RELATION BETWEEN SEISMIC GROUND MOTION AND STRUCTURAL DAMAGES & FUNCTION LOSS

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Executive Summary

A fundamental issue that arises in the framework of Probabilistic Seismic Risk Analysis is the choice of ground motion Intensity Measures (IMs). Based on the Floor Response Spectrum method, the present report focuses on the ability of IMs to predict Non-Structural Components (NSCs) horizontal acceleration demand. The best-performing IMs with respect to structural-demand prediction are examined and a new IM, namely $E\text{-}ASA_R$ ((Equipment) Relative Average Spectral Acceleration) is proposed for the purpose of NSCs acceleration demand prediction. The IMs efficiency and sufficiency comparisons are based on: a) the use of a large dataset of recorded earthquake signals; b) numerical analyses executed with three-dimensional MDOF models, representing actual structural-wall and frame buildings; and c) systematic statistical analysis of the results. From the comparative study, the herein introduced $E\text{-}ASA_R$ shows high efficiency with respect to the estimation of maximum floor response spectra ordinates. Such efficiency is particularly remarkable in case of structural wall buildings. Besides, the sufficiency and the simple formulation, which allows the use of existing ground motion prediction models, make the $E\text{-}ASA_R$ a promising IM for Performances Based Seismic Design/Assessment.

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Résumé : The aim of the work is the evaluation of Intensity Measures 'performance with respect to the prediction of non-structural-components acceleration demand. A new Intensity Measure, particularly suited for non-structural-components housed on structural wall buildings, is proposed. The work is performed in the framework of the SIGMA (Seismic Ground Motion Assessment) project.

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SUMMARY

A fundamental issue that arises in the framework of Probabilistic Seismic Risk Analysis is the choice of ground motion Intensity Measures (IMs). Based on the Floor Response Spectrum method, the present report focuses on the ability of IMs to predict Non-Structural Components (NSCs) horizontal acceleration demand. The best-performing IMs with respect to structural-demand prediction are examined and a new IM, namely $E\text{-}ASA_R$ ((Equipment) Relative Average Spectral Acceleration) is proposed for the purpose of NSCs acceleration demand prediction. The IMs efficiency and sufficiency comparisons are based on: a) the use of a large dataset of recorded earthquake signals; b) numerical analyses executed with three-dimensional MDOF models, representing actual structural-wall and frame buildings; and c) systematic statistical analysis of the results. From the comparative study, the herein introduced $E\text{-}ASA_R$ shows high efficiency with respect to the estimation of maximum floor response spectra ordinates. Such efficiency is particularly remarkable in case of structural wall buildings. Besides, the sufficiency and the simple formulation, which allows the use of existing ground motion prediction models, make the $E\text{-}ASA_R$ a promising IM for Performances Based Seismic Design/Assessment.

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1. Introduction

Secondary Systems or Nonstructural Components (NSCs) are those systems and elements housed on or attached to the floors, roofs and walls of a building that are not part of their main structural system. It is recognized that, in the event of an earthquake, the economical losses are primarily produced by NSCs damage (Taghavi and Miranda, 2003). Moreover, the survival of NSCs during an earthquake, important for maintaining the operation of emergency services and/or the continuing functionality of critical facilities (e.g. Hospitals, Nuclear Power Plants, etc.), is a life-saving issue. The importance of NSCs seismic assessment is also highlighted by the fact that damage to NSCs usually initiates at levels of ground shaking much smaller than those required causing structural damage. This means that with respect to structural-damaging earthquake events, larger geographical areas are affected. Besides the NSCs damage risk can be considered higher, being low-to-moderate earthquake events more frequent than large ones.

The Seismic Probabilistic Risk Assessment (SPRA) methodology (Kennedy et al. (1980); Wakefield et al., 2003) is the most commonly used approach for evaluating the seismic safety of nuclear engineering structures, and in recent years it has also become popular for characterizing seismic behavior of civil structures. In the form developed by the Pacific Earthquake Engineering Research (PEER) Center, and here assumed as reference, the Performance Based Earthquake Engineering (PBEE) methodology is articulated in four processes (Porter, 2003): hazard analysis, structural analysis, damage analysis, and loss analysis.

In the Hazard Analysis, one evaluates the seismic hazard at the facility considering the seismic environment, the location and the structural features. The Hazard Curve describes the annual frequency with which seismic excitation is estimated to exceed various levels. Excitation is parameterized via an IM usually the *PGA* or the $S_{pa}(f_1)$, the damped elastic spectral acceleration at the fundamental frequency of the structure. In addition to quantifying IM, the hazard analysis leads to the selection of a sufficient number of appropriate (in terms of IM) ground-motion records for time history analyses. Therefore, the IM is the primary parameter by which the seismic hazard is defined. The choice of IM has a deep impact on the simplifying assumptions and methods that can be used to evaluate accurately and efficiently the risk integral, which aggregates the results of the four sub-tasks of the PBEE process (Conte et al., 2003). An improved IM (i.e. able to better capture the damaging features of a record and the site hazard), other than to reduce the record-to-record variability, makes criteria for selecting input ground motions for inelastic time-history analyses become less strict (Cornell, 2004).

The two main characteristics defining IMs are efficiency and sufficiency: an IM is defined efficient if it allows, for a given value, to obtain a reduced variability in the structural response; a sufficient IM, on the other hand, is defined as the one that for a given value renders the structural response conditionally independent of earthquake magnitude and source-to-site distance (Luco and Cornell, 2007). Furthermore, an effective and practically exploitable IM is one for which it is realistic to compute probabilistic seismic hazard (i.e. Ground Motion Prediction Equations have to be easily computable for such an IM).

It's a fact that most of the research about Intensity Measures has been focused on IMs to estimate structural-deformation-demand neglecting the floor-acceleration-demand. Being a sizeable part of NSCs sensible to inertial failure (i.e. electronic devices, piping systems, ceiling systems, ventilation ducts, machinery, bookcases, etc.), the aim of the present work is to propose a new Intensity Measure and to compare its efficiency and sufficiency to the ones of well known IMs, i.e.: 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), Effective Peak Acceleration *EPA* (ATC, 1978), Acceleration Spectral Intensity *ASI* (Von Thun et al., 1988), 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 Table 1.

The comparative study is performed following three main steps: a) computer analyses of the ground motions dataset's accelerograms to provide the values of the selected IMs; b) dynamic (mode-superposition method) analyses to provide the structural response, of the chosen reinforced concrete structures, for the given seismic excitations; c) statistical analysis of the outputs of the aforementioned two steps to provide the grade of efficiency and sufficiency of the examined IMs.

Table 1. IMs, from literature, compared in the study

TYPE	IM	NOTES
Frequency-Response Based	$S_{pa}(f_1)$	S_{pa} = pseudo-spectral acceleration f_1 = fundamental frequency
	$ASA_{40}(f_1)=\frac{2.5}{f_1}\int_{0.6\cdot f_1}^{f_1} S_{pa}(f,\xi)df$	S_{pa} = pseudo-spectral acceleration f_1 = fundamental frequency ξ = damping ratio
	$EPA=\frac{1}{2.5}\int_{0.1}^{2.5} S_{pa}(T,\xi)\ dT$	S_{pa} = pseudo-spectral acceleration ξ = damping ratio
	$ASI=\int_{0.1}^{0.5} S_{pa}(T,\xi)\ dT$	S_{pa} = pseudo-spectral acceleration ξ = damping ratio
	$S^*=S_{pa}(T_1)\left(\frac{S_{pa}(T_2)}{S_{pa}(T_1)}\right)^{0.5}$	S_{pa} = pseudo-spectral acceleration T_1 = fundamental period $T_2 = 2 * T_1$
	$I_{NP}=S_{pa}(T_1)\left(\frac{S_{paAV}(T_1,\ldots,T_2)}{S_{pa}(T_1)}\right)^{0.4}$	S_{paAV} = Averaged pseudo-spectral acceleration between T_1 and T_2 T_1 = fundamental period $T_2 = 2 * T_1$
Amplitude-Based	$PGA=\max\lvert a(t)\rvert$	*a(t) = acceleration time history*
	$PGV=\max\lvert v(t)\rvert$	*v(t) = velocity time history*
Duration-Based	$I_A=\frac{\pi}{2g}\int_0^{t_f} a(t)^2\,dt$	*a(t) = acceleration time history* *t(f) = total duration of the record*
	$CAV=\int_0^{t_f}\lvert a(t)\rvert dt$	*a(t) = acceleration time history* *t(f) = total duration of the record*
	$SCAV=CAV_i+\int_{i-1}^{t_i}\lvert a(t)\rvert dt$	*a(t) = acceleration values in one-second interval where at least one value exceeds 0.025 g;* *i =1,…,n with n equal to the record length in seconds*

2. Intensity Measure for NSCs acceleration demand

Despite the large amount of research focused on IMs to estimate structural-deformation-demand, the specific literature about IMs for floor-acceleration-demand estimation is quite limited. The few studies available (ex. Taghavi and Miranda, 2006; Zentner et al., 2011), are mostly realized considering low-frequency frame structures and denote the *PGA* as the most efficient IM. For instance, in their comprehensive study, Taghavi and Miranda (2006) considering few IMs and numerical models with fundamental frequency ranging from 0.25 Hz to 2 Hz, conclude that the *PGA* is more efficient than the IMs based on the spectral acceleration at the fundamental frequency of the structure, despite these last are accepted as the most efficient with respect to structural demand (NUREG, 1986; Buratti, 2012, De Biasio et al., 2014). The lack of performance of the $S_{pa}(f_1)$-based IMs is generally justified saying that differently from structural-demand, which is mainly leaded by the structure's first mode of vibration, floor-acceleration-demand is also strongly dependent on the higher vibration modes.

2.1 Factors affecting NSCs acceleration demand

In order to emphasize the ground motions' features affecting the NSCs acceleration demand, herein is studied the modal recombination of the response of a generic linear MDOF system having a linear SDOF system attached to the structural node *k*. The weight of the SDOF is assumed to be negligible with respect to the weight of the structure, i.e. the dynamic interaction between the primary (MDOF) system and the secondary (SDOF) system is neglected and the secondary system is considered as representing a light Non-Structural Component located in the structure.

By means of the Complete Quadratic Combination (CQC) rule (Wilson et al., 1981) the max acceleration of the secondary system (i.e. NSC) can be written (Igusa and Der Kiureghian, 1985) as (1):

$$\max(\ddot{u}_k) = \left(\sum_{i=1}^{N} \sum_{j=1}^{N} A_{ik} \cdot \rho_{ij} \cdot A_{jk} \cdot S_{pa}(f_i, \xi_i^*) \cdot S_{pa}(f_j, \xi_j^*) \right)^{1/2} \tag{1}$$

Where, ρ_{ij} represents the cross-correlation between modes *i* and *j* and where, being φ_{ik} the modal displacement of the structural node *k* and being Γ_i the modal participation factors, the coefficients A_{ik} stands (2):

$$A_{ik} = \alpha_i \cdot \varphi_{ik} \cdot \Gamma_i \tag{2}$$

The α_i are amplification factors accounting for the dynamic interaction of the equipment system with the supporting structure. These are expressed (3) as function of the equipment's fundamental frequency f_e and the structure's natural frequencies f_i.

$$\alpha_i = \frac{f_e^2}{f_e^2 - f_i^2} \tag{3}$$

S_{pa} is the spectral pseudo-acceleration and the damping ratio ξ^* (4) is equal to the average of the modal ξ_i and the equipment ξ_e damping ratios.

$$\xi_i^* = \frac{\xi_e + \xi_i}{2} \tag{4}$$

The equation (1), other than to show excellent predictive capability of the Floor Response Spectra (EPRI, 1989), gives useful insight on the factors affecting equipment response and how the different structural modes contribute to this last. From (2), the contribution of each structural mode to the equipment's response is equal to the contribution of that mode to the acceleration of the attachment node multiplied by an amplification factor that depends on the natural frequencies of the mode and the equipment. From (3), if one of the modal frequencies f_i is very close to the equipment frequency then α_i is very large, this indicates that the equipment amplifies the motion of mode i. If f_i is much higher than the equipment frequency, α_i is smaller than the unity in absolute value, which indicates that the equipment de-amplifies the motion of mode i. If f_i is much lower than the equipment frequency, α_i value is approximately one, which indicates that the equipment responds statically to the motion of mode i.

Thus, the presence of the factors α_i in equation (2) highlights the importance of the superior order vibration modes and the impossibility to neglect them (at least "a priori") in the evaluation of NSCs acceleration demand.

Nevertheless, it must be considered that the participation factors Γ_i are usually negligible for higher (horizontal) vibration modes. Then in order to "weight" the combined effect of the α_i and Γ_i, a factor λ_i can be defined as (5):

$$\lambda_i = \alpha_i \cdot \Gamma_i \tag{5}$$

The practical role of the factor λ_i is to discriminate, with respect to frequency, the ground motion spectral acceleration ordinates having (or not) influence on the Floor Response Spectra ordinates. In other words, when λ_i are negligible, their associated ground-motion response spectra ordinates (i.e. $S_{pa}(f_i, \xi)$) can be neglected in the computation of equation (1), vice versa in case of large λ_i values.

2.2 A new IM for NSCs acceleration demand

An attempt to identify an efficient IM for NSCs acceleration demand is done here. Based on the foregoing, it can appear intuitive to consider (as "significant") the ground motion spectral ordinates associates to the modal frequency giving the highest λ_i values. Following this thought, a new IM could be defined (6) under the simple form of a normalized summation of the n values of $S_{pa}(f_n, \xi)$ corresponding to the n highest λ_i values:

$$\lambda IM = \frac{1}{n}\sum_{i=1}^{n} S_{pa}(f_i, \xi) \tag{6}$$

Nevertheless an IM stated as in equation (6) requires for it definition the computation of the λ_i values. These last necessitate, in order to be computed, the knowledge of n-natural frequencies and participation factors of the supporting structure other than the fundamental frequency(s) of the secondary system(s). Thus, even if hypothetically high efficient, the "λIM" could not be practically implemented in PSHA and then employed in Probabilistic Seismic Risk Analysis.

For such a reason, here the intention is to define an IM that is independent from the dynamic characteristics of the NSCs and in which the only required structural characteristic is the fundamental frequency of the supporting structure.

Having in mind the aforementioned aim, the authors highlight three points: a) generally, the evolution with the frequency of the (horizontal) participation factors Γ_i is such that higher values of Γ_i appear for lower natural frequencies; b) the dynamic amplification factors α_i assume higher values in correspondence of the NSCs frequencies; c) usually the frequency content of earthquake ground motions is such that higher modes are less likely to be excited with respect to the lower ones. These facts suggest that for every structure it is possible to "roughly" identify a, here-called, "*dominant-frequencies interval*" such that internally to this, the factors λ_i (5) assume the highest (i.e. dominant) values.

Thus, the key-idea is to consider such a structure-relative's dominant-frequencies interval as the "core" of the IM. The dominant-frequencies interval can be "approximately" defined knowing only the fundamental frequency value: the lower bound corresponds to the fundamental frequency of the structure and the upper bound can be evaluated as percentage of the fundamental frequency value.

In practice, this can be done considering the recently introduced ASA_R (De Biasio et al., 2014), and modifying it in order to consider the structure-relative's dominant-frequencies interval. The ASA_R (7) has been conceived as IM aiming to predict structural demand, and it has revealed particularly efficient in case of non-linear structural behavior.

$$ASA_R(f_1) = \frac{1}{f_1(1-X_f)} \int_{X_f \cdot f_1}^{f_1} S_{pa}(f,\xi)\,df \qquad with\, X_f < 1 \tag{7}$$

In equation (7), f_1 is the fundamental frequency of the structure, X_f *<1* is a factor accounting for the drop of the fundamental frequency, S_{pa} is the spectral pseudo-acceleration and ξ is the structural damping value. The suffix R indicates the chosen percentage of drop of the fundamental frequency f_1 ($X_f = 1-(R/100)$). The recommended value of R, issued from numerical sensibility analyses and post-earthquakes observations, is 40% (i.e. ASA_{40}).

In order to consider the dominant-frequencies interval instead that the frequency drop one (Figure 1), the formulation (7) is kept identical but a modification is done by taking X_f>1. Now X_f represents a factor accounting for the width of the dominant-frequencies interval and the suffix R indicates the width of the dominant-frequencies interval as percentage of the fundamental frequency f_1 ($X_f = 1+(R/100)$).

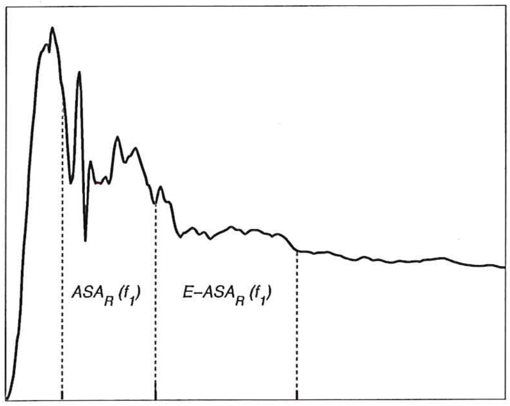


Figure 1. ASA_R and $E\text{-}ASA_R$ (f_1 is the structure's fundamental frequency)

Therefore, the herein proposed IM, named (Equipment) Relative Average-Spectral-Acceleration *($E\text{-}ASA_R$)*, is defined (8) as the average spectral pseudo-acceleration over the dominant-frequencies interval of the structure:

$$E-ASA_R(f_1)=\frac{1}{f_1(1-X_f)}\int_{X_f\cdot f_1}^{f_1} S_{pa}(f,\xi)df \qquad with\, X_f>1 \tag{8}$$

The formulation (8) of the $E\text{-}ASA_R$ captures the presence of significant spectral acceleration ordinates over the structure 's dominant-frequencies interval. According to (1) this last is a key feature that a seismic signal must have in order to produce high FRS ordinates.

In (8), the value of X_f (i.e. the value of R) depends on the dynamic characteristics of the structure that, in turn, depend on its design properties. A general, optimum, R-value (i.e. $E\text{-}ASA_{67}$), issued from numerical sensibility analyses, is suggested in the final part of the report.

Finally it is worth to note that the formulation (8) of the $E\text{-}ASA_R$, based on spectral pseudo-acceleration values, allows (Bazzurro and Cornell, 2002; Stewart et al., 2002; Baker and Cornell, 2006a; Inoue and Cornell, 1990) performing PSHA with respect to $E\text{-}ASA_R$ by means of widespread ground motion prediction models available for $S_{pa}(f_1)$.

3. Comparative Analyses

3.1 Test case structures and numerical models

In order to compare the performance of the IMs the numerical analyses are performed on three structures. Among these, two have been experimentally tested: this offers the advantage to dispose of a validation tool for the numerical models giving, under the condition of sufficiently precise agreement of the numerical simulations vs. experimental tests, value/authority to the results extracted from the numerical models. The chosen structures have different design characteristics (and then dynamic properties): two are stiff, high frequency, load-bearing walls structures, the SMART (CEA, 2013) mock-up and the, here called, TC3 (Test-Case n°3) building; the other one is a ductile, low frequency, frame structure the EC8-FRAME (JRC, 1994).

The SMART (Figure 2a) is a ¼ scaled model, designed accordance with the current French nuclear regulation, tested in 2013 on the shaking table of the French Atomic Agency (CEA) and also object of an international blind contest. The mock-up (Figure 1a) is a trapezoidal, three-story reinforced concrete structure representative of a typical simplified half part of an electrical nuclear building. The mock-up is designed and tested following precise similitude criteria that allow doing its behavior representative of a full-scale structure.

The EC8-FRAME (Figure 2b) is a full-scale, four-story, high-ductility R/C frame designed in accordance to European seismic code EC8 (EC8, 1988) and tested in 1994 on the reaction-wall of the European Joint Research Center (JRC). The structure (Figure 1b) is symmetric in one direction, with two equal spans of 5 m, whilst in the other direction is slightly irregular due to the different span lengths (6 and 4 m).

The TC3 (Figure 2c) is an existing thirteen-story, European, industrial building characterized by irregular plan/slabs distributions.

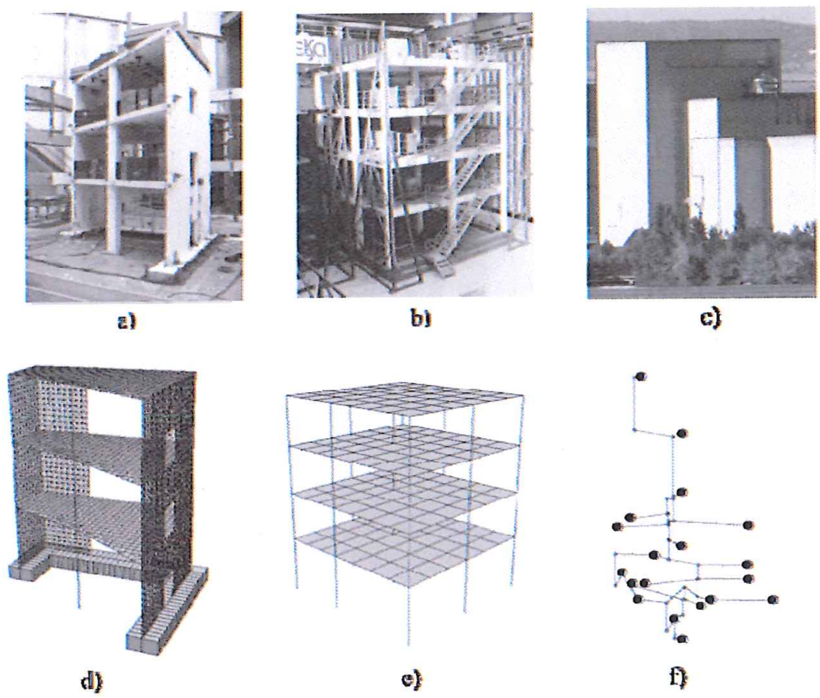


Figure 2. Test case structures and numerical models: (a, d) SMART; (b, e) EC8-FRAME; (c, f) TC3.

In this comparative study, linear-elastic modeling of the test-case structures has been adopted. This choice is justified by the fact that, as shown by Rodriguez et al. (2002), the maximum FRS acceleration, which is the selected demand parameter, occurs when the building is behaving elastically. Moreover, being the interest of the study pointed on the response of Non-Structural Components to low-to-moderate-earthquakes, it is assumed that such earthquakes are not able to significantly damage a well-engineered reinforced concrete structure.

Regarding the numerical FE discretization, a lattice modeling technique, derived from the approach of Kotronis et al. (2003) has been used to model the structural walls of the SMART building. For both SMART and the EC8-FRAME buildings the columns and beams have been modeled by means of Timoshenko beam elements, the slabs have been represented by means of shell elements and distributed masses have been adopted. Differently, the TC3 building has been modeled as a stick model able to take into account the torsional characteristics of the building. In this model the slabs are supposed rigid and the walls give in-plane and out-of-plane stiffness contribution to the equivalent beams, which axes pass through the storeys's gravity centers. The masses of the slabs are then represented by lumped masses (highlighted in Figure 2f) located at the slabs' gravity centers. The dynamic characteristics of the three numerical models (Figure 2d-2e-2f) are summarized in Tables 2-4 and, where the values of λ_i (eq. 5) are also given.

Table 2. EC8-FRAME: dynamic characteristics *(the modes higher than the 10th are not reported because they all give values of λ_i approaching zero).*

Mode	Frequency [Hz]	Γ_x	Γ_y	α_i				$\lambda_i = \alpha_i * (\Gamma_x + \Gamma_y)$			
				1hz	5hz	10hz	20hz	1hz	5hz	10hz	20hz
1	1.57	1.21	0.00	0.68	1.11	1.03	1.01	**0.82**	**1.34**	**1.24**	**1.21**
2	1.59	0.00	1.31	0.65	1.11	1.03	1.01	**0.85**	**1.45**	**1.34**	**1.31**
3	2.07	0.00	0.12	0.30	1.21	1.04	1.01	0.04	0.14	0.12	0.12
4	5.47	0.28	0.00	0.03	5.08	1.43	1.08	0.01	**1.44**	**0.40**	**0.31**
5	5.53	0.00	0.30	0.03	4.48	1.44	1.08	0.01	**1.35**	**0.43**	**0.33**
6	7.16	0.00	0.03	0.02	0.95	2.05	1.15	0.00	0.03	0.06	0.03
7	10.75	0.11	0.00	0.01	0.28	6.43	1.41	0.00	0.03	**0.70**	0.15
8	10.87	0.00	0.12	0.01	0.27	5.51	1.42	0.00	0.03	**0.64**	0.16
9	12.68	0.03	0.00	0.01	0.18	1.65	1.67	0.00	0.01	0.05	0.05
10	13.81	0.00	0.01	0.01	0.15	1.10	1.91	0.00	0.00	0.01	0.02

Table 3. SMART (full-scale): dynamic characteristics.

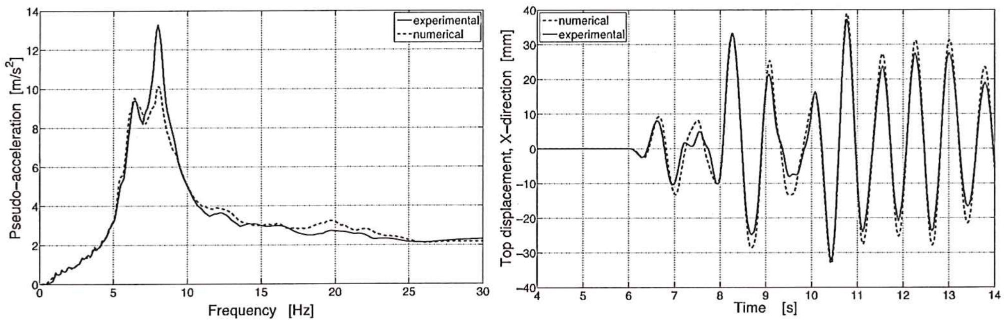
Mode	Frequency [Hz]	Γ_x	Γ_y	α_i				$\lambda_i = \alpha_i * (\Gamma_x + \Gamma_y)$			
				1hz	5hz	10hz	20hz	1hz	5hz	10hz	20hz
1	**5.35**	1.69	0.62	0.04	6.90	1.40	1.08	**0.08**	**15.97**	**3.24**	**2.49**
2	**9.54**	0.81	1.40	0.01	0.38	11.13	1.29	0.02	0.84	**24.60**	**2.86**
3	**16.76**	0.57	0.76	0.00	0.10	0.55	3.36	0.00	0.13	0.73	**4.46**
4	**19.74**	0.68	0.22	0.00	0.07	0.35	38.71	0.00	0.06	0.31	**34.73**
5	22.87	0.03	0.12	0.00	0.05	0.24	3.25	0.00	0.01	0.04	0.49
6	28.02	0.37	0.14	0.00	0.03	0.15	1.04	0.00	0.02	0.08	0.54
7	28.55	0.19	0.08	0.00	0.03	0.14	0.96	0.00	0.01	0.04	0.26
8	28.93	0.30	0.72	0.00	0.03	0.14	0.92	0.00	0.03	0.14	0.93
9	29.66	0.49	0.77	0.00	0.03	0.13	0.83	0.00	0.04	0.16	1.05
10	30.59	0.31	0.25	0.00	0.03	0.12	0.75	0.00	0.02	0.07	0.42

Table 4. TC3: dynamic characteristics.

Mode	*Frequency [Hz]*	Γ_x	Γ_y	α_i				$\lambda_i = \alpha_i * (\Gamma_x + \Gamma_y)$			
				1hz	*5hz*	*10hz*	*20hz*	*1hz*	*5hz*	*10hz*	*20hz*
1	6.42	1.80	0.11	0.02	1.54	1.70	1.11	0.05	2.94	3.25	2.13
2	8.03	0.12	1.90	0.02	0.63	2.82	1.19	0.03	1.28	5.69	2.41
3	10.78	0.08	0.11	0.01	0.27	6.17	1.41	0.00	0.05	1.17	0.27
4	15.87	1.30	0.09	0.00	0.11	0.66	2.70	0.01	0.15	0.91	3.74
5	17.56	0.08	0.89	0.00	0.09	0.48	4.36	0.00	0.09	0.47	4.24
6	18.09	0.06	0.66	0.00	0.08	0.44	5.50	0.00	0.06	0.32	3.94
7	24.96	0.05	0.24	0.00	0.04	0.19	1.79	0.00	0.01	0.06	0.52
8	26.93	0.55	0.04	0.00	0.04	0.16	1.23	0.00	0.02	0.09	0.73
9	32.53	0.19	0.48	0.00	0.02	0.10	0.61	0.00	0.02	0.07	0.41
10	33.79	0.63	0.30	0.00	0.02	0.10	0.54	0.00	0.02	0.09	0.50

The predictive capabilities of the SMART and EC8-FRAME numerical models have been checked with the results coming from the respective experimental campaigns of the two structures. The SMART numerical model has been checked with the results of shaking table tests, in terms of floor response spectra (Figure 3a). In the case of the EC8-FRAME, the predictive capabilities of the numerical model have been checked with the results coming from the (reaction-wall) pseudo-dynamic tests (Takanashy, 1975) in terms of roof displacement time history (Figure 3b). The comparison of numerical and experimental results has shown the ability of the aforementioned numerical models to predict with good agreement the structures' linear behavior qualitatively and quantitatively.

Let's note that the SMART numerical model has been validated with respect to the ¼ scale mock-up. Nevertheless, once validated, such a model can be rescaled to a full-scale: this step allows performing comparative analyses without needing to scale the entire ground motion dataset.



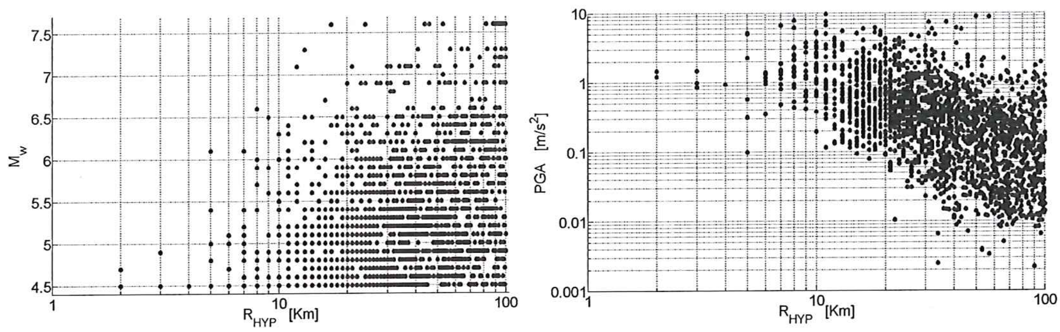
a) b)

Figure 3. Numerical model validation: (a) SMART, floor response spectrum at the roof; (b) EC8-FRAME, roof displacement time history

3.2 Ground motion dataset

The ground motion dataset employed as input for the FE simulations has been extracted from the 2013 version of the RESORCE database (Akkar et al., 2013) which compiles ground motions recorded in Europe and nearby countries during the past decades and related to events with a moment magnitude (M_w) lying between 2 and 8.

The records with M_w smaller than 4.5 and hypocentral distance (R_{hyp}) larger than 100 km have been excluded to focus on earthquake excitations that are of engineering significance. Selecting records with respect to soil type has not been considered an essential step (Singh, 1985; Boore, 2004; Bommer and Acevedo, 2004). Accordingly (Luco and Cornell, 2007), the conditional independence (i.e. sufficiency) of IMs with respect to the V_{S30} (i.e. shear wave velocity in the upper 30 m) of records will be discussed in the final part of this report.



a) b)

Figure 4. Ground motions used in the study: (a) moment Magnitude vs. Hypocentral distance; (b) PGA (geometrical mean of the horizontal components) vs. Hypocentral distance.

The records (Figure 4) have then been clustered for specific moment magnitude and hypocentral distance intervals. In theory, magnitude-dependent clustering implies a more realistic consideration of frequency content and strong-motion duration of ground motions (Bommer and Acevedo 2004, Stewart et al. 2002). Two magnitude groups are described to account for the above facts: moderate magnitude (MM, $4.5 \leq M_w \leq 6.0$), and large magnitude (LM, $6.0 < M_w \leq 7.6$). Records in the dataset are then classified in two R_{hyp} bins: short distance (SR, $0\text{km} < R_{hyp} \leq 20\text{km}$) and large distance (LR, $20\text{km} < R_{hyp} \leq 100\text{km}$). No screening has been done in order to isolate "pulse-like" records. Finally, the 2,045 records composing the dataset are divided into four bins of different magnitude and source-to-site distance interval (Table 5).

Table 5. Number of records within each M_w and R_{Hyp} interval pair.

	SR $0\ \text{km} < R_{hyp} \leq 20\ \text{km}$	LR $20\ \text{km} < R_{hyp} \leq 100\ \text{km}$
SM $4.5 \leq M_w \leq 6.0$	403	1,286
LM $6.0 < M_w \leq 7.6$	34	322

The IMs have been computed as the geometrical mean of the two horizontal components' IMs values (Baker and Cornell, 2006b).

3.3 Load and Demand Parameters

Regarding the application of the load, for all the test-case buildings three-dimensional load (two horizontal components and the vertical one) has been applied to the three-dimensional models clamped at the bases.

The (horizontal) NSCs acceleration demand has been measured in the numerical models as the maximum of the acceleration Floor Response Spectra (Villaverde, 2004) over all the floors and over four frequency ranges: 0.8 to 1.2 Hz, 4.5 to 5.5 Hz, 8 to 10 Hz and 18 to 22 Hz. These frequency ranges reflect the hypothetical presence of NSCs with fundamental frequency standing respectively 1 Hz, 5 Hz, 10 Hz, and 20 Hz, and with an interval of confidence of ± 10 % on such values.

The usage of the FRS method (i.e. to uncouple the response of the supporting structure from that of the non-structural component) is acceptable for NSCs whose mass is less than 1% of that of the structure supporting them (Singh and Ang, 1974; USNRC, 1978; Taghavi and Miranda, 2008).

4. Results

4.1 Analysis method

In order to evaluate the IMs' efficiency, the relationship between EDPs and IMs (which values have been normalized with respect to their maxima) has been written by means of logarithmic transformation as (9) where a_1 and a_2 are constant coefficients and e_i is a random variable representing the randomness in the relationship (Cornell et al., 2002).

$$\ln(EDP_i) = a_1 + a_2 \ln(IM_i) + e_i \tag{9}$$

Thus, Linear Least-Square regression (LLS) has been used to estimate the regression coefficients (a_1 and a_2) in eq. (9). In this last, the term e_i (called "residual") represents the error between the computed and the estimate value of EDP_i. The validity of the LLS method requires the satisfaction of the condition of normal distribution with constant variance of the residual e_i. Such condition for the data in the study has been examined by means of residual vs. fit plots and quantile-normal plots of the residuals, and found to be sound. Consequently, the efficiency of the IMs has been evaluated computing the standard deviation of the residual (e_i) between the computed and the estimated value of EDP_i (Baker and Cornell, 2004): lower the standard deviation, higher the efficiency of the IM.

On the other hand, the sufficiency of the IMs can be evaluated (Cornell, 2004) by verifying whether the residuals obtained from the regression carried out from the aforementioned statistical procedure show any dependency on other ground-motion parameters (i.e. magnitude, source-to-site distance and V_{S30}). If no dependency is found then it is possible to assume that they do not affect structural response for a given IM value. In other words it means that with respect to the EDP considered the selected IM provides a sufficient description of the features of the ground-motion affecting structural response.

Consequently to appraise the IMs sufficiency, the rank correlation coefficient after Spearman (1925) has been calculated between the residuals of the regression EDPs-IMs and the M_w, the R_{hyp} and the V_{S30}. The Spearman rank correlation coefficient between two variables X and Y, is given by the relation (10):

$$\rho_{Spearman} = 1 - \frac{6\sum_{i=1}^{N} D^2}{N(N^2-1)} \tag{10}$$

where D denotes the differences between the ranks of corresponding values of X_i and Y_i and N is the number of pairs of values (X,Y) in the data. Such a coefficient measures how well the data agree with monotonic (linear or not) ranking. In case of perfect positive correlation the coefficient assumes the value "1"; if perfect negative correlation it assumes the value "-1"; when the correlation is not perfect it lies in the interval [-1, 1]; hence, the further the absolute value of the correlation coefficient from unity, the more sufficient the IM.

4.2 IMs efficiency comparison

The results of the comparative statistical analysis about the IMs' efficiency are presented (Table 6) with respect to the 2,045 records composing the four ground-motions bins. These results do not show significant discrepancies with the results obtained with respect to the four ground-motions bins taken one-by-one (Appendix, Tables 9-12).

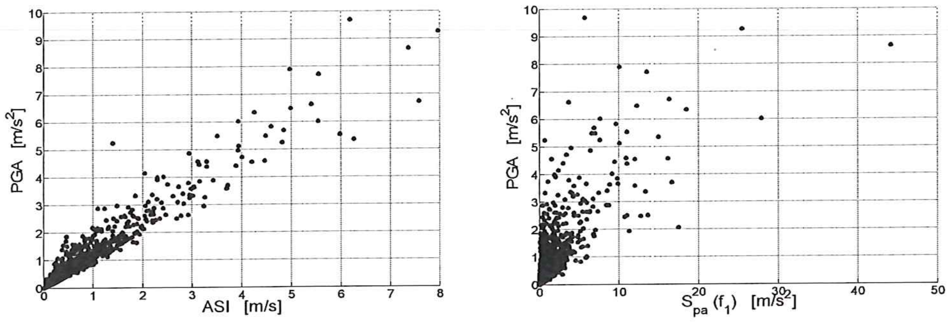
For the comparison with the other IMs, the *E-ASA$_R$* has been computed with an *R*-value of 67% (*E-ASA$_{67}$*). The choice of an "optimum *R*-value" is addressed in the final part of the dissertation. The *λIM* has been computed (distinctly for each EDP vs. test-case couple) considering the structural vibration modes highlighted in bold in the "λ_i columns" of Tables 2-4.

Table 6. Efficiency analysis on all the 2,045 ground motions: standard deviation of the residuals.

Max FRS at:	IMs												
	Frequency Response Based								*Peak Based*		*Duration Based*		
	λIM	*E-ASA$_{67}$*	*ASA$_{40}$*	*S_{pa}*	*S**	*I_{NP}*	*ASI*	*EPA*	*PGA*	*PGV*	*I_A*	*CAV*	*SCAV*
EC8 FRAME													
1Hz±10%	**0.27**	**0.49**	0.25	**0.25**	0.33	0.24	0.87	**0.51**	**1.01**	0.59	0.74	0.66	1.24
5Hz±10%	**0.25**	**0.59**	0.77	**0.73**	0.79	0.71	0.27	**0.53**	**0.35**	0.56	0.42	0.67	0.77
10Hz±10%	**0.19**	**0.58**	0.74	**0.69**	0.76	0.67	0.3	**0.49**	**0.22**	0.5	0.34	0.65	0.76
20Hz±10%	**0.29**	**0.51**	0.67	**0.62**	0.69	0.6	0.31	**0.43**	**0.28**	0.44	0.3	0.59	0.77
SMART													
1Hz±10%	**0.29**	**0.37**	0.34	**0.26**	0.36	0.25	0.35	**0.61**	**0.32**	0.62	0.46	0.75	0.81
5Hz±10%	**0.42**	**0.46**	0.53	**0.42**	0.59	0.45	0.61	**0.87**	**0.61**	0.89	0.75	0.97	0.94
10Hz±10%	**0.56**	**0.51**	0.81	**0.69**	0.88	0.73	0.85	**1.14**	**0.75**	1.14	0.99	1.22	1.06
20Hz±10%	**0.46**	**0.42**	0.72	**0.58**	0.79	0.62	0.77	**1.07**	**0.65**	1.07	0.91	1.16	1.00
TC3													
1Hz±10%	**0.22**	**0.28**	0.44	**0.35**	0.46	0.35	0.41	**0.67**	**0.3**	0.67	0.51	0.8	0.77
5Hz±10%	**0.35**	**0.35**	0.6	**0.46**	0.67	0.5	0.66	**0.97**	**0.6**	0.98	0.82	1.06	0.94
10Hz±10%	**0.48**	**0.39**	0.79	**0.67**	0.85	0.71	0.83	**1.12**	**0.73**	1.12	0.97	1.20	1.04
20Hz±10%	**0.55**	**0.49**	0.82	**0.72**	0.88	0.75	0.85	**1.10**	**0.73**	1.10	0.96	1.19	0.97

Considering Table 6:

In agreement with the literature, the present study finds a noticeable efficiency of the *PGA* in the case of the tested frame structure (Figure 6a). It is opinion of the authors that such efficiency originates from the observed (Figure 5a) strong correlation between the *PGA* and the spectral acceleration ordinates in the range 2-10 Hz. Indeed such frequency interval roughly coincides with the dominant-frequency interval of the EC8-FRAME (Table 2). Evidently the *PGA* is not well correlated (Figure 5b) with the $S_{pa}(f_1)$ which owns the highest efficiency for NSCs 'frequencies lower than the structure fundamental frequency. The same argumentation (i.e. Figure 5a) can be used to explain the efficiency of the *PGA* with respect to low-frequency FRS ordinates in the case of the SMART and TC3 test cases.



a) b)

Figure 5. Scatter plots, all 2,045 records: (a) *PGA* vs. *ASI* (i.e. spectral acceleration between 2 and 10 Hz); (b) *PGA* vs. $S_{pa}(f_1)$ (computed at the fundamental frequency of the EC8-FRAME).

The *PGV*'s efficiency is sensibly lower than the *PGA*'s. Among the duration based IMs, the I_A shows the better performance. Nevertheless, the efficiency of this class of IMs, which has been found slightly higher in the case of the EC8-FRAME structure, is generally lower than the one of the frequency-response and amplitude based ones.

The IMs based on the spectral acceleration at the fundamental frequency of the supporting structure $S_{pa}(f_1)$ exhibit high efficiency in predicting FRS ordinates at frequency lower than the fundamental one. Indeed in such case, based on equation (1), the equipment de-amplifies all the motions of the structure being the first-mode motion the less de-amplified one and therefore the one leading the NSCs response (i.e. higher λ_i values).

For NSCs frequencies higher than the structure fundamental frequency, the efficiency of the S_{pa} (f_1) is higher than the *PGA*'s for the structural-wall test case buildings, but is lower to this last for the EC8-FRAME (Figure 6b-6e-6h). Still, this last result can be explained considering the non-negligible values of λ_i for the modes 4-5 and 7-8 in the case of EC8-FRAME (Table 2). The contribution of these modes is not captured by the f_1-based IMs, as instead is done by the *ASI*, *EPA* and *PGA* that, thus, have high efficiency in the EC8-FRAME case.

In the case of S^* and I_{NP}, to couple to the S_{pa} (T_1) a factor accounting for the spectral shape in the period lengthening zone reveals to slightly decrease the performances of S_{pa} (T_1) considered alone. In the same trend, the ASA_{40_L} which considers spectral acceleration ordinates along the structure frequency drop interval, shows minor performance with respect to the S_{pa} (f_1). Nevertheless for both the structural-wall buildings tested, the ASA_{40} and the I_{NP} have efficiency comparable to the *PGA*'s.

The observation of the correlation coefficients of the *EPA* and *ASI* reveals the effect of the width of the dominant-frequencies interval: the *ASI* is computed on the range 2 to 10 Hz and the *EPA* is computed on the range 0.4 to 10 Hz. From Table 6, in the case of the EC8-FRAME the *ASI* has higher correlation with respect to the *EPA* being its interval of integration smaller and more centered around the dominant-frequencies interval of the structure. The same argumentation can be used with respect to the results relative to the SMART and the TC3 test cases.

As hypothesized, the *λIM* owns high efficiency in all the test cases and with respect to all the selected FRS frequency values (i.e. selected EDPs).

Finally, the herein introduced $E\text{-}ASA_{67}$ with respect to the *PGA* is slightly less efficient in the case of the frame structure. This is essentially due to the low value of *R (i.e. 67%)*, not able to cover the frequencies related to the modes 4-5 and 7-8 (Table 2), which are associated to high λ_i values. Nevertheless the $E\text{-}ASA_{67}$ is very efficient in the cases of structural wall buildings where its efficiency is up to 52% higher than the one of *PGA* (Figure 5f-5i).

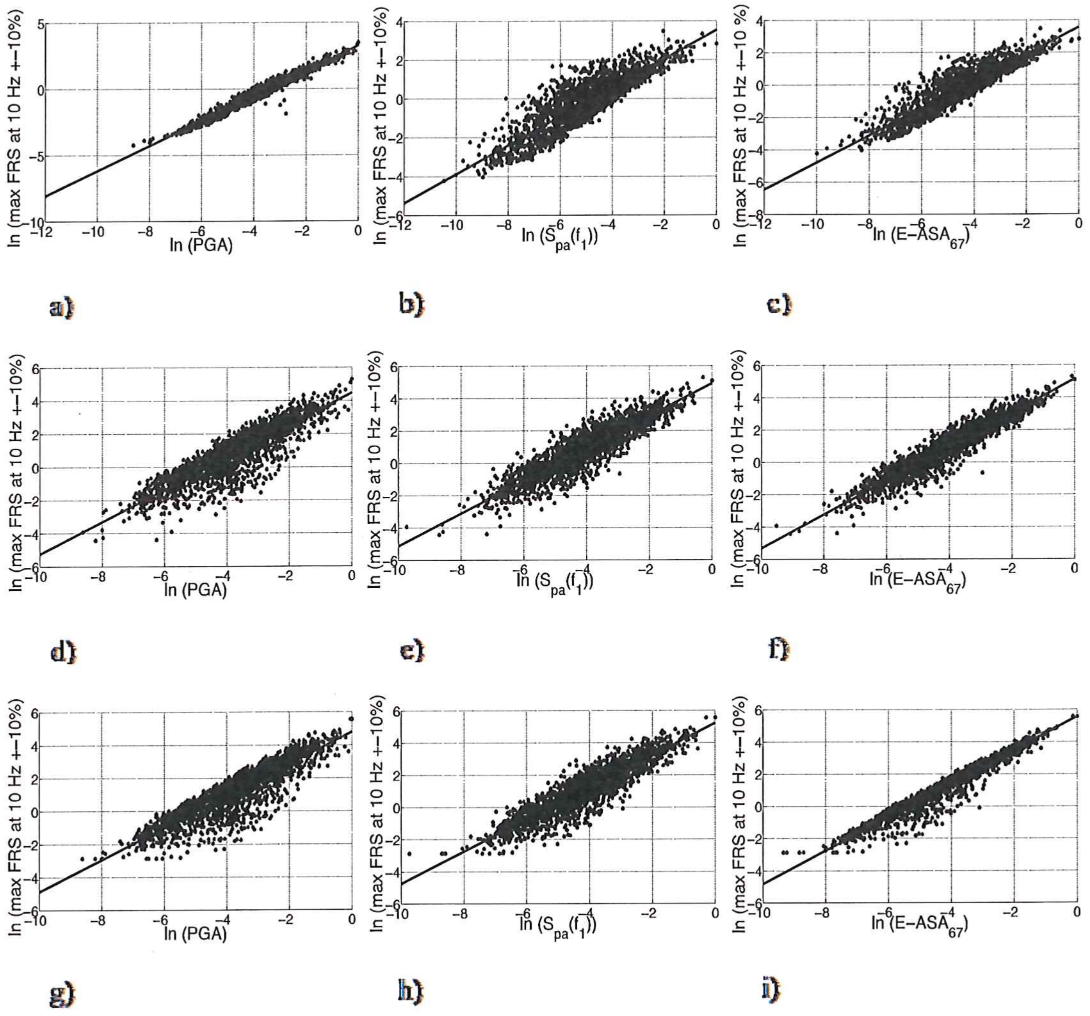


Figure 6. Regression plots-all 2,045 records, IMs vs. Max FRS in the range 10Hz±10% (a) EC8-FRAME, *PGA* b) EC8-FRAME, $S_{pa}(f_1)$; (c) EC8-FRAME, $E\text{-}ASA_{67}$; (d) SMART, *PGA*; (e) SMART, $S_{pa}(f_1)$; (f) SMART, $E\text{-}ASA_{67}$; (g) TC3, *PGA*; (h) TC3, $S_{pa}(f_1)$; (i) TC3, $E\text{-}ASA_{67}$.

4.3 IMs sufficiency evaluation

The results of the comparative statistical analysis concerning the IMs' sufficiency are presented (Table 7) for the 2,045 records composing the four ground-motions bins, with respect to the max of the FRS at 20 Hz ± 10% (analogous results have been found for the other considered EDPs).

Excluding a light correlation in the case of the EC8-FRAME with respect to the R_{hyp} (Figure 7b), the analysis of the results in Table 7 evidences no significant correlation between the proposed IM and the M_w or the R_{hyp}. Moreover, excluding the case of the EC8-FRAME, the $E\text{-}ASA_{67}$ shows always lower correlation than the $S_{pa}(f_1)$ and the *PGA*. The lack of correlation indicates the sufficiency of the ASA_R (Figure 7) with respect to the magnitude and the source-to-site distance.

A pronounced in-sufficiency of the *CAV* with respect to the R_{hyp} appears in all the test cases. Such an insufficiency of the *CAV* has also been noted by De Biasio et al. (2014) in considering structural demand parameters.

Lastly, it is important to notice that none of the considered IMs shows an appreciable degree of in-sufficiency with respect to the soil-type, i.e. V_{S30} (Figure 7c-7f-7i).

Table 7. IMs Sufficiency-all 2,045 records: Spearman rank correlation coefficient of residual 20 Hz

	IMs												
	Frequency Response Based								*Amplitude Based*		*Duration Based*		
	λIM	*E-ASA₆₇*	ASA_{40}	S_{pa}	S^*	I_{NP}	*ASI*	*EPA*	*PGA*	*PGV*	I_A	*CAV*	*SCAV*
EC8 FRAME													
M_w	0.08	-0.13	-0.18	-0.15	-0.2	-0.18	0.12	-0.31	0.36	-0.26	-0.29	-0.26	0.29
R_{hyp}	-0.07	-0.5	-0.6	-0.58	-0.61	-0.58	-0.06	-0.52	0.19	-0.5	-0.44	-0.66	-0.19
V_{S30}	0.01	0.22	0.27	0.26	0.24	0.26	0.01	0.22	-0.14	0.18	0.11	0.2	0.00
SMART													
M_w	-0.3	-0.28	-0.29	-0.28	-0.3	-0.3	-0.3	-0.26	-0.27	-0.25	-0.33	-0.23	-0.03
R_{hyp}	-0.21	-0.23	-0.41	-0.34	-0.43	-0.36	-0.41	-0.56	-0.31	-0.56	-0.55	-0.66	-0.44
V_{S30}	0.18	0.14	0.25	0.23	0.26	0.24	0.26	0.28	0.22	0.26	0.27	0.27	0.16
TC3													
M_w	-0.27	-0.26	-0.27	-0.26	-0.28	-0.27	-0.28	-0.24	-0.27	-0.24	-0.3	-0.21	-0.05
R_{hyp}	-0.28	-0.28	-0.44	-0.38	-0.46	-0.4	-0.45	-0.58	-0.37	-0.58	-0.56	-0.67	-0.5
V_{S30}	0.14	0.08	0.21	0.17	0.22	0.19	0.22	0.25	0.19	0.24	0.24	0.25	0.16

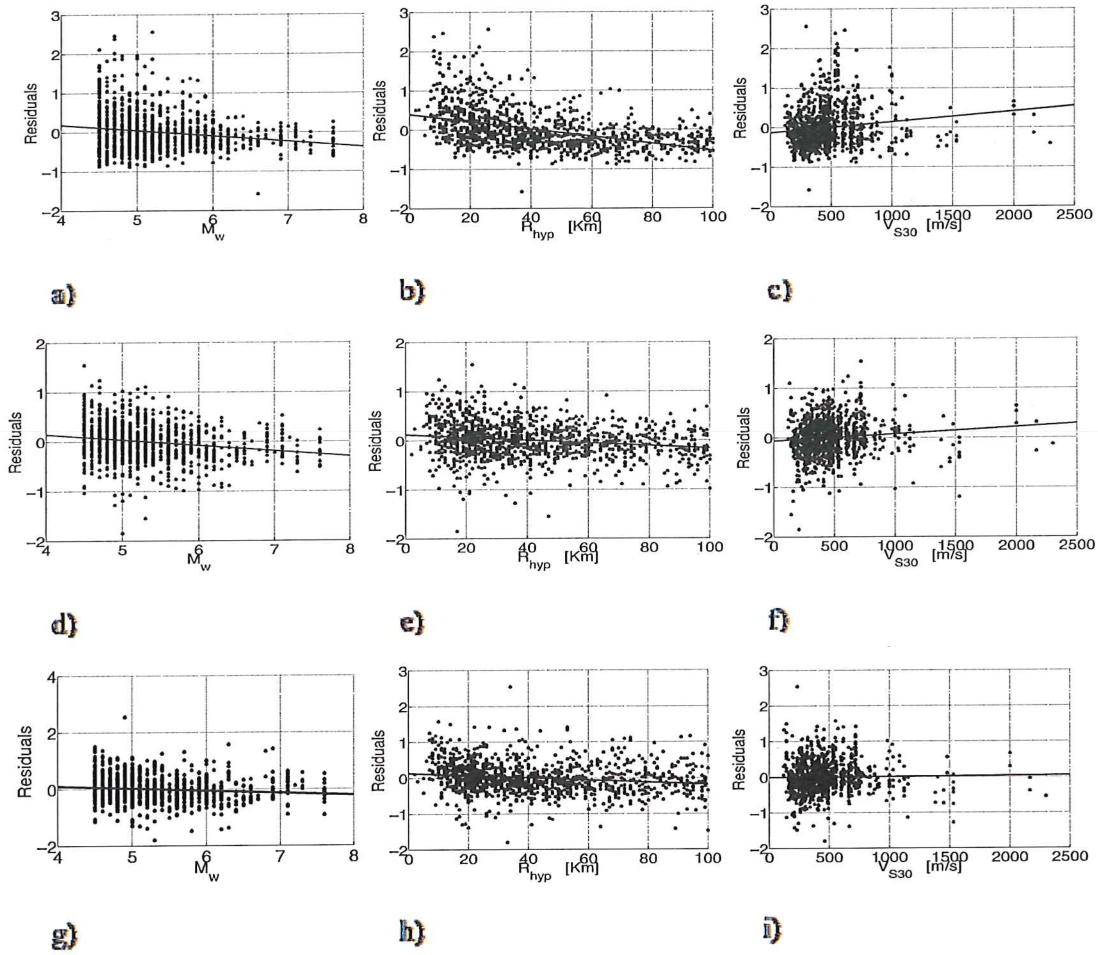


Figure 7. Sufficiency analysis, all 2,045 records. The sufficiency can be appreciated observing the slope of the regression line (lower the slope, higher the sufficiency): (a) EC8-FRAME, *E-ASA*$_{67}$ *vs.* M_w; (b) EC8-FRAME, *E-ASA*$_{67}$ *vs.* R_{hyp}; (c) EC8-FRAME, *E-ASA*$_{67}$ *vs.* V_{S30}; (d) SMART, *E-ASA*$_{67}$ *vs.* M_w; (e) SMART, *E-ASA*$_{67}$ *vs.* R_{hyp}; (f) SMART, *E-ASA*$_{67}$ *vs.* V_{S30}; (g) TC3, *E-ASA*$_{67}$ *vs.* M_w; (h) TC3, *E-ASA*$_{67}$ *vs.* R_{hyp}; (i) TC3, *E-ASA*$_{67}$ *vs.* V_{S30};

4.4 E-ASA$_R$ Optimum

In order to analyze the sensitivity to the choice of the width of the dominant-frequencies interval (i.e. R-value in equation (8)), the $E\text{-}ASA_R$ has been computed for several values of width, i.e. $E\text{-}ASA_{40}$, $E\text{-}ASA_{67}$ $E\text{-}ASA_{80}$, $E\text{-}ASA_{100}$, $E\text{-}ASA_{150}$ and $E\text{-}ASA_{200}$ (Table 8).

Table 8. $E\text{-}ASA_R$ sensibility study: efficiency analysis on all the 2,045 records (standard deviation of the residuals).

Test-Case	IM	Max FRS 1Hz±10%	Max FRS 5Hz±10%	Max FRS 10Hz±10%	Max FRS 20Hz±10%
EC8 FRAME	$E\text{-}ASA_{40}$	**0.41**	0.65	0.63	0.55
	$E\text{-}ASA_{67}$	0.49	0.59	0.58	0.51
	$E\text{-}ASA_{80}$	0.52	0.56	0.56	0.5
	$E\text{-}ASA_{100}$	0.57	0.53	0.54	0.47
	$E\text{-}ASA_{150}$	0.67	0.44	0.47	0.42
	$E\text{-}ASA_{200}$	0.77	**0.34**	**0.41**	**0.38**
SMART	$E\text{-}ASA_{40}$	**0.35**	**0.42**	0.56	0.46
	$E\text{-}ASA_{67}$	**0.37**	**0.46**	0.51	0.42
	$E\text{-}ASA_{80}$	0.38	0.47	0.49	0.4
	$E\text{-}ASA_{100}$	0.39	0.49	**0.46**	**0.38**
	$E\text{-}ASA_{150}$	0.41	0.51	**0.45**	**0.37**
	$E\text{-}ASA_{200}$	0.42	0.53	**0.47**	**0.37**
TC3	$E\text{-}ASA_{40}$	**0.26**	**0.32**	**0.4**	0.51
	$E\text{-}ASA_{67}$	**0.28**	**0.35**	**0.39**	**0.49**
	$E\text{-}ASA_{80}$	0.29	0.36	**0.39**	**0.48**
	$E\text{-}ASA_{100}$	0.3	0.38	0.4	**0.48**
	$E\text{-}ASA_{150}$	0.31	0.41	0.44	**0.47**
	$E\text{-}ASA_{200}$	0.31	0.43	0.47	**0.47**

It can be noticed (Tables 8) that for both the structural wall test cases the difference in terms of efficiency among the several values of R is negligible. Nevertheless in the case of the EC8-FRAME a sensible difference in the performance between the lowest and the highest values of R is observed. Such a difference is due to the inability of the lower R-values to cover the EC8-FRAME dominant-frequencies interval (Table 2). In such a case (i.e. low-frequency - frame buildings), a higher value of R, i.e. R =150-200%, is recommended.

Nevertheless, practical considerations lead to suggest the $E\text{-}ASA_{67}$ as general optimum IM for NSCs acceleration demand prediction. The mainly reason is due to the noticeable (Table 6) global performance (i.e. on the three structures and for the four FRS-EDPs) of the $E\text{-}ASA_{67}$. An additional reason comes from the direct relation (11) between $E\text{-}ASA_{67}$ (8) and ASA_{40} (7):

$$E-ASA_{67}(f_1)=ASA_{40}(1.67f_1) \qquad (11)$$

Indeed the ASA_{40} has shown to be an high efficiency IM with respect to structural damage prediction (De Biasio et al., 2014) and GMPEs in its terms are under developing, This leads to suggest that (through the relation (11)) PSHA could be advantageously performed by means of the same GMPE with the aim to specify both the structural and the non-structural seismic hazard.

5. Conclusions

In the present contribution the adaptation of an existing IM (the ASA_R (De Biasio et al., 2014) has been proposed in order to predict NSCs acceleration demand. The proposed IM, called Equipment Relative Average-Spectral-Acceleration (*E-ASA$_R$*), is based on the pseudo-spectral acceleration values along a definite, structure-specific, range of frequency. Such range corresponds to the here-called structure's "*dominant-frequencies interval*". This last has been defined as the range of frequency enclosing the first *n*-vibration modes such that the product between the participation factors of that modes and the dynamic amplification factors related to that modes assumes a dominant value. It has been shown that the dominant-frequencies interval can be effectively defined based on the fundamental frequency of the structure.

The efficiency of the *E-ASA$_R$* has been revealed by comparative statistical analysis of the results of linear dynamic simulations performed on three reinforced concrete structures over a database of 2,045 recorded seismic ground motions. The *E-ASA$_R$* exhibits high efficiency in predicting non-structural components' acceleration demand. It has been shown that such efficiency is particularly high in case of structural wall (i.e. high-frequency) buildings, and it is robust with respect to the fundamental frequencies of the non-structural components.

The sufficiency of the *E-ASA$_R$* with respect to moment magnitude, source-to-site distance and soil- type (V_{s30}) has also been shown by statistical analysis. Such sufficiency implies that if the *E-ASA$_R$* of interest is given (through Hazard Analysis), there is no need to be concerned about the M_w, R and V_{s30} of the records to be used in structural analyses provided that the selected records match the given *E-ASA$_R$* value. Although, the scaling robustness of the *E-ASA$_R$* has not been explicitly investigated in this paper.

The *E-ASA$_R$* can be computed, as the $S_{pa}(f_1)$, by only knowing the fundamental frequency of the structure. This represents a practical advantage, with respect to more complex structure-specific IMs: indeed, for an actual structure the fundamental frequency is usually known or easy to know by means of in situ tests or, for regular buildings, it can be roughly estimated by empirical code-based approach.

The simple formulation of the *E-ASA$_R$* based exclusively on spectral pseudo-acceleration values, allows performing PSHA by means of common ground motion prediction models currently available for $S_{pa}(T_1)$. Moreover, the straightforward link between the *E-ASA$_R$* and the ASA_{40}, which is a high-efficiency IM for structural demand prediction, enable to use the same GMPE form for both structural and non-structural demand oriented PSHA/SPRA studies.

Due to its robust efficiency, the usage of the *E-ASA$_R$* can be particularly advantageous when the earthquake engineer has to handle with more kinds of acceleration-sensitive non-structural components (i.e. characterized by different fundamental frequency values). In such case the use of the *E-ASA$_R$* turns in using a single high-efficiency IM for the whole panel of non-structural components.

Moreover, the ability of *E-ASA*$_R$ to predict the response of the acceleration sensitive NSCs with the smallest scatter using the smallest number of response analyses can be valuable in the formulation of fragility.

In conclusion, due to its proved efficiency, sufficiency, robustness and exploitable formulation, the *E-ASA*$_R$ can be considered as a worthy candidate to be used, in a near future, for NSCs seismic hazard definition in the framework of probabilistic seismic risk analysis.

REFERENCES

Akkar S., Sandıkkaya M.A., Şenyurt M., Azari Sisi A., Ay B.Ö., Traversa P., Douglas J., Cotton F., Luzi L., Hernandez B., Godey S. (2013). Reference database for seismic ground-motion in Europe (RESORCE), Bulletin of Earthquake Engineering, DOI: 10.1007/s10518-013-9506-8

Arias A., (1970). "A measure of earthquake intensity". *Seismic design for nuclear power plants*. Cambridge, MA: MIT Press, 1970:438–69.

ATC, (1978). "Tentative Provisions for the Development of Seismic Regulations for Buildings", ATC-3-06 Report, Applied Technology Council, Redwood City, CA.

Baker J., W., Cornell C., A., (2004). "Choice of a vector of ground motion intensity measures for seismic demand hazard analysis". *13th World Conference on Earthquake Engineering, Vancouver, Canada 2004.*

Baker J., W., Cornell C., A., (2006a). "Spectral shape, epsilon and record selection". *Earthquake Engineering and Structural Dynamics* . 2006; 35:1077–1095.

Baker J., W., Cornell C., A., (2006b). "Which spectral acceleration are you using?". *Earthquake Spectra*, Volume 22, No. 2, pages 293–312, May 2006; Earthquake Engineering Research Institute

Bazzurro P, Cornell CA. (2002). "Vector-valued probabilistic seismic hazard analysis. In: Proceedings of the 7th US national conference on earthquake engineering". Earthquake Engineering Research Institute, Boston, MA; 2002.

Bojorquez E., Iervolino I., (2011). "Spectral shape proxies and nonlinear structural response". *Soil Dynamics and Earthquake Engineering 31 (2011) 996–1008*

Bommer J. J. and A. B. Acevedo, (2004). "The Use of Real Earthquake Accelerograms as Input to Dynamic Analysis". *Journal of Earthquake Engineering, 8 (Special Issue 1), 43-9*

Boore D. M. (2004). "Can site response be predicted?". *Journal of Earthquake Engineering, 8 (Special Issue 1), 43-91.*

Buratti N., (2012) "A comparison of the performances of various ground–motion intensity measures". *15th World Conference on Earthquake Engineering*. Lisbon, Portugal

CEA, (2013)."Presentation of the SMART 2013 international benchmark", Report DEN/DANS/DM2S/SEMT/EMSI/ST/12-017/E, CEA, France

Conte, J., P., Pandit, H., Stewart, J., P., Wallace, J., W. (2003). "Ground motion intensity measures for performance based earthquake engineering". *Ninth International Conference* *on Applications of Statistics and Probability in Civil Engineering (ICASP9) July 6-9, 2003, San Francisco, USA.*

Cordova, P.P., Deierlein, G.G., Mehanny, S.S.F. and Cornell, C.A. (2001). "Development of a two-parameter seismic intensity measure and probabilistic assessment procedure". *2nd U.S.-JapanWorkshop on PBEE Methodology for Reinforced Concrete Building Structures.*

Cornell, C., A. (2004). "Hazard, ground motions and probabilistic assessment for PBSD". *Performances based seismic design concepts and implementation, International workshop. 28 June-1 July 2004, Bled, Slovenia.*

Cornell, C.A., Jalayer, F., Hamburger, R.O. and Foutch, D.A. (2002). Probabilistic basis for 2000 SAC federal emergency management agency steel moment frame guidelines. *Journal of Structural Engineering.* 128:4, 526 - 533.

De Biasio, M., Grange, S., Dufour, F., Allain, F., Petre-Lazar I. (2014). A simple and efficient intensity measure to account for nonlinear structural behavior. *Earthquake Spectra (Submitted).*

EC8, (1988). Eurocode No. 8 Structure in Seismic Regions -Design-), Part1, General and Building, May 1988 edition, Report EUR 12266 EN, Commission of the European Communities.

EPRI, (1988). "A criterion for determining exceedance of the operating basis earthquake". EPRI NP-5930. Electric Power Research Institute, USA.

EPRI, (1989). "Linear and nonlinear response of structures and equipment to California and Eastern United States earthquakes". EPRI NP-5566. Electric Power Research Institute, USA.

EPRI, (1991). "Standardization of the Cumulative Absolute Velocity". EPRI TR-100082. Electric Power Research Institute. USA.

IAEA, (2011). "Non-linear response to a type of seismic input motion". *IAEA-TECDOC-1655* International Atomic Energy Agency. Vienna, Austria.

Igusa T., Der Kiureghian A. (1985). "Dynamic characterization of two-degree-of freedom equipment structure systems". *Journal of Engineering Mechanics. ASCE, vol. 111, No. 1, January 1985.*

Inoue T, Cornell C. 1., (1990). Seismic hazard analysis of multi-degree-of-freedom structures. Reliability of marine structures, RMS-8. Stanford, CA; 1990.

JRC, (1994) - "Tests on a Four-Storey Full-Scale R/C Frame Designed According to Eurocodes 8 and 2: Preliminary Report, EUR 15879, 1994, Ispra, Italy.

Kennedy, R.P., et al., 1980. Probabilistic seismic safety of an existing nuclear power plant. Nuclear Engineering Design, 59, 315□338.

Kotronis P., Mazars J., Davenne L., (2003). "The equivalent reinforced concrete model for simulating the behaviour of walls under dynamic shear loading". *Engineering Fracture Mechanics.* Vol. 70, Issues 7-8, May 2003, Pages 1085-1097

Luco N., Cornell C. A. (2007) Structure-Specific Scalar Intensity Measures for Near-Source and Ordinary Earthquake Ground Motions. Earthquake Spectra: May 2007, Vol. 23, No. 2, pp. 357-392.

NUREG, 1986. "Engineering characterization of ground motion. Task 2:summary report" Office of Nuclear Regulatory Research. U.S. NUREG/CR-3805, Vol. 5

Porter, K. (2003). "An overview of PEER's performance-based earthquake engineering methodology". *Ninth International Conference on Applications of Statistics and Probability in Civil Engineering (ICASP9) July 6-9, 2003, San Francisco, USA.*

Rodriguez M., E., Restrepo J., I., Carr A., J., (2002). "Earthquake-induced floor horizontal accelerations in buildings". *Earthquake Engineering and Structural Dynamics* . 2002; 31:693–718.

Singh, A. K., Ang A. H. S. (1974). "Stochastic Prediction of Maximum Seismic Response of Light Secondary Systems." *Nuclear Engineering and Design* 29(2): 218-230.

Singh J.P. (1985) Earthquake Ground Motions: Implications for Designing Structures and Reconciling Structural Damage. Earthquake Spectra: February 1985, Vol. 1, No. 2, pp. 239-270.

Spearman, C. (1925). "Some Issues in the Theory of "g" (including the Law of Diminishing Returns)". *Nature* 116 (2916): 436

Stewart JP, Chiou SJ, Bray JD, Graves RW, Somerville PG, Abrahamson NA. (2002). "Ground motion evaluation procedures for performance-based design". *Soil Dynamics and Earthquake Engineering* 2002;22:765–72.

Taghavi, S., Miranda E. (2003)."Response assessment of nonstructural building elements". *Pacific Earthquake Engineering Research Center*. PEER 2003/05.

Taghavi, S., Miranda E. (2006)."Probabilistic seismic assessment of floor acceleration demands in multistory buildings". *Department of Civil and Environmental Engineering, Dtanford University*. Report No. 162.

Taghavi S, Miranda E (2008) "Effect of interaction between primary and secondary systems on floor response spectra". *14th World Conference on Earthquake Engineering*, October 12-17, 2008, Beijing, China.

Takanashi, K., (1975) "Nonlinear earthquake response analysis of structures by a computer-actuator on-line system," *Bull. Of Earthquake Resistant Structure Research Center,* 8, Inst. of Ind. Science, Univ. of Tokyo.

USNRC. 1978. Regulatory Guide 1.122. "Development of floor design response spectra for seismic design of floor-supported equipment or components." *U.S. Nuclear Regulatory Commission*

Villaverde, R. (2004). Seismic Analysis and Design of Nonstructural Components. *Earthquake Engineering: From Engineering Seismology to Performance-Based Engineering*. Y. Bozorgnia and V. V. Bertero, CRC.

Von Thun, J. L., Rochim, L. H., Scott, G. A., and Wilson, J. A. (1988). "Earthquake ground motions for design and analysis of dams." Earthquake Eng. Soil Dyn. II—Recent Advances in Ground-Motion Evaluation (GSP 20), ASCE, New York, 463–481.

Wakefield, D., Ravindra, M., Merz, K., and Hardy, G., (2003). Seismic probabilistic risk assessment implementation guide. Final Report 1002989, EPRI.

Wilson, E., L., Der Kiureghian A., Bayo, E., R., (1981) "A Replacement for the SRSS Method in Seismic Analysis," *Earthquake Engineering and Structural Dynamics*, Vol. 9, pp. l87-l92, 1981.

Zentner, I., Humbert, N., Ravet, S., and Viallet, E. (2011). "Numerical methods for seismic fragility analysis of structures and components in nuclear industry - Application to a reactor coolant system." Georisk: Assessment and Management of Risk for Engineering Systems and Geohazards, 5:2, 99-109.

APPENDIX

Table 9. Efficiency analysis on the LM-SR ground motion bin: standard deviation of the residuals.

Max FRS at:	IMs												
	Frequency Response Based								*Peak Based*		*Duration Based*		
	λIM	$ASA_{E66.7}$	ASA_{40}	S_{pa}	S^*	I_{NP}	ASI	EPA	PGA	PGV	I_A	CAV	$SCAV$
EC8 FRAME													
1Hz±10%	**0.4**	**0.47**	0.32	**0.39**	0.36	0.37	0.56	0.4	**0.56**	0.46	0.5	0.56	0.49
5Hz±10%	**0.26**	**0.3**	0.42	**0.38**	0.42	0.39	0.17	0.36	**0.25**	0.38	0.27	0.4	0.34
10Hz±10%	**0.17**	**0.33**	0.44	**0.38**	0.44	0.4	0.23	0.38	**0.19**	0.4	0.32	0.45	0.4
20Hz±10%	**0.2**	**0.26**	0.38	**0.31**	0.38	0.33	0.17	0.32	**0.15**	0.35	0.26	0.41	0.35
SMART													
1Hz±10%	**0.25**	**0.32**	0.29	**0.23**	0.25	0.22	0.25	0.41	**0.25**	0.47	0.31	0.52	0.43
5Hz±10%	**0.35**	**0.37**	0.41	**0.35**	0.43	0.37	0.42	0.64	**0.42**	0.69	0.52	0.70	0.63
10Hz±10%	**0.38**	**0.28**	0.60	**0.54**	0.59	0.55	0.56	0.76	**0.51**	0.77	0.67	0.80	0.75
20Hz±10%	**0.28**	**0.26**	0.53	**0.48**	0.55	0.49	0.52	0.75	**0.47**	0.77	0.64	0.78	0.73
TC3													
1Hz±10%	**0.13**	**0.17**	0.33	**0.25**	0.3	0.26	0.28	0.44	**0.24**	0.48	0.37	0.51	0.45
5Hz±10%	**0.2**	**0.23**	0.58	**0.39**	0.48	0.41	0.45	0.66	**0.42**	0.68	0.57	0.71	0.65
10Hz±10%	**0.27**	**0.21**	0.58	**0.53**	0.58	0.54	0.55	0.77	**0.54**	0.79	0.68	0.8	0.75
20Hz±10%	**0.43**	**0.4**	0.69	**0.68**	0.72	0.69	0.68	0.88	**0.66**	0.91	0.77	0.86	0.83

Table 10. Efficiency analysis on the LM-LR ground motion bin: standard deviation of the residuals.

Max FRS at:	IMs												
	Frequency Response Based								*Peak Based*		*Duration Based*		
	λIM	$ASA_{E66.7}$	ASA_{40}	S_{pa}	S^*	I_{NP}	ASI	EPA	PGA	PGV	I_A	CAV	$SCAV$
EC8 FRAME													
1Hz±10%	**0.33**	**0.47**	0.31	**0.34**	0.39	0.3	0.64	0.43	**0.64**	0.6	0.51	0.62	0.84
5Hz±10%	**0.26**	**0.43**	0.59	**0.55**	0.62	0.55	0.25	0.5	**0.31**	0.58	0.42	0.62	0.51
10Hz±10%	**0.19**	**0.3**	0.44	**0.39**	0.48	0.39	0.24	0.36	**0.23**	0.47	0.31	0.54	0.51
20Hz±10%	**0.20**	**0.24**	0.37	**0.31**	0.41	0.32	0.24	0.32	**0.25**	0.44	0.27	0.48	0.51
SMART													
1Hz±10%	**0.23**	**0.3**	0.32	**0.23**	0.29	0.23	0.31	0.52	**0.29**	0.59	0.46	0.711	0.61
5Hz±10%	**0.44**	**0.43**	0.53	**0.44**	0.6	0.48	0.64	0.91	**0.68**	0.94	0.83	1.03	0.84
10Hz±10%	**0.58**	**0.48**	0.72	**0.64**	0.8	0.68	0.83	1.14	**0.84**	1.15	1.07	1.3	1.02
20Hz±10%	**0.52**	**0.38**	0.6	**0.5**	0.67	0.54	0.71	1.02	**0.72**	1.04	0.95	1.18	0.94
TC3													
1Hz±10%	**0.2**	**0.24**	0.35	**0.3**	0.32	0.29	0.31	0.51	**0.26**	0.56	0.46	0.72	0.6
5Hz±10%	**0.43**	**0.4**	0.54	**0.47**	0.63	0.51	0.66	0.98	**0.71**	1.01	0.91	1.13	0.9
10Hz±10%	**0.57**	**0.48**	0.72	**0.65**	0.8	0.69	0.82	1.13	**0.85**	1.15	1.06	1.28	1.03
20Hz±10%	**0.6**	**0.46**	0.73	**0.66**	0.79	0.69	0.81	1.1	**0.81**	1.11	1.04	1.28	1.05

Table 11. Efficiency analysis on the SM-SR ground motion bin: standard deviation of the residuals.

Max FRS at:	IMs												
	Frequency Response Based								*Peak Based*		*Duration Based*		
	λIM	$ASA_{E66.7}$	ASA_{40}	S_{pa}	S^*	I_{NP}	ASI	EPA	PGA	PGV	I_A	CAV	$SCAV$
EC8 FRAME													
1Hz±10%	**0.28**	**0.53**	0.22	**0.26**	0.29	0.25	0.89	0.55	**1.07**	0.62	0.87	0.78	1.15
5Hz±10%	**0.24**	**0.54**	0.66	**0.64**	0.67	0.62	0.27	0.46	**0.39**	0.48	0.38	0.52	0.55
10Hz±10%	**0.15**	**0.55**	0.64	**0.62**	0.64	0.6	0.3	0.45	**0.19**	0.43	0.28	0.48	0.47
20Hz±10%	**0.35**	**0.55**	0.63	**0.61**	0.63	0.59	0.33	0.44	**0.22**	0.42	0.28	0.48	0.5
SMART													
1Hz±10%	**0.31**	**0.37**	0.33	**0.26**	0.36	0.26	0.33	0.51	**0.35**	0.51	0.38	0.55	0.58
5Hz±10%	**0.43**	**0.49**	0.53	**0.43**	0.58	0.46	0.58	0.76	**0.61**	0.78	0.66	0.78	0.77
10Hz±10%	**0.5**	**0.49**	0.73	**0.65**	0.77	0.67	0.74	0.89	**0.62**	0.89	0.76	0.89	0.78
20Hz±10%	**0.41**	**0.44**	0.69	**0.59**	0.73	0.62	0.7	0.87	**0.56**	0.87	0.72	0.87	0.75
TC3													
1Hz±10%	**0.17**	**0.26**	0.43	**0.34**	0.45	0.35	0.38	0.56	**0.28**	0.55	0.38	0.56	0.54
5Hz±10%	**0.29**	**0.32**	0.55	**0.41**	0.6	0.45	0.57	0.77	**0.49**	0.78	0.62	0.78	0.69
10Hz±10%	**0.4**	**0.33**	0.72	**0.63**	0.77	0.66	0.73	0.9	**0.6**	0.9	0.75	0.89	0.77
20Hz±10%	**0.46**	**0.46**	0.79	**0.7**	0.82	0.73	0.79	0.94	**0.64**	0.94	0.79	0.94	0.81

Table 12. Efficiency analysis on the SM-LR ground motion bin: standard deviation of the residuals.

Max FRS at:	IMs												
	Frequency Response Based								*Peak Based*		*Duration Based*		
	λIM	$ASA_{E66.7}$	ASA_{40}	S_{pa}	S^*	I_{NP}	ASI	EPA	PGA	PGV	I_A	CAV	$SCAV$
EC8 FRAME													
1Hz±10%	**0.22**	**0.46**	0.2	**0.18**	0.28	0.19	0.8	0.48	**0.92**	0.55	0.71	0.62	1.18
5Hz±10%	**0.23**	**0.58**	0.76	**0.72**	0.77	0.7	0.27	0.49	**0.33**	0.51	0.37	0.59	0.84
10Hz±10%	**0.20**	**0.57**	0.71	**0.66**	0.72	0.64	0.3	0.45	**0.22**	0.44	0.29	0.56	0.81
20Hz±10%	**0.28**	**0.48**	0.62	**0.57**	0.64	0.55	0.31	0.37	**0.29**	0.36	0.27	0.51	0.82
SMART													
1Hz±10%	**0.28**	**0.36**	0.35	**0.25**	0.37	0.25	0.35	0.58	**0.32**	0.58	0.43	0.68	0.87
5Hz±10%	**0.41**	**0.45**	0.52	**0.41**	0.58	0.44	0.59	0.83	**0.59**	0.84	0.69	0.88	0.99
10Hz±10%	**0.53**	**0.48**	0.76	**0.65**	0.82	0.68	0.8	1.04	**0.7**	1.04	0.88	1.08	1.08
20Hz±10%	**0.42**	**0.39**	0.66	**0.53**	0.73	0.57	0.71	0.97	**0.6**	0.96	0.8	1.02	1.02
TC3													
1Hz±10%	**0.23**	**0.28**	0.45	**0.36**	0.47	0.36	0.42	0.63	**0.31**	0.63	0.47	0.72	0.82
5Hz±10%	**0.32**	**0.33**	0.57	**0.43**	0.63	0.47	0.62	0.88	**0.55**	0.89	0.73	0.94	0.97
10Hz±10%	**0.42**	**0.34**	0.72	**0.61**	0.79	0.64	0.75	1.00	**0.66**	1.00	0.84	1.05	1.04
20Hz±10%	**0.47**	**0.44**	0.73	**0.63**	0.77	0.66	0.75	0.94	**0.63**	0.93	0.8	0.99	0.9

Project SIGMA

Review of:

RELATION BETWEEN SEISMIC GROUND MOTION AND STRUCTURAL DAMAGES & FUNCTION LOSS

(Ref : SIGMA-2014-D5-119)

by : Jean B. Savy
May 21, 2014

This is a review of the research work done by M. DiBiasio and documented in EDF Ref: SIGMA-2014- D5-119. This work is to be presented at the CS7 of June 4 th to the 6th, 2014, in Cadarache, France.

1. Purpose and Scope of the study

The purpose of the study was to select the most appropriate, efficient and sufficient, seismic damage Intensity Measures (IM), with respect to its ability to predict non-structural-components (NCSs) acceleration demand. The first phase of the work consisted in a thorough collection, and evaluation of existing IMs, and in the second phase a new IM is proposed and tested on a set of three typical types of structures relevant to the SIGMA project. The goal of the study was to formulate this new proposed IM and to demonstrate its efficiency and its sufficiency.

2. Review approach

Most IMs present some advantages, but all are known to be very imperfect in predicting structural damages and a task such as the one of this study, with the premises of improvement, is very worthwhile and important. Thus my review concentrated on trying to answer the following questions:

- Did the author(s) thoroughly examine previous work?
- Is the overall approach to developing the new IM scientifically sound?
- Is the proposed IM well designed? Is the demonstration of improvements valid and convincing?
  - The proposed IM Model
  - Data
  - Statistical analyses
- Are the conclusions supported by the results of the analysis?
- How relevant is this work in the context of SIGMA's goals?
- How does it relate to other WPs?
- What future work do the authors propose, and/or could be done?

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- Comments on the general form of the document reviewed.

3. General review conclusions

The reviewed study achieved the goal of introducing a potentially useful predictor of acceleration demand in NCSs. The arguments that lead to the formulation of the proposed IM are well constructed and well presented. The demonstration of its superiority over some other IMs is good but not entirely convincing. In effect, Table 6 shows that better efficiency can still be obtained with our old friend “PGA” in certain cases of specific structures (The EC8 Frame test structure in this case), and certain ranges of frequencies (High frequency in this case).

One possible limitation that I see in this study is in the limited choice of test structures that are used to demonstrate the efficiency of the proposed IM. Although the numerical test models are well constructed, and are well calibrated by laboratory testing, they nevertheless are limited in the range of cases they include. In addition, the analyses performed do not show a fully convincing case for using the proposed IM as its efficiency varies considerably between the three cases of structures considered, even for this limited and to a certain extent similar types of structures. As a matter of fact, the authors conclude that for some types of structures the proposed IM should or could be modified to better fit the structure.

Although I think the conclusion is a bit too enthusiastic in saying that this new IM is a great improvement over existing IMs, because the authors did not really demonstrate its universality over many types of structures, the fact that it works well for a range of structures, and that it can be used for both structures and NCSs with a small conversion, and the fact that it can be derived easily from the information generated in a hazard study, makes it a real contender for general implementation, and possibly introduction in building codes. This is actually natural since recent developments in regulations and building codes internationally are moving in the same direction.

The reviewed document is well structured and contains all the information to understand the method and the details of the analyses performed, but it needs editing. Detailed suggestions for editing are given in attachment, for the authors use. (Not given here)

- *Review of previous work:*
  The review of existing work is satisfactory. It includes the most recent contributions, and is adequately referenced.

- *Overall strategy/approach:*
  The general approach consists of selecting a set of the most relevant IMs published in the literature and test them concurrently with the proposed IM whose construction is done with sound considerations of structural dynamics, well documented in the report.

- *Proposed IM and demonstration of improvements:*
  -Proposed IM:
  The structural dynamics arguments used to identify the dominant parameters that influence the NCSs acceleration demand make sense and are well presented. One possible caveat is in the fact that the structural dynamics analyses were only linear. Although it is certainly true that the greatest acceleration demand for the NCSs is when the supporting structures remain linear, the fact that only linear analyses were performed will tend to over-predict the actual NCSs demand. This would evidently occur only for the larger earthquakes. For those earthquakes where the supporting structures would possibly behave non-linearly, the actual NCSs demand would be less than that predicted by assuming linearity.

-Data:
The ground-motion dataset used is the last version of RESORCE (Akkar 2013), which is quite appropriate and the limitation to magnitudes greater than 4.5 and distances less than 100km is also appropriate. The number of remaining events is still sufficient (2045 ground-motion values).
-Statistical analysis:
The statistical analysis is well conducted, and the choice of the Spearman Ranking test is a good one as it does not assume normality in the data, unlike the often used Pearson's test.

- *The conclusions*
  The conclusions are based on a subjective interpretation of statistical results, basically by a visual inspection of standard deviation values (table 6) for conclusions on efficiency, and use of the Spearman rank correlations test for sufficiency (table 7). The authors note that in spite of their preference for the proposed IM as exhibiting the highest number of bins (in Table 6) where standard deviation is the lowest, there are a good number of cases where they are not. Particularly for the case of the EC8 Frame test structure. So, in my view, the statement of superiority in efficiency should be somewhat tempered. At a minimum, Table 6 shows that different IMs work best for different supporting structures and different ranges of frequencies. But one thing that makes the proposed IM more powerful is that, recognizing this fact, the R value can be adjusted to the specifics of the supporting structure, and possibly to the type of NCSs.

- *Relevance to SIGMA*
  The document does not have a discussion on the relevance of this work to SIGMA. However, I believe that it is very important and could be used extensively in the future. EDF's stock of buildings and special facilities is limited in types and the IM definition could be adjusted for each type and therefore allowing for better prediction of the NCSs demand.

- *Contextualization, relation to other WPs*
  There was no attempt to relate this work with tasks in other WPs, or other tasks within WP5, such as the generation of design time-histories for specific return periods.

- *Future work*
  The next step should be the testing of this approach on real structures… if and when the demand data become available.

4. Detailed comments

Executive Summary
- No comments.

Summary
- No comments.

1. Introduction
- As mentioned above, the linear analysis of the structures precludes possible non-linear degradation for larger earthquakes. This assumption of linearity would be fine if it were confirmed. How much

difference would that make is not clear, but it would be worth doing at least a
few non-linear calculations to find out. A linear calculation will possibly lead to much greater NCSs acceleration

demand than a non-linear calculation. A non-linear calculation will have the effect of de-correlating the ground-motion with the NCSs demand. The prediction with a linear model will likely be over-estimated.

2. Intensity Measure for NSCs acceleration demand

- It is stated on page 10 that the $ASA_R(f1)$ is particularly efficient for non-linear structures. What is the (missing) reference to support this statement?

- The extension of eq. 7 to eq. 8 for NCSs appears naturally and is well supported.
.

3. Comparative Analyses

- The descriptions of the test structures, the numerical models and the dataset are appropriate.

4. Results

- The choice of using the Spearman rank correlation test is good to test the sufficiency of IM with respect to M_W, R_{hyp} or V_{S30}. A minor comment is that the equation given in the report applies only to data where ranks are all distinct. This does not seem to be our case, since many magnitude values will be clustered in equal values, and this is possibly also true for V_{S30}. In those cases, the weight of each couple should be given a value such that the cluster has weight one, and not each couple. I suppose the calculations where done with a standard software (SAS, SPSS, NCSS or the like), and this is done automatically then. Otherwise the formula given in the text should be updated.

- On page 17, section 4.2, it is stated:
  "The results of the comparative statistical analysis about the IMs' efficiency are presented (Table 6) with respect to the 2,045 records composing the four ground-motions bins. These results do not show significant discrepancies with the results obtained with respect to the four ground-motions bins taken one-by-one (Appendix, Tables 9-12)."
  This is not really a fair statement in regard to the information shown in Table 6. Although the table shows a good agreement of $E\text{-}ASA_{67}$ for the TC3 structure and for high frequencies in the SMART structure, it does not show a similar good agreement for the low frequency range of SMART, and not a good agreement for the EC8 FRAME.
  It would be more realistic to say that this IM works well for a range of structures but has limitations. Furthermore the three structures selected do not represent a universal range of structures and that remaining space of structures has not been tested.
  It is appropriately noted that PGA works best in some cases.

- *Page 22 section 4.4 E-ASAR Optimum:*
  Same comment as above. The selection of 67% from a visual inspection of Table 8, does not seem to be 67% as an obvious optimum. In fact among the 12 cases shown in the table, only 1 has 67% as the best choice, 200% appears to be the best in 4 cases, 40% in 5 cases, 80% in 2 cases, and 100% in 1 case, and 150% in 2 cases (Total more than 12 because of 3 equal values).
  Rather, it is the explanations given below the table that give a fair statement of how this value was selected. It therefore appears that the 67% is not really supported by the analysis, but rather subjectively by practical considerations, albeit quite appropriate.

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This indicates that more research should be done to better develop impartial and objective arguments and criteria, based on real data to support the choice of R.

- *Page 22, equation 11:*
  Unless I completely misinterpreted eq. 7, and eq. 8, it seems that eq. 11 should be:
  From eq. 7, replacing f1 by 1.67.f1,

$$\text{E-ASA}_{67}(f1) = 1.5\ \text{ASA}_{40}(1.67.f1)$$

5. Conclusions

- No additional comments.

Respectfully submitted, May 21, 2014

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REVIEW ON BEHALF OF SIGMA PROJECT

Reference: SIGMA-2014-D5-119

Title: Relation between Seismic Ground Motion and Structural Damages & Function Loss. Part 2: Acceleration-sensitive Equipment.

Author: M. Di Biasio

Reviewer: Philippe Renault (supported by Luis A. Dalguer)

Date: 26.05.2014.

Review comments:

The author proposes a new ground motion intensity measure (IM) to predict horizontal acceleration demand of non-structural components (NSCs) attached to the main structure. This new IM (named E-ASA_R) is a modification of the recently IM ASA_R proposed by the same author (De Biasio et al, 2014) and developed to predict structural demands of the main structure. The main modification with respect to ASA_R is that the new IM is represented as the average spectral pseudo-acceleration over the dominant frequency interval, in which the lower bound of this interval is the fundamental frequency of the main structure, and the upper bound is a percentage (R) of this fundamental frequency. An optimum value R = 67% is found after a numerical sensitivity analyses. This proposed E-ASA_R sounds promising to assess damage of NSCs. In general the report reads well, the goal and the scope are well defined. I just have some minor comments that I think would be beneficial to be addressed to improve the quality of the report.

1) The author develops a comparative statistical analysis (with other IMs) by estimating the efficiency and sufficiency for each IM. Though the idea sounds good, well done and very informative to verify the performance of the proposed IM with respect to other IMs, the comparison is not fair and not symmetric with respect to the others IMs. This is because the other IMs were not designed to predict the demand of NSCs but for the main structure (actually most of the IMs listed in the report are used to predict deformation demand and not acceleration demand). In that sense it is expected that the proposed IM (E-ASA_R) should be by character better than the other IMs to predict demands of NSCs, as shown in the report, but it does not automatically mean that the proposed E-ASA_R performs better than other IMs designed for the same purpose. This aspect should maybe be discussed in the final version of the report. The only symmetric and fair comparison would be with the λIM. Though λIM is not practical, the proposed E-ASA_R performs badly when compared to it. It would be good to estimate quantitatively how far E-ASAR is with respect to λIM. It should be noted that the term "λIM" caused confusion the first time I read the report, as it was understood as a multiplication of the parameter lambda and IM and λ is used in the previous equation. Thus, a suggestion might be to put "IM_λ " instead.

2) Equation 3 defines the amplification factors α_i as squares of the ratio of the frequencies. The report should give a brief explanation why the squares are preferred (e.g. more robust) than the unsquared values.

3) How is the modal participation factors Γ_i of equation (1) calculated? It is not clear to me when seeing the numbers in Table 2-4. Commonly engineers would talk about the modal mass participation factor. From my understanding the Γ_i would be a percentage of the contribution of each mode (*i*) to the defined target (in this case to the structural node *k*), then the summation of all Γ_i would be 1.0. I may be missing something, so I think in the report need to be clearly explained how Γ_i are calculated and what are the meanings of the obtained results.

4) In Chapter 3.3 ''Load and Demand Parameters'' (second block) are the frequency ranges defined, in which the NSCs acceleration demand is calculated. Is the rage 8 to 10 Hz correct? I ask it because the corresponding NSCs fundamental frequency for this rage is 10Hz (see next two lines and tables 2-6, 8). This is no consistent with the other ranges and their respective hypothetical NSC fundamental frequencies, i.e., the hypothetical NSC fundamental frequency is supposed to fall in the mean of the frequency range.

5) Within the conclusions or maybe earlier in the discussion it should be highlighted why the separation in four magnitude and distance bins does not significantly impact the result of the study: Up to my understanding this is because the new intensity measure is thought to relate to non-structural components which are susceptible to high frequency motion. The high frequency motion is not so sensitive to the selected bins and thus, there is no obvious dependency. The reader might be interested in this more clear explanation, as from experience on the evaluation of the main structural components it is expected to get some dependencies on M-R bins and the risk assessment methods sometimes also benefit from fragility curves derived on M-R bins.

It the same way it might be worth to shade some light on why the CAV (or S-CAV) is not performing well for this specific evaluation. CAV is indirectly related to the velocities of an event which is governed by low frequency behavior. Again, here we are looking for the NSCs which are affected by the high frequency content and thus, is becomes obvious why CAV would fail as a good indicator for this case.

6) Editorial comments: Acronyms such as FRS, EDP (eq. 9), PGA and others should to be defined earlier before using in the text. Even though the meaning of them may be obvious, for formality and clarity of whatever document they need to be defined.

The figure 1 should get a label of what is represented on the horizontal and vertical axes (SA vs. frequency). Furthermore, the central dashed line should at least get the label “f_1“ on the horizontal axis so that it is consistent with the text.