Prediction of Seismic Damage Using Scalar Intensity Measures Based On Integration of Spectral Values

Konstantinos G. Kostinakis, Asimina M. Athanatopoulou

***Abstract*—**A key issue in seismic risk analysis within the context of Performance-Based Earthquake Engineering is the evaluation of the expected seismic damage of structures under a specific earthquake ground motion. The assessment of the seismic performance strongly depends on the choice of the seismic Intensity Measure (IM), which quantifies the characteristics of a ground motion that are important to the nonlinear structural response. Several conventional IMs of ground motion have been used to estimate their damage potential to structures. Yet, none of them has been proved to be able to predict adequately the seismic damage. Therefore, alternative, scalar intensity measures, which take into account not only ground motion characteristics but also structural information have been proposed. Some of these IMs are based on integration of spectral values over a range of periods, in an attempt to account for the information that the shape of the acceleration, velocity or displacement spectrum provides. The adequacy of a number of these IMs in predicting the structural damage of 3D R/C buildings is investigated in the present paper. The investigated IMs, some of which are structure specific and some are non structure-specific, are defined via integration of spectral values. To achieve this purpose three symmetric in plan R/C buildings are studied. The buildings are subjected to 59 bidirectional earthquake ground motions. The two horizontal accelerograms of each ground motion are applied along the structural axes. The response is determined by nonlinear time history analysis. The structural damage is expressed in terms of the maximum interstory drift as well as the overall structural damage index. The values of the aforementioned seismic damage measures are correlated with seven scalar ground motion IMs. The comparative assessment of the results revealed that the structure-specific IMs present higher correlation with the seismic damage of the three buildings. However, the adequacy of the IMs for estimation of the structural damage depends on the response parameter adopted. Furthermore, it was confirmed that the widely used spectral acceleration at the fundamental period of the structure is a good indicator of the expected earthquake damage level.

***Keywords*—**Damage measures, Bidirectional excitation, Spectral based IMs, R/C buildings.

I. Introduction

An important phase of the performance-based seismic evaluation is the calculation of the mean annual frequency of exceeding specific structural response levels for a given structure and site. This is achieved with the aid of the Probabilistic Seismic Demand Analysis (PSDA) [1]-[3], which combines the seismic hazard for the structure considered and the response of the structure subjected to a set of seismic motions. In order to estimate the structural damage potential of an earthquake it is necessary to introduce two intermediate variables, one describing the structural performance and the other describing the ground motion intensity. A successful correlation of the aforementioned variables ensures more accurate evaluation of seismic performance and a sufficient reduction in the variability of structural response prediction. Consequently, the identification of an optimal Intensity Measure (IM), which sufficiently correlates with an appropriate Engineering Demand Parameter (EDP), is of great importance

The expected seismic performance is usually described by displacement demands, such as interstory drift as well as deformation demands in the structural elements. On the other hand, several simple-to-elaborate conventional IMs have been used to estimate the damage potential of ground motions (e.g. [4], [5]). Yet, none of them was proved to be able to predict adequately the seismic damage, since their computation is based on ground motion parameters only and ignores the special characteristics of the structure. Therefore, alternative advanced IMs have been proposed. These IMs are structure-specific, since they take into account not only ground motion characteristics but also structural information (e.g. modal vibration properties or even data from pushover curve) in order to reduce the scatter of the selected damage response parameter.

Many researchers proposed structure-specific IMs and they investigated the ability of them in predicting the structural response (e.g. [6], [7]). Fontara et al. examined the correlation between a number of advanced, structure-specific ground motion IMs and the structural damage of multistory R/C regular and irregular frames [8]. It was shown that the intensity measures which take into consideration the effects of inelastic behavior through the spectral shape indicate the strongest correlation with the structural damage for low as well as high nonlinear behavior.

However, it must be noted that all the investigations were restricted to planar R/C frames, thus they took into account only one component of the strong motion records. Modern seismic codes [9]-[13] suggest that structures shall be designed for the two horizontal translational components of ground motion (in the majority of buildings the vertical component can be neglected). In a preliminary study, Kostinakis et al. [14] investigated the adequacy of structure-specific IMs as descriptors of the seismic damage of 3D buildings under earthquake records of arbitrary direction. The research identified certain intensity measures which exhibited strong correlation with the seismic damage of the two buildings. However, their adequacy for estimation of the structural damage depends on the response parameter adopted.

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The above study was restricted to two 5-story R/C buildings under 20 pairs of accelerograms. Furthermore, only two IMs based on integration of spectral values were evaluated.

The objective of the present paper is to investigate the correlation between seven scalar ground motion IMs (four structure- and three non structure-specific) and the structural response of 3D R/C buildings. The examined IMs are based on integration of spectral values over a range of periods, attempting by this way to quantify the information that the shape of the acceleration, velocity or displacement spectrum contains. For this purpose three R/C buildings (one 3-story, one 5-story and one 8-story) are studied. The buildings' structural system consists of vertical elements in two perpendicular directions (axes X and Y).The structures, which have been designed on the basis of EC8 and EC2 provisions, are analyzed by means of Nonlinear Time History Analysis (NTHA) for 59 bidirectional strong motions. For the evaluation of the expected structural damage state of each building the Park and Ang overall structural damage index, as well as the maximum interstory drift are determined. The results show that the interdependency between the IMs and the expected seismic damage depends on the special structural characteristics and on the damage measure adopted. The structure-specific IMs exhibit higher correlation compared to the IMs which do not account for any structural information.

II. Description of the Analyses Procedure

A. *Description, Design and Modeling of the Nonlinear Behavior of the Buildings*

For the purposes of the present investigation, three double symmetric R/C buildings, with data supplied in Fig. 1 and Table I, are chosen. All three buildings have a structural system that consists of R/C frames in two orthogonal directions (axes X and Y). Along X-axis there are two R/C walls that receive approximately 60% of the base shear. According to the structural types described in EC8 [10], all buildings belong to the type of frame systems along the Y-axis and to the type of wall-equivalent dual systems along X-axis. Therefore, their horizontal stiffness along the Y-axis is roughly equal to 65% of their horizontal stiffness along the X-axis.

All three buildings are regular in plan and elevation according to the criteria set by EC8 [10] and were designed as medium ductility class (DMC) buildings. Based on the above data, the process of calculating the upper limit value of the behavior factor q of EC8 led, in all three cases, to the values $maxq_x=3$ and $maxq_Y=3.9$. However, a unique value for the behavior factor in X and Y axes was considered for the analysis and design. That is $q=\min(maxq_X, maxq_Y)=3$. In the modeling of the buildings, all basic recommendations of EC8 [10] were taken into consideration, such as the diaphragmatic behavior of the plates, the rigid zones in the joint regions of beams/columns and beams/walls, and the values of flexural and shear stiffness corresponding to cracked R/C elements. All buildings were considered to be fully fixed to the ground.

The three structures were analyzed using the modal response spectrum analysis, as described in EC8. The R/C structural elements were designed following the clauses of EC2 [15] and EC8 [10]. It should be noted that the choice of the dimensions of the structural element cross-sections as well as of their reinforcement was made bearing in mind the optimum exploitation of the structural materials (steel and concrete). Therefore, the capacity ratios (CRs) of all critical cross-sections due to bending and shear are close to 1.0. The professional computer program RA.F. Reference [16] was used for the design of the buildings. In Table II all the common design data of the examined buildings are presented. The first 6 natural periods as well as the corresponding modal participating mass ratios of all models are given in Table III.

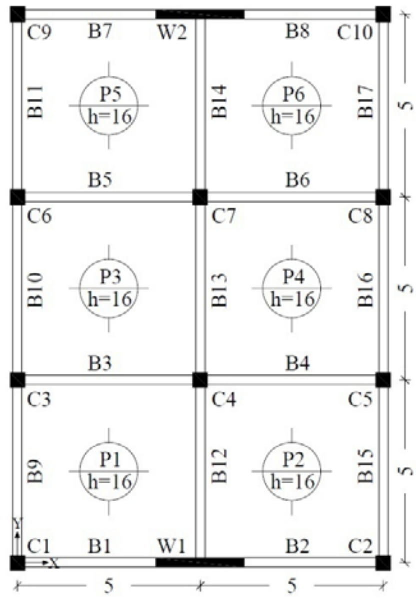


Fig. 1 Plan view and geometrical parameters of the examined buildings

For the modeling of the buildings' nonlinear behavior, plastic hinges located at the column and beam ends as well as at the base of the walls were used. The material inelasticity of the structural members was modeled by means of the Modified Takeda hysteresis rule [17]. It is important to notice that the effects of axial load-biaxial bending moment (P-M_1-M_2) interaction at column and wall hinges were taken into consideration by means of the P-M_1-M_2 interaction diagram which is implemented in the software used to conduct the analyses [18]. The yield moments as well as the parameters needed to determine the P-M_1-M_2 interaction diagram of the vertical elements' cross sections were determined using appropriate software [19].

TABLE I
DIMENSIONS OF THE CROSS SECTIONS

	3-Story Building (3SB)			5-Story Building (5SB)			8-Story Building (8SB)		
Storey	Beams	Columns	Walls	Beams	Columns	Walls	Beams	Columns	Walls
1st	25/45	35/35	115/25	25/55	40/40	150/25	25/55	45/45	160/25
2nd	25/45	35/35	115/25	25/55	40/40	150/25	25/55	45/45	160/25
3rd	20/45	30/30	115/25	25/50	35/35	150/25	25/55	40/40	160/25
4th	-	-	-	25/45	35/35	150/25	25/50	40/40	160/25
5th	-	-	-	20/45	30/30	150/25	25/50	35/35	160/25
6th	-	-	-	-	-	-	25/50	35/35	160/25
7th	-	-	-	-	-	-	25/45	30/30	160/25
8th	-	-	-	-	-	-	20/45	30/30	160/25

TABLE II
COMMON DESIGN DATA FOR THE THREE BUILDINGS

Stories' heights	Concrete	Steel	Design spectrum (EC8)
3.2m	*C20/25* $E_c=3 \cdot 10^7 kN/m^2$ $v=0.2$ $w=25kN/m^3$	*S500B* $E_s=2 \cdot 10^8 kN/m^2$ $v=0.3$ $w=78.5kN/m^3$	*Reference PGA*: $a_{gR}=0.24g$ *Importance class*: II $\rightarrow \gamma_I=1$ *Ground type*: C

TABLE III
FIRST 6 NATURAL PERIODS AND CORRESPONDING MODAL PARTICIPATING MASS RATIOS

	3SB		
Mode	Period T (sec)	X-axis (%)	Y-axis (%)
1	0.66	0	87
2	0.51	81	0
3	0.37	0	0
4	0.24	0	10
5	0.16	14	0
6	0.15	0	3
	5SB		
Mode	Period T (sec)	X-axis (%)	Y-axis (%)
1	0.89	0	80
2	0.70	76	0
3	0.51	0	0
4	0.34	0	13
5	0.23	15	0
6	0.21	0	5
	8SB		
Mode	Period T (sec)	X-axis (%)	Y-axis (%)
1	1.37	0	76
2	1.13	77	0
3	0.81	0	0
4	0.52	0	13
5	0.39	14	0
6	0.31	0	5

B. Ground Motions

A suite of 59 pairs of horizontal bidirectional far-fault earthquake excitations obtained from the PEER [20] and the European [21] strong motion database was used as input ground motion for the analyses. The seismic excitations, which have been chosen from worldwide well known sites with strong seismic activity, were recorded on Soil Type C according to EC8 [10]. The ground motion set employed was intended to cover a variety of conditions regarding tectonic environment, modified Mercalli intensity and closest distance to fault rapture, thus representing a wide range of intensities and frequency content. Another important aspect considering the selection of the seismic excitations is that they provided a wide spectrum of structural damage, from negligible to severe, to the buildings investigated in the present study.

The horizontal recorded accelerograms of each ground motion were transformed to the corresponding uncorrelated ones rotating them about the vertical axis by the angle θ_o (1) [22]. Then, the pairs of the uncorrelated accelerograms have been used as seismic input for the analyses of the structures, as ASCE 41-06 [9] proposes. The characteristics of the input ground motions are shown in Table IV along with the correlation factor of the recorded components p [22], which is given by (1):

$$p = \frac{\sigma_{xy}}{\left(\sigma_{xx} \cdot \sigma_{yy}\right)^{1/2}}, \ \tan 2\theta_o = \frac{2\sigma_{xy}}{\sigma_{xx} - \sigma_{yy}} \tag{1}$$

with

$$\sigma_{ij} = \frac{1}{t_{tot}} \cdot \left(\int_0^{t_{tot}} \alpha_i(t) \cdot \alpha_j(t) dt \right) \quad i = x, y$$

where $\alpha_x(t)$ and $\alpha_y(t)$ are the recorded ground acceleration histories along the two horizontal directions of the ground motion; σ_{xx}, σ_{yy} are the quadratic intensities of $\alpha_x(t)$ and $\alpha_y(t)$ respectively; σ_{xy} is the corresponding cross-term; t_{tot} is the duration of the motion.

C. Scalar Intensity Measures Based On Integration of Spectral Values

Several researchers proposed scalar IMs which are based on integration of spectral values over a range of periods, attempting by this way to quantify the information that the shape of the acceleration, velocity or displacement spectrum contains. In the present study both non structure-specific as well as structure-specific IMs are evaluated. The examined ground motion IMs are intended to avoid the major shortcomings associated with the widely used first-mode acceleration $S_a(T_1)$, namely ignoring both the contribution of higher modes to the overall dynamic response and the increase of the fundamental period of the structure (period elongation) associated with non-linear behavior. Therefore, all the following IMs are assessed with respect to $S_a(T_1)$ efficiency.

More specifically, the following scalar structure-specific IMs are considered:

- IM proposed by Housner [23], [24] (HI).

$$HI = \int_{0.1}^{2.5} PS_V(T,\zeta)\,dT \tag{2}$$

where PS_v is the spectrum pseudovelocity curve.

TABLE IV
GROUND MOTIONS RECORDED ON SOIL TYPE C ACCORDING TO EC8

No	Date	Earthquake name	Station name	p (%)
1	15/10/1979	Imperial Valley	Chihuahua	-18.63
2	28/06/1992	Landers	Coachella Canal	18.52
3	18/10/1989	Loma Prieta	Halls Valley	3.61
4	18/10/1989	Loma Prieta	Agnews State Hospital	15.29
5	17/01/1994	Northridge	LA - Saturn St	-6.36
6	17/01/1994	Northridge	Sun Valley - Roscoe Blvd	-3.28
7	01/10/1987	Whittier Narrows	Bell Gardens - Jaboneria	-2.13
8	01/10/1987	Whittier Narrows	El Monte - Fairview Av	22.70
9	01/10/1987	Whittier Narrows	Santa Fe Springs - E Joslin	-8.09
10	19/05/1940	Imperial Valley	El Centro Array #9	-13.03
11	28/06/1966	Parkfield	Cholame #5	-15.36
12	20/09/1999	Chi-Chi, Taiwan	TCU	-32.97
13	20/09/1999	Chi-Chi, Taiwan	TCU	-10.49
14	20/09/1999	Chi-Chi, Taiwan	TCU	-8.25
15	20/09/1999	Chi-Chi, Taiwan	TCU	-15.17
16	20/09/1999	Chi-Chi, Taiwan	TCU	-25.55
17	18/10/1989	Loma Prieta	Gilroy Array #3	4.51
18	18/10/1989	Loma Prieta	Capitola	-22.57
19	01/10/1987	Whittier Narrows	LA - Fletcher Dr	-4.19
20	07/12/1988	Spitak	Gukasian	-4.54
21	20/06/1990	Manjil (Iran)	Abhar	-33.38
22	17/08/1999	Izmit (Turkey)	Iznik-Karayollari Sefligi Muracaati	1.75
23	17/08/1999	Izmit (Turkey)	Istanbul-Zeytinburnu	5.34
24	11/09/1976	Friuli (Italy)	Buia	3.80
25	20/06/1978	Volvi (Greece)	Thessaloniki-City Hotel	16.39
26	24/02/1981	Aktion (Greece)	Korinthos-OTE Building	-28.07
27	26/09/1997	Umbria Marche (Italy)	Colfiorito	-10.76
28	12/11/1999	Duzce Turkey)	LDEO Station No. C1062 FI	12.82
29	15/10/1979	Imperial Valley	Delta	5.92
30	27/01/1980	Livermore	San Ramon - Eastman Kodak	-23.09
31	18/10/1989	Loma Prieta	Gilroy Array #4	5.98
32	18/10/1989	Loma Prieta	SF Intern. Airport	19.31
33	18/10/1989	Loma Prieta	Oakland - Title & Trust	2.85
34	18/10/1989	Loma Prieta	Sunnyvale - Colton Ave.	-9.66
35	17/01/1994	Northridge	Downey - Co Maint Bldg	-2.57
36	17/01/1994	Northridge	LA - Centinela St	-10.16
37	17/01/1994	Northridge	LA - Fletcher Dr	16.52
38	17/01/1994	Northridge	LA - N Faring Rd	-17.96
39	17/01/1994	Northridge	LA - S Grand Ave	-6.95
40	17/01/1994	Northridge	Point Mugu - Laguna Peak	2.76
41	09/02/1971	San Fernando	LA - Hollywood Stor Lot	18.12
42	01/10/1987	Whittier Narrows	Compton - Castlegate St	-36.20
43	01/10/1987	Whittier Narrows	Downey - Birchdale	-6.38
44	01/10/1987	Whittier Narrows	Downey - Co Maint Bldg	45.73
45	01/10/1987	Whittier Narrows	Lakewood - Del Amo Blvd	15.08
46	17/01/1994	Northridge	LA - Pico & Sentous	-4.68
47	24/11/1987	Superstitn Hills	Plaster City	27.34
48	24/01/1980	Livermore	San Ramon - Eastman Kodak	-34.55
49	17/01/1994	Northridge	Elizabeth Lake	-18.46
50	15/10/1979	Imperial Valley	Aeropuerto Mexicali	-6.94
51	15/10/1979	Imperial Valley	Calexico Fire Station	4.09
52	15/10/1979	Imperial Valley	EC County Center FF	-19.05
53	15/10/1979	Imperial Valley	El Centro Array #10	13.42
54	15/10/1979	Imperial Valley	Holtville Post Office	1.88
55	24/11/1987	Superstitn Hills	El Centro Imp. Co. Cent	9.45
56	24/11/1987	Superstitn Hills	Westmorland Fire Sta	8.15
57	24/04/1984	Morgan Hill	Gilroy Array #4	-36.09
58	15/10/1979	Imperial Valley	El Centro Array #11	34.00
59	24/04/1984	Morgan Hill	Halls Valley	16.11

- Velocity Spectrum Intensity (VSI) proposed by Von Thun et al. [25].

$$\mathrm{VSI} = \int_{0.1}^{2.5} S_V(T,\zeta)\,dT \tag{3}$$

where S_v is the spectrum velocity curve.

- Acceleration Spectral Intensity (ASI) proposed by Von Thun et al. [25].

$$\mathrm{ASI} = \int_{0.1}^{0.5} S_a(T,\zeta)\,dT \tag{4}$$

where S_a is the spectrum acceleration curve.

- IM proposed by Kappos [6] (IM_{Kappos}).

$$IM_{Kappos} = \int_{T_1-t}^{T_1+t} S_V(T,\zeta)\,dt \tag{5}$$

where T_1 fundamental period of the structure and $t=0.2T_1$.

- IM proposed by Matsumura [7] ($IM_{Matsumura}$).

$$IM_{Matsumura} = \frac{1}{T_y}\int_{T_y}^{2T_y} S_V(T,\zeta)\,dt \tag{6}$$

where T_y is the yield period of the equivalent SDOF system, which is determined via Pushover Analysis.

- IM proposed by Hutchinson et al. [26] ($IM_{Hutch\ et\ al}$).

$$IM_{Hutch\ et\ al} = \frac{1}{T_{sec}-T_y}\int_{T_y}^{T_{sec}} S_d(T,\zeta)\,dT \tag{7}$$

where S_d is the spectrum displacement curve and T_{sec} is the secant period of the equivalent SDOF system, which is determined via Pushover Analysis.

- IM proposed by Kadas *et al.* [27] ($IM_{Kadas\ et\ al}$).

$$IM_{Kadas\ et\ al} = \frac{1}{(T_f-T_y)a_y}\int_{T_y}^{T_f} S_a(T,\zeta)(T-T_y)\,dT \tag{8}$$

where a_y is the yield acceleration and T_f is the softened period, which is given by the following relation:

$$T_f = 1.07\cdot T_y\cdot\left[\frac{S_a(T_y)}{a_y}\right]^{0.45} \le 2.0T_y \tag{9}$$

The aforementioned IMs are determined for each one of the two horizontal components of the 59 bidirectional strong motions. However, in order to study the correlation of the IMs with the structural damage of the buildings, it is necessary to represent the intensity parameters corresponding to the two horizontal components by a single value. To achieve this, the following relations, which are common both in seismic codes and in literature for the definition of horizontal bidirectional ground motion characteristics ([9], [12], [13], [28]) are used for each seismic excitation:

- Arithmetic Mean Value (AMV):

$$IM_{AMV} = \frac{IM_1 + IM_2}{2} \tag{10}$$

- Geometric Mean Value (GMV):

$$IM_{GMV} = \sqrt{IM_1 \cdot IM_2} \tag{11}$$

- Square Root of the Sum of the Squares (SRSS):

$$IM_{SRSS} = \sqrt{IM_1^2 + IM_2^2} \tag{12}$$

- Maximum Value:

$$IM_{MAX} = \max(IM_1, IM_2) \tag{13}$$

where IM_1 and IM_2: values of the IMs determined for each one of the two horizontal components of the ground motion.

D. Non-Linear Time History Analyses - Damage Indices

The three buildings presented in Section II.*A* are analyzed by Nonlinear Time History Analysis (NTHA) for each one of the 59 earthquake ground motions taking into account the vertical loads corresponding to the seismic design combination. The analyses are performed with the aid of the computer program Ruaumoko [18]. The two uncorrelated horizontal accelerograms of each ground motion are applied along the structural axes X and Y of the buildings.

For each ground motion the damage state of the three buildings is determined. The seismic performance is expressed in the form of the following parameters: i) the Maximum Interstory Drift Ratio (MIDR) and iii) the Overall Structural Damage Index (OSDI). The aforementioned structural response parameters have been chosen, since they lump the existing damage in all the cross-sections in a single value, which can be easily correlated to scalar seismic IMs. So, they have been used by many researchers for the inelastic assessment of structures (e.g. [4], [5], [29]).

The MIDR, which is generally considered an effective indicator of global structural and nonstructural damage of R/C buildings [30] corresponds to the maximum drift among the four perimeter frames. Moreover, in the present study, the OSDI is computed as a weighted average of the local damage indices at the ends of each structural element. The dissipated energy is used as a weight factor (14) ([4], [29], [31]):

$$\mathrm{OSDI} = \sum_{i=1}^{n}\left[\mathrm{LDI}_i \cdot \left(\mathrm{E}_{Ti} \Big/ \sum_{i=1}^{n}\mathrm{E}_{Ti}\right)\right] \quad (14)$$

where LDI_i is the local damage index at cross section i (15), E_{Ti} is the energy dissipated at the cross section i and n is the number of cross sections at which the local damage is computed. For the LDI, the widely used Park and Ang damage index [32] modified by Kunnath et al. [33] has been used. At a given cross section the local damage index (LDI) is given by (15):

$$\mathrm{LDI} = \frac{\varphi_m - \varphi_y}{\varphi_u - \varphi_y} + \left(\frac{\beta}{M_y \cdot \varphi_u}\right) \cdot E_T \quad (15)$$

where φ_m is the maximum curvature observed during the load history, φ_u is the ultimate curvature capacity, φ_y is the yield curvature, E_T is the dissipated hysteretic energy, M_y is the yield moment of the cross section and β is a dimensionless constant determining the contribution of cyclic loading to damage, which is taken equal to 0.5 for the analyses conducted.

In the present study three damage degrees are defined based on the following values of OSDI [31]: 1) minor for OSDI<0.25, 2) moderate for 0.25<OSDI<0.4 and 3) severe for OSDI>0.4. The number of records which cause minor, moderate and severe damage in the examined buildings are shown in Fig. 2. We should note that no record caused elastic behavior to anyone of the three buildings.

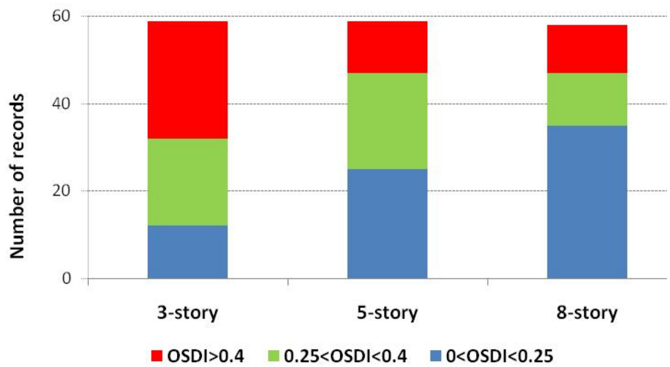


Fig. 2 Number of records corresponding to each damage degree

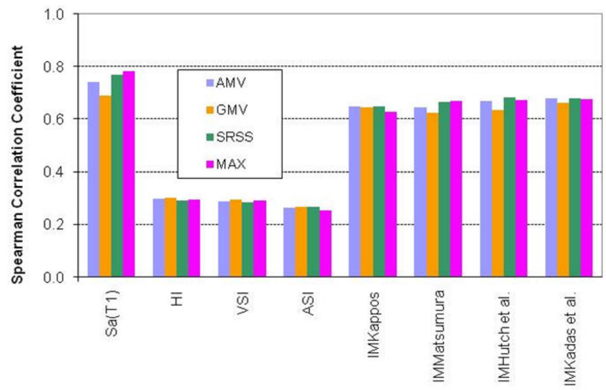
III. Comparative Assessment of the Results

Correlation coefficients between the examined ground motion IMs and the damage measures of the four buildings are determined. They express the grade of interdependency between the above mentioned parameters and they quantify the effectiveness of an IM. As a first step, the Kolmogorov–Smirnov test was used in order to identify whether the input parameters follow a normal distribution. For the selected set of ground motions, this test showed that, with a 5% error, the examined quantities do not follow the normal distribution. So, for the evaluation of the correlation between the investigated parameters, the Spearman rank correlation coefficient is adopted.

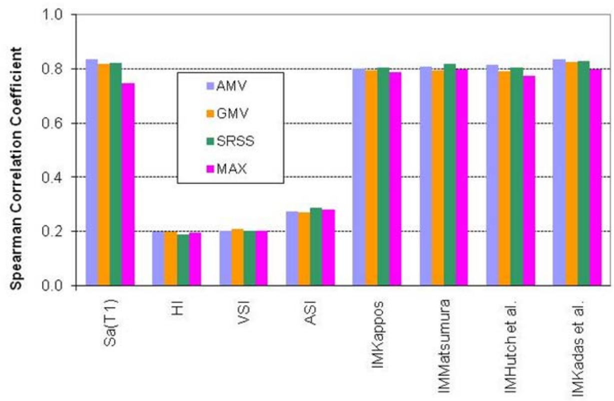
The Spearman's rank correlation coefficient is used as an index to assess how well the relationship between two variables X and Y can be described using a monotonic function. Its value ranges from −1 to 1. The values 1 and -1 indicate that each of the variables is a perfect monotone function of the other while 0 indicates no association between the ranks of the two variables. The Spearman rank correlation coefficient between two variables X and Y is given by (16):

$$\rho_{\mathrm{Spearman}} = 1 - \frac{6\sum_{i=1}^{N} D^2}{N\left(N^2 - 1\right)} \quad (16)$$

where: D are the differences between the ranks of corresponding values of X_i and Y_i and N is number of pairs of values (X,Y) in the data.



(a)



(b)

Fig. 3 Spearman correlation coefficients between IMs based on integration of spectral values and OSDI (a) or MIDR (b) for the 3-story building (3SB)

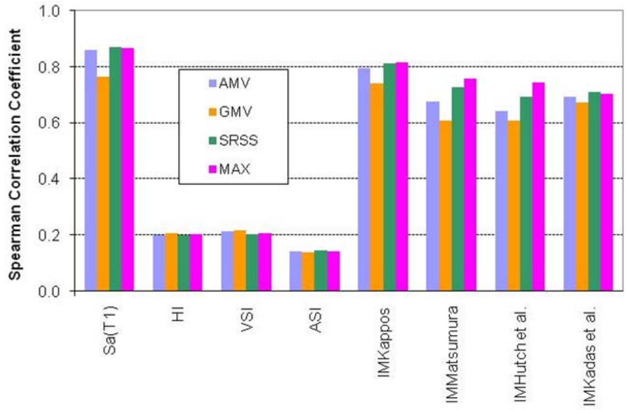
Figs. 3-5 illustrate the correlation coefficients after Spearman between the two damage indices (OSDI and MIDR) investigated in the present study and the seismic IMs considered for the three buildings. The figures show the results produced by the four different expressions used to combine in a single parameter the two separate values of IMs corresponding to the two horizontal seismic components ((10)-13)).

From these figures it can be seen that the correlation coefficients depend on the building and on the IM adopted. A comparison among the correlation coefficients produced for the three buildings under investigation reveals that they are higher for the 8-story building. This can be attributed to the fact that the 3- and 5-story buildings suffered moderate or severe damage for a large number of earthquake records (Fig. 2). On the other hand, the damage induced by the majority of the strong motions to the 8-story building was minor. As a consequence, the IMs evaluated in the present study, which approximately account for the inelastic behavior (except the IM proposed by Kadas et al. (see Section II.*C*) are able to better capture the performance of the 8-story structure (relatively to the other buildings), since it does not undergo significant inelastic response. Moreover, as it can be seen form Table III, the response of the 8SB, as well as of the 3SB and 5SB, is not dominated by higher mode effects, which can reduce the effectiveness of the IMs.

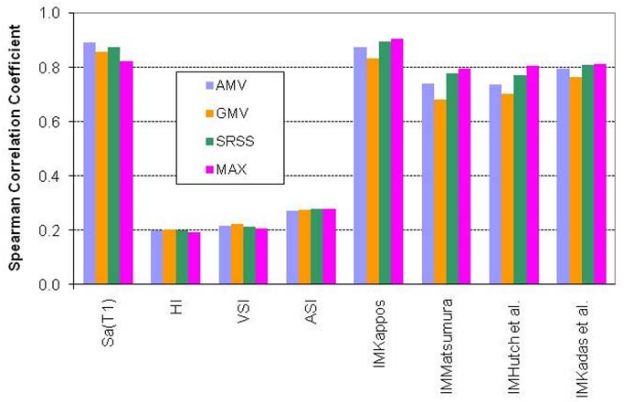
As a general observation from Figs. 3-5, we can see that the structure-specific IMs have led to significantly stronger correlation with the seismic damage of the buildings compared to the three non structure-specific IMs (HI, VSI and ASI). This observation was more or less expected, since the IMs which account for the structural characteristics are able to better capture the response of the structures (e.g. [6], [27]).

Among the four structure-specific IMs that are based on integration of spectral values it is difficult to choose a single IM as the best indicator of structural damage. The relative adequacy of the IMs as predictors of the seismic response depends on the natural period and on the damage response measure adopted (OSDI or MIDR). For example, the IM proposed by Kappos produces high values of correlation coefficients (relatively to the values corresponding to the other three IMs) for the 5-story building, but lower values for the 3-story one. The relationship between IM_{Kappos} and OSDI for these buildings is shown in Figs. 6 and 7. We can see that the dispersion is smaller for the 5-story building. Moreover, it can be seen that when the IM_{Kappos} is correlated with the OSDI of building 8SB, the Spearman's rank correlation coefficient attain smaller values compared to the other structure-specific IMs (Fig. 5 (a)). On the other hand, the adoption of the above IM for the MIDR of the same building leads to larger values of the correlation coefficients (Fig. 5 (b)).

Another significant observation concerning the four IMs that are based on integration of spectral values is that the correlation with the structural damage state is stronger when the MIDR is used as response parameter (Figs. 3-5). We can notice that, regarding the building 3SB, the values of the correlation coefficients range between 0.63 and 0.68 when the OSDI is adopted as damage measure and between 0.78 and 0.84 when the MIDR is used. Similarly, with regard to the 5-story building, the correlation coefficients attain values between 0.61 and 0.82 in the case of OSDI and between 0.68 and 0.91 when the MIDR is adopted. The difference between the values of the correlation coefficients produced using the two damage response parameters depend on the IM and the building.

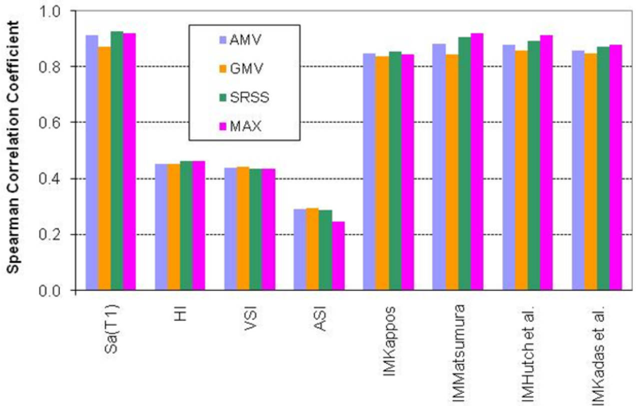


(a)

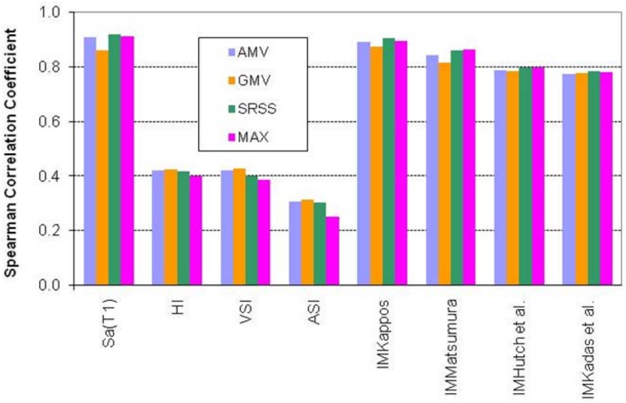


(b)

Fig. 4 Spearman correlation coefficients between IMs based on integration of spectral values and OSDI (a) or MIDR (b) for the 5-story building (5SB)



(a)



(b)

Fig. 5 Spearman correlation coefficients between IMs based on integration of spectral values and OSDI (a) or MIDR (b) for the 8-story building (8SB)

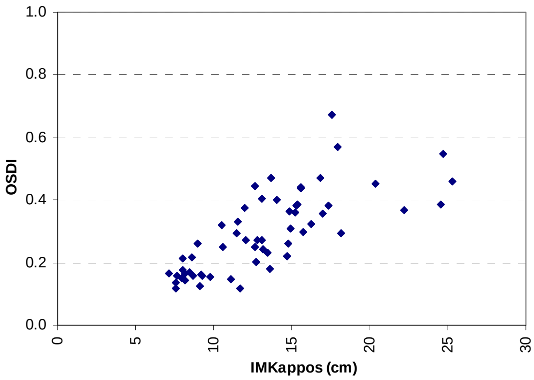


Fig. 6 Relationship between IM_{Kappos} and OSDI for the 5-story building

Of particular importance is the fact that the widely used spectral acceleration at the fundamental mode period of the structure is a relatively good predictor of the structural performance, since it shows strong enough correlation with the seismic damage. This observation, which is valid for the three buildings investigated as well as for both response indicators OSDI and MIDR, can be attributed to the fact that the performance of the three buildings is dominated by the first modes of vibration, as it can be shown from the modal participating mass ratios given in Table III. Note that the Spearman's correlation coefficient corresponding to $S_a(T_1)$ reaches the value of 0.93 when the OSDI of the 8-story building is used (Fig. 5 (a)). The relationship between $S_a(T_1)$ and OSDI for this building is shown in Fig. 8. We can see that the dispersion is small when the values of $S_a(T_1)$ range between 0.1g and 0.2g.

Of particular interest is also the fact that the influence of the definition of a single intensity parameter corresponding to the two horizontal components on the correlation coefficients between IMs and damage measures is almost negligible. The above conclusion also applies for all the IMs, damage measures and buildings considered in the present study, with the exception of the 5-story building.

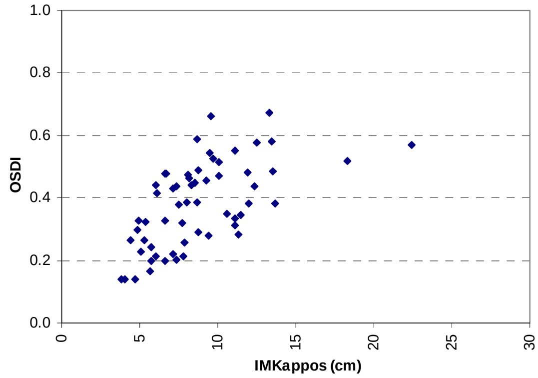


Fig. 7 Relationship between IM_{Kappos} and OSDI for the 3-story building

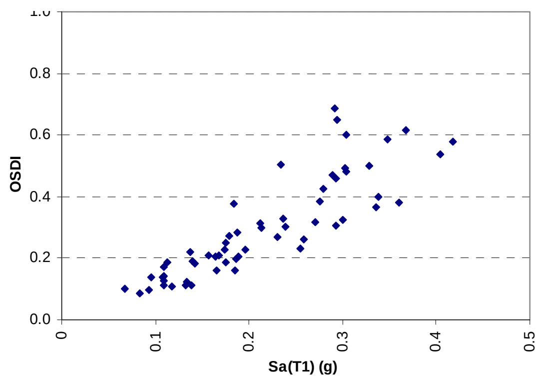


Fig. 8 Relationship between $S_a(T_1)$ and OSDI for the 8-story building

IV. Conclusion

The aim of the present paper is to examine the interdependency between the seismic damage of 3D R/C buildings and seven scalar earthquake IMs based on integration of spectral values. To achieve this, three R/C buildings (a 3-story, a 5-story and an 8-story) are investigated. The buildings are subjected to 59 bidirectional earthquake ground motions for which nonlinear time history analyses are conducted. The evaluation of the expected structural damage state of each building is made by using the Park and Ang Overall Structural Damage Index (OSDI), as well as the Maximum Interstory Drift Ratio (MIDR). The comparative assessment of the results has led to the following conclusions:

- The adequacy of the IMs for estimation of the expected seismic damage depends on the response parameter adopted and on the special building's characteristics.
- The correlation coefficients are higher in the 8-story building for the most IMs. This can be attributed to the fact that it suffered less damage compared to the other two buildings under investigation.
- The IMs that utilize in their definition characteristics of the building show stronger correlation than the IMs that are not based on any structural information.
- When the structure-specific IMs are used the correlation between them and MIDR is higher than the correlation between the IMs and OSDI.

- The widely used spectral acceleration at the fundamental mode period is a relatively good indicator of the structural damage for the three buildings under investigation irrespective of the number of stories, since it shows high correlation with OSDI and MIDR.
- The definition of a single intensity parameter corresponding to the two horizontal components has no strong impact on the correlation coefficients between the IMs and the damage measures.

It must be noted that the aforementioned conclusions are valid for the buildings and ground motions used in the present study. However they provide a good insight into the effectiveness of the spectral based IMs for 3D, R/C buildings under bidirectional excitation. In order to expand them to other structural systems, further investigation is necessary.

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References

[1] CA. Cornell and H. Krawinkler, "Progress and challenges in seismic performance assessment", PEER Center News, vol. 3, no. 2, 2000
[2] N. Shome, "Probabilistic seismic demand analysis of nonlinear structures", Ph.D. Dissertation, Department of Civil and Environmental Engineering, Stanford University, Stanford, CA, 1999
[3] FEMA-355, *State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking*, SAC Joint Venture, Sacramento, CA, 2000
[4] A. Elenas, and K. Meskouris, "Correlation study between seismic acceleration parameters and damage indices of structure", *Engineering Structures,* vol. 23, pp. 698-704, 2001.
[5] A. Yakut, and H. Yilmaz. "Correlation of deformation demands with ground motion intensity", *Journal of Structural Engineering, ASCE,* vol. 134, no. 12, pp. 1818-1828, 2008.
[6] A.J. Kappos, "Sensitivity of calculated inelastic seismic response to input motion characteristics", in *Proc. 4th U.S. National Conference on Earthquake Engineering*, Palm Springs, California. 1990, vol. 2, pp. 25-34, 1990.
[7] K. Matsumura, "On the intensity measure of strong motion related to structural failures" in *Proc. 10th WCEE*, Rotterdam, vol. I, pp. 375-80, 1992
[8] I.-K. Fontara, A. Athanatopoulou, and I. Avramidis, "Correlation between advanced, structure-specific ground motion intensity measures and damage indices", in*Proc. 15th World Conference on Earthquake Engineering*, Lisbon, Portugal, September 24-28, 2012, Paper No: 3718.
[9] ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings*, American Society of Civil Engineers, Reston, VA, 2008.
[10] Eurocode 8, *Design provisions for earthquake resistance of structures*, European Committee for Standardization, 2003.
[11] UBC Vol. 2: *Structural Engineering Design Provisions*. International Conference of Building Officials (ICBO), Whittier, CA, 1997.
[12] FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, Federal Emergency Management Agency, Washington, DC, 2000.
[13] NEHRP, *Recommended provisions for seismic regulations for new buildings and other Structures*, FEMA450, Building Seismic Safety Council, Washington, DC, 2003.
[14] K. Kostinakis, M. Papadopoulos and A. Athanatopoulou, "Adequacy of advanced earthquake intensity measures for estimation of damage under seismic excitation with arbitrary orientation", in Proc. *International Conference on Civil, Structural and Earthquake Engineering*, Paris, France, April 28-29, 2014, Paper No:214
[15] Eurocode 2, "Design of concrete structures. 1: General rule and rules for buildings", Brussels, 1991.
[16] RA.F.-Structural Analysis and Design Software v.3.3.2, TOL (Engineering Software House) Iraklion, Crete, Greece, 2012
[17] A. Otani, "Inelastic Analysis of RC frame structures", *J Struct Div (ASCE),* vol. 100, no. 7, pp. 1433–1449, 1974.
[18] A. Carr, "Ruaumoko – a program for inelastic time-history analysis, Program manual", Department of Civil Engineering, University of Canterbury, New Zealand, 2004.
[19] Imbsen Software Systems, XTRACT: Version 3.0.5. Cross-sectional structural analysis of components, Sacramento, CA, 2006.
[20] Pacific Earthquake Engineering Research Centre (PEER), Strong Motion Database. http://peer.berkeley.edu/smcat/, 2003
[21] European Strong-Motion Database, http://www.isesd.hi.is/ESD Local/frameset.htm, 2003
[22] J. Penzien, and M. Watabe, "Characteristics of 3-D Earthquake Ground Motions", *Earthquake Eng Struct Dyn*, vol. 3, pp. 365-373, 1975.
[23] GW. Housner, "Spectrum intensity of strong-motion earthquakes", in Proc. *Symposium on Earthquakes and Blast Effects on Structures*, EERI, 1952
[24] GW. Housner, "Behavior of structures during earthquakes", *Journal of the engineering mechanics division*, *(ASCE)*, vol. 85, no. EM14, pp. 109-129, 1959
[25] JL. Von Thun, LH. Rochim, GA. Scott and JA. Wilson, "Earthquake ground motions for design and analysis of dams", in Proc. *Earthquake Eng. Soil Dyn. II - Recent Advances in Ground-Motion Evaluation (Geotechnical Special Publication 20)*, ASCE, New York, pp. 463–481, 1988.
[26] TC. Hutchinson, YH. Chai, RW. Boulanger and IM. Idriss, "Estimating Inelastic Displacements for Design: Extended Pile-Shaft-Supported Bridge Structures", *Earthquake Spectra*, vol. 20, no. 4, pp. 1081–1094, 2004
[27] K. Kadas, A. Yakut and I. Kazaz, "Spectral ground motion intensity based on capacity and period elongation", *Journal of Structural Engineering, (ASCE),* vol.137, no. 3, pp.401-409, 2011
[28] K. Beyer, and J. Bommer," Relationships between median values and between aleatory variabilities for different definitions of the horizontal component of motion", *Bulletin of the Seismological Society of America*, vol. 96, no. 4A, pp. 1512–22, 2006.
[29] S.L. Dimova, and P. Negro, "Seismic assessment of an industrial frame structure designed according to Eurocodes. Part 2: Capacity and vulnerability", *Engineering Structures,* vol. 27, no. 5, pp. 724-735, 2005.
[30] F. Naeim, "The seismic design handbook", Kluwer Academic, Boston. 2nd Ed., 2001.
[31] Y.J. Park, and A.H-S. Ang, and Y.K. Wen, "Damage-limiting aseismic design of buildings", *Earthq Spectra*, vol. 3, no. 1, pp. 1-26, 1987.
[32] Y. J. Park, and A.H.-S. Ang, "Mechanistic Seismic Damage Model for Reinforced-Concrete" *J Struct Eng-ASCE*, vol. 111, no. 4, pp. 722-739, 1985
[33] S.K. Kunnath, A.M. Reinhorn, and R.F. Lobo, "IDARC Version 3: A program for the inelastic damage analysis of RC structures", Technical Report NCEER-92-0022. National Centre for Earthquake Engineering Research, State University of New York, Buffalo NY, 1992.