Correlation Between Advanced, Structure-Specific Ground Motion Intensity Measures and Damage Indices

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SUMMARY:
The present paper investigates the correlation between a number of advanced, structure-specific ground motion intensity measures and the structural damage to multistorey r/c regular and irregular frames. The examined intensity measures are determined via eigenvalue and pushover analyses and they reflect the effects of inelastic behavior. The structural damage is expressed by means of: i) the overall structural damage index, ii) the peak roof drift ratio and iii) the peak interstorey drift ratio. Nonlinear dynamic analyses for thirty three ground motions and three intensity levels are performed. The results show that the overall structural damage index seems to be the more appropriate engineering demand parameter to correlate with ground motion intensity measures. Furthermore, 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. This is valid for regular as well as irregular frames.

Keywords: non-linear response, ground motion intensity, structure-specific intensity measure, damage indices

1. INTRODUCTION

One of the objectives in Performance-Based Seismic Design (PBSD) is to estimate the mean annual frequency of exceeding specified limit states for a given structure and site. In order to estimate this frequency it is necessary to introduce two intermediate variables, one describing the structural demand and the other describing the ground motion intensity measure. A successful correlation of the aforementioned variables ensures more accurate evaluation of seismic performance and in particular a sufficient reduction in the variability of structural response prediction. Consequently, the development of an optimal intensity measure (IM), which sufficiently correlates with an appropriate engineering demand parameter (EDP), is of great importance for a successful PBSD. Optimal was defined as being practical, sufficient, effective, efficient and robust (Mackie and Stojadinovic, 2005).

Several researchers developed advanced, structure-specific intensity measures that incorporate not only ground motion characteristics (elastic or inelastic spectral intensity) but also structural information (e.g. modal vibration properties or even data from pushover curve) in order to reduce the scatter of the selected EDP. Note that a successful correlation between the IM and EDP depends also on the selection of an appropriate EDP, which should be a reliable indicator of the structural damage.

Several studies investigated the efficiency of advanced, structure-specific ground motion intensity measures. Luco et al (2005) studied the correlation between several ground motion intensity measures and a vector consisting of various response measures. They studied three 3D-frames using multivariate multiple linear regression. The examined ground motion parameters account for the effects of inelasticity, higher modes of vibration and energy considerations. The results of this study demonstrated that a vector ground motion parameter that includes higher-mode elastic spectral acceleration and first-mode inelastic spectral acceleration in addition to the first-mode elastic spectral acceleration correlates better with nonlinear structural response than $S_a(T_1)$ alone. Another study carried out by Lucchini et al. (2011) examined the correlation of five different ground motion intensity measures with the peak interstorey drift ratio and the peak roof drift ratio of an in-plan irregular three-dimensional building subjected to bi-directional earthquake motion. They observed that the two-parameter scalar intensity measure given as a combination of the pseudo-accelerations at the first elastic period and the damage-elongated first period (Cordova et al. 2000) indicates an improved prediction of structural damage. Furthermore, the above mentioned vector of intensity measures including spectral accelerations at higher periods (Luco et al. 2005) was found to improve the prediction capability though it is not account for the torsional-traslational coupling and the effects of nonlinearity. It is noted that all the above investigations have used drifts or drift ratios as engineering demand parameters for predicting the correlation with the examined intensity measures.

The objective of the present paper is to investigate the correlation between eight structure-specific ground motion intensity measures with the following EDP: i) the overall damage index, ii) the roof drift ratio and iii) the peak inter-storey drift ratio to multi storey reinforced concrete planar frames. For this purpose two R/C frames are analyzed by nonlinear dynamic analyses for thirty three ground motions and three intensity levels.

2. CASE STUDY

2.1. Ground Motions

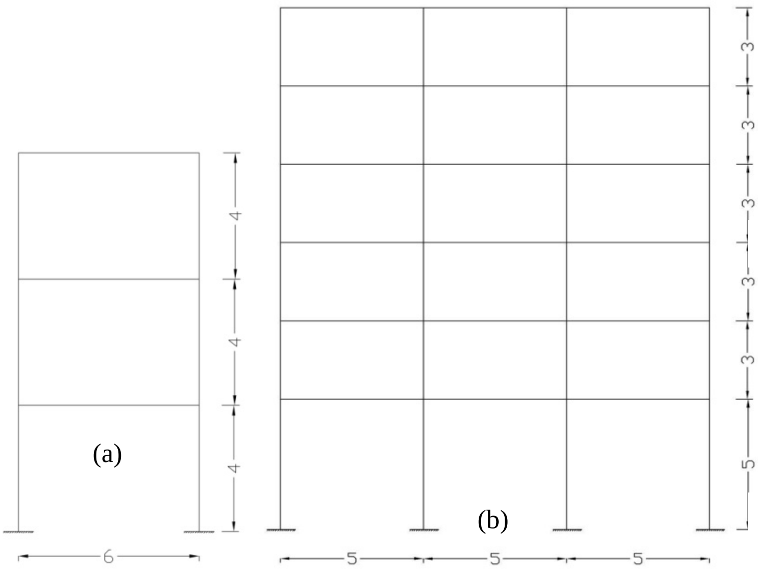
A suite of thirty three earthquake records is obtained from the PEER strong motion database according to magnitude, closest distance to fault rupture and site class. In particular, ground motions are selected to fall into the following bins: M_W=[5.7, 7.30], R_{rup}=[6, 54.10] (M_W: seismic magnitude, R_{rup} (km): closest distance to fault rupture) and recorded on site class C and D in accordance to FEMA classification. Attempt was made to avoid directivity effects. Note that for initial studies of ground motions intensity measures, it is desirable to have earthquake ground motions with broad bandwidth to the values of intensity measure. Thus, as it is shown in Table 2.1 the records are selected to have a range of magnitude and distance values.

2.2. Description of Structures

The two reinforced concrete planar frames shown in Fig. 2.1 are investigated in this paper. The first one is a three storey regular and the second one is a six storey irregular frame. The design is performed on the basis of the Greek Code for the Design and Construction of Concrete Works and the seismic analysis is conducted according to Greek Seismic Code (EAK) for seismic zone I (0.16g) and site class B, which corresponds to site class D according to FEMA. Detailed descriptions of the examined structures can be found in Tsiggelis (2009) and Manoukas (2010).

The features of the examined frames are given in Table 2.2, where the periods (T_i), the modal participation ratios (Γ_i) and the modal mass contribution factors (s_i) for the first two modes as well as the yield displacement (d_y) associated with the first mode are listed. The yield displacement along with all the data obtained from a roof-displacement versus base shear curve is determined via pushover analyses of the structures under consideration. Note that all the above information of the structures is used to calculate the intensity measures described in the following sections.

The reinforced concrete frames are designed for gravity and earthquake loads. The seismic response is computed by the response spectrum method. Plastic hinges are assigned at the ends of the frame elements. Nonlinear dynamic analyses are carried out for the thirty three ground motions presented in Table 2.1 for three seismic intensity levels. Particularly, each ground motion is multiplied by a factor equal to 0.8, 1.0 and 1.5 in order to produce three different levels of inelastic response. The nonlinear analyses are performed by the aid of computer program Ruaumoko 3D (Carr, 2004).



Columns	35/35(cm)
Beams	25/50(cm)
E	32GPa
C30 (f_{ck}=30MPa)	
S500 (f_{yk}=500MPa)	

(a) 3-storey-frame

Columns	50/50(cm)
Beams	25/40(cm)
E	27.5GPa
C16 (f_{ck}=16MPa)	
S400 (f_{yk}=400MPa)	

(b) 6-storey-frame

Figure 2.1 (a) 3-storey regular frame and (b) 6-storey irregular frame

2.3. Engineering Demand Parameters

In this study three EDPs are evaluated; peak interstorey drift ratio (IDR), roof drift ratio (RDR) and overall structural damage index (OSDI). All the above EDPs express the inelastic structural behaviour. In particular, peak interstorey drift ratio is the peak interstorey drift over all stories normalized by storey height and roof drift ratio is the peak lateral roof displacement divided by the building height. The damage index estimates quantitatively the degree of seismic damage that a cross-section as well as a whole structure has suffered. In general, damage index is a quantity with zero value when no damage occurs and a value equal to 1 when failure or collapse occurs. However, the damage index referring to the whole structure may exceed the value of 1 (Park and Ang, 1985). In the present paper, the modified Park and Ang [1985] damage index, given by Eqn 2.1, has been used:

$$\mathrm{DI} = \frac{\varphi_m - \varphi_y}{\varphi_u - \varphi_y} + \frac{\beta}{M_y \cdot \varphi_u} \cdot E_T \qquad (2.1)$$

where DI is the local damage index, φ_m the maximum curvature attained during the load history, φ_u the ultimate curvature capacity of the section, φ_y the yield curvature, β a strength degrading parameter, M_y the yield moment of the section and E_T the dissipated hysteric energy. Eqn. 2.1 gives the local damage index (cross-section damage). This research addresses the overall structural damage index (OSDI) computed as the mean value of all stories damage indices weighted by the energy absorption of all storeys (Eqn. 2.3.). The storey damage index (DI_{storey}) is calculated as the average value of the local damage indices at each storey weighted by the sum of the local energy absorptions (Eqn. 2.2).

$$\mathrm{DI}_{storey} = \frac{\sum_{i=1}^{n}[\mathrm{DI}_{col,i} \cdot E_{col,i}] + \sum_{i=1}^{m}[\mathrm{DI}_{beam,i} \cdot E_{beam,i}]}{\sum_{i=1}^{n} E_{col,i} + \sum_{i=1}^{m} E_{beam,i}} \qquad (2.2)$$

$$\mathrm{OSDI} = \frac{\sum_{i=1}^{N}[\mathrm{DI}_{storey,i} \cdot E_{storey,i}]}{\sum_{i=1}^{N} E_{storey,i}} \qquad (2.3)$$

where $D.I._{col,i}$ is the column damage index, $D.I._{beam,i}$ the beam damage index, E the dissipated energy, N the number of stories and n, m the number of columns and beams of the storey respectively. Since the locations having high damage indices will also be the ones which absorb large amounts of energy, the weighted damage index assigns a higher weight to the more heavily damaged members. Thus, to a first approximation, the weighted damage index reflects the state of the most heavily damaged members.

Table 2.1. Data of Earthquake Records

No	Event	Year	Station Name	Magnitude (Mw)	Closest distance to fault rupture (Km)	s (sec)
1	Loma Prieta	1989	Agnews State Hospital	6.9	28.2	40
2	Loma Prieta	1989	Aderson Dam(Downstream)	6.9	21.4	39.605
3	Northridge	1994	Arleta - Nordhoff Fire Sta	6.7	9.2	40
4	Landers	1992	Coolwater	7.3	21.2	27.965
5	Landers	1992	Desert Hot Springs	7.3	23.2	50
6	Cape Mendocino	1992	Fortuna - Fortuna Blvd	7.1	23.6	44
7	Imperial Valley	1979	Aeropuerto Mexicali	6.5	8.5	11.15
8	Imperial Valley	1979	Agrarias	6.5	12.9	28.36
9	Imperial Valley	1979	Chihuahua	6.5	28.7	40
10	Imperial Valley	1979	El Centro Array #3	6.5	21.9	39.505
11	Imperial Valley	1979	Hotville Post Office	6.5	7.5	37.745
12	Coalinga	1983	Pleasant Valley P.P. -yard	6.4	8.5	39.96
13	Loma Prieta	1989	Hollister - South and Pine	6.9	28.8	59.955
14	Imperial Valley	1940	El Centro Array #9	7	8.3	40
15	Landers	1992	Joshua Tree	7.3	11.6	44
16	Landers	1992	North Palm Springs	7.3	24.2	70
17	N Palm Springs	1986	Palm Springs Airport	6	16.6	30
18	Northridge	1994	Sun Valley-Roscoe Blvd	6.7	12.3	30.28
19	Loma Prieta	1989	Sunnyvale - Colton Ave.	6.9	28.8	39.25
20	Landers	1992	Yermo Fire Station	7.3	24.9	44
21	Borrego	1942	El Centro Array #9	6.5	49	40
22	Coalinga	1983	Parkfield- Cholame 8W	6.4	50.7	32
23	Coalinga	1983	Parkfield - Gold Hill 1W	6.4	46.5	40
24	Coalinga	1983	Parkfield - Fault Zone 3	6.4	36.4	40
25	Imperial Valley	1979	Compuertas	6.5	32.6	36
26	Imperial Valley	1979	Victoria	6.5	54.1	40
27	Coyote Lake	1979	Gilroy Array #2	5.7	7.5	26.86
28	Coyote Lake	1979	Gilroy Array #3	5.7	6	26.805
29	Coyote Lake	1979	San Juan Bautista	5.7	15.6	28.46
30	Livermore	1980	Antioch - 510 G St	5.8	20.3	39.995
31	Whittier Narrows	1987	Arcadia - Campus Dr	6	12.2	34.92
32	Whittier Narrows	1987	LA - 116th St School	6	22.5	39.995
33	Whittier Narrows	1987	Carson - Water St	6	24.5	29.7

Table 2.2. Features of Frames

Frame	Number of bays	Storey height	T_1 (sec)	T_2 (sec)	T_{eff} (sec)	mass storey (t)	Γ_1	Γ_2	s_1	s_2	d_y (cm)
3-storey regular	1	4m	0.89	0.28	1	30	1.25	-0.32	0.89	0.11	6
6-storey irregular	3	5m 1st storey 3m 2st-6th storey	0.74	0.22	1.07	30	1.27	-0.40	0.88	0.09	3.2

In the present study four damage degrees are defined based on the values of OSDI: 1) low for OSDI<0.11, 2) medium for 0.11<OSDI<0.4, 3) large for 0.4<OSDI<0.77 and 4) total for 0.77<OSDI. The number of records which cause low, medium, large and total damage for the three seismic intensity levels are shown in Figure 2.2. We should note that the records that cause elastic behavior to the 3-storey frame are not taken into consideration for the correlation with the examined IMs.

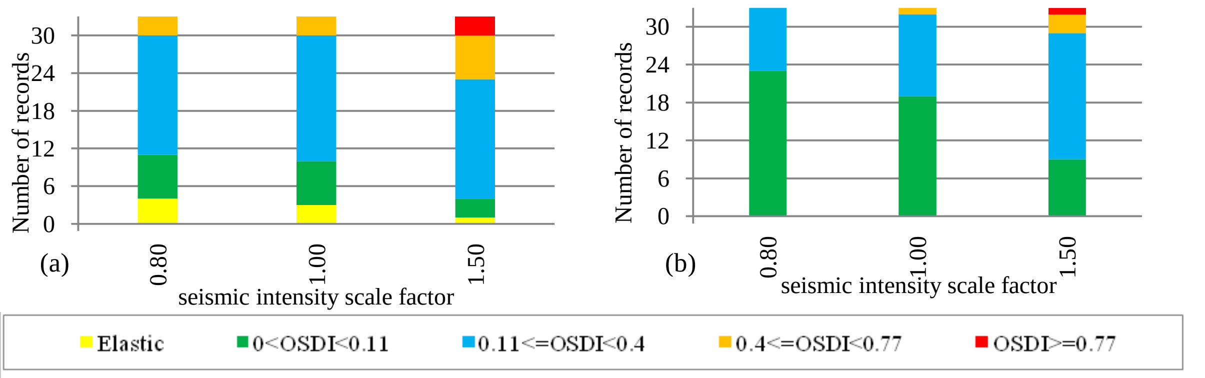


Figure 2.2 Number of records corresponding to each damage degree.
(a) 3-storey regular frame and (b) 6-storey irregular frame

2.4. Intensity Measures

In the present paper the evaluated ground motion intensity measures are multiplicative modifications of the pseudo-acceleration corresponding to the first mode period of the structure ($Sa(T_1)$) and they are determined via eigenvalue or pushover analyses. The examined IMs were proposed by researchers in an attempt to avoid the major shortcomings associated with $Sa(T_1)$; 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 behaviour. Therefore, all the following IMs are assessed with respect to $Sa(T_1)$ efficiency.

Cordova et al. (2000) proposed a two-parameter scalar IM that reflects both spectral intensity and spectral shape, thus it accounts for the period elongation. It is formulated as follows:

$$IM_{Cordova} = Sa(T_1) \cdot \left[\frac{Sa(c \cdot T_1)}{Sa(T_1)}\right]^{\alpha}, c = 2 \text{ and } \alpha = 0.5 \quad (2.4)$$

Furthermore, Mehanny (2009) built on and generalized the above IM by substituting the multiplier c in Eqn. 2.4. by a self-adaptive multiplier to the fundamental period of the structure which is a function of the relative lateral strength ($R=V_e/V_y$, where V_e and V_y are determined by pushover analysis). For the here examined structures, for which the Equal Displacement Rule does hold true, a multiplier equal to $\sqrt{R}$ is used ($c=\sqrt{R}$). In order to examine various earthquake scenarios and the corresponding levels of nonlinear demand of the structures, the design spectrum of Greek Seismic Code for seismic zone I (0.16g), II (0.24g) and III (0.36g) is considered. We should recall that the structures under consideration were designed for seismic zone I. The corresponding R values associated with each earthquake scenario are presented in Table 2.3.

Table 2.3. R values corresponding to three different earthquake scenarios

R	seismic zone		
	I(0.16g)	II(0.24g)	III(0.36g)
3-storey regular	1.23	1.85	2.78
6-storey irregular	2.54	3.82	5.73

Luco and Cornell (2007) focused on an alternative IM that is intended to reflect both the contribution of the second mode (in addition to the first one) and the effects of nonlinearity as well (Eqn. 2.5).

$$\mathrm{IM}_{\mathrm{Luco}\ \&\ Cornell} = \frac{\mathrm{S}_\mathrm{d}^\mathrm{I}(\mathrm{T}_1,\zeta_1,\mathrm{d}_\mathrm{y})}{\mathrm{S}_\mathrm{d}(\mathrm{T}_1,\zeta_1)} \cdot \mathrm{IM}_{1\mathrm{E}\&2E} \tag{2.5}$$

$$\mathrm{IM}_{1\mathrm{E}\&2E} = \sqrt{[\mathrm{PF}_1^{[2]} \cdot \mathrm{S}_\mathrm{d}(\mathrm{T}_1,\zeta_1)]^2 + [\mathrm{PF}_2^{[2]} \cdot \mathrm{S}_\mathrm{d}(\mathrm{T}_2,\zeta_2)]^2}$$

where $S_d(T_1,\zeta_1)$ and $S_d(T_2,\zeta_2)$ is the spectral displacement of the ground motion for the first- and the second-mode period of the structure respectively, $S^I{}_d(T_1,\zeta_1,d_y)$ the inelastic spectral displacement and PF_1 and PF_2 the model structure's first- and second-mode participation factor for maximum peak storey drift angle.

Yahyaabadei and Tehranizadeh (2011) proposed an improved IM based on optimal combinations of spectral values at elastic and inelastic elongated modal periods for non-collapse (Eqn. 2.6) and collapse (Eqn. 2.7) capacity prediction.

$$\mathrm{IM}_{\mathrm{Yahyaab}\ .\&Terh.,NC} = (\Gamma_1^2 \cdot \left[0.8 \cdot \mathrm{S}_\mathrm{d}^2(\mathrm{T}_1) + 0.2 \cdot \mathrm{S}_\mathrm{d}^2(1.2 \cdot \mathrm{T}_1)\right] + \Gamma_2^2 \mathrm{S}_\mathrm{d}^2(\mathrm{T}_2))^{0.5} \tag{2.6}$$

where $\Gamma_\mathrm{i} = \mathrm{PF}_\mathrm{i} \times \mathrm{ID}_\mathrm{i}$

$$\begin{aligned}\mathrm{IM}_{\mathrm{Yahyaab}\ .\&Tehr.,C} = {} & (\Gamma_1^2 \cdot \left[0.4 \cdot \mathrm{S}_\mathrm{d}^2(\mathrm{T}_1) + 0.4 \cdot \mathrm{S}_\mathrm{d}^2(1.2 \cdot \mathrm{T}_1) + 0.2 \cdot \mathrm{S}_\mathrm{d}^2(1.6 \cdot \mathrm{T}_1)\right] + \\ & \Gamma_2^2 \mathrm{S}_\mathrm{d}^2(\mathrm{T}_2))^{0.5}\end{aligned} \tag{2.7}$$

where $S_d(T_1)$ and $S_d(T_2)$ is the spectral displacement for the first- and the second-mode period of the structure respectively, PF_i is the i^{th}-mode participation factor and ID_i is the i^{th}-mode interstorey drift that corresponds to the storey at which the parameter A is maximized.

$$\mathrm{A} = \sqrt{[\mathrm{PF}_1 \cdot \mathrm{ID}_1 \cdot \mathrm{S}_\mathrm{d}(\mathrm{T}_1)]^2 + [\mathrm{PF}_2 \cdot \mathrm{ID}_2 \cdot \mathrm{S}_\mathrm{d}(\mathrm{T}_2)]^2}$$

Kappos (1990) in an attempt to refine the definition of spectrum intensity introduced by Housner, proposed a modified velocity spectrum IM defined as:

$$\mathrm{IM}_{\mathrm{Kappos}} = \int_{\mathrm{T}_1-\mathrm{t}}^{\mathrm{T}_1+\mathrm{t}} \mathrm{S}_\mathrm{v}(\mathrm{T},\zeta)\mathrm{dt} \tag{2.8}$$

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

Matsumura (1992) developed a new velocity spectrum IM given by:

$$\mathrm{IM}_{\mathrm{Matsumura}} = \frac{1}{\mathrm{T}_\mathrm{y}} \int_{\mathrm{T}_\mathrm{y}}^{2\mathrm{T}_\mathrm{y}} \mathrm{S}_\mathrm{v}(\mathrm{T},\zeta)\mathrm{dt} \tag{2.9}$$

where T_y is the yield period of the structure.

Martinez-Rueda (1996) introduced an alternative velocity spectrum IM defined as:

$$\mathrm{IM}_{\mathrm{Martinez\ -Rueda}} = \frac{1}{\mathrm{T}_\mathrm{y}-\mathrm{T}_1} \int_{\mathrm{T}_1}^{\mathrm{T}_\mathrm{y}} \mathrm{S}_\mathrm{v}(\mathrm{T},\zeta)\mathrm{dt} \tag{2.10}$$

The adopted integration intervals in the Matsumura IM $[T_y\text{-}2T_y]$ and in the Martinez-Rueda IM $[T_1\text{-}T_y]$ are associated with the resonance of the structure linked to excitation frequencies close to the natural frequency of the structure.

3. CORRELATION STUDY 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 EDPs, is computed using the Pearson correlation coefficient (Eqn. 3.1). The Pearson correlation coefficient shows how well the data fit a linear relationship and ranges between -1 and 1.

$$\rho = \frac{\sum_{i=1}^{N}(X_i - \overline{X})\cdot(Y_i - \overline{Y})}{\sqrt{\sum_{i=1}^{N}(X_i - \overline{X})^2 \cdot \sum_{i=1}^{N}(Y_i - \overline{Y})^2}} \tag{3.1}$$

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.

Tables 3.1, 3.2, and 3.3 present the correlation coefficients between the EDPs under consideration and the evaluated IMs for seismic intensity scale factor 0.8, 1.0 and 1.5 respectively. Table 3.4 presents the correlation coefficients of the above mentioned parameters but only for the ground motion records that produce high nonlinearity (OSDI>0.3) to the examined structures. Note that the Romanian numerals next to Mehanny denote the three different earthquake scenarios that were investigated (I[0.16g], II[0.24g] and III[0.36g]) and the letters next to Yahyaabadei and Tehranizadeh represent the non-collapse (NC) and the collapse (C) structural performance state.

Observe that the overall structural damage index (OSDI) exhibit in general better correlation with the IMs than the roof drift ratio (RDR) and the peak interstorey drift ratio (IDR). Furthermore, the RDR correlates better with the IMs than the IDR. Consequently, the OSDI appears to be the appropriate engineering demand parameter to correlate with ground motion intensity measures.

Table 3.1. Correlation coefficients between Overall Structural Damage Index (OSDI), Roof Drift Ratio (RDR) and Peak Interstorey Drift Ratio (IDR) and the evaluated IMs for scale factor 0.8.

3-storey regular	OSDI	RDR	IDR
$Sa(T_1)$	0.96	0.95	0.87
Cordova et al.	0.92	0.88	0.91
Mehanny I	0.98	0.97	0.91
Mehanny II	0.95	0.94	0.87
Mehanny III	0.95	0.94	0.89
Luco & Cornell	0.92	0.94	0.96
Yah. & Tehr. NC	0.98	0.97	0.90
Yah. & Tehr. C	0.96	0.95	0.90
Matsumura	0.93	0.92	0.88
Martinez-Rueda	0.97	0.96	0.91
Kappos	0.98	0.97	0.88

6-storey irregular	OSDI	RDR	IDR
$Sa(T_1)$	0.93	0.90	0.72
Cordova et al.	0.98	0.96	0.83
Mehanny I	0.97	0.94	0.77
Mehanny II	0.98	0.96	0.84
Mehanny III	0.97	0.93	0.85
Luco & Cornell	0.92	0.88	0.81
Yah. & Tehr. NC	0.94	0.91	0.73
Yah. & Tehr. C	0.96	0.94	0.76
Matsumura	0.96	0.94	0.85
Martinez-Rueda	0.94	0.91	0.75
Kappos	0.91	0.88	0.69

Moreover, one can see that spectral acceleration at the fundamental period of the structure $Sa(T_1)$ is relatively efficient to correlate with structural damage for the 3-storey regular frame and for low seismic intensity. Note that $Sa(T_1)$ provides higher degree of correlation compared with some of the advanced complicate-to-elaborate IMs. On the contrary, $Sa(T_1)$ shows a rather weak correlation for the 6-storey irregular frame and for high seismic intensity (Table 3.4 and Fig. 3.1). $Sa(T_1)$ fails to account for the effects of inelasticity on structural demand. This finding is in general in line with previous studies which demonstrated that $Sa(T_1)$ may not be efficient for tall, long period buildings (Shome and Cornell, 1999) and also for near source ground motions (Luco and Cornell, 2007).

Table 3.2. Correlation coefficients between Overall Structural Damage Index (OSDI), Roof Drift Ratio (RDR) and Peak Interstorey Drift Ratio (IDR) and the evaluated IMs for scale factor 1.0.

3-storey regular	OSDI	RDR	IDR
$Sa(T_1)$	0.96	0.94	0.83
Cordova et al.	0.92	0.88	0.87
Mehanny I	0.98	0.96	0.86
Mehanny II	0.95	0.94	0.84
Mehanny III	0.94	0.94	0.84
Luco & Cornell	0.94	0.94	0.90
Yah. & Tehr. NC	0.98	0.96	0.85
Yah. & Tehr. C	0.96	0.95	0.86
Matsumura	0.92	0.92	0.83
Martinez-Rueda	0.98	0.96	0.86
Kappos	0.98	0.97	0.83

6-storey irregular	OSDI	RDR	IDR
$Sa(T_1)$	0.91	0.87	0.81
Cordova et al.	0.97	0.95	0.91
Mehanny I	0.96	0.92	0.86
Mehanny II	0.98	0.95	0.91
Mehanny III	0.96	0.94	0.91
Luco & Cornell	0.89	0.85	0.82
Yah. & Tehr. NC	0.92	0.88	0.82
Yah. & Tehr. C	0.95	0.92	0.86
Matsumura	0.96	0.95	0.90
Martinez-Rueda	0.92	0.89	0.83
Kappos	0.89	0.84	0.77

The correlation capability is improved with the IM proposed by Cordova et al. and its modification proposed by Mehanny. According to this modification the period elongation associated with non-linear structural behavior is expressed through the acceleration spectral shape. It is worth mentioning that even for the 6-storey irregular frame and for high nonlinear level the IM developed by Mehanny exhibits a strong correlation with OSDI (Table 3.4). Observe in Fig. 3.1 that the dispersion of the Mehanny-IM is relatively low compared to the dispersion of $Sa(T_1)$. The study of the results presented in Tables 3.1, 3.2, 3.3 and 3.4 leads to the conclusion that for all the three earthquake scenarios the Mehanny-IM shows in general the same trend of correlation with the OSDI, except for the case of the 6-storey irregular frame for high nonlinear level.

Table 3.3. Correlation coefficients between Overall Structural Damage Index (OSDI), Roof Drift Ratio (RDR) and Peak Interstorey Drift Ratio (IDR) and the evaluated IMs for scale factor 1.5.

3-storey regular	OSDI	RDR	IDR
$Sa(T_1)$	0.96	0.94	0.84
Cordova et al.	0.96	0.93	0.91
Mehanny I	0.97	0.95	0.85
Mehanny II	0.97	0.95	0.86
Mehanny III	0.98	0.96	0.89
Luco & Cornell	0.92	0.92	0.83
Yah. & Tehr. NC	0.97	0.95	0.85
Yah. & Tehr. C	0.97	0.96	0.88
Matsumura	0.96	0.95	0.88
Martinez-Rueda	0.96	0.94	0.84
Kappos	0.95	0.94	0.81

6-storey irregular	OSDI	RDR	IDR
$Sa(T_1)$	0.88	0.82	0.70
Cordova et al.	0.96	0.93	0.84
Mehanny I	0.94	0.89	0.78
Mehanny II	0.96	0.94	0.84
Mehanny III	0.96	0.93	0.87
Luco & Cornell	0.90	0.78	0.74
Yah. & Tehr. NC	0.89	0.83	0.72
Yah. & Tehr. C	0.93	0.88	0.78
Matsumura	0.96	0.95	0.89
Martinez-Rueda	0.91	0.83	0.74
Kappos	0.85	0.78	0.67

The displacement spectral IM proposed by Luco and Cornell exhibit poor correlation with respect to $Sa(T_1)$ for the 3-storey regular frame for low nonlinearity. However, for high nonlinearity the Luco and Cornell-IM correlates better with OSDI than $Sa(T_1)$. Furthermore, the Yahyaabadei and Tehranizadeh-IMs show in general good correlation with OSDI only for the 3-storey regular frame and for relatively low nonlinear behavior of the 6-storey irregular structure.

Table 3.4. Correlation coefficients between Overall Structural Damage Index (OSDI), Roof Drift Ratio (RDR) and Peak Interstorey Drift Ratio (IDR) and the evaluated IMs for ground motions that produce high nonlinearity.

3-storey regular)	OSDI	RDR	IDR
$Sa(T_1)$	0.92	0.86	0.67
Cordova et al.	0.91	0.86	0.84
Mehanny I	0.94	0.90	0.70
Mehanny II	0.94	0.90	0.71
Mehanny III	0.95	0.92	0.80
Luco & Cornell	0.94	0.90	0.71
Yah. & Tehr. NC	0.94	0.89	0.69
Yah. & Tehr. C	0.94	0.92	0.76
Matsumura	0.92	0.90	0.78
Martinez-Rueda	0.92	0.88	0.68
Kappos	0.92	0.88	0.62

6-storey irregular)	OSDI	RDR	IDR
$Sa(T_1)$	0.59	0.20	0.10
Cordova et al.	0.86	0.68	0.52
Mehanny I	0.78	0.44	0.30
Mehanny II	0.86	0.70	0.54
Mehanny III	0.90	0.74	0.69
Luco & Cornell	0.82	0.46	0.42
Yah. & Tehr. NC	0.65	0.25	0.14
Yah. & Tehr. C	0.78	0.44	0.31
Matsumura	0.88	0.81	0.71
Martinez-Rueda	0.76	0.37	0.29
Kappos	0.55	0.16	0.07

The velocity spectral IM developed by Matsumura correlates better than $Sa(T_1)$ only to the 6-storey irregular frame. Moreover, the IM proposed by Martinez-Rueda and Kappos show a small improvement with respect to $Sa(T_1)$ mainly for the 3-storey regular frame.

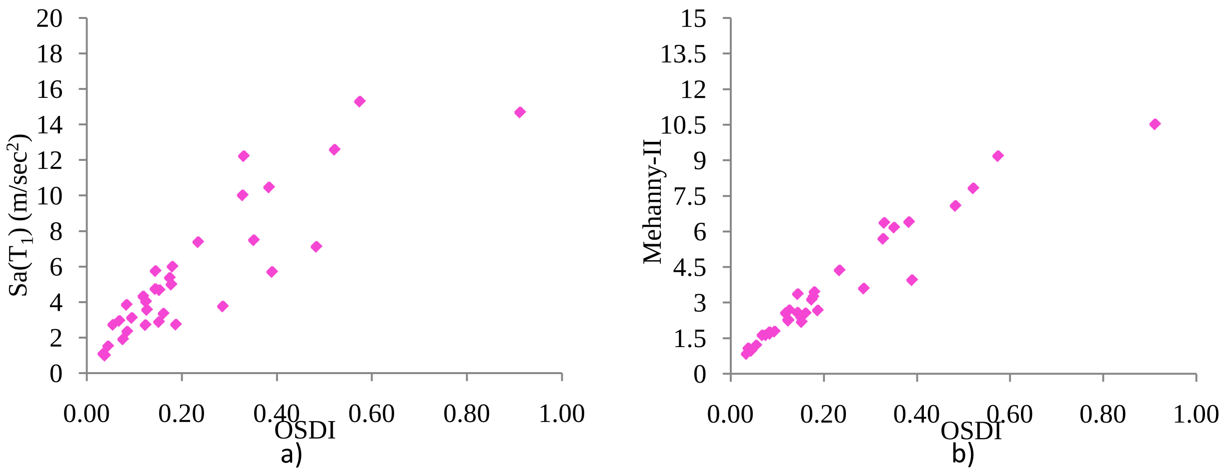


Figure 3.1. 6-storey irregular frame for scale factor 1.5 and earthquake scenario of seismic zone II. a) $Sa(T_1)$ versus Overall Structural Damage Index (OSDI) and b) Mehanny's IM versus Overall Structural Damage Index (OSDI).

4. CONCLUSIONS

In this paper the correlation between a number of advanced, structure-specific ground motion intensity measures and some engineering demand parameters of multi storey reinforced concrete regular and irregular planar frames is investigated. Nonlinear dynamic analyses using thirty three ground motion records for three intensity levels are performed. The evaluation of the correlation coefficients has led to the following conclusions:

- The overall structural damage index appears to be the appropriate engineering demand parameter to correlate with ground motion intensity measures compared to the roof drift ratio and the peak interstorey drift ratio.
- The spectral acceleration computed at the fundamental period of the structure is an efficient parameter to correlate with structural damage only for low nonlinear range and for regular

structures.

- The intensity measures that take into consideration the effects of inelasticity through the acceleration spectral shape indicate the strongest correlation with structural damage for low as well as high nonlinear range. This is valid for both regular and irregular structures.
- The intensity measures that account for the period elongation through an integration of the spectrum velocity curve in specific intervals or through an appropriate combination of the elastic and inelastic spectral displacement values do not indicate high correlation with structural damage for all levels of nonlinearity. This is true for both regular and irregular structures.

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