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RELATION BETWEEN SEISMIC GROUND MOTIONS AND STRUCTURAL DAMAGE

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ABSTRACT

This paper handles the problem of the seismic ground motions' destructiveness quantification. A new Intensity Measure, namely Cumulative Pseudo Acceleration *(CPA)*, is proposed and a comparison with respect to the most accredited IMs present in literature has been done, based on: a) the use of a large dataset of recorded earthquake signals (more than 4000 records); b) numerical analyses executed with a state-of-art FE model, representing an actual structure, validated by tests on shaking table and taking into account Soil-Structure Interaction effects; c) systematic statistical analysis of the results. The study shows that the IMs built starting from the pseudo acceleration response spectrum exhibit higher efficiency compared to others and that, between these, herein introduced *CPA* shows the highest and most robust efficiency. The simplicity of computation and the efficiency of the *CPA* lead to suggest its use, in the framework of performances based seismic design/assessment, for seismic hazard definition and fragility functions definition.

INTRODUCTION

This contribution aims at identifying the characteristics of ground motions (i.e. Intensity Measures or IMs) determining their destructiveness with respect to building structures.

Ground-motion IMs are the link between seismic hazard and seismic demand analysis: they play a key role in ground-motion selection procedures, seismic hazard definition, fragility functions definition and operating basis earthquake (OBE) exceedance definition. 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. Furthermore, an effective IM is one for which it is practically realistic to compute probabilistic seismic hazard (i.e. attenuation relationships have to be easily computable for such an IM).

Many studies have proposed IMs and investigated their performances: unfortunately most of these analyses are based on limited case studies that make use of small sets of ground motions and simplistic test case structures (this can explain the heterogeneity of the conclusions of most of such studies). Therefore their use for engineering studies/analyses is limited and necessitates large safety coefficients. The ambition of the present work is to overcome these points by using a large dataset of real ground motions and a state-of-art numerical model validated by experimental results.

The study is performed following three main steps: a) computer analysis of the 4031 accelerograms to provide the values of the selected IMs; b) non-linear dynamic (time history) analyses to provide the structural response, of the chosen reinforced concrete structure, for the given 4031 seismic excitations; c) statistical analysis of the outputs of the aforementioned two steps to provide the grade of interdependency between seismic acceleration parameters and the selected damage indices (structural response).

Finally, a new IM, the Cumulative Pseudo-Acceleration (*CPA),* is introduced and its efficiency is compared to the one of well known IMs: peak ground acceleration and velocity *(PGA, PGV)*, 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), RMS acceleration a_{RMS} (Housner et al. 1964), Characteristic Intensity I_C (Park et al., 1985) and Japan Meteorological Agency index *JMA* (Shabestari et al., 2001). The definition of each of these IMs is presented in literature and will not be repeated here.

INTENSITY MEASURES

Improving IMs Efficiency

Experience shows that not all the structures have the same sensitivity, in terms of experienced damage, to the same ground motion: large earthquakes appear to heavily damage specific classes of structures and produce little damage on others (typical example is the 1985 Mexico earthquake where the most damaged buildings were those with fundamental frequency around 0.5 Hz whatever the construction age).

The dependence on the structure of the correlation IM vs. damage-of-the-building is conceptually evident: a ground motion includes a certain amount of energy; a part of this energy is transferred to the buildings that can, or not, experience relevant damage. What determines the amount of damage that the building experience is: a) the amount of energy that the ground motion transfers to the building; b) the energy dissipation property of the building. Both these properties do not depend exclusively on the building but they also depend on the non-linear dynamic interaction between building and forcing input (earthquake): the first one is essentially related to the frequency content of the earthquake with respect to the natural frequencies of the structure (i.e. resonance effect); the second one is strongly dependent on the building design and material characteristics. At this point it is evident that IMs built exclusively on the characteristics of the ground motion have a limit and they will always contain a non-eliminable degree of uncertainty due to the absence of information about the dynamic behaviour of the structure: in fact such IMs (for example I_A and *CAV*) attempt to measure the amount of energy released by the ground motion, but they can not isolate the amount of energy that “enter” in the structure, being this last the only one able to produce damage.

At this point, the structure’s fundamental frequency appears to be the key parameter able to discriminate the destructiveness of the ground motion when compared to the frequency content/spectral response of this last. Nevertheless, for an actual structure such a quantity is usually known or easy to know by means of in situ tests (Michel et al., 2010) or, for regular buildings, it can be roughly estimated by empirical code-based approach.

The simplest IM able to consider the relation structure’s-fundamental-frequency vs. ground-motion’s-frequency-content is the well-known spectral acceleration computed at the fundamental frequency of the structure (S_{pa} (f_1)). However, this has two major shortcomings, it ignores both the contributions of higher modes to the overall dynamic response and the decrease of the fundamental frequency associated with accumulation of damage. In fact, an inelastic behaving structure is not characterized by a “single-value” fundamental frequency: when the elastic limit is reached irreversible processes (concrete cracking, steel yielding, joint failure, etc.) enable a progressive loss of stiffness with consequent decrease of frequency.

Efforts to overcome the drawbacks of the S_{pa} (f_1), specially regarding the decrease of frequency aspect, have already been made, for example Cordova et al. (2001), Mehanny (2009) and Kadas et al. (2011). However, most of the cases, the complex formulation of such IMs avoids their implementation and diffusion in engineering practice.

A New Intensity Measure: the CPA (Cumulative Pseudo Acceleration)

An attempt to create a more efficient IM is done here: the key-idea is to consider the structure's frequency drop interval in the definition of the IM. The range of frequencies to take into account has the upper bound at the fundamental frequency and the lower one at the maximum expected "damaged" frequency. This last is evaluated as percentage of decrease with respect to the fundamental value. Therefore we propose a new IM, named Cumulative Pseudo-Acceleration *(CPA)*, that is the integral of the pseudo-acceleration over the interval of variation of the fundamental frequency:

$$CPA = \frac{1}{f_F}\int_{f_D}^{f_F} S_{pa}(f,\xi)\ df \qquad (1)$$

where f_F is the fundamental frequency of the structure, f_D is the frequency of the structure at the expected damaged state, S_{pa} is the pseudo-acceleration and ξ is the damping value. The frequency has been chosen as integration variable, contrary to the commonly used period *(T):* this makes a fundamental difference between the *CPA* and apparently similar IMs as the *EPA* and the *ASI* (Von Thun et al., 1988). In fact the integration over the frequency gives equal weight to the spectral ordinate values; instead (2), the integration over the period gives higher weight to the lower frequency spectral values.

$$\int_{T_1}^{T_2} S_{pa}(T,\xi)\,dT = \int_{\frac{1}{T_2}}^{\frac{1}{T_1}} \frac{S_{pa}(f,\xi)}{f^2}\,df \neq \int_{\frac{1}{T_2}}^{\frac{1}{T_1}} S_{pa}(f,\xi)\,df \qquad (2)$$

Therefore, the formulation (1) of the *CPA* captures, in the simplest way, the presence of high spectral acceleration ordinates over the range of evolution of the fundamental frequency: this last is the key characteristic that a seismic signal must have in order to be destructive, see NUREG (1986) and Kennedy et al. (1988).

The value of f_D as percentage of reduction with respect to f_F can be evaluated based on the ability of the structure object of the analysis to lose (or not) stiffness when it undergoes inelastic deformation. Values of reduction with respect to f_F are in the range of 15-65 % (CALTECH, 1975). Note that f_D can be also chosen based on the level of damage assumed as tolerable for the examined structure (i.e. by using the frequency drop as Engineering Demand Parameter (EDP)): for example if a small damage is tolerated (5-10 % frequency drop) f_D can be assumed as a reduction of 5-10 % with respect to f_F.

Moreover, another way to specify the f_F and f_D boundaries is to assign them based on the class of structure to analyse: in such a way the dependency on the specific structure is overcame (differently from the S_{pa} (f_1)), and the decrease of the IM efficiency can be limited if the interval of integration is not excessively large. For instance, based on the knowledge of the authors, the range 0.5 – 2.5 Hz is representative of the most of European residential buildings and is small enough to assure high efficiency to the *CPA*. In such a way, overcoming the single structure dependency, the aforementioned enable the *CPA* to be easily employed in Ground Motion Prediction Equations (GMPE) in order to build more accurate probabilistic seismic hazard maps.

COMPARATIVE ANALYSES

Test Case Structure

In order to test the performances of the IMs, the choice of the test case was a structure object of experimental tests: this offers the advantage to dispose of a validation tool for the numerical model giving, under the condition of sufficiently precise agreement of the numerical simulation vs. experimental tests, value/authority to the results extracted from the numerical model. The test case structure chosen for this study is the CAMUS 1 (IAEA, 2011) mock-up: this is a 1/3 scaled model tested in 1998 on the

Azalee shaking table (CEA, France), also object of an international blind contest. Nevertheless, the mock-up is dimensioned and tested following precise similitude criteria that allow doing its behavior representative of a full-scale structure.

The mock-up (Figure 1) is composed by two parallel 5-floor R/C walls without openings linked by 6 square floors. The total height of the structure is 5.10 m and the total mass is around 36 tons.

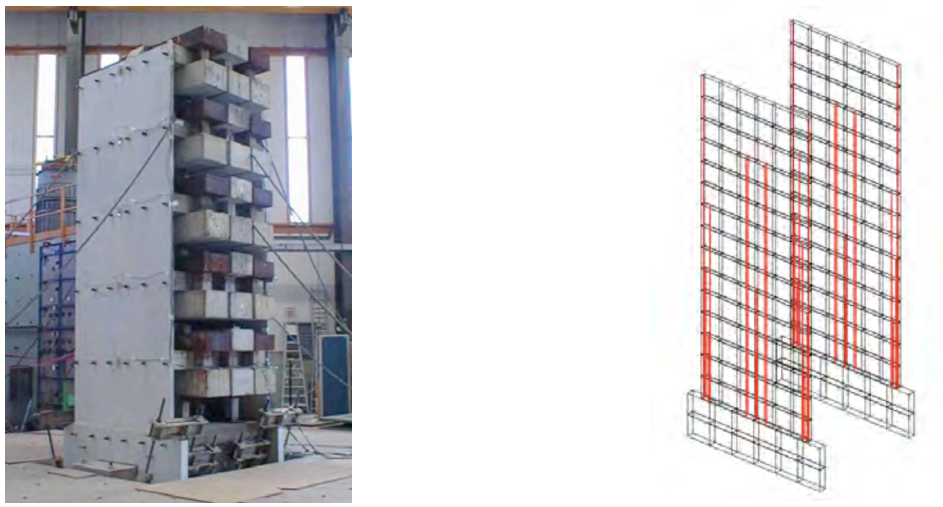

Figure 1. CAMUS1: left, mock up on the shaking table; right, numerical model

Numerical Model

In order to be adequate for this study, the numerical model must have three fundamental characteristics: a) good representation of the linear dynamic behaviour of the structure, i.e. natural modes and frequencies; b) good representation of the non-linear behavior of the structure, in order to accurately reproduce the key damage mechanisms under seismic loading; c) low computational cost, in order to allow, in a reasonable time, to extend the analysis to a large number of seismic signals providing statistical value to the study.

The numerical modelling technique (adequate to the chosen test-case structure) that can match these requisites is the, here called, "semi-global" modelling approach. This approach allows using comprehensive local behavior laws in the framework of a simplified kinematic associated to finite beam elements (multifiber modelling) (Guedes et al. 1994).

The structure has been modelled as a single multifiber beam model representing the two structural walls. The masses have been assigned to the numerical model as concentrated masses. The position, the dimension and the number of steel reinforcement bars of the mock-up have been scrupulously respected in the numerical modelling (Figure 1).

The constitutive law chosen to represent the concrete behaviour (La Borderie et al., 1994), is able to take into account the decrease in stiffness due to cracking, the stiffness recovery that occurs at crack closure and the inelastic strains concomitant to damage. For the steel reinforcement bars a constitutive relation including kinematic hardening (Menegotto et al., 1973) has been chosen. The mechanical characteristics of the mock-up's materials have been assigned based on elementary tests on both concrete and steel.

The predictive capabilities of the numerical model have been checked with the results coming from shaking table tests, in terms of: a) natural frequencies; b) roof displacement time history (Figure 2); c) max inter-storey drift; d) floor response spectrum at the roof (Figure 2); e) damage pattern in the concrete. The comparison of numerical and experimental results has shown the ability of the numerical model to predict with accuracy the mock-up's linear behavior as well as its non-linear behavior, qualitatively and quantitatively.

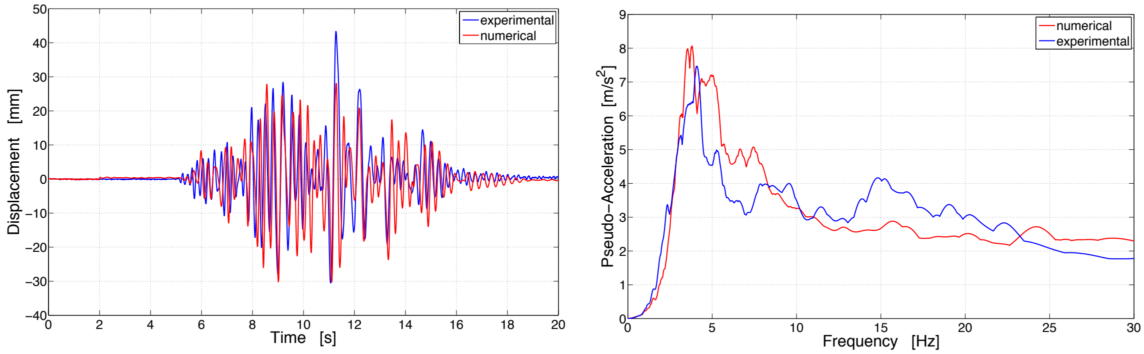

Figure 2. Numerical model validation: left, top displacement time history; right, floor response spectrum at roof

The validated 1/3 numerical model has been rescaled to full scale in order to avoid the scaling of the whole ground motion dataset. The horizontal unidirectional load has been applied in the plane of the walls; the self-weight of the structure (important for concrete crack-closure phenomena) has been taken into account.

In order to analyze the effect of Soil-Structure Interaction on the structural response, two different boundary conditions have been employed: a) structure clamped at the base (i.e. foundation on stiff rock, V_{s30} >900 m/s); b) structure posed on a soft soil (V_{s30} =270 m/s). The presence of the soil has been taken into account by means of an experimentally-validated discrete model, Wolf (1988), able to reproduce the kinematic soil-structure interaction (i.e. inability of the foundation to match the free field motion) as well as the inertial soil-structure interaction (i.e. the masses of the super-structure and the foundation transmit the inertial force to the soil causing further deformation in the soil) as well as the radiative damping (i.e. radiation of the waves from the foundation to infinite). These three SSI effects reflect the possibility of displacement at the foundation level, natural frequencies change and energy dissipation capability.

The fundamental frequency of the full-scale model clamped at the base is 5.85 Hz; the one of the full-scale model on soft soil is 3.80 Hz.

Damage Measure

The response of the structure (structural demand) has been measured in the numerical model as: a) maximum top displacement; b) maximum inter-storey drift at the first storey, where the non-linearities are concentrated; c) fundamental frequency drop. This last has been evaluated by means of Fourier transform analysis of the post-earthquake response of the structure under white-noise signal.

Ground Motion Dataset

The ground motion dataset employed as input for the FE simulations is constituted by the first horizontal component of all the 4031 three-dimensional records of the 2013 version of the SIGMA RESORCE database (www.resorce-portal.eu). This last regroups ground motions recorded in Europe and nearby countries (i.e. Iran, Uzbekistan, Georgia, Egypt, Algeria, Armenia, Syria, Kyrgyzstan, Israel, Lebanon) during the last decades. The database is composed by records of magnitude between 2 and 8. It must be highlighted that, due to the geographical provenience, most of the earthquakes records have very low intensity, so they do not impact the structure in terms of damage.

RESULTS

Results Analysis Method

To emphasize the grade of relation between IMs and Damage Measures (DMs), two different methods are employed: a) the rank correlation coefficient after Spearman (1925) has been calculated for the couples IMs-DMs: such a coefficient measures how well the data agree with monotonic (linear or not) ranking (i.e. if IM increases, DM increases as well and vice versa); b) regression analysis is used on the couples IMs-DMs in order to estimate the standard deviation of DM given IM (i.e. how far the data points are from the data best-fit curve); the employed regression technique considers a linear relationship between the natural logarithms of the couples IMs (normalized)-DMs (Cornell et al., 2002).

Results

The results of the statistical analysis on the relation IMs-DMs are presented in Tables 1-2 in terms of Spearman rank correlation coefficient. Table 1 shows the results of the statistical analysis in the case where all the 4031 ground motion signals are considered. Tables 2 shows the results of the statistical analysis in the case where only the signals able to produce at least a 20 % of frequency drop have been retained.

The results in terms of standard deviation give coherent indication with respect to the ones in terms of Spearman correlation coefficient. Therefore, for the sake of brevity, they are not presented in this paper.

In order to analyze the sensitivity to the choice of the reduced fundamental frequency (*f_D)* of the *CPA* indicator, it has been computed for both 20% and 40% drop, i.e. CPA20 and CPA40 respectively.

Table 1: Spearman’s correlation coefficient (the higher, the more efficient)

	STIFF SOIL (V_{s30} >900 m/s)			SOFT SOIL (V_{s30} = 270 m/s)		
	Max top displacement	Max drift	Frequency drop	Max top displacement	Max drift	Frequency drop
CPA20	**0.99**	**0.99**	**0.35**	**0.99**	**0.99**	**0.39**
CPA40	**0.98**	**0.98**	**0.34**	**0.98**	**0.99**	**0.39**
S_{pa}	0.99	0.99	0.35	0.99	0.99	0.39
Cordova	0.97	0.97	0.34	0. 97	0.97	0.39
PGA	0.96	0.96	0.34	0.94	0.94	0.38
CAV	0.80	0.81	0.31	0.88	0.88	0.35
EPA	0.85	0.86	0.33	0.93	0.92	0.38
PGV	0.85	0.86	0.32	0.91	0.91	0.37
I_H	0.76	0.77	0.30	0.86	0.86	0.36
I_A	0.90	0.91	0.34	0.94	0.94	0.38
S-CAV	0.82	0.82	0.39	0.81	0.81	0.44
I_C	0.67	0.6	0.50	0.66	0.66	0.54
a_{RMS}	0.66	0.67	0.48	0.65	0.65	0.51
JMA	0.41	0.42	0.17	0.42	0.42	0.19

Table 2: Spearman's correlation coefficient (the higher, the more efficient)
(Analysis limited to the signals (about 200) able to reduce the fundamental frequency at least of 20 %)

	STIFF SOIL (V_{s30} >900 m/s)			SOFT SOIL (V_{s30} = 270 m/s)		
	Max top displacement	Max drift	Frequency drop	Max top displacement	Max drift	Frequency drop
CPA20	**0.80**	**0.79**	**0.80**	**0.89**	**0.88**	**0.88**
CPA40	**0.89**	**0.89**	**0.89**	**0.92**	**0.91**	**0.92**
S_{pa}	0.66	0.63	0.64	0.77	0.76	0.73
Cordova	0.80	0.81	0.77	0.75	0.74	0.72
PGA	0.70	0.70	0.67	0.67	0.65	0.67
CAV	0.55	0.56	0.51	0.55	0.53	0.54
EPA	0.60	0.63	0.56	0.63	0.62	0.60
PGV	0.55	0.58	0.51	0.53	0.53	0.50
I_H	0.50	0.53	0.46	0.49	0.48	0.44
I_A	0.68	0.69	0.64	0.70	0.68	0.71
S-CAV	0.58	0.60	0.55	0.59	0.57	0.59
I_C	0.72	0.73	0.69	0.73	0.71	0.74
a_{RMS}	0.60	0.60	0.59	0.50	0.47	0.54
JMA	0.36	0.38	0.36	0.34	0.34	0.30

It can be noticed in Tables 1-2, that the three selected DMs provide coherent information: generally this is not obvious, in the present case this is due to the structure employed as test-case of which the dynamic behavior is dominated by the first natural mode.

The results synthetized in Tables 1-2 highlight the following points: a) among the IMs not based on the spectral acceleration, the *PGA* is the most efficient; b) the IMs based on spectral acceleration exhibit the higher efficiency *(CPA, S_{pa}, Cordova, EPA)*: this agrees with literature and post-earthquakes observations (Tan et al. 2012); c) between these last, the IMs carrying information about the fundamental frequency of the structure have the higher efficiency *(CPA, S_{pa}, Cordova)*; d) the here introduced *CPA* has the highest efficiency: such efficiency is robust with respect to the boundary conditions (soil type) and with respect to the range of variation of the fundamental frequency *(CPA20* vs. *CPA40)*.

CONCLUSIONS

In the present contribution, a new IM has been proposed based on the spectral acceleration of the seismic signal and is relation with the natural frequencies of the structure. Its effectiveness has been shown by means of a comparative statistical analysis of the results of non-linear dynamic simulations over a database of 4031 recorded seismic ground motions. The current study highlights that, the IMs exhibiting higher efficiency are the ones including information about the structure's fundamental frequency: IMs built only on the ground motions time history data show a lower degree of accuracy. This has a clear explication: what is important (to damage the structure) is not the total energy of the earthquake signal but the amount of energy that the earthquake signal is able to transmit to the structure. Therefore, the elastic acceleration response spectrum reveals to be an excellent base to build efficient IMs: such response quantity is extremely fast to compute (a few minutes for thousands of signals) and, with respect to other IMs, has the tremendous advantage to represent "already" the response of a structure (the simplest one, a SDOF).

The here-introduced Cumulative Pseudo Acceleration *(CPA)* exhibits the highest efficiency with respect to the other IMs. It has been shown that the *CPA* efficiency is robust with respect to the characteristics of the soil (V_{s30}) and with respect to the range of variation of frequency chosen in its definition. This robustness enables, without significant efficiency loss, to generalize the *CPA* avoiding the dependency on the single structure's dynamic characteristics: this can be done by generalizing the frequency variation's range (defining the *CPA*) to that typical of class of structures (for example 0.5 – 2.5 Hz for European residential buildings or 5.0 - 9.0 Hz for Nuclear Power Plant facilities). In the same way the range of frequencies can be widened in order to take into account higher modes for structures particularly sensible to that ones. In conclusion, due to its efficiency, robustness and simple/versatile formulation the *CPA* is a worthy candidate to be used in performance based seismic design/assessment, i.e. seismic hazard definition and fragility functions definition.

Nevertheless, the development of the *CPA* is still at the initial phase. The proposition of a clear method to select the *CPA* integration boundaries and the testing/validation of the *CPA* efficiency on multimodal structures will be the next steps of the study.

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