the use of *FIV3* and equals to 61%. This means that the same level of accuracy in the estimation of the probability of collapse using S_a as the IM can be achieved with approximately one sixth of the number of NRHA if *FIV3* is the IM. Similarly, if one uses *FIV3* in lieu of Sa_{avg} or IM_{comb}, the computational effort is reduced by approximately 35%.

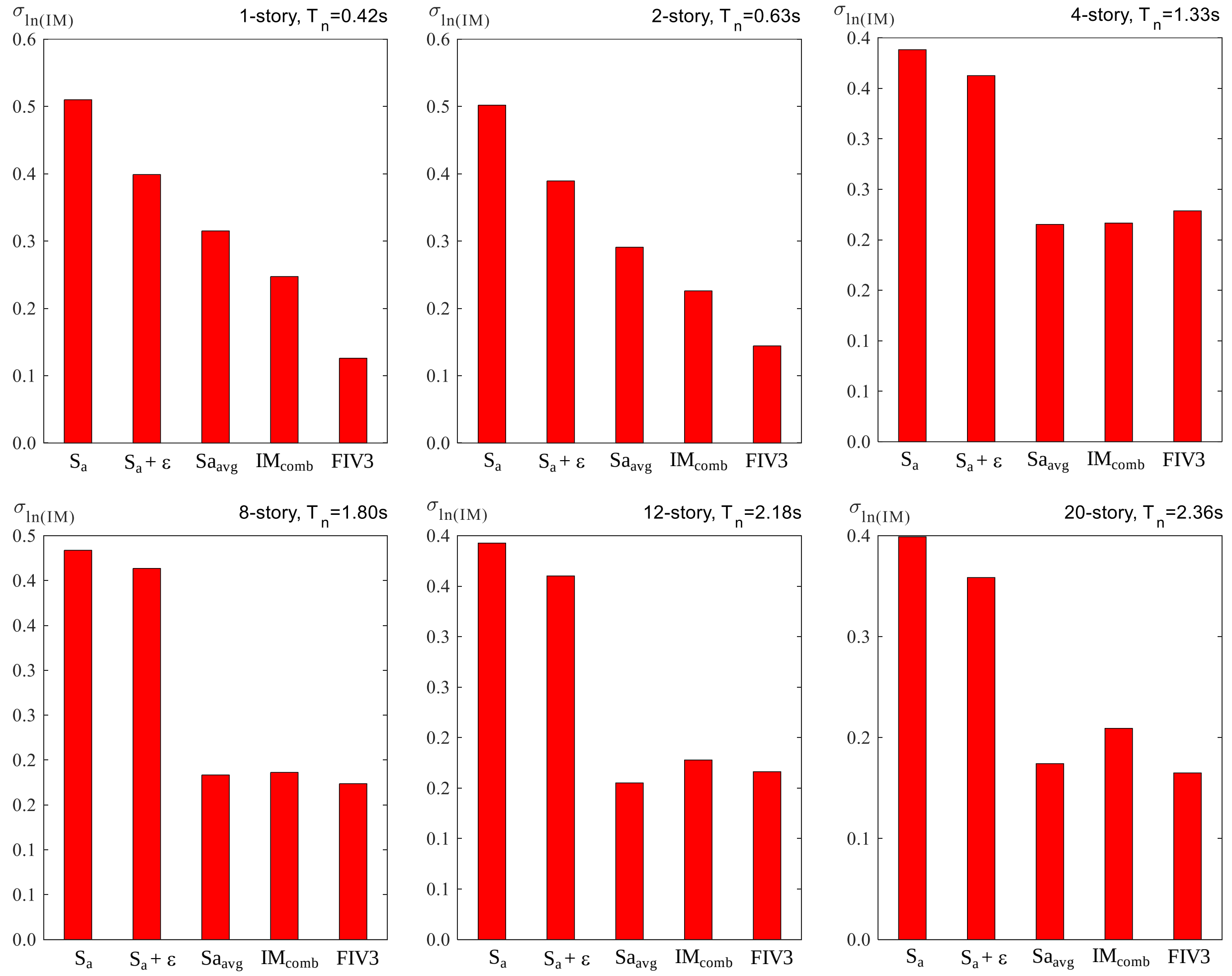

Fig. 3. Efficiency comparison among five different IMs on the six moment-resisting-frame buildings.

4. Sufficiency evaluation of *FIV3*

A simple method to compare the relative sufficiency among IMs and ground motion parameters termed simplified relative sufficiency was proposed by the authors in [17]. That same approach is used here and it consists on conducting a standard linear regression on the normalized collapse intensities (with respect to its median) of a structural model against the desired ground motion parameter. Then, the absolute value of the slope is used to assess the IM's sufficiency. The smaller the absolute value of the slope (S), the higher the IM's sufficiency. Note that its value provides a direct measure of the level of bias that can be introduced in the structural collapse capacity with a unitary change in the ground motion characteristic.

4.1 Sufficiency with respect to magnitude

Figure 4 presents the evaluation of the simplified relative sufficiency (SRS) of the five IMs with respect to moment magnitude M_w. In all cases, the largest absolute slope correspond to either S_a or $S_a+\varepsilon$ whereas the remaining three IMs have the smaller slopes. This means that by using either Sa_{avg}, IM_{comb}, or $FIV3$ collapse capacities are practically unaffected by changes in the M_w range considered here.

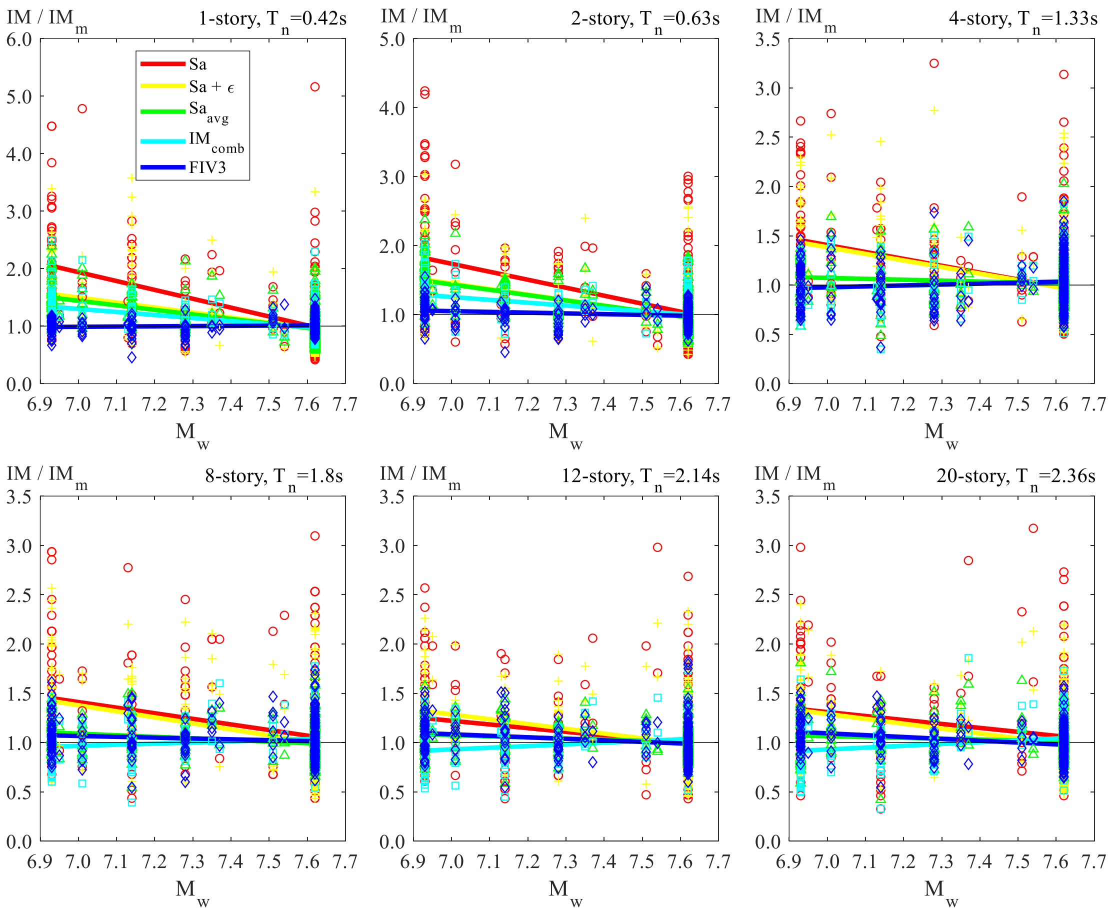

Fig. 4 – Simplified relative sufficiency comparison among five different IMs with respect to magnitude.

While the performance of Sa_{avg}, IM_{comb}, and *FIV3* is relatively similar for the longer period structures, for the two short-period cases *FIV3* is much more sufficient. The insufficiency of Sa_{avg} for short-period structures is in agreement with previous observations by Eads et al. [13].

The largest differences in the slope values occurs in the short-period structures and the slope tends to decrease with increasing period of vibration. For example, for the structure with the shortest period of vibration, the difference in slopes indicates that for a M_w value of 6.93, the intensities that trigger collapse measured using S_a are on average 2.03 times larger than the median collapse intensities computed using all the records. On the other hand, collapse intensities are very similar across the whole range of magnitudes considered if *FIV3* is used as the IM. In this and all the *SRS* assessments that follow, the term under- or overestimation is used assuming that the 'target' collapse intensity corresponds to the median value computed using all the ground motions.

Considering the six structures, mean overestimations at the lowest M_w considered are on average 56%, 43%, 22%, 8%, and 6% for S_a, $S_a + \varepsilon$, Sa_{avg}, IM_{comb}, and *FIV3*, respectively. These results indicate that, in terms of sufficiency with respect to M_w, *FIV3* is the most sufficient IM from those analyzed.

4.2 Sufficiency with respect to spectral shape

It is well known that a strong bias in collapse estimates can be introduced if during record selection the expected spectral shape of records controlling the collapse risk of the structure is ignored [7, 9, 20, 21]. This study uses the parameter *SaRatio* [27], defined as the ratio of S_a to Sa_{avg}, to evaluate the simplified relative sufficiency of the IMs with respect to spectral shape. This was decided because *SaRatio* has an outstanding correlation with collapse intensities and therefore is much better suited for this assessment than ε.

Figure 5 presents the evaluation of the simplified relative sufficiency (SRS) of the five IMs with respect to *SaRatio*. In all cases the largest absolute slope, and therefore the least sufficient IM, corresponds to S_a. In other words, S_a-based collapse intensities are strongly dependent on the value of *SaRatio*. Similary, collapse capacities adjusted by considering ε seem to be strongly dependent on *SaRatio*. In these cases, if *SaRatio* is large, collapse intensities tend to be overestimated. If we consider only the two shortest period structures, *FIV3* clearly outperforms all the other five intensity measures as it has the smallest normalized slope. Sa_{avg}, IM_{comb}, and *FIV3* are similarly sufficient with respect to *SaRatio* in the four longest period structures. In this case, Sa_{avg} is a slightly better IM as its normalized slope is closer to zero, which is expected because of the close relation between Sa_{avg} and *SaRatio*.

Considering all the structures, the overestimation of the median collapse capacity at an *SaRatio* = 2.0 are on average 71%, 44%, 13%, 15%, and 2% for S_a, S_a+ε, Sa_{avg}, IM_{comb}, and *FIV3*, respectively.

4.3 Sufficiency with respect to duration

Several recent studies have reported an influence of strong motion duration on structural collapse capacity (e.g., [28-30]). The SRS assessment of the five IMs with respect to the 5-95% significant duration is presented in Figure 6.

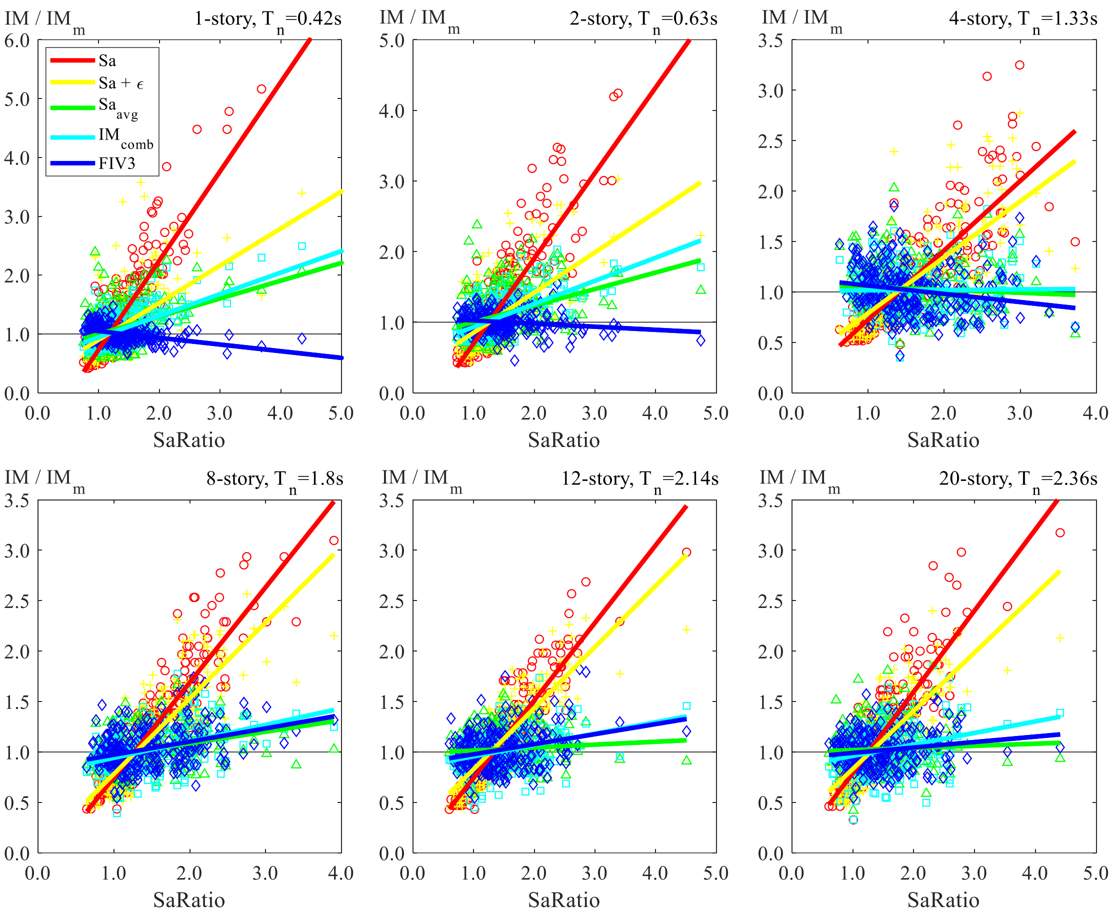

Fig. 5 – Simplified relative sufficiency comparison among five different IMs with respect to spectral shape.

Again, S_a or S_a adjusted by considering ε result, on average, the least sufficient IMs. In all the structures, using S_a or the correction of S_a collapse capacities considering the ε of each record results in computing significantly lower collapse capacities as $SD_{5\text{-}75}$ increases, which is in agreement from previous research [28-30]. *FIV3* seems to be a much better IM than any other option considered in the case of the two structures with shortest periods of vibration. In these cases the median collapse capacity using *FIV3* is practically unaffected by the value of $SD_{5\text{-}75}$. Conversely, the other four IMs tend to estimate lower collapse intensities as $SD_{5\text{-}75}$ increases. For the rest of the structures, as expected due to the inclusion of $SD_{5\text{-}75}$ in the definition, IM_{comb} is the best IM. Nonetheless, Sa_{avg} and *FIV3* perform adequately, both having a slight trend to estimate lower collapse intensities as $SD_{5\text{-}75}$ increases. Considering the average results from the six structures, median overestimations at $SD_{5\text{-}75}$ = 5s are on average 47%, 33%, 20%, 8%, and 6% for S_a, S_a+ε, Sa_{avg}, IM_{comb}, and *FIV3*, respectively. If we consider an $SD_{5\text{-}75}$ = 30, mean underestimations from the six structures correspond to 36%, 29%, 20%, 5%, and 6%, respectively.

It is worth noticing that by only taking into consideration information from the three largest period-dependent and equally-polarized acceleration pulse segments, *FIV3* seems to be relatively sufficient with respect to $SD_{5\text{-}75}$.

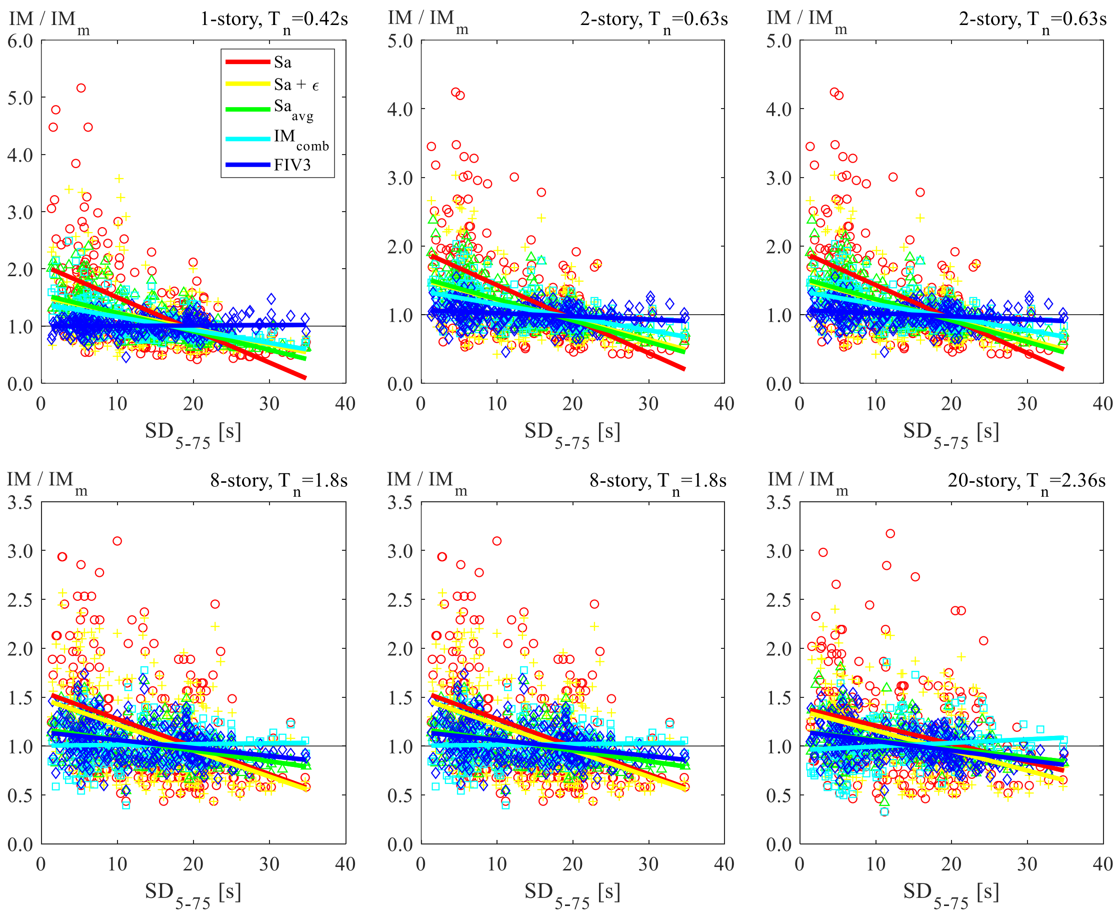

Fig. 6 – Simplified relative sufficiency comparison among five different IMs with respect to significant duration.

6. Conclusions

This study presented an evaluation of *FIV3* as an intensity measure for seismic collapse estimation using six different moment frame structures. The efficiency and sufficiency of the IM was compared with other four intensity measures, namely, S_a, S_a and its adjustment for collapse estimation by considering the spectral shape proxy ε, Sa_{avg}, and $IM_{comb.}$

From the results, the following conclusions can be drawn:

1) *FIV3* outperformed all the IMs considered in this study in terms of efficiency for structures with relatively short periods of vibration (e.g., T_1 between 0.4s and 0.7s). The high efficiency of this IMs means that, for the same level of confidence, the collapse capacity of moment frame structures can be estimated with a much smaller number of ground motions than those required when S_a or $S_a+\varepsilon$ are used as the IMs. In structures with periods of vibration longer than 1s, *FIV3*, Sa_{avg} and IM_{comb} have similar efficiencies, all of them considerably higher than that of S_a or $S_a+\varepsilon$.

2) The simplified relative sufficiency of *FIV3* with respect to M_w, spectral shape, and duration was also higher than the other considered IMs in the two shortest period structures. This implies that changes in these ground motion characteristics have a minor impact on the estimated collapse capacities, thus, the record selection process can be greatly simplified by not having to attempt to estimate the expected ground motion characteristics from future earthquake events. The sufficiency of *FIV3* in the four structures with the longest periods of vibration was again comparable with that of Sa_{avg} and IM_{comb} and clearly higher than those computed using S_a or $S_a+\varepsilon$.

3) Overall, *FIV3*, Sa_{avg}, and IM_{comb} are much better options for collapse estimation of moment frame structures than either S_a or the correction to S_a collapse intensities using the spectral shape proxy, ε.

4) Based on these results, an updated definition of *FIV3* is recommended as an IM for seismic collapse risk assessment of moment frame buildings with fundamental periods of vibration between 0.4s and 2.4s.

7. Acknowledgements

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