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CORRELATION BETWEEN SEISMIC GROUND MOTIONS AND FLOOR ACCELERATION DEMAND FOR FRAME AND STRUCTURAL-WALL BUILDINGS

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ABSTRACT

This paper focuses on the ability of Intensity Measures (IMs) to predict horizontal floor acceleration demand of multi-story buildings. The efficiency of a broad panel of IMs is compared, based on: a) the use of a large dataset of recorded European earthquake ground motions (over 4,000 records); b) numerical analyses performed with three-dimensional models, representing actual frame and structural wall buildings and validated against experimental test results; c) statistical analyses of results. The study finds that the efficiency of current IMs depends on the type of building being examined. In agreement with the literature, the *PGA* has proven to be efficient for low-frequency frame buildings. In contrast, when considering stiff structural wall buildings, the IMs based on low-damped spectral acceleration at the fundamental frequency of the structure have demonstrated the highest efficiency with respect to predicting the ordinates of floor response spectra.

INTRODUCTION

Secondary Systems or Non-Structural Components (NSCs) refer to the components of a building that are not part of the primary seismic load-resisting system. The importance of considering NSCs in seismic risk analyses is supported by several reasons. First of all, in most types of buildings, Non-Structural Components account for a major share of total building costs and therefore represent a large proportion of potential losses to owners, occupants and insurance companies (Taghavi and Miranda, 2003). Second, damage to most types of NSCs in buildings is usually initiated at levels of deformation much lower than those required to initiate structural damage (i.e. high accelerations associated with small drifts can damage ceilings, piping and other nonstructural components, yet with little or no damage to structural members). This condition implies that with respect to structurally damaging earthquake events, larger geographic areas are affected (as low-to-moderate earthquake events are more diffuse than more major events). Third, if the nonstructural damage is substantial, then significant economic losses can be caused by a temporary loss of building functionality. Moreover, the ability of NSCs to remain intact during an earthquake, which is critical to maintaining the operations of emergency services and/or continuing functionality of key facilities (e.g. nuclear power plants, chemical plants, hospitals), can be life-saving.

Ground motion Intensity Measures (IMs) constitute the link between seismic hazard and seismic demand analysis; they play a key role in: ground motion selection procedures, seismic hazard definition, fragility function definition, and Operating Basis Earthquake (OBE) exceedance definition.

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The two main characteristics defining IMs are efficiency and sufficiency. An IM is deemed to be efficient if it yields, for a given value, reduced variability in the structural response; on the other hand, a sufficient IM is defined by its ability to yield, for a given value, a structural response conditionally independent of both earthquake magnitude and source-to-site distance. The present paper focuses exclusively on the efficiency aspect of IMs.

Many studies have proposed IMs and investigated their efficiency, yet it is well known that the majority of such research has been directed at IMs seeking to estimate structural deformation demand in neglecting the floor acceleration demand, despite the fact that a sizable part of NSCs is sensitive to inertial failure. The studies available, most of which have been conducted in assuming low-frequency frame structures, rate the *PGA* as the most efficient Intensity Measure. For instance, in their comprehensive study, Taghavi and Miranda (2006) considered numerical models with fundamental frequency ranging from 0.25 Hz to 2 Hz and a selection of a few IMs; they concluded that the *PGA* is more efficient than IMs based on spectral acceleration at the fundamental frequency of the structure $S_{pa}(f_1)$, even though these latter IMs are generally accepted as the most efficient with respect to structural demand (Buratti, 2012; De Biasio *et al.*, 2014). The lack of performance provided by $S_{pa}(f_1)$-based IMs is typically justified by the fact that as opposed to structural demand, which is mainly dictated by the structure's first vibration mode, floor acceleration demand is also strongly dependent on the higher vibration modes.

The aim of the present work therefore is to compare the efficiency of a wide array of well-known IMs with respect to the prediction of floor acceleration demand, with special emphasis placed on high-frequency structural wall buildings.

To overcome the limitations (often present in this type of work) due to the use of small ground motion datasets and simplistic test case structures, the present study makes use of a large dataset of recorded ground motions and state-of-the-art numerical models that have been validated against experimental results. This study is performed in following three main steps: a) computer analysis of the 4,031 accelerograms in order to generate the values of the selected IMs; b) linear dynamic analyses to provide the structural response of the chosen reinforced concrete structures for the given 4,031 seismic excitations; and c) a statistical analysis of the outputs of steps a) and b) to produce the interdependency rating between seismic acceleration parameters and the selected floor acceleration demand parameters.

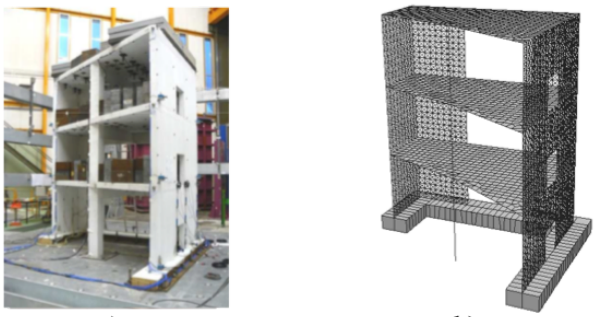
The following IMs are compared in this study: peak ground acceleration and velocity *(PGA, PGV)*, low-damped spectral acceleration at fundamental frequency $S_{pa}(f_1)$, Arias intensity I_A (Arias, 1970), Cumulative Absolute Velocity *CAV* (EPRI, 1988), Standardized Cumulative Absolute Velocity *S-CAV* (EPRI, 1991), Housner Intensity I_H (Housner, 1959), Effective Peak Acceleration *EPA* (ATC, 1978), Acceleration Spectral Intensity *ASI* (Von Thun *et al.*, 1988), Root Mean Square acceleration a_{RMS} (Housner *et al.*, 1964), Characteristic Intensity I_C (Park *et al.*, 1985), S^* (Cordova *et al.*, 2001), I_{NP} (Bojorquez *et al.*, 2011), and ASA_{40} (De Biasio *et al.*, 2014). The definition of each of these IMs is presented in the literature and will not be repeated here.

COMPARATIVE ANALYSES

Test-case structures and numerical models

In order to compare the performance of these various IMs, the numerical analyses are performed on two experimentally tested structures. This set-up offers the advantage of a validation tool for the numerical models, which under the condition of sufficiently close agreement between numerical simulation runs and experimental testing, lends credence to the results derived from numerical models.

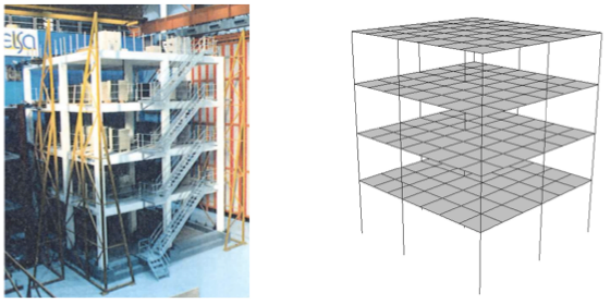
SMART-2013 (CEA, 2013) is a ¼-scale model designed in accordance with current French nuclear regulations and tested in 2013 on the shaking table at the French Atomic Energy Agency (this structure is also object of an international blind contest). The mock-up (Fig. 1) is a trapezoidal, three-story reinforced concrete structure representative of a typical simplified half-section of a nuclear plant building. The mock-up is designed and tested according to precise similitude criteria that ensure its behavior is representative of a full-scale structure. Nevertheless, once created and validated, the SMART-2013 numerical model has been rescaled to full size in order to avoid having to scale the entire ground motion dataset.



a) b)

Figure 1: SMART-2013 test case building: a) structure, b) numerical model

EC8-FRAME (JRC, 1994) is a full-scale, four-story, high-ductility R/C frame designed in accordance with European Seismic Code EC8 (EC8, 1988) and tested on the reaction wall of the European Joint Research Center (JRC). The structure (Fig. 2) is symmetrical in one direction, with two equal 5-m spans, while slightly irregular in the other direction due to the different span lengths (6 and 4 m).



a) b)

Figure 2: EC8-FRAME test case building: a) structure, b) numerical model

In this comparative study, linear-elastic modeling of the test case buildings has been adopted. This choice is justified by the fact that, as shown by Rodriguez *et al.* (2002), the maximum Floor Response Spectrum (FRS) acceleration ordinates (which correspond to the selected demand parameters) occur when the building is behaving elastically. Moreover, given that the study is focused on the structure's floor acceleration demand during low-to-moderate earthquakes, it is assumed that such earthquakes are not able to significantly damage a well-engineered reinforced concrete structure.

Regarding the FE numerical discretization, a lattice modeling technique derived from the approach developed by Kotronis *et al.* (2003) has been used to model the structural walls of the SMART-2013 building. For both the test-case structures (SMART-2013 and EC8-FRAME), the columns and beams have been modeled by means of Timoshenko beam elements, with slabs being represented by shell elements and the incorporation of distributed masses. The mechanical characteristics of the structural materials have been assigned based on elementary tests on the concrete and steel of both test case structures. The dynamic characteristics of the numerical models are summarized in Table 1.

Table 1: Test case structures: natural frequencies of numerical models

	Frequency [Hz]		
Mode	***SMART-2013***		***EC8-FRAME***
1	5.35		1.57
2	9.54		1.59
3	16.76		2.07

The predictive capabilities of these numerical models have been checked vs. results stemming from the respective experimental campaigns. The EC8-FRAME has been corroborated with the results of pseudo-dynamic (reaction wall) tests in terms of roof displacement time history. The SMART-2013 numerical model has been checked against the results of shaking table tests in terms of floor response spectra (Fig. 3). This comparison of numerical and experimental results has indicated the ability of both numerical models to predict with good agreement the structures' linear behavior both qualitatively and quantitatively.

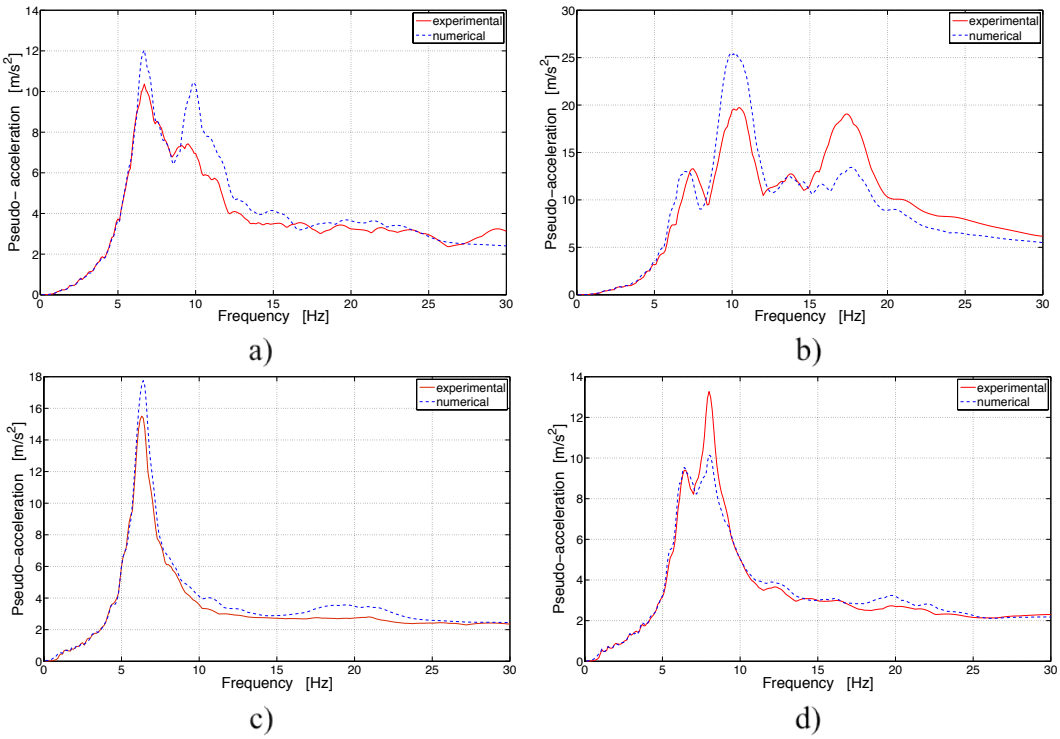


Figure 3: SMART-2013 numerical model validation: a) shaking table test1: FRS at the roof along the X-direction; b) shaking table test1: FRS at the roof along the Y-direction; c) shaking table test2: FRS at the roof along the X-direction; and d) shaking table test2: FRS at the roof along the Y-direction

Load and demand parameters

A three-dimensional load (i.e. two horizontal components and one vertical component) has been applied to the three-dimensional models clamped at the base.

The ground motion dataset used as input for the FE simulations is composed of the 4,031 three-dimensional records from the 2013 version of the RESORCE database (Akkar *et al.*, 2013), which combines ground motions recorded in Europe and nearby countries over the past decades relative to events with moment magnitude lying between 2 and 8.

The corresponding IMs have been computed as the geometric mean of IM values of the two horizontal components (Baker and Cornell, 2006).

The (horizontal) NSC acceleration demand has been measured in the numerical model as the maximum of the (horizontal) acceleration Floor Response Spectra (Villaverde, 2004) over all floors and over four frequency ranges: 0.9-1.1 Hz, 4.5-5.5 Hz, 9-11 Hz, and 18-22 Hz. These ranges reflect the hypothetical presence of NSCs with a fundamental frequency equal respectively to 1 Hz, 5 Hz, 10 Hz and 20 Hz, with a confidence interval of ±10% on these values. Use of the FRS method (to decouple the response of the supporting structure from that of the non-structural component) is acceptable for NSCs with a mass less than 1% of the supporting structure mass (USNRC, 1978).

RESULTS

Analysis method

To obtain the extent of the correlation between IMs and Demand Parameters (EDPs), the rank correlation coefficient, according to Spearman (1925), has been calculated for the IM-EDP pairs. Such a coefficient measures how well the data agree with monotonic (whether linear or not) ranking. The Spearman rank correlation coefficient between two variables X and Y is given by the following relation (1):

$$\rho_{Spearman} = 1 - \frac{6\sum_{i=1}^{N} D^2}{N(N^2-1)} \tag{1}$$

where D denotes the differences between the ranks of corresponding values of X_i and Y_i, and N the number of pairs of values (X,Y) in the dataset.

Results

The results of the statistical analysis performed on the IM-EDP relation are presented in Table 2 in terms of Spearman rank correlation coefficient and in Figures 4-5 in terms of scatter plots.

Table 2: Spearman's rank correlation coefficient results

IM		***SMART-2013***				***EC8-FRAME***			
		Max FRS at:							
IM type	IM	1 Hz ±10%	5 Hz ±10%	10Hz ±10%	20Hz ±10%	1 Hz ±10%	5 Hz ±10%	10Hz ±10%	20Hz ±10%
Frequency Response based	**$S_{pa}(f_1)$**	**0.99**	**0.96**	**0.9**	**0.93**	**0.99**	**0.86**	**0.86**	**0.89**
	ASA_{40}	0.98	0.94	0.87	0.89	0.99	0.82	0.83	0.86
	S^*	0.97	0.92	0.83	0.86	0.99	0.8	0.81	0.85
	I_{NP}	0.99	0.96	0.89	0.91	0.99	0.86	0.86	0.89
	EPA	0.9	0.79	0.67	0.7	0.96	0.92	0.92	0.94
	ASI	0.98	0.92	0.84	0.87	0.87	0.99	0.98	0.97
	I_H	0.81	0.68	0.54	0.57	0.98	0.83	0.84	0.87
Amplitude based	**PGA**	**0.98**	**0.91**	**0.88**	**0.9**	**0.81**	**0.97**	**0.99**	**0.98**
	PGV	0.89	0.77	0.66	0.69	0.95	0.9	0.92	0.94
Duration Based	I_A	0.94	0.85	0.76	0.79	0.91	0.95	0.97	0.97
	CAV	0.85	0.74	0.62	0.64	0.94	0.87	0.88	0.9
	S-CAV	0.8	0.74	0.71	0.73	0.67	0.79	0.81	0.81
	a_{RMS}	0.62	0.59	0.58	0.59	0.52	0.62	0.63	0.63
	I_C	0.63	0.6	0.58	0.59	0.54	0.62	0.64	0.63

The results summarized in Table 2 highlight the following points:

a) The indication derived from the analyses performed on the frame structure contrasts with that stemming from analyses performed on the structural wall building. In other words, among the examined test cases, no one single IM is clearly superior to the others (and suitable for both types of structures);

b) In agreement with the literature, the *PGA* shows the highest efficiency in the case of a low-frequency frame structure (Fig. 4). However, when the structural wall building is considered, the *PGA* reveals a lower performance with respect to $S_{pa}(f_1)$ (Fig. 5). Regarding *PGV*, its efficiency is always below that of *PGA*;

c) Among the duration-based IMs, Arias Intensity (I_A) displays the best performance. Nevertheless, the efficiency of this class of IMs is typically lower than that of frequency response-based IMs;

d) In the case of the frame structure, all IMs based on spectral acceleration at the fundamental frequency of the structure, $(S_{pa}(T_1), S^*, I_{NP}, ASA_{40})$, exhibit lower efficiency in predicting FRS ordinates than the *PGA*. The efficiency of such IMs is however comparable or higher than that of the *PGA* when the structural wall building is taken into consideration;

e) The IMs accounting for the drop in structural frequency (S^*, I_{NP} and ASA_{40}) exhibit an efficiency that is (always) slightly less than that of $S_{pa}(T_1)$, which among all IMs considered is the one revealing the highest efficiency in the case of the structural wall building.

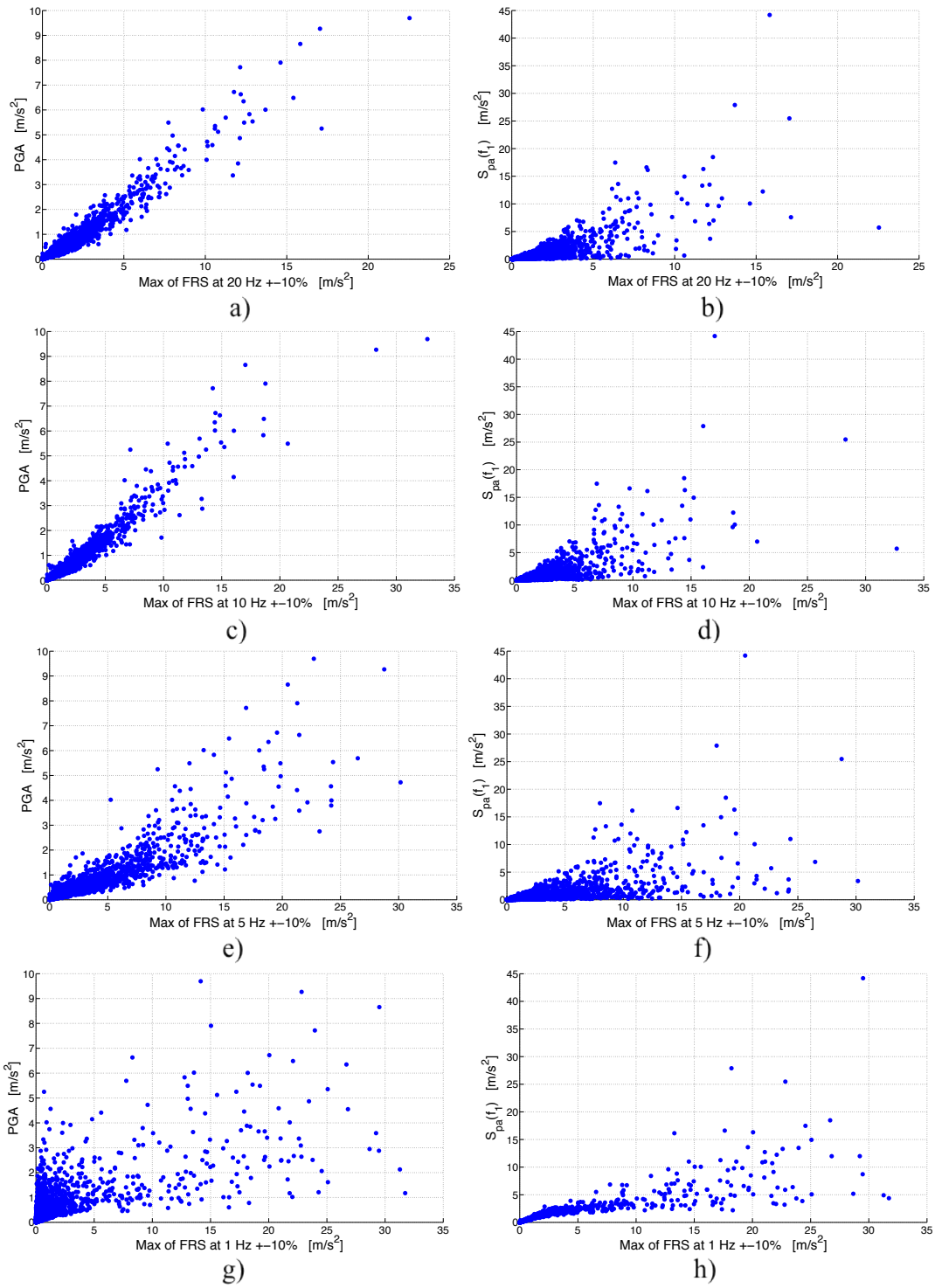


Figure 4. EC8-FRAME, scatter plots: (a) Max FRS at 20Hz vs. *PGA*; (b) Max FRS at 20Hz vs. $S_{pa}(f_1)$; (c) Max FRS at 10Hz vs. *PGA*; (d) Max FRS at 10Hz vs. $S_{pa}(f_1)$; (e) Max FRS at 50Hz vs. *PGA*; (f) Max FRS at 50Hz vs. $S_{pa}(f_1)$; (g) Max FRS at 1Hz vs. *PGA*; (h) Max FRS at 1Hz vs. $S_{pa}(f_1)$.

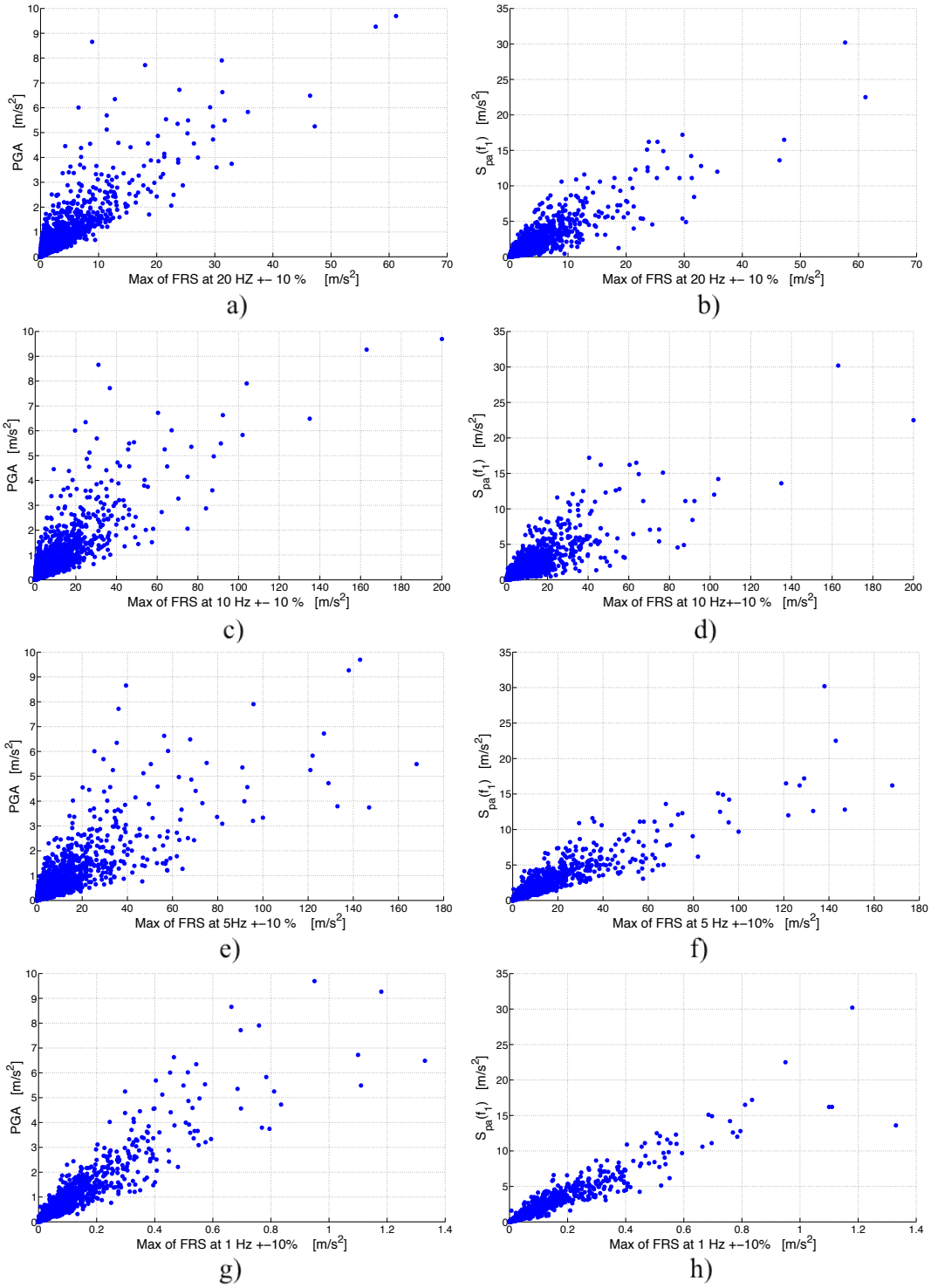


Figure 5: SMART-2013, scatter plots: a) Max FRS at 20 Hz vs. *PGA*; b) Max FRS at 20 Hz vs. $S_{pa}(f_1)$; c) Max FRS at 10 Hz vs. *PGA*; d) Max FRS at 10 Hz vs. $S_{pa}(f_1)$; e) Max FRS at 50 Hz vs. *PGA*; f) Max FRS at 50 Hz vs. $S_{pa}(f_1)$; g) Max FRS at 1 Hz vs. *PGA*; and h) Max FRS at 1 Hz vs. $S_{pa}(f_1)$

CONCLUSIONS

In the present paper, the efficiency of a large panel of IMs has been compared with respect to the ability to predict Floor Response Spectra ordinates in both frame and structural wall buildings. This comparative study has been based on linear dynamic simulations conducted on (experimentally validated) numerical models of existing buildings subjected to an extensive dataset of recorded European seismic ground motions.

This study has highlighted differences in the performance of IMs, based on the type of building being examined. Among the considered IMs, those based on spectral acceleration at the fundamental frequency of the structure, along with the *PGA*, prove to be the most efficient with respect to floor acceleration demand prediction. In agreement with the literature, the *PGA* offers the highest efficiency in the case of low-frequency frame structures. When high-frequency structural wall buildings are taken into consideration however, the *PGA* performance dips below that of $S_{pa}(f_1)$-based IMs. Within a procedural simplification framework, this last result could have interesting practical implications. The $S_{pa}(f_1)$-based IMs are widely recognized as the most efficient with respect to structural damage prediction; therefore, if their validity relative to NSC acceleration demand is proven, they could be used for both structural and non-structural acceleration demand, which in turn would imply, for instance, a reduction in the number of numerical simulations input into probabilistic seismic risk assessment and design.

Nevertheless, this study has demonstrated that further research is indeed necessary, in the aim of defining a suitable IM to predict floor acceleration demand. Such a predictive IM would ideally be characterized both by a validity not limited to a single class of structures and by a higher and more robust efficiency (with respect to NSCs frequencies) than that displayed by current IMs.

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