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CORRELATION BETWEEN STRUCTURE-SPECIFIC GROUND MOTION INTENSITY MEASURES AND SEISMIC RESPONSE OF 3D R/C BUILDINGS

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ABSTRACT

The present paper investigates the correlation between a number of structure-specific ground motion intensity measures and the damage state of 3D R/C buildings. To accomplish this purposefour 5-story R/C buildings are studied. The buildings are analyzed by nonlinear time history analysis using 20 bidirectional earthquake ground motions. The two horizontal accelerograms of each ground motion are applied along the structural axes of the buildings. The structural damage is expressed in terms of the maximum interstory drift as well as the overall structural damage index. For each individual pair of accelerograms the values of the aforementioned seismic damage measures are determined. Then, they are correlated with severalstrong motion intensity measures that take into account both earthquake and structural characteristics. The research identified certain intensity measures which exhibited strong correlation with the seismic damage of the four buildings. However, the degree of correlation between them and the seismic 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.

INTRODUCTION

An important area of research in seismic risk analysis is the evaluation of expected seismic damage of structures under a specific earthquake ground motion. 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, is of great importance.

The expected seismic performance is usually described by displacement demands, such as maximum interstory drift as well as deformation demands in the structural elements. On the other hand, several simple-to-elaborate conventional intensity measures have been used to estimate the damage potential of ground motions (e.g. Elenas and Meskouris, 2001, Yakut and Yilmaz, 2008). Yet, none of them was proved to be able to predict adequately the seismic damage of any structural system, since their computation is based on ground motion parameters only and ignores the special

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characteristics of the structure. Therefore, alternative advanced intensity measures 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. Cordova et al., 2000, Mehanny, 2009, Yahyaabadi and Tehranizadeh, 2011). Moreover, Fontara et al. (2012) examined the correlation between a number of advanced, structure-specific ground motion intensity measures and the structural damage of multistory R/C regular and irregular frames. 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 range. However, it must be noted that all the above investigations were restricted to planar R/C frames, thus accounting for only one component of the strong motion records. Modern seismic codes (ASCE/SEI 41-06, EC8, FEMA 356, NEHRP, UBC) 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).

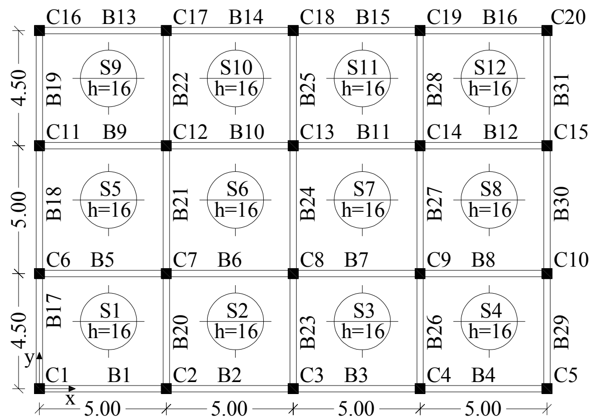
The objective of the present paper is to investigate the correlation between 9 structure-specific ground motion intensity measures and the structural response of 3D R/C buildings. For this purpose four medium-rise 3D R/C buildings are studied. All buildings have five stories and their structural systems consist of vertical elements in two perpendicular directions (axes x and y).The buildings, which have been designed on the basis of EC8 and EC2 provisions, were analyzed by means of Nonlinear Time History Analysis (NTHA) for 20 bidirectional strong motions. For the evaluation of the expected structural damage state of each building the Park and Ang overall structural damage index (Park and Ang, 1985), as well as the maximum interstory drift were 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. Moreover, the widely used spectral acceleration at the fundamental period is a relatively good indicator of the structural damage for medium-rise R/C buildings.

DESCRIPTION AND MODELING OF THE BUILDINGS ' NONLINEAR BEHAVIOR

For the purposes of the present investigation four 3D R/C buildings, with data supplied in Fig.1 and 2, are studied. All buildings have five stories and their structural systems consist of vertical elements in two perpendicular directions (axes x and y). More specifically, the following buildings are investigated:

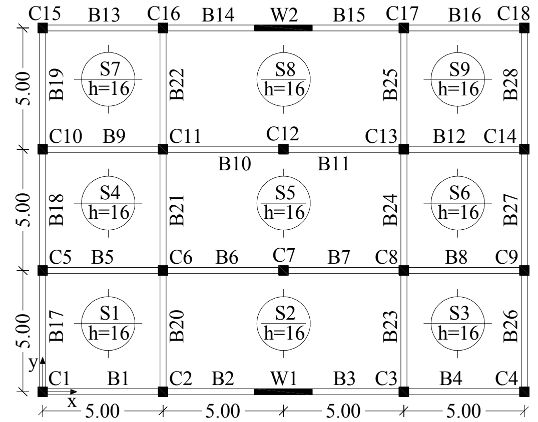
- Symmetric Frame System (according to the structural types prescribed in EC8) along both axes x and y. This building is denoted in the following as SFxy (Fig.1(a)).
- Symmetric Wall System along x axis and Frame System along y axis (according to the structural types prescribed in EC8). This building is denoted in the following as SWxFy (Fig.1(b)).
- Asymmetric Frame System (according to the structural types prescribed in EC8) along both axes x and y. This building is denoted in the following as AFxy (Fig.1(c)).
- Asymmetric Frame System along x axis and Wall System along y axis (according to the structural types prescribed in EC8). This building is denoted in the following as AFxWy (Fig.1(d)).

All buildings were designed using the assumption that they behave as medium ductility class (DMC) buildings. For the elastic modeling and design of the buildings, all basic recommendations of EC8were taken into consideration. The four 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 and EC8. 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. was used for the design of the buildings. In Table.1 all the common design data of the examined buildings are presented.



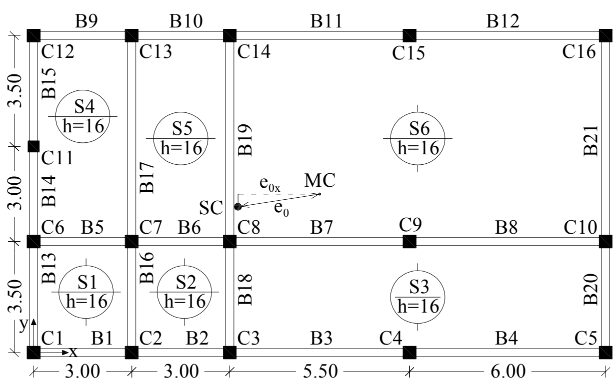
Storey	Beams	Columns
1st	25/65	45/45
2nd	25/65	45/45
3rd	25/60	40/40
4th	25/55	35/35
5th	20/45	35/35

(a)



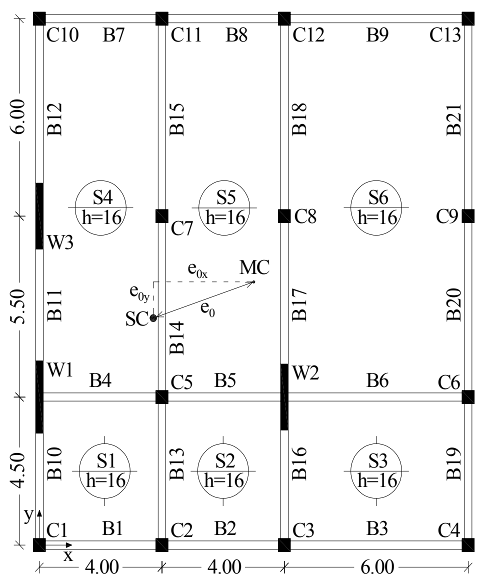
Storey	Beams	Columns	Walls
1st-2nd	25/55	40/40 (C7,C12:35/35)	W1,W2 240/25
3rd	25/50		
4th	25/50	35/35	
5th	20/45	30/30	

(b)



Storey	Beams	Columns
1st-2nd	25/60	40/40 (C11: 35/35)
3rd	25/55	35/35
4th	25/50	35/35 (C11: 30/30)
5th	20/45	30/30

(c)



Storey	Beams	Columns	Walls
1st	25/55	40/40	W1 220/25 W2,W3 200/25
2nd	25/55	40/40	
3rd	25/50	35/35	
4th	25/50	35/35	
5th	20/45	30/30	

(d)

Figure 1. 5-story buildings. Plan view and geometrical parameters(MC: Mass Centre, SC: Stiffness Centre)

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 (Otani, 1974). 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 (Carr, 2004). 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 (Imbsen Software Systems, 2006).

Table 1. Common design data for all buildings

Stories' heights	Concrete	Steel	Slab loads	Masonry loads	Design spectrum (EC8)
3.2m	C20/25 $E_c=3\cdot10^7$kN/m^2 $\nu=0.2$ w=25kN/m^3	S500B $E_s=2\cdot10^8$kN/m^2 $\nu=0.3$ w=78.5kN/m^3	Dead: G=1.0kN/m^2 Live: Q=2.0kN/m^2	Perimetric beams: 3.6kN/m^2 Internal beams: 2.1kN/m^2	Reference PGA: $a_{gR}=0.24$g Importance class: II $\rightarrow\gamma_I=1$ Ground type: C

GROUND MOTIONS

A suite of 20 pairs of horizontal bidirectional earthquake excitations obtained from the PEER (2003) and the European(2003) strong motion database is 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 and have magnitudes (M_s) between 5.5 and 7.8. 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 provide 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 (Eq.(1)) (Penzien and Watabe, 1975). Then, the pairs of the uncorrelated accelerograms have been used as seismic input for the analyses of the structures, as ASCE 41-06 proposes. The characteristics of the input ground motions are shown in Table.2 along with the correlation factor of the recorded components ρ(Penzien and Watabe, 1975), which is given by Eq.(1):

$$\rho = \frac{\sigma_{xy}}{\left(\sigma_{xx}\cdot\sigma_{yy}\right)^{1/2}},\ \tan\theta_o = \frac{2\sigma_{xy}}{\sigma_{xx}-\sigma_{yy}} \quad \text{with} \quad \sigma_{ij} = \frac{1}{t_{tot}}\cdot\left(\int_0^{t_{tot}} \alpha_i(t)\cdot\alpha_j(t)dt\right) \quad i = x, y \tag{1}$$

where $\alpha_x(t)$ and $\alpha_y(t)$ are the recorded ground accelerations along two horizontal directions, 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.

STRUCTURE-SPECIFIC GROUND MOTION INTENSITY MEASURES

In the present paper the evaluated ground motion intensity measures are determined via eigenvalue or pushover analyses. The examined IMs were proposed by researchers in an attempt to avoid the major shortcomings associated with $S_a(T_1)$; 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 advanced, structure-specific IMs are considered:

- IM proposed by Cordova et al. (2000) ($IM_{Cordova\ et\ al}$).

$$IM_{Cordova\ et\ al} = S_a(T_1) \cdot \left[\frac{S_a(2T_1)}{S_a(T_1)} \right]^{0.5} \tag{2}$$

Table 2.Ground Motions Recorded on Soil Type C according to EC8

No	Date	Earthquake name	Station name	Closest distance (Km)	ρ (%)
1	15/10/1979	Imperial Valley	Coachella Canal #4	49.3	53.33
2	17/08/1999	Kocaeli, Turkey	Cekmece	76.1	12.25
3	28/06/1992	Landers	Coachella Canal	55.7	18.52
4	18/10/1989	Loma Prieta	Halls Valley	31.6	3.61
5	18/10/1989	Loma Prieta	Agnews State Hospital	28.2	15.29
6	18/10/1989	Loma Prieta	Gilroy Array #7	24.2	-30.08
7	24/04/1984	Morgan Hill	Hollister City Hall	32.5	-15.34
8	17/01/1994	Northridge	Glendale - Las Palmas	25.4	-4.80
9	02/05/1983	Coalinga	Parkfield - Cholame 5W	47.3	-9.62
10	02/05/1983	Coalinga	Parkfield - Cholame 8W	50.7	-27.60
11	07/12/1988	Spitak	Gukasian	20	-4.54
12	17/08/1999	Izmit (Turkey)	Iznik-Karayollari Sefligi Muracaati	29	1.75
13	17/08/1999	Izmit (Turkey)	Istanbul-Zeytinburnu	80	5.34
14	27/01/1980	Livermore	San Ramon - Eastman Kodak	17.6	-23.09
15	18/10/1989	Loma Prieta	Gilroy Array #4	16.1	5.98
16	18/10/1989	Loma Prieta	SF Intern. Airport	64.4	19.31
17	18/10/1989	Loma Prieta	Sunnyvale - Colton Ave.	28.2	-9.66
18	17/01/1994	Northridge	Downey - Co Maint Bldg	47.6	-2.57
19	17/01/1994	Northridge	LA - N Faring Rd	23.9	-17.96
20	17/01/1994	Northridge	LA - S Grand Ave	36.9	-6.95

- IM proposed by Mehanny (2009) ($IM_{Mehanny}$). It must be noticed that Mehanny introduced the above structure-specific IM in an attempt to improve the adequacy of the $IM_{Cordova\ et\ al}$.

$$IM_{Mehanny} = S_a(T_1) \cdot \left[\frac{S_a(\sqrt{R} \cdot T_1)}{S_a(T_1)} \right]^{0.5} \tag{3}$$

where $R=V_e/V_y$. V_e is the lateral strength required to maintain the system elastic. It is defined for Peak Ground Acceleration 0.24g that corresponds to seismic zone II according to the Greek Seismic Code (EAK, 2003). V_y is the lateral yielding strength of the equivalent SDOF system and it is determined by Pushover Analysis.

- IM proposed by Yahyaabadi and Tehranizadeh (2011) for Non-Collapse seismic demand prediction ($IM_{Yah\ \&\ Tehr,\ NC}$).

$$IM_{Yah\ \&\ Tehr,\ NC} = \left[0.8S_d^2(T_1) + 0.2S_d^2(1.2T_1)\right]^{0.5} \tag{4}$$

where $S_d(T_1)$ is the spectral displacement for the first mode period of the structure.

- IM proposed by Yahyaabadi and Tehranizadeh (2011) for Collapse seismic demand prediction ($IM_{Yah\ \&\ Tehr,\ C}$).

$$IM_{Yah\ \&\ Tehr,\ C} = \left[0.4S_d^2(T_1) + 0.4S_d^2(1.2T_1) + 0.2S_d^2(1.6T_1)\right]^{0.5} \tag{5}$$

- IM proposed by Kappos (1990) (IM_{Kappos}).

$$IM_{Kappos} = \int_{T_{1-t}}^{T_{1+t}} S_V(T,\xi)dT \tag{6}$$

where S_v is the spectrum velocity curve, $T_1$the fundamental period of the structure and $t=0.2T_1$.

- IM proposed by Matsumura (1992) ($IM_{Matsumura}$).

$$IM_{Matsumura} = \frac{1}{T_y}\int_{T_y}^{2T_y} S_V(T,\xi)dT \tag{7}$$

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

- IM proposed by Lin et al.(2011) ($IM_{Lin\ et\ al}$).

$$IM_{Lin\ et\ al} = S_a(T_1)^{0.5} \cdot S_a(1.5T_1)^{0.5} \tag{8}$$

- IM proposed by Bojorquez & Iervolino (2011) ($IM_{Boj\ \&\ Ier}$).

$$IM_{Boj\ \&\ Ier} = S_a(T_1) \cdot \left[\frac{GMV\left(S_a(T_1 ... 2T_1)\right)}{S_a(T_1)}\right]^{0.4} \tag{9}$$

where $GMV(S_a(T_1)..S_a(2T_1))$ is Geometric Mean Value of the spectral acceleration over a range of periods between T_1 and $2T_1$.

The aforementioned IMs are determined for each one of the two components of the 20 bidirectional strong motions. However, in order to study the correlation of the IMs with the structural damage of the buildings, it was necessary to represent the intensity parameters corresponding to the two horizontal components by a single value. To achieve this, the Geometric Mean Value (GMV), which is the most widely used expression for the definition of horizontal bidirectional ground motion characteristics(Beyer and Bommer, 2006) was used for each seismic excitation:

$$IM_{GMV} = \sqrt{IM_1 \cdot IM_2} \tag{10}$$

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

DAMAGE INDICES - NON LINEAR TIME HISTORY ANALYSES

The four buildings presented above were analyzed by Nonlinear Time History Analysis (NTHA) for each one of the 20 earthquake ground motions taking into account the design vertical loads of the structures. The accelerograms of each earthquake record were applied along the structural axes x and y of the buildings. The analyses were performed with the aid of the computer program Ruaumoko (Carr, 2004).For each ground motion, the damage state of the buildings was determined. The seismic performance was expressed in the form of the following parameters: i) the Maximum Interstory Drift Ratio (MIDR) and ii) 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 intensity measures. So, they have been used by many researchers for the inelastic assessment of structures (Elenas and Meskouris, 2001, Yakut and Yilmaz, 2008, Dimova and Negro, 2005).

The MIDR, which is generally considered an effective indicator of global structural and nonstructural damage of R/C buildings (Naeim, 2001) corresponds to the maximum drift among the four perimetric frames. The values of this damage indicator have been classified according tothe European Macroseismic Scale (1998), by considering the following damage levels: 1) slight for MIDR<0.5%, 2) moderate for 0.5%<MIDR<1.0% and 3) heavy for MIDR>1.0%. The number of records which cause slight, moderate and heavy damage in the examined buildings are shown in Fig.2(a).

Moreover, in the present study, the OSDI was computed as a weighted average of the local damage indices at the ends of each structural element. The dissipated energy was used as a weight factor (Eq.(11)) (Elenas and Meskouris, 2001, Yakut and Yilmaz, 2008, Dimova and Negro, 2005, Park et al., 1987):

$$OSDI = \sum_{i=1}^{n} \left[LDI_i \cdot \left(E_{Ti} \Big/ \sum_{i=1}^{n} E_{Ti} \right) \right] \tag{11}$$

where LDI_i is the local damage index at cross section i (Ep.(12)), 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 (Park and Ang, 1985) modified by Kunnath et al. (1992) has been used. At a given cross section the local damage index (LDI) is given by Eq.(12):

$$LDI = \frac{\varphi_m - \varphi_y}{\varphi_u - \varphi_y} + \left(\frac{\beta}{M_y \cdot \varphi_u} \right) \cdot E_T \tag{12}$$

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 values of OSDI (Park et al., 1987): 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(b). We should note that no record caused elastic behavior to anyone of the four buildings.

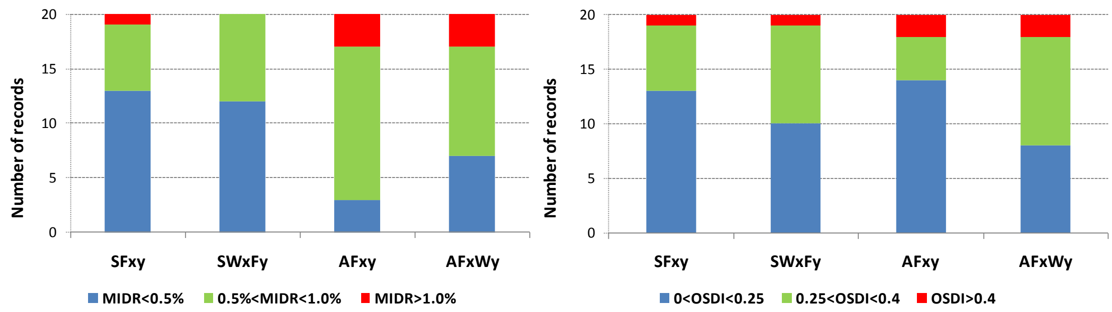


Figure 2. Number of records corresponding to each damage degree

COMPARATIVE ASSESSMENT OF THE RESULTS

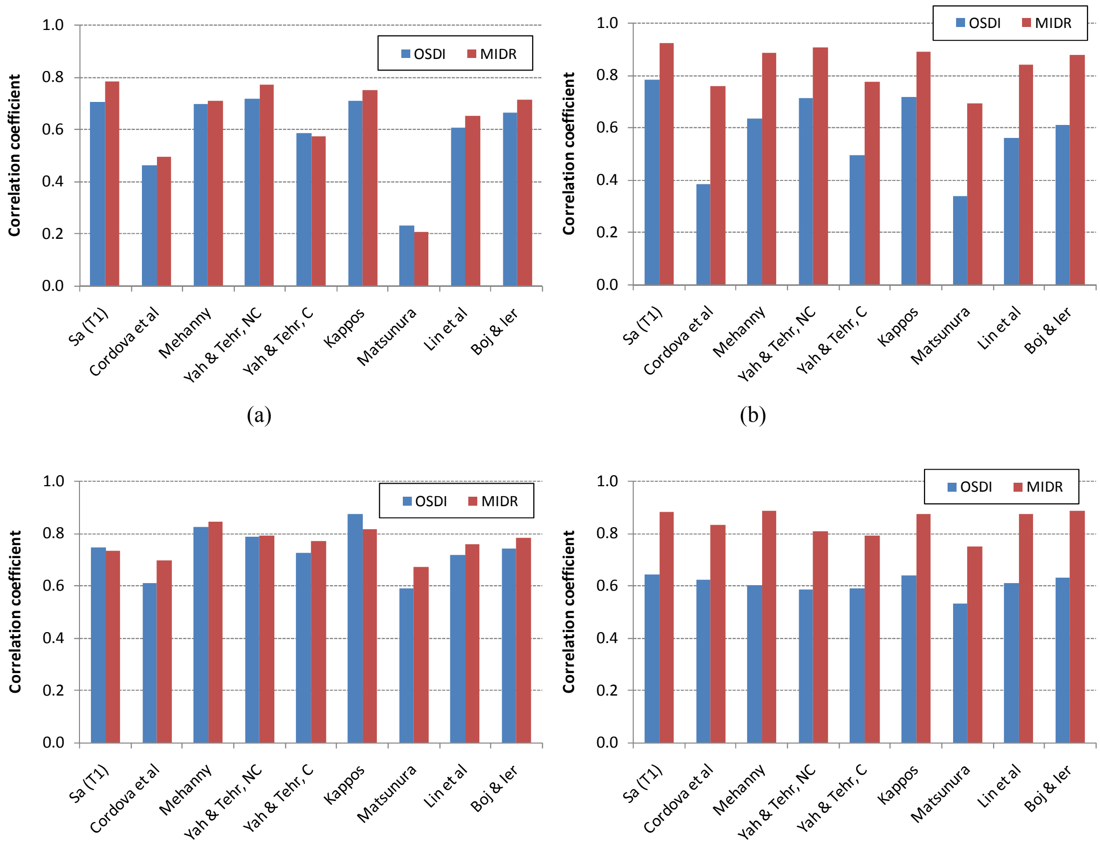
In order to evaluate the relative adequacy of the examined IMs, the correlation between the intensity measures corresponding to each ground motion and the produced damage response parameters is computed using the Pearson correlation coefficient (Eq.(13)). The Pearson correlation coefficient shows how well the data fit a linear relationship and ranges between -1 and 1.

$$p = \frac{\sum_{i=1}^{N}\left(X_i - \overline{X}\right)\cdot\left(Y_i - \overline{Y}\right)}{\sqrt{\sum_{i=1}^{N}\left(X_i - \overline{X}\right)^2 \cdot \sum_{i=1}^{N}\left(Y_i - \overline{Y}\right)^2}} \tag{13}$$

where: $\overline{X}$ and $\overline{Y}$ are the mean values of X_i and Y_i data respectively and N is the number of pairs of values X_i,Y_i in the data.

Fig.3 illustrates the correlation coefficients between the two damage indices investigated in the present study and the advanced seismic intensity measures considered for the four buildings. From the figure it is obvious that the correlation with the structural damage state is affected by the damage measure used. More specifically, we can notice that, with a few exceptions, the correlation is better when the maximum interstory drift (MIDR) is adopted as response parameter. The above observation is more intense in the case of the buildings with wall structural systems (Fig.3(b) and 2(d)). Note that, concerning building SWxFy, the values of the correlation coefficients range between 0.69 and 0.92 when the MIDR is adopted as damage measure and between 0.34 and 0.78 when the OSDI is used. Similarly, with regard to building AFxWy, the correlation coefficients attain values between 0.75 and 0.89 in the case of MIDR and between 0.53 and 0.64 when the OSDI is adopted.As an example, the relationship between $IM_{\text{Cordova et al}}$ and OSDI of building SWxFy is shown in Fig.4(a). In this case, the correlation is rather poor, since the correlation coefficient is only 0.38. On the other hand, the correlation coefficient between the same intensity measure ($IM_{\text{Cordoca et al}}$) and MIDR reaches the value of 0.76, thus indicating a medium-to-strong interdependence between the two parameters. Fig.4(b) illustrates the relationship between $IM_{\text{Cordova et al}}$ and MIDR for the building SWxFy.

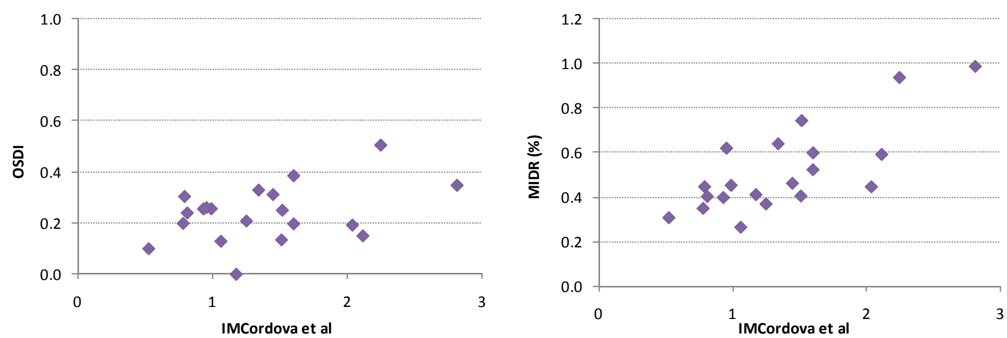
The small values of correlation coefficients in the case of OSDI of buildings SWxFy and AFxWy indicate its inadequacy to describe the expected overall damage state of these buildings in a reliable way. This can be attributed to the assumptions and the inherent uncertainties of the definition of this damage measure. In particular, the analyses showed that, for a large number of earthquakes, the damage observed in the frame-wall systems investigated was restricted to a single column or wall, although the rest structural elements remained elastic. In this case, the value of the OSDI is very large, since its computation (Eq.(11)) takes into account only the damaged cross sections and ignores the elastic frame elements, the dissipated energy in which is zero. Such a result is misleading, because in this case large OSDI indicates very significant structural damage of the whole building, whereas, in fact the damage is restricted to a single structural element only. On the contrary, the MIDR, which is based on displacements' demands, is not sensitive to the above described problem.



(a) (b)

(c) (d)

Figure 3. Pearson correlation coefficients between structure-specific IMs and damage response parameters (OSDI and MIDR) for buildings SFxy (a), SWxFy (b), AFxy (c) and AFxWy (d)



(a) (b)

Figure 4. Relationship between $IM_{Cordova\ et\ al}$ and OSDI (a) or MIDR (b)for the building SWxFy

From Fig.3we can also notice that the correlation between the damage level of the buildings and the structure-specific seismic intensity measures investigated in the present study, apart from the damage measure, depends also on the building and the intensity measure adopted, hence it is difficult to choose a single IM as the best indicator of structural damage for the four buildings. For example, the intensity measure proposed by Yahyaabadi and Tehranizadeh for Non-Collapse seismic demand prediction produces high values of correlation coefficients (relatively to the values corresponding to the other IMs) for the building SWxFy, but lower values for the building AFxWy. Moreover, it can be seen that when the IMs proposed by Cordova et al and by Matsumura are used as predictors of seismic damage of building SWxFy, the correlation coefficients attain small values (0.34-0.38 for OSDI and 0.69-0.76 for MIDR), thus indicating poor and moderate correlation with OSDI and MIDR respectively. On the other hand, the adoption of the $IM_{Mehanny}$, $IM_{Yah\ \&\ Tehr,\ NC}$ or IM_{Kappos} leads to moderate correlation with OSDI ($0.63<p<0.72$) and to strong correlation with MIDR ($0.88<p<0.91$). The influence of the choice of the IM on the correlation coefficients is smaller in the case of the asymmetric buildings (AFxy and AFxWy).

However, a careful observation of the figures leads to the conclusion that the intensity measures proposed by Mehanny, Kappos, Bojorquez and Iervolino as well as by Yahyaabadi and Tehranizadeh for Non-Collapse seismic demand prediction exhibit the highest correlation with the expected damage for the most cases. For these IMs the values of the Pearson's correlation coefficients reveal that the interdependency is moderate-to-strong in the case of MIDR (correlation coefficients range between 0.71 and 0.91) and poor-to-moderate in the case of OSDI (correlation coefficients range between 0.58 and 0.87).

Moreover, comparing the IMs proposed by Yahyaabadi and Tehranizadeh for Non-Collapse and for Collapse demand prediction, it can be seen that the first one leads to larger correlation coefficients. This observation can be explained on the basis of Fig.2. From this figure it is obvious that the vast majority of the earthquake records used in the present study produced minor or moderate damage to the four buildings and only a very small number of them led to severe damage or collapse, thus indicating that $IM_{Yah\ \&\ Tehr,\ NC}$ is more appropriate to describe the seismic damage of the structures than $IM_{Yah\ \&\ Tehr,\ C}$. Furthermore, Fig.3 shows that the structural damage level (MIDR and OSDI) exhibited stronger correlation with the IM introduced by Mehanny than with the one proposed by Cordova et al. (with the exception of OSDI of the building AFxWy (Fig.3(d))). This was expected since $IM_{Mehanny}$ is an improved version of the $IM_{Cordova\ et\ al}$.

Of particular importance is also the fact that the widely used spectral acceleration at the fundamental mode period of the structure $S_a(T_1)$ is a good predictor of the structural damage, since it produces values of correlation coefficients which are large relatively to the values corresponding to the other IMs. This observation is valid for the four buildings investigated as well as for both OSDI and MIDR. Note for example that the Pearson's correlation coefficient reaches the value of 0.89 when the MIDR of building AFxWy is used (Fig.3(d)). The relationship between $S_a(T_1)$ and MIDR for the asymmetric frame-wall system is shown in Fig.5.

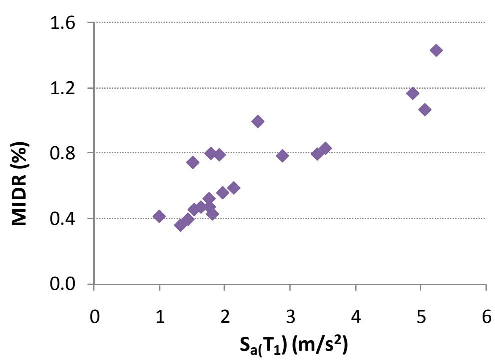


Figure 5. Relationship between $S_a(T_1)$ and MIDR for the building AFxWy

CONCLUSIONS

The aim of the present paper is to examine the interdependency between 9 structure-specific ground motion intensity measures and the seismic damage of 3D R/C buildings. To achieve this, four medium-rise R/C buildings with different structural systems are investigated. The buildings aresubjected to 20 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 correlation between the IMs and the expected seismic damage depends on the damage measure (MIDR and OSDI) adopted as well as on the special building's characteristics.
- The correlation between IMs and MIDR is better than the correlation between IMs and OSDI.
- The IMs that show the strongest correlation with the structural damage level of the four buildings investigated are those proposed by Mehanny, Kappos, Bojorquez and Iervolino, as well as by Yahyaabadi and Tehranizadeh for Non-Collapse seismic demand prediction.
- The widely used spectral acceleration at the fundamental mode period is a relatively good indicator of the structural damage for medium-rise R/C buildings, since it shows high correlation with OSDI and MIDR for the most cases.

It must be noted that the aforementioned conclusions are valid for the buildings and ground motions used in the present study. In order to expand them to other structural systems, further investigation is necessary.

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