CORRELATION STUDY ON STRUCTURAL RESPONSE AND GROUND MOTION INTENSITY PARAMETERS

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Abstract

This paper presents the results of an extensive correlation study between ground motion intensity parameters (non-structure specific and structure specific) and structural response. Promising eleven ground motion intensity parameters from the literature with varying computational effort were picked and their respective correlation performance was assessed in linear and nonlinear range. A total of 372 nonlinear time history analyses were performed for six reinforced concrete frames selected from the project-database of Obermeyer Planen + Beraten GmbH. The selected frames fall into the low-to-mid-rise category and are designed/engineered in different countries. A set of ground motions was compiled in order to represent a wide range of magnitudes and peak ground values. Using the results of the correlation performance, two acceleration and velocity related intensity measures were combined. The purpose of this combination was to attain encouraging correlation results ($R^2 > 0.80$) with less computational effort. Furthermore, a new seismic intensity measure was introduced which improves the correlation performance of the spectral acceleration at the fundamental period. Both combined and the new intensity measure provided stable results in all period ranges of this study.

Keywords: ground motion intensity parameters, reinforced concrete, nonlinear analysis, correlation study

1. Introduction

An important area of research in performance-based earthquake engineering is the assessment of the expected seismic response of structures under a specific earthquake ground motion. In order to define the damage potential of a seismic event, various earthquake intensity measures (IM) were introduced by researchers. An optimal choice of an earthquake intensity parameter is not only significant for the assessment of the seismic performance of structures, but also for selection and scaling procedures of ground motion records prior to the structural analysis.

Many current conventional IMs can easily be determined from the trace or response spectra of ground motions with the drawback of not considering structural information. In the past years, several studies have been carried out in order to verify the performance of intensity measures which are only based on ground motion quantities (Yakut and Yilmaz [1]; Akkar and Ozen [2]; Cabanas et al. [3]; Liao et al. [4]). The outcome of these studies suggest that the correlation performance of non-structure-specific ground motion parameters highly depend on the structural system in terms of the fundamental period. As a consequence, several advanced IMs have been proposed by researchers, which are taking into account both structural information and ground motion characteristics to provide sufficient correlation with the seismic response of various structures.

As first part of this study, the correlation strength of the above mentioned IMs were checked. Although some of these IMs correlated well with the structural response, they required more computational effort and calculation time compared to non-structure specific IMs. Using the findings of the first part results, two new IMs are proposed in the second part. The goal of the second part is to propose a simple and efficient IM that shows stable results in wide period ranges.

Among the available structure types, reinforced concrete (RC) frame structures are preferred for this study due to its common design and existence in seismically active regions, especially in Europe. The numerical computation of the structures is conducted using direct nonlinear time history. For the investigation of correlation, the Pearson's correlation coefficient, ρ, and the coefficient of determination, R^2 , are used in this study as appropriate statistical measures in order to assess the linear relationship between the parameters.

2. Ground Motion Intensity Parameters

In the following section, the ground motion intensity parameters utilized in this study are briefly described:

Peak Ground Acceleration (PGA) and peak ground velocity (PGV) are the simplest and most widely used intensity parameters in the field of earthquake engineering. Both intensity parameters, PGA and PGV, are often used to develop fragility curves for loss estimation studies. Cordova et al. [5] developed a two-parameter seismic intensity measure, $IM_{cor,}$ that reflects both spectral intensity and spectral shape. Their proposed seismic parameter considers the effect of period elongation resulting from inelastic strength and stiffness degradation as the structure enters nonlinear region using a mathematical relationship between the spectral acceleration at higher modes and at the fundamental period. Yahyaabadi and Tehranizadeh [6] defined two advanced, structure-specific intensity measures, IM_{YNC} and $IM_{YC,}$ based on optimal combinations of displacement response spectra values for non-collapse seismic demand and collapse capacity prediction of structures. According to the authors, both proposed IMs may estimate the nonlinear response of structures with higher accuracy by taking into account period softening and higher mode effects. Kadas et al. [7] developed a new spectral intensity measure, $IM_{Kad,}$ that relies on the capacity and period elongation of structures with the primary purpose to compile fragility curves for loss estimation studies. This IM represents an advanced modification of the spectral intensity parameter ASI proposed by Von Thun et al. [8]. The modification comprises the consideration of structural-specific information in terms of the yield acceleration strength (A_y), the initial period (T_i) and the softened period (T_f) of structures. In this manner, deformation demands of structures at nonlinear region are believed to be displayed more realistically. The initial period, T_i, and the yield capacity strength, A_y, may be obtained via period and nonlinear static (pushover) analysis. Lin et al [9] proposed an advanced intensity parameter, $IM_{Lin.}$ based on the spectral acceleration at the fundamental period of structures taking into account period softening effects by increasing the fundamental period and using the square root for the spectral combination.

Other common used intensity measures that are computed from the response spectra of the ground motion record are Housner intensity (HI), acceleration spectrum intensity (ASI) and the velocity spectrum intensity (VSI) (Von Thun et al.[8]). In this study, we integrate the pseudo-acceleration spectrum for ASI between the periods 0.1 – 2.5 s. Among the above-described parameters, PGA, PGV, ASI, VSI, and HI are non-structure specific IMs and IM_{Cor}, IM_{Lin}, IM_{YNC}, IM_{YC} and IM_{Kad} are structure-specific by considering structural information.

2.1 Computational Effort of the selected Intensity Measures

Since the selection of an appropriate ground motion intensity measure for structural response prediction does also depend on the related computational effort, an evaluation of the selected IMs in this regard is of primary interest. The outcome of the assessment is presented in Table 1 where the seismic parameters are ranked according to their computational effort. The simplest IMs requiring least computational effort are PGA and PGV. The remaining intensity measures are considered with higher computational effort since these parameters are based on response spectra of ground motions.

Although ASI, VSI and HI are period independent parameters, they have been ranked with moderate computational effort due to requiring computational expensive integration of response spectra which can only be conducted via numerical approach. IM_{Cor}, IM_{Lin}, IM_{YNC} and IM_{YC} are representing optimal multiplications or combinations of response spectra quantities at the first mode period of structures. Given the fact that these structure-specific IMs are expressed by comparatively simple equations, their computational effort has been ranked as moderate by the authors of this paper although the calculation of these measures is associated with the computation of response spectra and the fundamental period of structures. The intensity measure proposed by Kadas et al [7] requires numerical integration of the acceleration response spectrum, eigenvalue analysis, pushover analysis and idealization of the pushover curve and is therefore considered as the most complex seismic parameter among the selected IMs in this study. Hence, IM_{Kad} has been ranked accordingly with the highest computational effort.

Table 1 – Computational effort of selected IMs.

ID	Computational Effort
PGA	*
PGV	*
$Sa(T_1)$	**
ASI	***
VSI	***
HI	***
IM_{Cor}	***
IM_{Lin}	***
IM_{YNC}	***
IM_{YC}	***
IM_{Kad}	****

Additional to the ranking of computational effort, the number of the investigated frames and earthquake records in these correlation studies were also examined and summarized in Table 2. As it can be seen in Table 2, the quantities of the frames and earthquakes vary considerably.

Table 2 – Computational effort of selected IMs.

Literature	# of RC Frames	# of Ground Motions	EDP	2D/3D
Yakut and Yilmaz [1]	16	80	MIDR	2D
Fontara et al. [10]	2	33	MIDR / AIDR / OSDI	2D
Konstinakis et al. [11]	4	20	MIDR / OSDI	3D
Fontara et al. [12]	2	30	MIDR	2D
Elenas and Meskouris [13]	1	29	MIDR / OSDI	2D
Konstinakis and Athanatopoulou [14]	3	59	MIDR / OSDI	3D
Pejovic and Jankovic [15]	1	40	MIDR	3D
De Biasio et al. [16]	1	4000	MHFA	3D
Elenas and Nanos [17]	1	225	OSDI	2D

3. Description of Ground Motions and Frames

3.1 Ground Motions

In total, 62 individual unscaled earthquake records with moment magnitudes (M_w) ranging from 5.7 to 7.3, mostly recorded on alluvium sites, are selected in this study. The ground motion data set was extracted from the PEER [18] strong motion database to cover various PGA and PGV bins. In Fig. 1 the relationship between PGA and PGV of the ground motion set is presented. The distribution of PGA falls mostly in the range of 0.2 − 0.6 g while the distribution of PGV values covers a broad range between 10 and 115 cm/s, in particular within the range 10 − 50 cm/s. Moreover, among the 62 ground motions an outliner is included considering a PGA at value 1.78 g. According to this diagram, the employed frame structures are expected to experience various degrees of elastic as well as inelastic responses under the selected ground motion records.

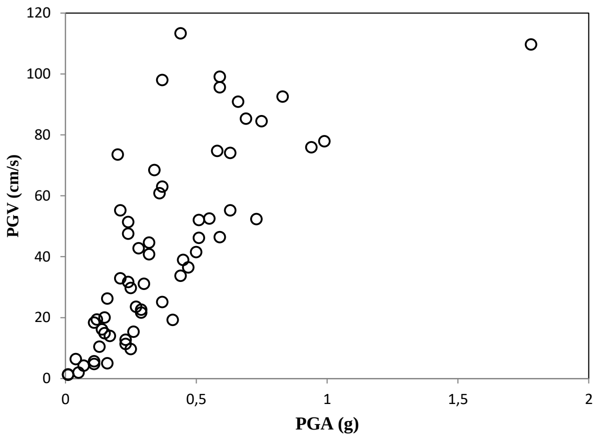

Fig. 1 – PGA versus PGV of the selected frames under the ground motion set.

3.2 Description of the Frames

Six RC frames were selected having fundamental periods between 0.29 and 1.0 s in order to represent general short-to-long period frame structures located at seismically active regions. The employed frames have 2 to 8 stories, three among them having 4 stories. Furthermore, the frames show no significant structural irregularity in terms of story height and bay width. The structures were modeled according to existing RC frames located in Haiti, Slovenia and Turkey which were extracted from the project-database provided by Obermeyer Planen + Beraten GmbH. The locations of the frame structures are depicted in Fig. 2 on a global map. The purpose of this selection was to cover different approaches to seismic design by practicing engineers.

FRM-1 is extracted from a non-engineered building in Turkey where only vertical loads were considered for the design of this structure. In order to observe the difference between the engineered and non-engineered structures, this frame was re-designed according to the European seismic code (EC8 [19]) where an effective peak ground acceleration of 0.4 g was considered for the design spectrum (Frame FRM-2). The frames FRM-3 and FRM-4 located in Haiti were designed according to ASCE 41-13 [20]. It is worth mentioning that both frames were designed after the 2010 Haiti earthquake. Compared to the existing structures built before this earthquake, seismic awareness aroused by the design and reinforcement detailing could be observed by the authors. The employed frames differ in story numbers and partly in reinforcement ratios considered for the column elements. Lastly, frames FRM-5 and FRM-6 were modeled according to two mid-rise buildings located in Slovenia. The structures were designed according to the European seismic standard (EC8 [19]). With a total number of eight and six stories, FRM-5 and FRM-6 are the tallest structures among the selected frames. A brief summary of the structural properties of the frames can be seen in Table 3.

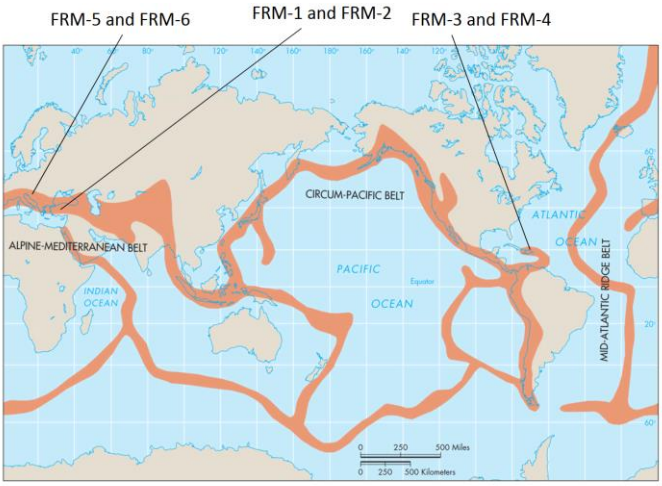

Fig. 2 – Location of frames from the project-database of Obermeyer Planen + Beraten GmbH / worldmap illustrating the major earthquake belts as shaded areas [21].

Table 3 – Structural properties of the frames FRM-1-6.

RC Frame	# of story	f_{ck} (MPa)	f_{yk} (MPa)	Height (m)	Fundamental Period T_1 (s)	Total Weight (kN)
FRM-1	4	20	500	12	0.58	1228.8
FRM-2	4	20	500	12	0.58	1228.8
FRM-3	4	25	413	13.2	0.55	1864.8
FRM-4	2	25	413	6.8	0.29	932.4
FRM-5	8	25	500	24.8	1.00	4531.2
FRM-6	6	25	500	18.6	0.82	3398.4

4. Nonlinear Analysis of the Frames

4.1 Nonlinear Static (Pushover) Analysis

Nonlinear static pushover analyses of 2D MDOF numerical models were performed using the well-known integrated software for structural analysis and design, SAP2000. The employed frames were modeled as planar two-dimensional structures consisting of column and beam elements. Column and beam elements were modeled considering flexural and shear deformations, respectively. Adaptive force based pushover analyses of the frames were carried out under the action of gravity loads and a representative lateral load pattern, proportional to the fundamental period of the structures.

The obtained pushover curves of the MDOF models using SAP2000 are depicted in Fig. 3 in form of base shear coefficient, V/W, versus average roof drift. The base shear coefficient, V/W, describes the ratio of the base shear to the total weight of frames and the average roof drift is the roof displacement divided by the total height. This form represents the dimensionless pushover curve which is a more practical description of the structural capacity in order to estimate the seismic response of structures.

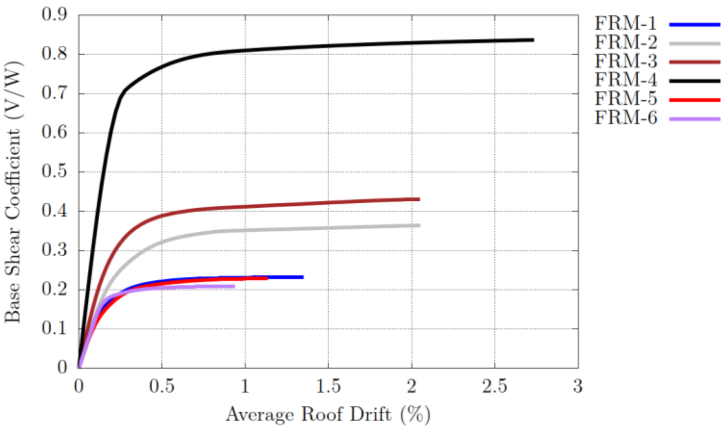

Fig. 3 – Comparison of the obtained pushover results.

As described in previous section, different design approaches were used in FRM-1 and FRM-2. Their effect on cost (concrete and reinforcement steel) and performance of the frames was here investigated. As it can be seen from the Table 4, seismic re-design of the frame increases the overall material cost around 23 %, whereas the yield point increases around 67%. Contrary to common belief in practice, seismic design effects the total cost (when all the costs groups included) negligibly.

Table 4 – Cost comparison between FRM-1 and FRM-2 considering the respective design code.

Frame	Material Cost	A_y (g)	Comment
FRM-1	2.937,03 €	0,18	Designed for vertical loads
FRM-2	3.616,80 €	0,3	Designed according to EC8, PGA=0,4 g
Increase	23%	67%	

4.2 Nonlinear Time History Analysis

The transient analyses of the frames were carried out using the finite element software SOFiSTiK. The pre-processing of the 2D frames was realized using a parametric approach where the geometry, material laws, constrains, loads and meshing were parametrized for frame generation. This approach was conducted in order to simplify time-consuming modeling work.

The selected frames were modeled consisting of beam and column elements considering elasto-plastic material law with kinematic hardening using rotational spring elements at node junctions and restrains. The corresponding rotational spring work laws of column and beam elements were determined according to FEMA-356 [22] in terms of moment-rotation relationship taking into account bilinear hysteretic behavior with no stiffness and strength degradation.

Nonlinear time history analyses of numerical models were performed using direct integration method that corresponds to a Newmark method with good numerical damping of higher frequencies for nonlinear analysis considering γ = 0.55 and β = 0.4 as input parameters. It is important to note that the effect of infill walls was not taken into account, but simply pure frame behavior ignoring P-Δ effects.

In this study, the average interstorey drift ratio (AIDR) is selected as damage parameter to evaluate the seismic response of the RC frame structures. The AIDR is the peak lateral roof displacement divided by the building height. Fig. 4 presents the distribution of the AIDR's obtained from 372 nonlinear time history analyses of the employed frames under the selected ground motion set. The computed AIDR's were compared with the selected intensity measures calculated for each ground motion.

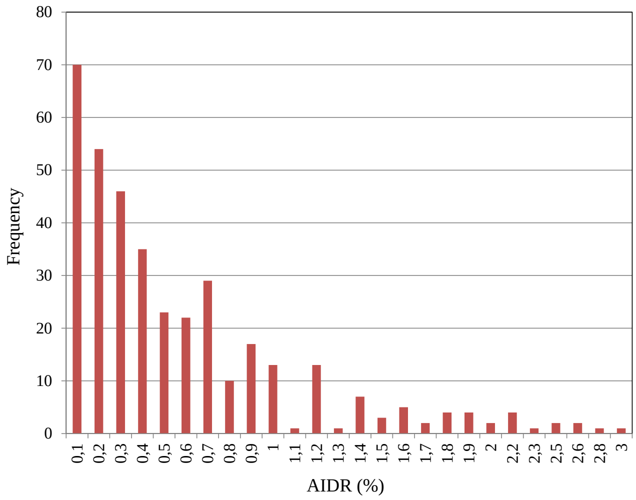

Fig. 4 – AIDR distribution of nonlinear time history analysis results.

5. Correlation Study

5.1 Correlation Measures

In order to evaluate the efficiency of each IM, the correlation between the intensity measures and the damage parameter of the frames is computed using Pearson's correlation coefficient, ρ, and the coefficient of determination, R^2. The Pearson correlation coefficient is a dimensionless measure from statistics which shows how well a dataset fits a linear relationship in ranges between -1 and 1. A correlation greater than 0.8 is generally considered as strong, whereas a correlation less than 0.5 is described as weak. The corresponding equation is expressed as follows:

$$\rho = \frac{\sum_{i=1}^{n}(X_i - \bar{X})(Y_i - \bar{Y})}{\sqrt{\sum_{i=1}^{n}(X_i - \bar{X})^2 \sum_{i=1}^{n}(Y_i - \bar{Y})^2}}, \tag{1}$$

where $\bar{X}$ and $\bar{Y}$ are the mean values of X_i and Y_i data, respectively, and n is the number of pairs of values X_i and Y_i in the dataset. In regression analysis, the square of Pearsons's correlation coefficient is denoted as R^2 and represents the coefficient of determination as shown in Eq. (2). The measure is expressed by a dimensionless value that describes the ratio of the regression sum of squares ($\mathbf{S}_{reg}$) to the variance of the data ($\mathbf{S}_{tot}$). R^2 can take values between 0 and 1, with 1 expressing a total correlation behavior and 0 no correlation

$$R^2 = \frac{\sum (\hat{Y}_i - \bar{Y})^2}{\sum (Y_i - \bar{Y})^2} = \frac{\mathbf{S}_{reg}}{\mathbf{S}_{tot}} \quad (2)$$

5.2 Correlation Study for MDOF Response

In order to provide an assessable overview of the obtained correlation results, coefficient of determination values and Pearson's correlation coefficients were averaged for each ground motion parameter in the three ranges, respectively. The average correlation results are presented in Table 5. As evidenced in Table 5, the selected ground motion parameters exhibit different degrees of correlation with MDOF responses depending on the examined range of interest. Overall, IM_{Cor}, IM_{YC} and IM_{Lin} turned out to be the intensity measures with the strongest AIDR correlation based on 372 nonlinear time history analyses of six RC frame structures. The strongest correlation in the linear response range is obtained with $S_a(T_1)$ and IM_{YNC} (R^2 values greater than 0.95). Considering the computational effort, $S_a(T_1)$ can highly be recommended for estimating the seismic response of MDOF systems within the linear range. It is apparent from the results that the advanced, structure-specific intensity measures yielded a higher degree of correlation in the nonlinear range, in particular IM_{Cor}, IM_{YC} and IM_{Kad}.

Table 5 – Average correlation between IMs and MDOF response in three ranges – ranked according to correlation performance (top down).

ID	Overall Resp. R^2	Linear Resp. R^2	Nonlinear Resp. R^2	Pearson ρ
IM_{Cor}	0.838	0.902	0.776	0.915
IM_{YC}	0.827	0.884	0.756	0.909
IM_{Lin}	0.806	0.765	0.722	0.898
VSI	0.799	0.719	0.73	0.893
ASI	0.791	0.755	0.72	0.889
IM_{Kad}	0.779	0.704	0.773	0.882
HI	0.765	0.867	0.684	0.872
IM_{YNC}	0.754	0.957	0.651	0.868
$S_a(T_1)$	0.7	0.96	0.573	0.836
PGV	0.667	0.577	0.535	0.814
PGA	0.556	0.533	0.434	0.741

5.3 Performance of the Spectral Acceleration Parameter

Based on the results presented in Table 5, the spectral acceleration at the fundamental (first mode) period, $S_a(T_1)$, has proved high correlation with the deformation demand within the linear range. However, for the nonlinear range, poor correlation was obtained using the spectral acceleration parameter due to not considering inelastic structural effects, such as period softening.

The correlation performance of $S_a(T_1)$ was analyzed for each frame, assuming that the first fundamental period (T_1) can take any value between 0-2 seconds. It was aimed to cover many unknowns of modelling and nonlinear effects. The obtained results are depicted in Fig. 5.

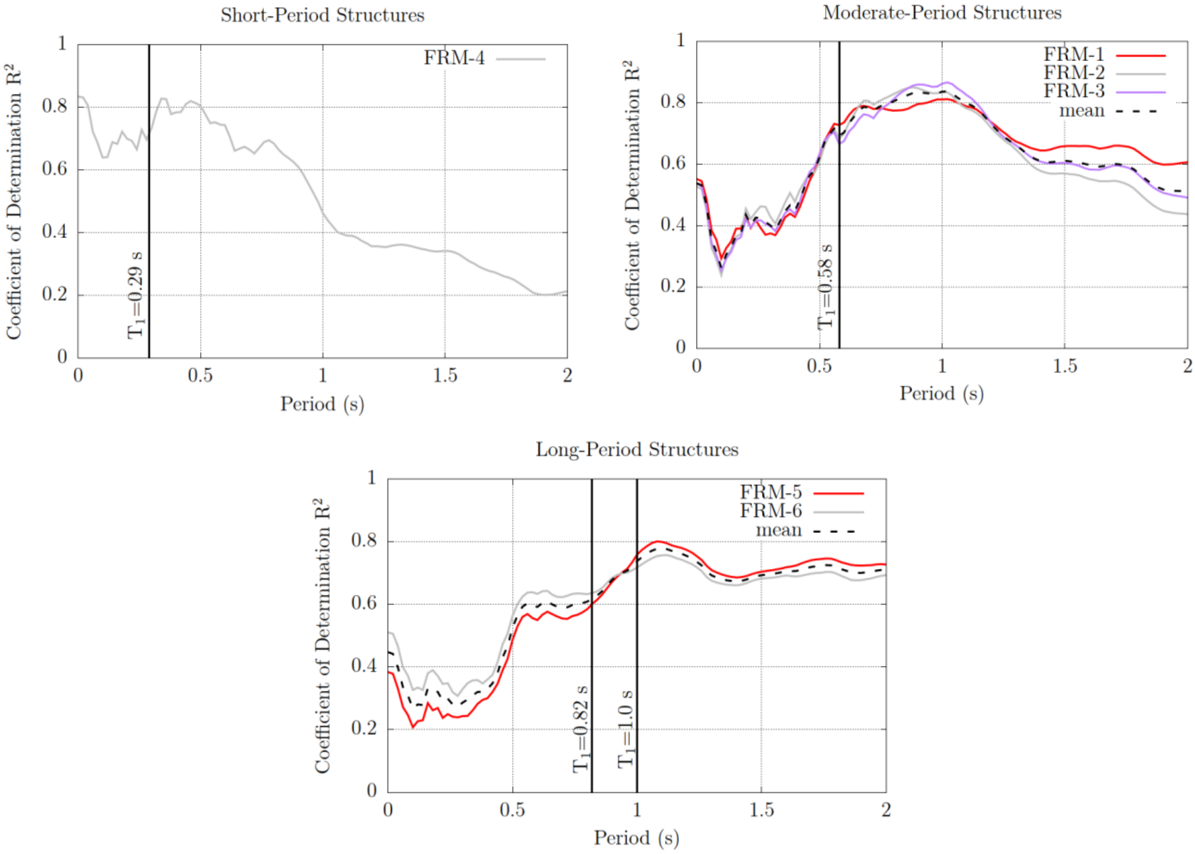

Fig. 5 – Coefficients of determination history of $S_a(T)$ for short-to-long period frames.

As shown in Fig. 5, the correlation increases with the fundamental period which supports the period elongation phenomena during a seismic event. The presented figure also provides a good insight to the maximum achievable correlation. The maximum achievable correlation was around R^2=0,80 for all the frames.

5.4 Proposed Intensity Measure

The degree of correlation between acceleration related ground motion parameters and the damage parameter, AIDR, decreases as the fundamental period of the frames increases. In reverse order, the same correlation trend is observed for velocity related parameters. Hence, both intensity measures lack in consistency with regards to the accuracy of structural response prediction. However, these shortcomings might be remedied using an optimal combination approach taking into account the correlation trends of PGA and VSI, respectively. Additionally, the structure related parameter T_1 is implemented (see Eq. (3)).

$$\mathrm{IM_{comb}} = \mathrm{PGA}\left(\frac{c}{T_1}\right)^2 + a \cdot \mathrm{VSI}\left(\frac{T_1}{c}\right)^2 \tag{3}$$

An optimal amplification factor, a, corresponding to the highest attainable correlation values were determined to be a = 0.5 for $\mathrm{IM_{comb}}$. Based on statistical analyses, c = 0.25 s was found to be the coefficient value providing optimal correlation results for $\mathrm{IM_{comb}}$ within the considered period range.

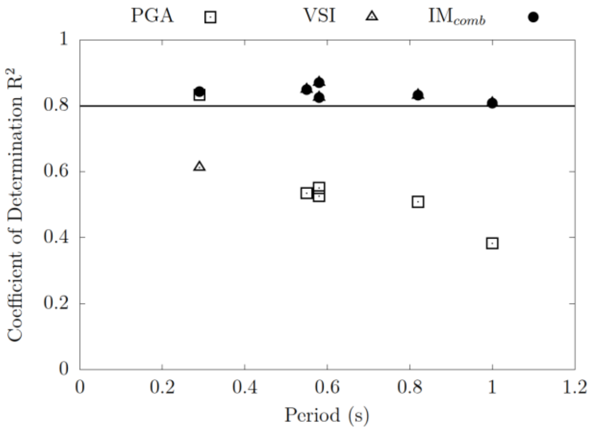

Fig. 6 – Coefficients of determination with respect to the period of frames with threshold line at $R^2 = 0.80$.

The coefficient of determination values, R^2, computed for PGA, VSI and IM_{comb} with respect to the period, T, are displayed in Figure 6. A remarkably strong correlation with the deformation demand of the frames is achieved for IM_{comb} where all coefficient of determination values are above the threshold line (R^2=0.80). Hence, IM_{comb} might potentially be a comparatively simple structure-specific ground motion parameter that is able to successfully predict the seismic response of short-to-long period RC frame structures with a high degree of accuracy.

5.5 Improved Spectral Acceleration Related Measure

As evidenced in Section 5.2, $S_a(T_1)$ yields strong correlation with the deformation parameter in the linear range. However, the post-linear response of structures under severe ground motions cannot be captured sufficiently by solely considering the spectral acceleration at the first mode period. This can be attributed to the fact that $S_a(T_1)$ does not take into account period softening effects and the shape of the acceleration response spectrum.

As mentioned above, the softened period, T_{soft}, and the shape of the acceleration response spectrum are considered to be important parameters for estimating the inelastic response of structures. The cracked state of reinforced concrete structures can simply be captured by reducing the elastic flexural and shear stiffness of concrete to 50% ($0.5EI_c$) which increases the fundamental first mode period, T_1, to 40%.

The shape of the acceleration response spectrum along the elongated period path must be taken into account for ascending, balanced and descending spectrum characteristics, respectively, according to Fig. 7. Based on the above-described conditions, a new seismic intensity measure was developed that relies on the spectral acceleration at the first mode period of structures and the shape of the acceleration response spectrum. The new parameter is structured as follows:

$$\mu(S_a(T)) = S_a(T_1) + \frac{\Delta T}{0.4T_1} \cdot \sum_{T=T_1}^{T=1.4T_1} (S_a(T) - S_a(T_1)). \tag{5}$$

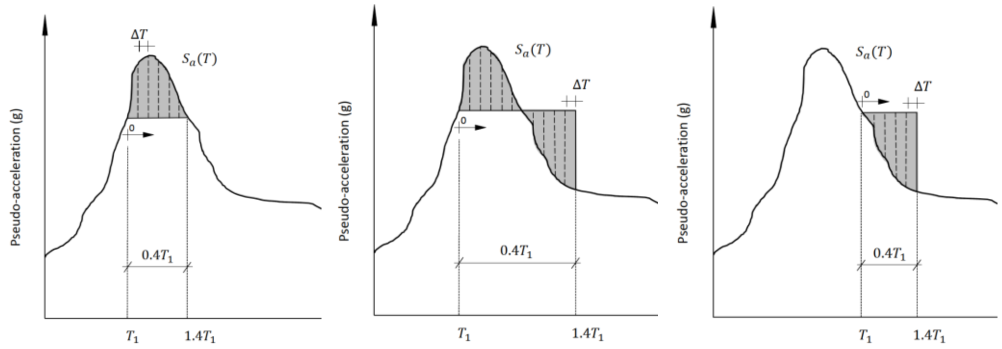

Fig. 7 – Ascending, balanced and descending spectra (left to right).

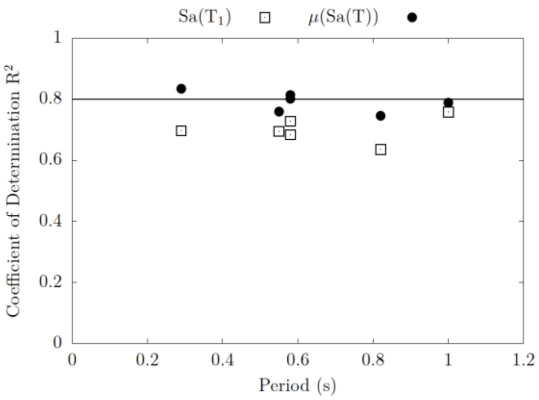

Fig. 8 – Coefficients results with respect to the period of frames for μ(Sa(T)) with threshold line at $R^2 = 0.80$.

The term in front of the summation refers to the length of the period elongation process and the period step size used for averaging the spectral acceleration values within the elongated period range as shown in Fig. 7. In this manner, an efficient consideration of the response spectrum shape for μ(Sa(T)) is ensured. The comparison of the correlation results obtained with $S_a(T_1)$ and the new intensity measure, μ(Sa(T)), shown in Fig. 8, reveal an improvement of the spectral acceleration parameter. The spectral intensity measure defined in Eq. (5) proved good correlation performance based on 372 nonlinear time history analyses of six reinforced concrete frame structures, having fundamental periods ranging from 0.29 − 1.0 s.

6. Summary / Conclusions

The results of an extensive correlation study between structural response and ground motion intensity parameters are represented. Eleven intensity measures from the literature were selected and their correlation strength was investigated with the average interstorey drift ratio selected as damage parameter. Existing frames presenting diverse engineering approaches from different seismic regions were used. After studying the correlation strength of eleven IMs with six frames under 62 ground motion records, the outcomes were used in order to propose two new intensity measures which require relatively less computational effort and deliver sufficient correlation in a wide range of structural periods. This aim is reached based on the results of nonlinear time history analysis. These two new parameters can be further used in performance-based earthquake engineering field and ground motion selection processes.

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