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PREDICTING FLOOR RESPONSE SPECTRA FOR RC FRAME STRUCTURES

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Abstract. *In the framework of performance-based earthquake engineering, a fundamental step in the assessment of buildings is the definition of models for the prediction of the seismic demand, namely, the definition of functional relationships between Engineering Demand Parameters of interest (EDPs) and an earthquake Intensity Measure (IM). While many studies focused on EDP representative of the structural response, only few investigated EDP well correlated with the non-structural response. The objective of this paper is to evaluate commonly used IMs, which are currently available in the literature, with respect to their capability to predict Floor Response Spectra, EDPs usually adopted for defining the seismic demand in acceleration-sensitive non-structural elements. Selected for the study are two RC frame structures, characterized by different number of stories and masonry infill wall configurations, subjected to a large number of ordinary natural accelerograms.*

1. INTRODUCTION

Non-structural elements are those buildings' components supported by the structure, such as architectural elements (e.g., ceilings, partitions and paneling) and mechanical and electrical equipment (e.g., elevators, tanks, pipes, antennas, transformers and emergency systems), that do not contribute to carrying gravity loads. Reducing seismic damage to these elements is of primary importance not only for economic reasons, since in most buildings they account for a large percentage of the building total replacement cost, but also for maintaining the functionality of the building immediately after the earthquake. Non-structural damage, in fact, can both produce significant economic losses and make the building unusable for a period of time that can vary from few weeks to several months. The usability aspect is crucial especially for buildings with emergency management functions, such as hospitals or police stations, which need to remain fully operational after frequent and rare seismic events. In addition, damage to non-structural elements caused by earthquakes can increase casualty risk. Potential threats to the safety of the occupants can be caused, for example, by falling of chimneys and ceiling panels and also by fire triggered by the pipe gas leaks that may ignite and explode. Based on these premises it is apparent that an accurate evaluation of the dynamic behavior of non-structural elements during earthquake events is essential.

In the framework of performance-based earthquake engineering, a fundamental step in the assessment of buildings is the definition of probabilistic seismic demand models (PSDM), namely, of functional relationships between Engineering Demand Parameters of interest (EDPs) and an earthquake Intensity Measure (IM). Most of the research work on PSDM that can be found in the literature focuses on the response prediction of regular structures and on EDPs well correlated to structural damage only. Recently investigations have been carried out also on types of structures different than regular moment resisting frames. Among these studies deserve to be mentioned those of Lucchini et al. [9], [10] and Asgarian et al. [1] on torsional and tall buildings, respectively, and the work of Mollaioli et al. on base-isolated frames [14]. Only a few studies, however, has focused on the prediction of EDPs well correlated to non-structural damage. In such works, the response parameters usually adopted for defining the demand in acceleration-sensitive non-structural elements are the floor response spectra. In Clayton and Medina [2] only the spectral acceleration of the ground motion at the first-mode period of the supporting structure is investigated. In Taghavi and Miranda [17] the PGA and the second-mode ordinate of the spectrum are also considered as well as a combination of them. Elenas and Meskouris [7] evaluated several IMs from the literature, but using in the analyses a limited number of ground motions and focusing on peak floor accelerations only.

The objective of this paper is to investigate the floor response spectra predictive capability of IMs that are commonly used in seismic assessment of buildings. Selected for the study are two RC frame structures characterized by different number of stories and masonry infill wall configurations. Studies [5] [6] have shown, in fact, that infills walls may affect significantly the seismic response of framed systems, and can modify both spectral shape and amplitude of the floor response spectra [12] [13]. A large number of ordinary natural records are used for exciting the buildings. For each considered IM, the predictive capability is evaluated as follows. First, a large number of ordinary natural records are used for exciting the buildings. Then, the floor response spectra are calculated using the obtained floor acceleration responses of the structures, and their ordinate values correlated to the values of the ground motion IMs. The predictive capability of the IMs is finally estimated by evaluating the results of the regression/correlation analyses.

2. CASE STUDIES

2.1 Studied structures

The selected buildings are a 2-bay 4-storey and a 2-bay 6-storey ductile reinforced concrete frame structures (see Figure 1), which are characterized by strong beams and weak columns. The frames are designed according to the seismic code in force in Italy between 1996 and 2008 that lacks any capacity design provision. The two frames are characterized by two infill walls configurations: bare and with brick infill walls at all stories. The infill walls are meant to represent internal partitions or weak perimeter walls. The four frames will be called 4b, 4w, 6b, and 6w, where the number indicates the stories and the letters “b” and “w” stands for bare and walled, respectively. Both buildings are characterized by a regular stiffness and mass distribution, and by a design lateral base shear capacity equal to 35% of the total weight. The masses of the walls are included also in the bare frame models. The length of each bay and the height of each story are identical in all four models. Table 1 shows the results of modal analyses run with linear models of the bare frames built by using for structural members the effective stiffness to yielding.

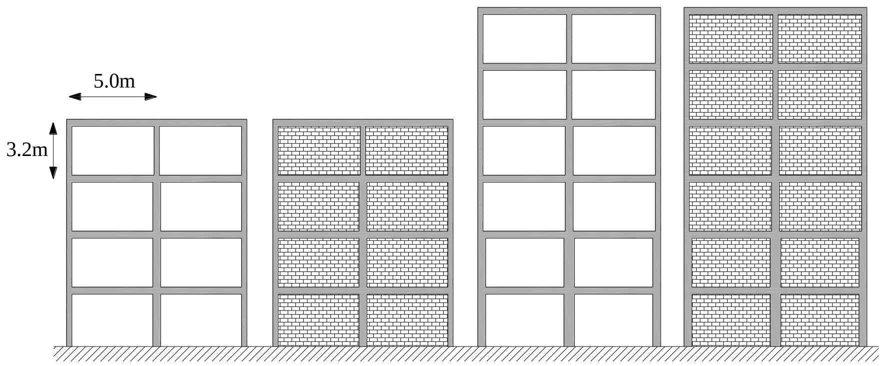

Figure 1: Schematic front view of the studied bare and infilled frame RC structures of 4- and 6-storey buildings.

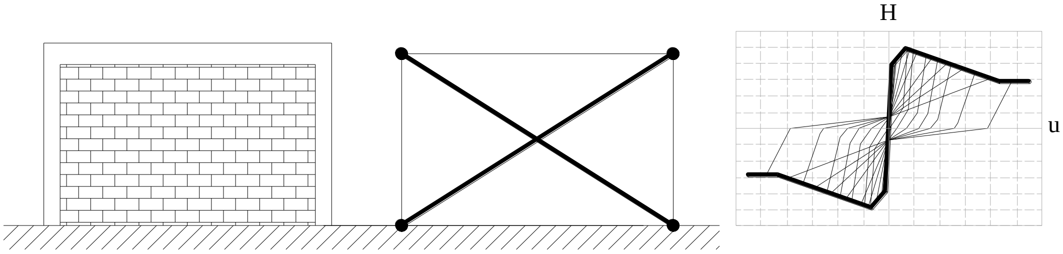

Figure 2: Infill wall model: struts system and horizontal force-displacement (H-u) constitutive behavior.

	4b		6b	
Mode	T	PMR	T	PMR
1	1.30s	80%	1.50s	75%
2	0.49s	11%	0.54s	11%
3	0.29s	5%	0.32s	5%

Table 1: Periods (T) and participating mass ratios (PMR) of the first three modes of vibration obtained with models built with the effective stiffness to yielding of structural members.

The nonlinear seismic response of the buildings is evaluated using finite element models built in OpenSees [11]. Structural members are modeled with force-based nonlinear elements characterized by distributed inelasticity, while contribution of the infill walls is represented using the equivalent strut model proposed by Decanini and Fantin [3]. The model consists of a system of diagonal struts connected to the nodes of the frame that act only in compression and with a hysteretic behavior characterized by stiffness degradation, strength deterioration, and loop pinching (Figure 2). The parameter values were calibrated after the work by Decanini et al. [4] on typical infill walls frequently used in Italy. These walls are made of 120 mm thick hollow bricks and a mixture of cement, sand and lime mortar, characterized by a compressive and shear strength (the latter evaluated through diagonal compressive test) equal to 1.2 and 0.2 MPa, respectively, and an initial elastic modulus of 1,050 MPa. A Rayleigh damping proportional to the mass and tangent stiffness matrix is considered with coefficients calibrated to provide a 5% damping at the first and third mode periods of the undamaged structures. The effects of geometric nonlinearities are not considered.

2.2 Ground motion database

The dynamic response of the studied buildings is evaluated via time-history analyses that use an ensemble of 72 ground motions from 26 worldwide earthquakes with magnitude ranging from 5.0 to 7.6 (see Figure 3). All the selected accelerograms are extracted from the Pacific Earthquake Engineering Research (PEER) Next Generation Attenuation (NGA) database [15]. All of them have a “usable” frequency range that brackets the frequency range of the response of these buildings. They are characterized by the same NEHRP soil condition type C-D. In general, they consist of far-field “standard” records that do not include any recognizable pulse in the velocity trace.

Not all obtained responses are used in the following correlation analyses. Responses corresponding to maximum inter-storey drift ratio values greater than 5% are excluded because considered to be associated, for the studied buildings, to deformations close to those expected at the onset of collapse. For this range of responses, the seismic demand evaluation of non-structural elements is, of course, not a major concern and, thus, the prediction of the floor response spectra is not of interest. As a consequence, in the case of the bare frames the results of only 65 time-histories are used in the regression analyses, while in the case of the infilled frames the used results are only those from 71 time-histories.

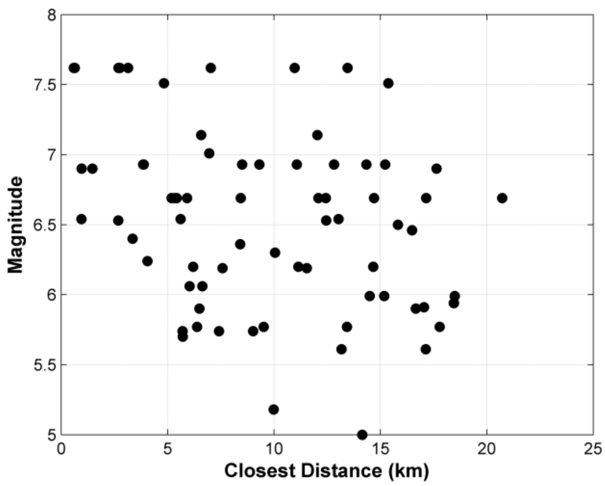

Figure 3: Earthquake magnitude and distance range for the 72 ordinary ground motions used in the analyses.

3. INTENSITY MEASURES AND ENGINEERING DEMAND PAPAREMTERS

3.1 Intensity measures

The IMs investigated in the present paper are: the Peak Ground Acceleration (PGA) and the Peak Ground Velocity (PGV); the compound acceleration-related IM (Ia), proposed by Riddell and Garcia [16], whose definition is given in Equation (1); the pseudo-spectral acceleration at the first- and second-mode period of the structure ($S_{pa}(T_1)$ and $S_{pa}(T_2)$, respectively); the Housner Intensity (I_H) [8], as defined in Equation (2); the Velocity and the Acceleration Spectrum Intensity (VSI and ASI, respectively) [19], derived from the absolute velocity and pseudo-acceleration response spectra as described in Equation (3) and (4).

$$\begin{gathered} I_a = PGA \cdot t_d^{1/3} \\ t_d = t_2 - t_1 ;\, t_1 = t(5\%AI);\; t_2 = t(95\%AI) \\ AI = \text{Arias Intensity} \end{gathered} \tag{1}$$

$$\begin{gathered} I_H = \int_{0.1}^{2.5} S_{pv}\, dT \\ S_{pv} = 5\%\ \text{damp. pseudo-velocity spectrum} \end{gathered} \tag{2}$$

$$\begin{gathered} VSI = \int_{0.1}^{2.5} S_v\, dT \\ S_v = 5\%\ \text{damp. absolute velocity spectrum} \end{gathered} \tag{3}$$

$$\begin{gathered} ASI = \int_{0.1}^{0.5} S_{pa}\, dT \\ S_{pa} = 5\%\ \text{damp. pseudo-acceleration spectrum} \end{gathered} \tag{4}$$

A modified version of the ASI (MASI), obtained by simply changing the periods range of integration from 0.1s-0.5s to 0.1s-2.5s, is investigated as well. This different range is considered so as to include periods of the pseudo-acceleration response spectrum that can significantly affect the response of the studied RC frames. The definition of ASI given in Equation (4), in fact, was proposed for predicting the response of dams, structures which are characterized by values of the fundamental period that are usually lower than 0.5s.

3.2 Engineering demand parameters

A flexible non-structural element experiences maximum acceleration when its period of vibration is close to one of those of the modes of vibration of the supporting structure. Based on this observation, the parameters selected in this study to be representative of the non-structural seismic demand are the following: the Peak Floor Acceleration (PFA), used to represent infinitely (or almost infinitely) rigid elements, and the maximum ordinate of the Floor Acceleration response Spectrum within the short-period region (FAS_2) and the fundamental-region (FAS_1). In accordance with the approach adopted in Clayton and Medina [2], the boundaries of these two regions are 0, $0.5T_1$ and $2.0T_1$, where T_1 denotes the fundamental period of the structure.

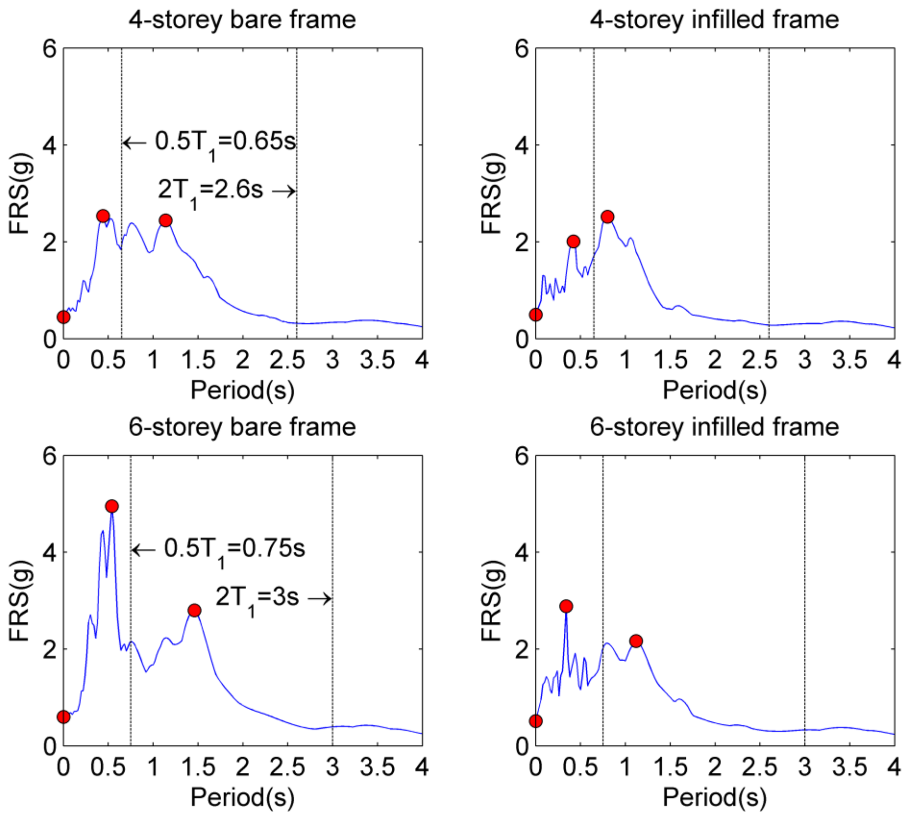

Figure 4: Floor Response Spectra obtained with one record of the ensemble. In red the spectral ordinates corresponding to PFA, FAS_1 FAS_2.

Many studies have shown that in medium- and low-rise buildings maximum non-structural accelerations usually occur at top floor. For such a reason, in the present investigation only the Floor Response Spectra (FRS) at the roof are calculated. In Figure 4 those obtained with one record from the used ensemble are shown. In the same Figure the considered fundamental- and short-period regions and ordinates of the spectra corresponding to PFA, FAS_2 and FAS_1 are also reported.

4. REGRESSION ANALYSES

4.1 Predictive models

Several are the properties that are usually investigated for evaluating the predictive capabilities of an IM (e.g., see Tothong and Luco [18]). Those considered in this study are the efficiency and the sufficiency. These properties are evaluated by first running non-linear dynamic analyses on the structures, and then by carrying out regression analyses between the obtained EDP values and the IM values of the used earthquake records. In all the regression analyses the used functional form is the following:

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

Among the statistical parameters calculated in the regressions, the standard error of residuals σ_ε is used for measuring the predictive efficiency of the IM. IMs resulting in EDPs standard errors of the order of 0.20-0.30 are normally considered as having a good efficiency, while the range 0.30-0.40 is still considered as reasonably acceptable. The regression residuals $\varepsilon|_{IM}$, instead, are used for evaluating the IM sufficiency. If the following predictive models are used for correlating $\varepsilon|_{IM}$ with magnitude (M) and distance (R):

$$\varepsilon|_{IM} = \alpha_M + \beta_M \cdot M,\ \varepsilon|_{IM} = \alpha_R + \beta_R \cdot R \tag{6}$$

the sufficiency can be directly measured by the p-value for the estimated slope coefficient β. A smaller p-value of β indicates a less sufficient IM, where M or R have significant influence on residuals of EDP. Generally, IM is considered sufficient when the p-value is more than 0.05.

4.2 Results

By looking at the results of the correlation analyses between IMs and PFA reported in Figure 5, it can be observed that in general the most efficient predictor for this EDP is MASI. The values of the standard error of residuals obtained with this IM are lower than 0.2 for all the studied buildings. Only for the 4w, the σ_ε values of the PGA-PFA regressions are slightly lower. This finding is in accordance with those of other studies from the literature that have demonstrated how efficient the PGA can be in predicting the response of structures with fundamental periods in the short-period (acceleration-controlled) region of the spectrum. The results of the present work show that in case of the PFA prediction, MASI can become much more efficient than the PGA with the increase of the flexibility of the structure (see in particular the results obtained with the 6b building).

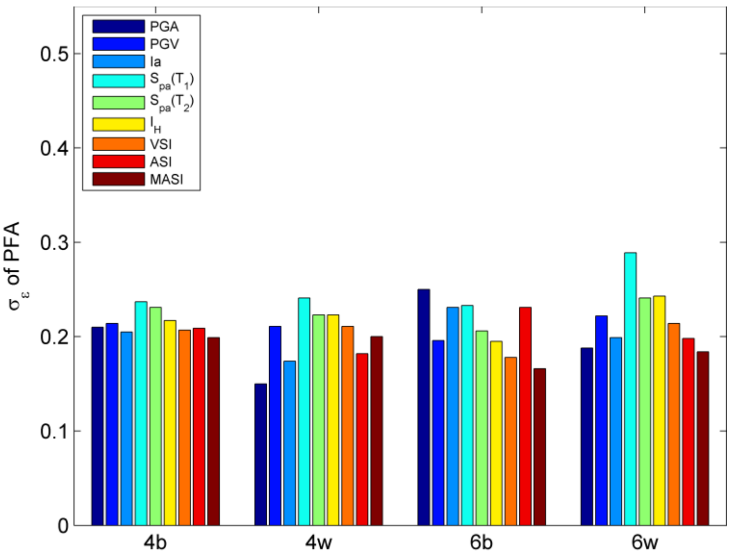

Figure 5: Standard error of residuals σ_ε obtained in the IMs-PFA correlation analyses.

In Figure 6, the standard errors of residuals of the IMs-FAS_1 regressions are reported. By comparing the plots of Figure 5 and Figure 6, it can be noticed that in the FAS_1 predictions the efficiency of all the IMs gets significantly worse. If the σ_ε values obtained with the different predictors are compared to each other, it can be found that the best IM is $S_{pa}(T_1)$. Also the efficiency of I_H, VSI and MASI is reasonably acceptable, being very close to that of $S_{pa}(T_1)$ for all the studied buildings except the 6b. About ASI and MASI, it can be observed that for the case of the 4-storey structures the predictive efficiency is almost the same, while for the case of the more flexible 6-storey buildings by using the modified IM than the existing one significant improvement in the FAS_1 predictions can be obtained.

The analyses results of the IMs-FAS_2 regressions are given in Figure 7. Analogously to what observed in the FAS_1 predictions, the most efficient predictor for this EDP is $S_{pa}(T_2)$. The standard errors of residuals obtained with $S_{pa}(T_2)$ are always close to the minimum observed values, being lower than 0.3 in all of the studied cases. For the 4-storey buildings, the σ_ε obtained with all the IMs are actually very similar. For the 6-storey frames, instead, the predictors efficiencies are much more different: in particular, for the bare frames the most efficient IM results to be $S_{pa}(T_2)$ while for the infilled frames is I_a. The predictive efficiency of ASI and MASI is in this case pretty the same.

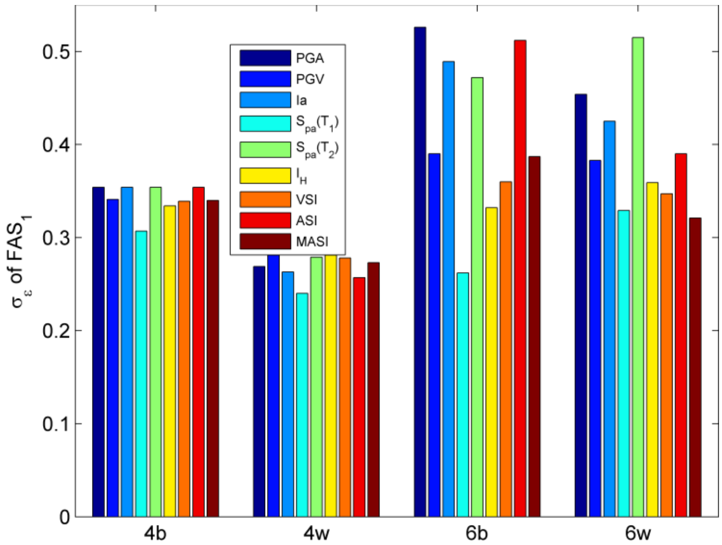

Figure 6: Standard error of residuals σ_ε obtained in the IMs- FAS_1 correlation analyses.

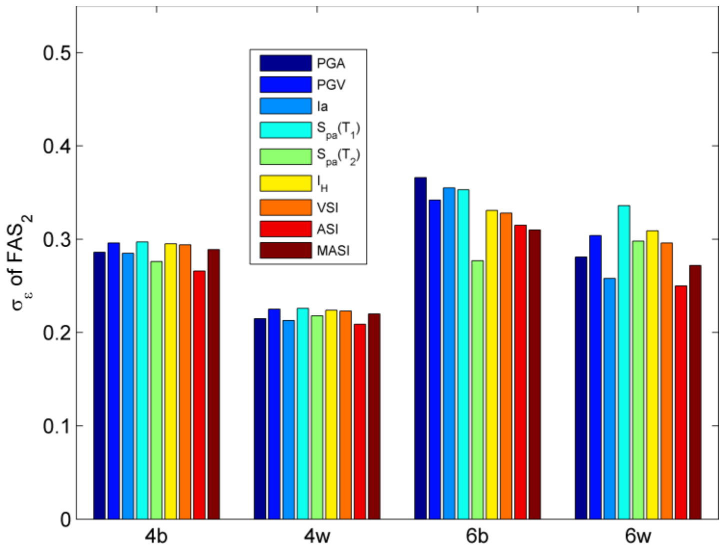

Figure 7. Standard error of residuals σ_ε obtained in the IMs- FAS_2 correlation analyses.

In Table 2 and Table 3 the p-values of β_M and β_R calculated for the evaluation of the IMs sufficiency are reported. In general, it can be observed that the only IMs that are sufficient for all the studied cases with respect to both magnitude and distance and for all the considered EDPs are $S_{pa}(T_1)$ and I_H. Also PGV and VSI are characterized in many cases by high p-values. The prediction errors obtained with MASI are much less correlated to magnitude and distance than those obtained with ASI.

In particular, by observing the results of the IMs-PFA regressions only, it can be observed that between the two most efficient IMs, namely, PGA and MASI, only the latter is sufficient for all the studied cases. The PGA is sufficient only when predicting the response of the 4w building. In the FAS_1 predictions, $S_{pa}(T_1)$ and I_H are always sufficient while VSI and MASI are not sufficient in two cases only. In the FAS_2 predictions, both $S_{pa}(T_2)$ and I_a result to be sufficient.

p-value for β_M		PGA	PGV	I_a	$S_{pa}(T_1)$	$S_{pa}(T_2)$	I_H	VSI	ASI	MASI
Prediction of PFA	4b	**0.00**	0.79	0.13	0.54	**0.02**	0.96	0.96	**0.01**	0.95
	4w	0.06	0.51	0.47	0.68	0.20	0.70	0.48	0.20	0.34
	6b	**0.00**	0.99	**0.03**	0.69	**0.00**	0.65	0.61	**0.00**	0.76
	6w	**0.00**	0.27	0.91	0.90	0.14	0.32	0.15	**0.01**	0.06
Prediction of FAS_1	4b	**0.00**	**0.01**	**0.00**	0.18	**0.00**	0.05	**0.02**	**0.00**	**0.01**
	4w	**0.00**	0.35	**0.02**	0.47	**0.00**	0.77	0.56	**0.00**	0.50
	6b	**0.00**	**0.00**	**0.00**	0.24	**0.00**	0.10	**0.03**	**0.00**	**0.00**
	6w	**0.00**	0.22	**0.00**	0.50	**0.00**	0.78	0.46	**0.00**	0.23
Prediction of FAS_2	4b	0.13	0.16	0.38	0.06	0.45	0.19	0.24	0.47	0.45
	4w	0.66	0.44	0.66	0.42	0.83	0.69	0.88	1.00	0.88
	6b	**0.00**	0.52	0.09	0.42	0.30	0.88	0.83	**0.03**	0.98
	6w	**0.00**	0.66	0.56	0.28	0.09	0.74	0.87	**0.02**	0.85

Table 2: Prediction of the $\varepsilon|_{IM}$ residuals obtained in the IMs-EDPs regressions: p-values for the slope coefficient β_M (in bold the values lower than 0.05).

p-value for β_R		PGA	PGV	I_a	$S_{pa}(T_1)$	$S_{pa}(T_2)$	I_H	VSI	ASI	MASI
Prediction of PFA	4b	0.98	0.52	0.39	0.91	0.53	0.57	0.51	0.78	0.33
	4w	0.80	0.80	0.39	0.33	0.09	0.88	0.90	0.21	0.74
	6b	0.48	0.15	0.39	0.93	0.24	0.19	0.13	0.24	0.10
	6w	0.08	0.78	0.89	0.18	**0.01**	0.89	0.84	0.01	0.62
Prediction of FAS_1	4b	0.26	0.87	0.59	0.73	0.55	0.81	0.99	0.42	0.97
	4w	**0.03**	0.66	0.28	0.29	**0.01**	0.79	0.72	**0.02**	0.71
	6b	**0.01**	0.64	0.18	0.53	**0.02**	0.81	0.87	**0.01**	0.61
	6w	**0.01**	0.96	0.20	0.34	**0.01**	0.74	0.86	**0.00**	0.93
Prediction of FAS_2	4b	0.79	0.58	1.00	0.41	0.81	0.59	0.66	0.79	0.87
	4w	0.53	0.98	0.24	0.97	0.79	0.78	0.64	0.52	0.48
	6b	0.46	0.78	0.97	0.75	0.72	0.68	0.67	0.59	0.50
	6w	0.22	0.77	0.86	0.20	0.07	0.67	0.80	0.09	0.96

Table 3: Prediction of the $\varepsilon|_{IM}$ residuals obtained in the IMs-EDPs regressions: p-values for slope coefficient β_R (in bold the values lower than 0.05).

5. SUMMARY AND CONCLUSIONS

In the present work, the seismic response of four buildings characterized by different number of stories and infill wall configurations were analyzed by using a large number of ground motions. Floor response spectra at top floor were evaluated with the objective of evaluating the capability of several intensity measures from the literature in predicting the acceleration response of non-structural elements. Different regions of the spectra were investigated by using the three following engineering demand parameters:

- PFA, the spectral ordinate of the spectrum corresponding to a period of vibration of the non-structural element T equal to 0;
- FAS_2, the maximum ordinate of the spectrum in the short-period region $0s<T<0.5T_1$, where T_1 is the fundamental period of the structure;
- FAS_1, the maximum ordinate of the spectrum in the fundamental-period region $0.5 T_1<T<2.0T_1$.

For each intensity measure both the predictive efficiency and sufficiency were investigated. Such properties were estimated by evaluating the results of correlation analyses between the values of the ground motion intensity measures and the obtained values of the considered engineering demand parameters. The main findings of the analyses can be summarized in the following:

- intensity measures are in general much less efficient in predicting the acceleration response of non-structural elements than that of the structure;
- the predictive capability of the acceleration spectrum intensity ASI can be significantly improved by changing the periods range of integration from 0.1s-0.5s to 0.1s-2.5s;
- the best intensity measure for predicting PFA is MASI, namely, the modified version of the acceleration spectrum intensity investigated in this study;
- for short-period structures only, such as the infilled frames investigated in this work, peak ground acceleration is also a good predictor for PFA;
- the best intensity measure for predicting FAS_1 is $S_{pa}(T_1)$, namely, the pseudo-spectral acceleration at the first-mode period of the structure;
- the best predictor for FAS_2 is $S_{pa}(T_2)$, namely, the pseudo-spectral acceleration at the second-mode period of the structure;
- the best option for predicting all of the regions of the floor response spectra with one intensity measure only is to use $S_{pa}(T_1)$, I_H or MASI;
- $S_{pa}(T_1)$, even if not always among the most efficient predictors, was found to be sufficient for all of the studied cases;
- Housner intensity I_H and MASI were found to be the most robust predictors, that is, the most efficient ones regardless of the considered engineering demand parameter and of the supporting structure properties;
- among I_H and MASI, the first one were found to be in general more sufficient while the latter slightly more efficient.

The above listed findings can be used as a guide for the selection of the best intensity measure for predicting floor response spectra of frame buildings. It is important to underline that the results obtained in the present investigation can be applied, in principle, only to structures with dynamic properties similar to those of the studied frames.

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