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THE ROLE OF SEISMIC POUNDING IN THE OPTIMAL SELECTION OF GROUND-MOTION INTENSITY MEASURES

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ABSTRACT

To grant quantitative estimates of the expected levels of seismic ground motion as the primary input to seismic hazard assessments, it is vital to characterize the complicated nature of strong motion accelerograms using simple indices. Over the years, numerous ground-motion parameters have been suggested by researchers for that purpose, and to be used as indices of a ground motion's damage potential. Finding a best correlated ground-motion parameter with the damage index, is a main goal of such type of studies. Minimizing the variability in this correlation is of great importance to determine the expected damage with a higher degree of accuracy. This paper presents an analysis of different ground-motion intensity measures (IMs) that can be used in assessing the performance of reinforced concrete buildings to test the impact of pounding on the optimal selection of ground-motion IMs. The occurrence of structural pounding in metropolitan cities is caused by the inadequate gap between adjacent buildings. Identifying the function in which the seismic pounding performs in selecting the most appropriate ground-motion IM, as an illustration of seismic action in a region of interest, is a main objective of the current study. Special cases of typical two-dimensional adjacent multi-story reinforced concrete buildings are analyzed using a number of natural earthquake time histories. The results indicated that, based on the number of records, the variability in the gap distance between buildings may lead to the selection of different IMs.

***KEYWORD*:** Seismic Assessment, Time History Analysis, Structural Pounding, Ground-Motion Intensity Measure, Reinforced Concrete Building

1. INTRODUCTION

Recent earthquakes such as the 1998 Northridge, 1999 Loma Prieta, 2003 Bam, and 2011 Van demonstrated that the no-damage based design of structures, which has been conventionally applied, is not an appropriate and sufficient design philosophy to provide a quantitative basis for evaluating the seismic performance of structures. To achieve the expected adequate performance levels which are essential to stakeholders, in terms of estimating life loss, economic loss and temporary loss of applications during probable future earthquakes, a new proficient method proposed by the Pacific Earthquake Engineering Research (PEER) Center and known as performance-based earthquake engineering (PBEE), has received much attention in recent years. PBEE as a probabilistic methodology is divided into four main analytical stages: hazard analysis (identifying the ground-motion hazard of the site); structural analysis (determining the structural response of the building); damage analysis (finding the damage of the building components); and loss analysis (obtaining the repair cost of the building). During the first (hazard analysis) stage, a seismic hazard curve, which quantifies the probability of exceeding a seismic ground-motion IM from a certain value, is generated for the specific site. In the second stage of PBEE, an analytical method such as nonlinear time-history analysis is performed to estimate the building's response to ground motions of a given IM in terms of roof displacement, inter-story drift, or other Engineering Demand Parameters (EDPs); the building's capacity curve is obtained by this stage. Damage measures (DMs) are then produced in the third stage using fragility functions. Fragility curves as the results of this stage are generally used to represent the frequency of exceeding a damage state with respect to various levels of ground-motion IMs. Lastly, a set of decision variables (DVs), such as repair costs and economic losses, can be evaluated on the basis of the DMs obtained in the third stage. Vulnerability

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curves are outcomes of this fourth stage. The aforementioned four-phase process involves uncertainties within each stage and needs to be treated probabilistically (Ramirez & Miranda, 2009; Zareian & Krawinkler, 2012). Each relationship, from site to IM, IM to EDP, EDP to DM, and DM to DV, depends mainly on the appropriate selection of ground-motion IMs.

To grant quantitative estimates of the expected levels of seismic ground motion as the primary input to seismic hazard assessments, it is vital to characterize the complicated nature of strong-motion time histories using simple indices. For that purpose, and over the years, numerous ground-motion parameters have been proposed by researchers (e.g., Housner, 1952; Arias, 1970; Shome et al., 1998; Fajfar et al., 1990; Bojórquez & Iervolino, 2011). One of the most important goals put forth in these studies was, once, to pick out the ground-motion parameter that used to be best correlated with the damage index. It is desirable to minimize the variability in this correlation to determine the expected damage for improved accuracy.

A good IM should be capable of capturing all ground-motion characteristics, namely the amplitude, duration, or frequency content of ground motion. Such a measure would satisfy the requirements of efficiency and sufficiency that are necessary to diminish the uncertainty in the predicted structural response (Krawinkler et al., 2003). Efficiency and sufficiency are achieved by measuring the strength and standard deviation of the relation between the IM and the structural response (higher correlation and lower standard deviations are preferable) (Padgett et al., 2008).

Because only one ground-motion parameter is used during the implementation of PBEE methodology, all the other available IMs are ignored, making it difficult to pick out the optimal parameter. Numerous studies (e.g., Riddell, 2006; Alvanitopoulos et al., 2010; Nanos, 2011; Buratti, 2012; Elenas, 2013) have shown that various IMs may have different abilities in assessing structural responses when being used as damage descriptors. Several other studies (e.g., Shome et al., 1998; Kafali & Grigoriu, 2004; Seyedi et al., 2010; Gehl et al., 2011; Xu & Wen, 2012; Gehl et al., 2013; Yaseen et al., 2015) have asserted that a single parameter cannot satisfactorily represent the seismic action in a specific region and express its complex nature. Therefore, the use of two IMs rather than a single parameter has recently been regarded as the main concern of such studies.

Different types of uncertainties were taken into consideration by the aforementioned studies. These uncertainties arise from a variety of sources, such as seismic hazards, ground motions and their features, site and soil conditions, mechanical properties of materials, damage state definition, ground-motion IMs, ground-motion suite selection, model error, and lack of knowledge (Mackie & Nielson, 2009; Gehl et al., 2013). Nevertheless, almost all the aforementioned studies were conducted on single detached structures/buildings with open space around. This can be correct if applied to rural areas. However, in urban areas such as large cities and metropolitan areas, this is not an appropriate simulation of attached structures and buildings. In view of that, the collision of adjacent buildings during earthquakes (known as seismic pounding) and the effect of this on the response of buildings has also recently been taken into consideration by several researchers (e.g., Tubaldi, 2012; Jameel, 2013; Licari et al., 2015; Chujo et al., 2016; Ghandil & Aldaikh, 2016). However, the author could not find helpful studies in the literature to clarify the role of seismic pounding in the optimal selection of ground-motion IMs. The main purpose of the current study is, therefore, to determine how seismic pounding affects ground-motion IM selection. For that purpose, special cases of typical two-dimensional adjacent multi-story reinforced concrete buildings with different height, floor levels, and gaps are analyzed, using a number of natural earthquake time histories.

2. STRUCTURAL MODELING AND ANALYSIS

To describe the actual behavior of a structure that is affected by an earthquake, a nonlinear dynamic analysis is an optimal and accurate

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choice (Erbay, 2007). This method employs the direct numerical integration of the differential equations of motion by considering the elastoplastic deformation of the structural members. Nonlinear dynamic analyses are also known as time-history analyses. To perform such an analysis, and to investigate which ground-motion IMs are most important, several time histories that are appropriate for the seismic hazard analysis of the selected region are required for the structure being evaluated. It is subsequently necessary to obtain the correlation between representative parameters of each accelerogram and the induced damage corresponding to the related accelerogram on the tested structure. A higher correlation coefficient is an indicator of a better ground motion IM.

To create a 2D model and run all required analyses, the general purpose finite element analysis software ETABS 2016 (Computers and Structures, Inc., 2016) is utilized in the current study. The software is able to assess the nonlinear behavior of frames under static or dynamic loadings, taking into account both material and geometric nonlinearities.

The 2D models of typical cases of attached and detached building frames are modeled using the two-node frame elements (three degrees of freedom in each end) employed in the ETABS. A six-story reinforced concrete building with no structure and geometric irregularities is considered as the main building under study. The building is fixed and the adjacent buildings are modeled to be variable in their story numbers, being two and four-story buildings (Figures 1 and 2). All buildings have a first-story height of 4.5 m, and of 3 m for the other floors. Buildings have three spans on X direction and four spans in the Y direction (rectangular plan of 15m x 20m). The assessed RC buildings are designed in such a way that they are only able to resist gravity loads, which is the model widely used in larger cities in the Kurdistan region of Iraq. The slabs of the buildings are considered to be 0.15m thick. Figure 3 shows a two-dimensional view of the selected six-story RC building frame. Sectional dimensions of columns and beams with the number of longitudinal reinforcement bars are also represented in Figure 4. The concrete strength f'c at twenty-eight days is 25.0 N/mm^2 and the reinforcing steel used is high-yield-strength deformed bars with fy = 415 N/mm^2.

In ETABS, a nonlinear time history analysis can be performed using either user-defined nonlinear hinge properties, default hinge properties, or automated hinge properties. Automated hinge properties are calculated automatically from the frame element material and section properties according to ACI 318-02 (ACI, 2002) criteria. Hinges are assigned at both ends of each element, beams, and columns. The concrete moment (M) hinge type and the concrete axial force-biaxial moment (P-M-M) hinge type are respectively used to realize the behavior of hinges formed in the beams and columns. The material properties assigned to the frame element are used to predict the plastic behavior of the hinges, whereas the elastic behavior of the frame elements is determined by the frame sections assigned to the elements. In the work under study, all the non-linear dynamic analyses were conducted as Direct Integration Transient time history analyses using Direct Integration in Newmark method by consideration of damping ratio for all modes equal to 5% and P-Δ effects. Furthermore, Takeda and Kinematic 's models were respectively used to describe the hysteretic behavior of concrete and steel materials. Mander and Simple parametric definitions were also used to define stress-strain curves of concrete and steel materials, respectively.

A number of given inputs that are represented by ground-motion time histories are required by ETABS to successfully perform the time history analyses and predict the structural response of the buildings under consideration. Despite the high variability in ground motions, it is preferable to select as few records as possible for these types of analyses and design purposes. This is mainly because the nonlinear modeling and dynamic analysis are computationally expensive and highly time-consuming.

Although the appropriate number of records is still a topic into which research should be done, in

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practice, it is typical to use seven motions according to EC8 (CEN, 2003) and ASCE/SEI-7 (ASCE, 2010) and eleven ground motions as specified by ATC-58 (2011). The average response of the structure is the outcome of the analysis if the aforementioned number of ground motions takes as input to the analysis. Likewise, Shome et al. (1998) and Shome (1999) asserted that for a mid-rise building, ten to twenty records are sufficient to assess its seismic demand with great confidence.

In this study, and based on the ASCE (2010) and ATC-58 (2011) recommendations, two suites of precisely selected seven and eleven motions are chosen in such a way as to be compatible with the seismic characteristics of the Kurdistan region of Iraq. Selected motion records were derived from a bin of recorded motions, including PEER Next Generation Attenuation NGA Strong Motion Database (available at http://peer.berkeley.edu/assets/NGA_Flatfile.xls). Tables 1 and 2 present specifications of the selected ground motions. With respect to the seismological characteristics of these records and structural configurations of models, some frequently used IMs that have recently been worked on in many studies are also presented in Table 3. An explicit explanation of the examined IMs, has been given by Kramer (1996). The following section (section 3) thoroughly details the outcomes of the study and discusses the significance of the results.

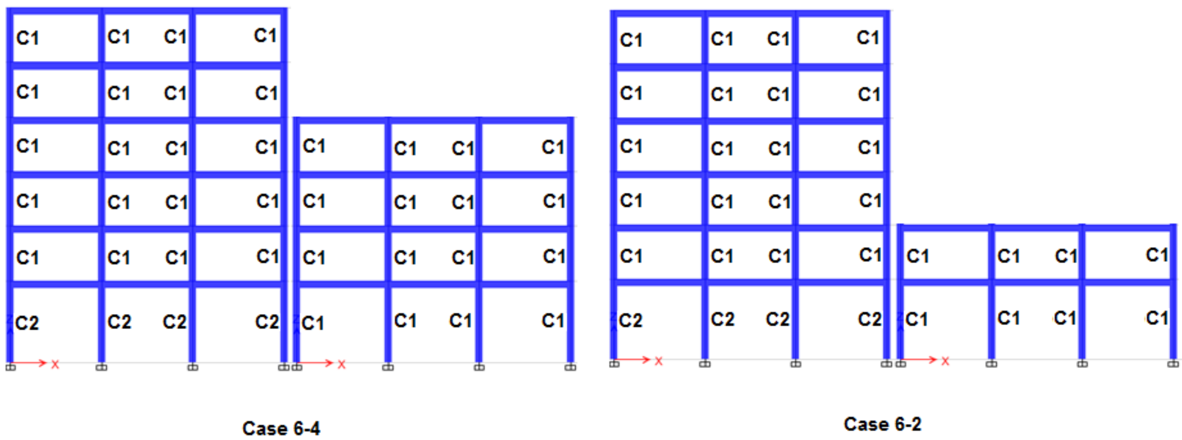

Fig. (1): Different cases of two adjacent buildings considered in the current study with zero and 30 mm gap between them

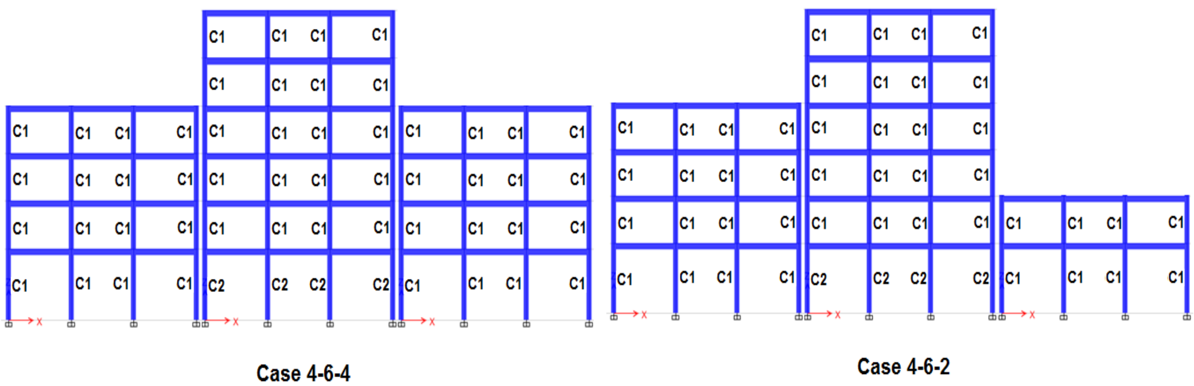

Fig. (2): Different cases of three adjacent buildings considered in the current study with zero and 30 mm gap between them

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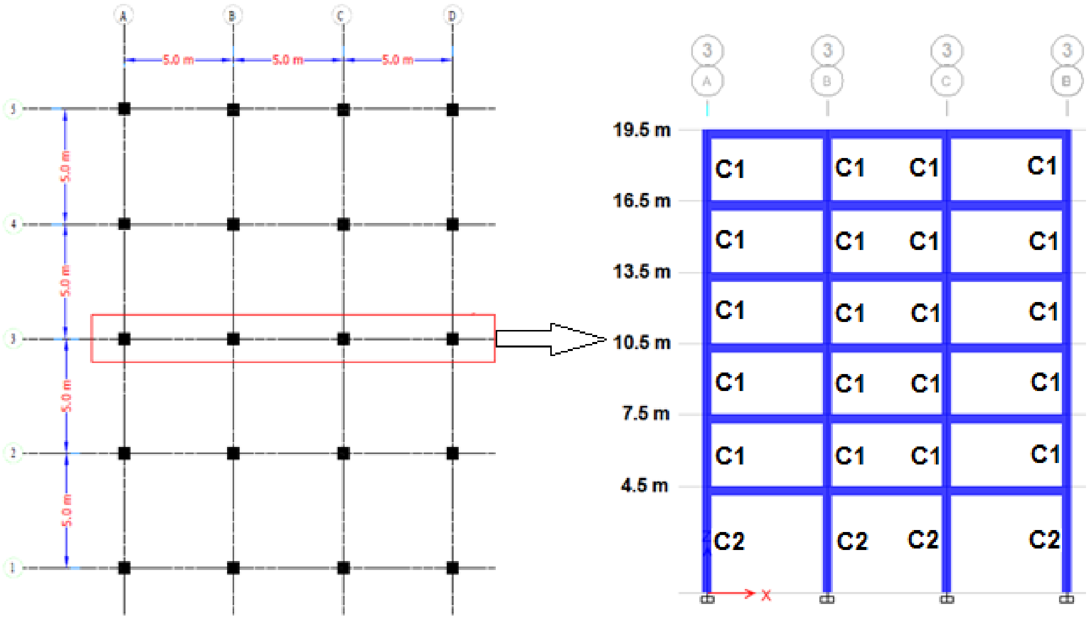

Fig. (3): 2D view of the frame considered in the study (right) and its layout (left)

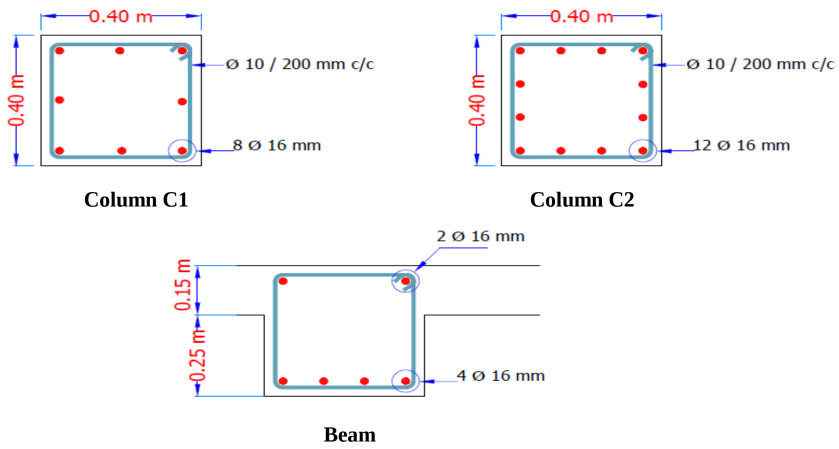

Fig. (4): Sectional dimensions of columns and beams with the number of longitudinal reinforcement bars

Table (1): Specifications of a suite of seven ground motions considered in the current study

NGA Record Number	Earthquake Name	Magnitude	Epicentral Distance (km)	PGA (g)	PGV (cm/s)	PGD (cm)	VSI (cm)	EDA (m/s^2)	A95 (m/sec^2)
126	Gazli, USSR	6.8	12.82	0.6	65.378	25.378	230.179	5.276	5.892
179	Imperial Valley-06	6.53	27.13	0.36	76.596	59.091	148.551	3.345	3.507
779	Loma Prieta	6.93	18.46	0.97	108.542	65.814	430.265	4.976	9.36
963	Northridge-01	6.69	40.68	0.57	51.828	9.014	212.358	5.927	5.505
983	Northridge-01	6.69	13.00	0.57	76.042	42.417	244.035	5.452	5.556
1085	Northridge-01	6.69	13.6	0.83	117.496	34.451	306.808	8.353	8.064
1086	Northridge-01	6.69	16.77	0.6	78.10	16.818	259.383	5.646	5.855

Table (2): Specifications of a suite of eleven ground motions considered in the current study

NGA Record Number	Earthquake Name	Magnitude	Epicentral Distance (km)	PGA (g)	PGV (cm/s)	PGD (cm)	VSI (cm)	EDA (m/s^2)	A95 (m/sec^2)
126	Gazli, USSR	6.8	12.82	0.6	65.378	25.378	230.179	5.276	5.892
143	Tabas, Iran	7.35	55.24	0.84	97.757	38.66	339.326	8.475	7.971
179	Imperial Valley-06	6.53	27.13	0.36	76.596	59.091	148.551	3.345	3.507
779	Loma Prieta	6.93	18.46	0.97	108.542	65.814	430.265	4.976	9.36
802	Loma Prieta	6.93	27.23	0.51	41.151	16.247	191.258	4.749	4.99
879	Landers	7.28	44.02	0.72	146.539	262.733	183.768	5.129	6.968
963	Northridge-01	6.69	40.68	0.57	51.828	9.014	212.358	5.927	5.505
983	Northridge-01	6.69	13.00	0.57	76.042	42.417	244.035	5.452	5.556
1085	Northridge-01	6.69	13.6	0.83	117.496	34.451	306.808	8.353	8.064
1086	Northridge-01	6.69	16.77	0.6	78.10	16.818	259.383	5.646	5.855
1508	Chi-Chi, Taiwan	7.62	21.42	0.49	71.70	38.711	246.095	4.716	4.659

Table (3): Ground-motion IMs considered in the current study

IMs	Name
Acceleration-based	Peak ground acceleration (PGA), root mean square of acceleration (ARMS), Arias intensity (IA), characteristic intensity (IC), cumulative absolute velocity (CAV), acceleration spectrum intensity (ASI), sustained maximum acceleration (SMA), effective design acceleration (EDA), A95 parameter, and spectral acceleration at different periods Sa(T1), Sa(T2), and Sa(T3), where T1, T2, and T3 are the periods of the first, second and third mode shapes of the structure
Velocity-based	Peak ground velocity (PGV), root mean square of velocity (VRMS), specific energy density (SED), velocity spectrum intensity (VSI), sustained maximum velocity (SMV), and Housner intensity (IH)
Displacement-based	Peak ground displacement (PGD), root mean square of displacement (DRMS)
Duration	Predominant period (Tp) and mean period (Tm)

3. RESULTS AND DISCUSSION

Several time-history analyses (more than 100 analyses) were applied to the four different cases of attached buildings, in addition to the case of a detached building represented by a six-story building. The pounding effect was assessed by the modeling of two and three adjacent buildings having a variable number of stories (two, four, and six stories) with zero gap and/or 30 mm gap between them. All analyses were performed in ETABS 2016. More than one hundred time history analyses were undertaken and, consequently, the maximum roof displacements (in the x-direction) of the six-story building were recorded as a main EDP in this study. The results are presented in Tables 4 and 5 for two sets of seven and eleven ground-motion records and different cases of pounding effect are considered. Furthermore, statistical analysis of the results to find the strength of correlation between ground-motion IMs and the structural response of the six-story building are also illustrated in Tables 6 and 7 for the aforementioned ground motions and for the different cases of pounding effect. Graphical representations of these correlations are shown in Figures 5-8 for the six-story building case and eleven time-history analyses. However, because of page limitations, the cases of other scenarios are not presented here. The graph shows the different strengths of correlations in seismic responses, namely displacements with intensity measures. As illustrated in Figures 5-8, the VSI predict the response of six-story building more accurately than the other IMs because its regression line has a steeper slope (i.e., less uncertainty) than the other

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IM regression graphs. The r^2 coefficient presented in Tables 6 and 7 is used to determine how poorly an IM fits the data. Table 7 demonstrates that the VSI is the optimal ground-motion IM for all scenarios considered in this study because it has a bigger r^2 measure than the other IMs. Even though VSI is found to be the optimal ground-motion IM but still there is a corresponding poor fit with the data in terms of r^2. This result agrees with the findings of studies such as Shome et al. (1998), Kafali & Grigoriu (2004), Xu & Wen (2012), Gehl et al. (2013), Yaseen et al. (2015), who asserted that a single parameter cannot satisfactorily represent the seismic action in a specific region and express its complex nature. Therefore, the use of two IMs rather than a single parameter is recommended. The velocity spectral intensity (VSI) was found to be the best correlated parameter for the main case considered in this study, which is a six-storey building with no adjacent buildings and as shown in Tables 6 and 7. VSI parameter considers the spectral velocity over a wide range of periods; notably, this range can account for increases in a building's natural period due to a loss in rigidity and progressive degradation. This result agrees with the findings of the Riddell and Garcia (2001), Riddell (2006), and Ye et al. (2011), who mentioned that the acceleration-based IMs (e.g. PGA, EDA, A95) are most suitable for the short period structures, IMs in terms of the velocity (e.g. PGV, VSI) are most appropriate for the intermediate period structures, and displacement-based parameters (e.g. PGD, DRMS) are mainly applicable to the long period structures.

However, with respect to Table 6, which presents the results of regression analysis of the data obtained from the application of a suite of seven time histories to the different cases of buildings, it can clearly be seen that the pounding effect has a great effect on the accuracy of the IM selection process. Despite being the most efficient parameter in the six-story building case, VSI is inefficient for other cases that take the pounding effect into consideration. Based on the availability of the gap between buildings, the EDA and A95 are the best choices for selection as the optimal ground-motion parameters (see Table 6).

4. CONCLUSIONS

The central objective of this study is to investigate the role of seismic pounding effect on the accurate selection of the ground motion IM. Four different cases of attached buildings along with a detached case of a six-toy building considered in the current study and analysed in ETABS software using several numbers of ground motion time histories. The correlation strength between ground motion IM (22 IMs used in this study) and structural response of the building (roof displacement) is then investigated. It is shown that while some ground-motion IMs are computationally complex or typically unsuitable, there are others available that could be used practically, if selected properly by considering the structure's typology.

For an intermediate period structure like the one used in this study, a detached six-story reinforced concrete building, it can be argued that the velocity based IMs perform better and the VSI is a powerful IM for predicting the nonlinear behavior of the building's response. This can be explained by the fact that VSI is able to represent several vibration frequencies, and hence the ground-motion characteristics and properties are intensively collected through this parameter.

Furthermore, the results indicated that, based on the number of records, the variability in the gap distance between buildings may lead to the selection of different IMs. Increasing the number of records from seven to eleven mitigated the impact of the pounding effect represented by the insufficiency gap between buildings. Thus, it can be concluded that the effect of seismic pounding on the optimal selection of ground-motion IMs increases with decreasing number of ground motions. Accordingly the use of eleven ground motions (mentioned by ATC-58) is highly recommended.

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Table (4): Structural response of buildings to a suite of seven ground-motion time histories

Roof Displacement (mm)				
Adjacent Buildings	NGA Record Number	No Gap Between buildings	30 mm Gap between buildings	Detached case
Detached six-story building (Case 6)	126			70.848
	179			46.896
	779			154.94
	963			72.021
	983			77.273
	1085			103.448
	1086			94.378
Attached six-story to the four-story building (Case 6-4)	126	55.368	69.845	
	179	38.322	46.896	
	779	100.787	87.661	
	963	54.504	69.532	
	983	64.082	77.271	
	1085	86.878	103.259	
	1086	76.768	96.052	
Attached six-story to the two-story building (Case 6-2)	126	53.182	68.623	
	179	37.665	46.896	
	779	87.06	89.657	
	963	52.635	72.178	
	983	62.447	77.278	
	1085	84.02	103.448	
	1086	74.467	93.881	
Attached four-story to six-story to the four-story building (Case 4-6-4)	126	49.533	70.793	
	179	35.752	46.896	
	779	84.824	85.449	
	963	48.646	69.533	
	983	59.338	77.215	
	1085	80.594	100.663	
	1086	71.597	76.139	
Attached four-story to six-story to the two-Story building (Case 4-6-2)	126	46.962	67.9	
	179	34.395	46.896	
	779	54.346	87.915	
	963	46.284	71.938	
	983	56.441	77.215	
	1085	76.729	100.617	
	1086	66.804	80.46	

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Table (5): Structural response of buildings to a suite of eleven ground-motion time histories

Roof Displacement (mm)				
Adjacent Buildings	NGA Record Number	No Gap Between buildings	30 mm Gap between buildings	Detached case
Detached six-story building (Case 6)	126			70.848
	143			89.085
	179			46.896
	779			154.94
	802			55.198
	879			59.823
	963			72.021
	983			77.273
	1085			103.448
	1086			94.378
	1508			98.347
Attached six-story to the four-story building (Case 6-4)	126	55.368	69.845	
	143	68.108	77.525	
	179	38.322	46.896	
	779	100.787	87.661	
	802	44.845	55.198	
	879	44.756	59.003	
	963	54.504	69.532	
	983	64.082	77.271	
	1085	86.878	103.259	
	1086	76.768	96.052	
	1508	79.286	77.833	
Attached six-story to the two-story building (Case 6-2)	126	53.182	68.623	
	143	63.669	89.595	
	179	37.665	46.896	
	779	87.06	89.657	
	802	43.341	55.198	
	879	43.32	59.823	
	963	52.635	72.178	
	983	62.447	77.278	
	1085	84.02	103.448	
	1086	74.467	93.881	
	1508	73.816	76.65	
Attached four-story to the six-story to the four-story building (Case 4-6-4)	126	49.533	70.793	
	143	71.84	73.063	
	179	35.752	46.896	
	779	84.824	85.449	
	802	40.046	55.198	
	879	40.341	59.003	
	963	48.646	69.533	
	983	59.338	77.215	
	1085	80.594	100.663	
	1086	71.597	76.139	
	1508	68.068	84.411	
Attached four-story to the six-story to the two-story building	126	46.962	67.9	
	143	60.077	86.095	

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(Case 4-6-2)	179	34.395	46.896
	779	54.346	87.915
	802	37.123	55.2
	879	37.796	59.823
	963	46.284	71.938
	983	56.441	77.215
	1085	76.729	100.617
	1086	66.804	80.46
	1508	60.55	95.489

Table (6): Correlation coefficients (r^2) between the ground-motion IMs and structural responses (roof displacement in x- direction) of different cases of adjacent buildings for the seven time histories considered in the current study

IMs	Case 6 only	Case 6-4		Case 6-2		Case 4-6-4		Case 4-6-2	
		No Gap	30 mm Gap	No Gap	30 mm Gap	No Gap	30 mm Gap	No Gap	30 mm Gap
PGA	0.894	0.894	0.567	0.829	0.620	0.824	0.721	0.367	0.730
PGV	0.489	0.600	0.388	0.597	0.400	0.621	0.466	0.383	0.455
PGD	0.110	0.044	0.044	0.013	0.038	0.021	0.009	0.057	0.012
ARMS	0.432	0.315	0.096	0.235	0.106	0.230	0.218	0.017	0.173
VRMS	0.438	0.268	0.006	0.163	0.008	0.181	0.043	0.005	0.032
DRMS	0.272	0.118	0.019	0.046	0.014	0.059	0.000	0.071	0.001
IA	0.802	0.648	0.240	0.523	0.276	0.525	0.397	0.088	0.381
IC	0.664	0.521	0.186	0.412	0.211	0.410	0.338	0.057	0.305
SED	0.548	0.360	0.018	0.237	0.027	0.264	0.052	0.000	0.057
CAV	0.863	0.704	0.283	0.578	0.334	0.577	0.434	0.109	0.444
ASI	0.430	0.498	0.492	0.512	0.552	0.478	0.752	0.366	0.727
VSI	0.984	0.895	0.473	0.808	0.519	0.817	0.551	0.265	0.585
IH	0.952	0.886	0.418	0.779	0.452	0.801	0.460	0.244	0.495
SMA	0.410	0.292	0.100	0.218	0.116	0.208	0.231	0.015	0.193
SMV	0.686	0.548	0.130	0.428	0.146	0.454	0.184	0.047	0.192
EDA	0.097	0.269	0.631	0.385	0.648	0.353	0.754	0.727	0.719
A95	0.892	0.912	0.569	0.831	0.623	0.826	0.726	0.371	0.734
TP	0.735	0.684	0.337	0.611	0.366	0.640	0.219	0.195	0.310
Tm	0.047	0.064	0.171	0.079	0.179	0.060	0.243	0.101	0.216
Sa(T1)	0.007	0.120	0.473	0.229	0.450	0.214	0.499	0.714	0.458
Sa(T2)	0.009	0.009	0.095	0.037	0.096	0.034	0.170	0.234	0.149
Sa(T3)	0.059	0.025	0.004	0.010	0.000	0.012	0.006	0.010	0.006

Table (7): Correlation coefficients (r^2) between the ground-motion IMs and structural responses (roof displacement in x- direction) of different cases of adjacent buildings for the eleven time histories considered in the current study

IMs	Case 6 only	Case 6-4		Case 6-2		Case 4-6-4		Case 4-6-2	
		No Gap	30 mm Gap	No Gap	30 mm Gap	No Gap	30 mm Gap	No Gap	30 mm Gap
PGA	0.537	0.442	0.357	0.394	0.471	0.488	0.346	0.215	0.327
PGV	0.102	0.083	0.077	0.077	0.098	0.100	0.076	0.051	0.069
PGD	0.025	0.065	0.090	0.082	0.085	0.081	0.073	0.130	0.075
ARMS	0.287	0.188	0.104	0.143	0.190	0.231	0.131	0.054	0.143
VRMS	0.224	0.099	0.011	0.055	0.032	0.098	0.015	0.000	0.010
DRMS	0.043	0.088	0.088	0.103	0.083	0.102	0.079	0.132	0.080
IA	0.273	0.202	0.085	0.157	0.197	0.273	0.127	0.084	0.244
IC	0.315	0.223	0.108	0.173	0.221	0.288	0.147	0.082	0.224
SED	0.133	0.046	0.000	0.019	0.005	0.040	0.001	0.009	0.004

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CAV	0.179	0.143	0.042	0.111	0.108	0.179	0.099	0.060	0.242
ASI	0.261	0.277	0.304	0.276	0.501	0.427	0.337	0.333	0.452
VSI	0.871	0.785	0.506	0.706	0.616	0.809	0.508	0.479	0.546
IH	0.825	0.724	0.453	0.645	0.550	0.753	0.417	0.324	0.441
SMA	0.135	0.064	0.031	0.040	0.090	0.092	0.049	0.008	0.074
SMV	0.288	0.171	0.054	0.123	0.075	0.151	0.059	0.010	0.055
EDA	0.085	0.165	0.393	0.214	0.554	0.323	0.360	0.375	0.406
A95	0.542	0.448	0.365	0.400	0.475	0.490	0.352	0.217	0.325
TP	0.633	0.674	0.349	0.636	0.293	0.545	0.377	0.312	0.457
Tm	0.000	0.000	0.011	0.000	0.026	0.001	0.030	0.004	0.044
Sa(T1)	0.022	0.117	0.445	0.198	0.368	0.165	0.381	0.420	0.220
Sa(T2)	0.000	0.015	0.118	0.040	0.129	0.050	0.119	0.187	0.068
Sa(T3)	0.006	0.000	0.048	0.003	0.024	0.001	0.003	0.035	0.001

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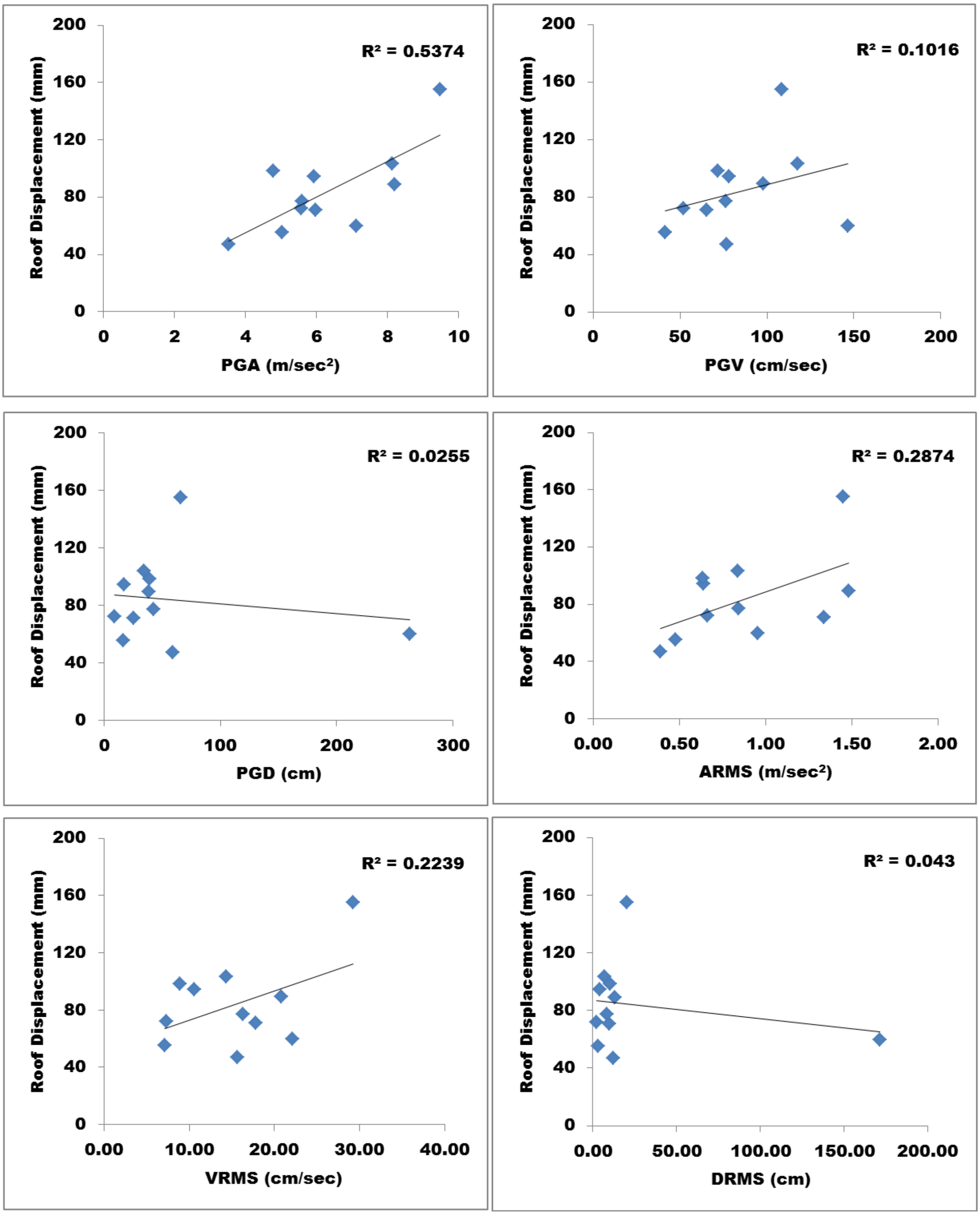

Fig. (5): Graphical representations of the correlation between ground-motion IMs (PGA, PGV, PGD, ARMS, VRMS, and DRMS) and the structural response of the six-story building using eleven ground-motion time histories

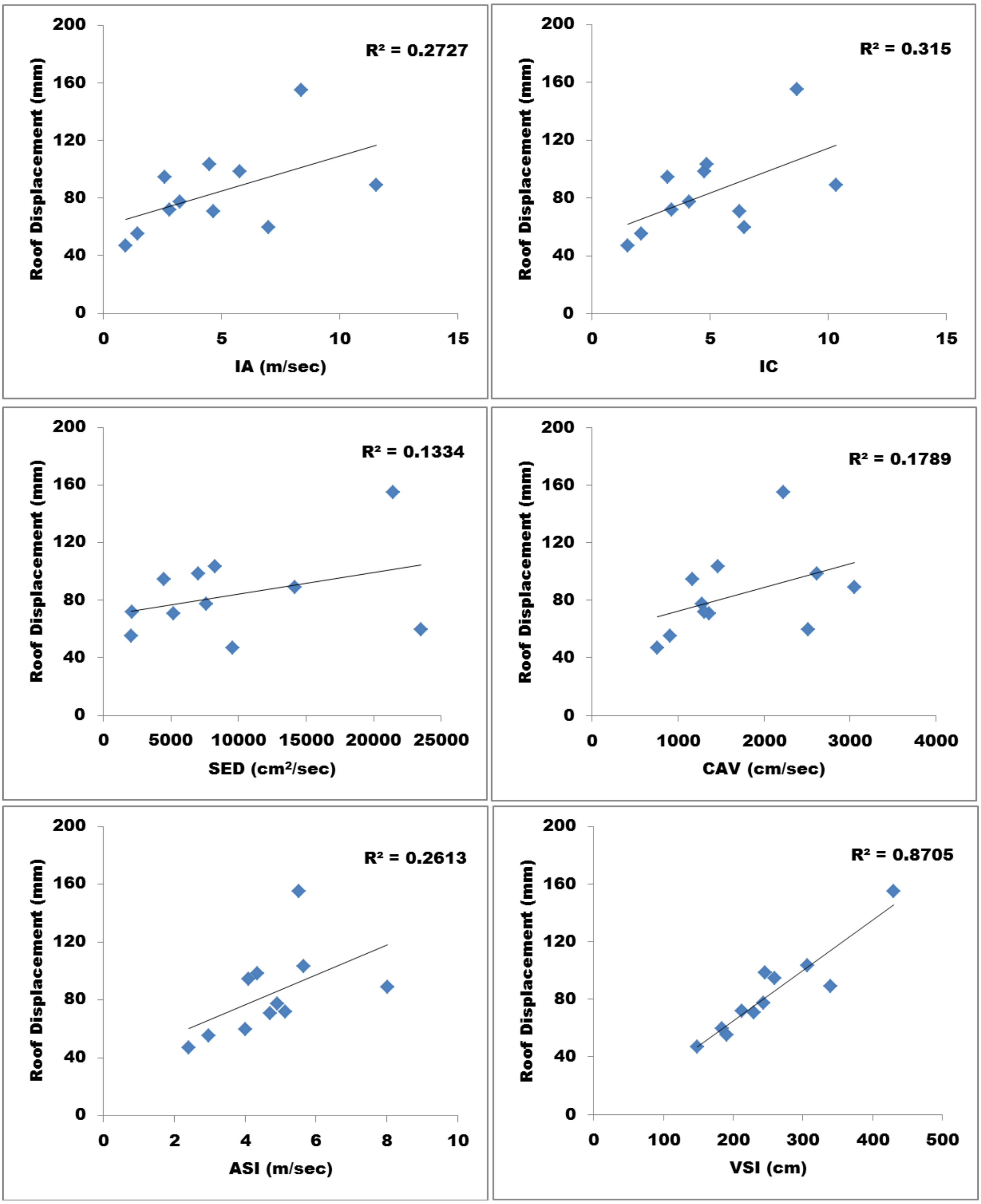

Fig. (6): Graphical representations of the correlation between ground-motion IMs (IA, IC, SED, CAV, ASI, and VSI) and the structural response of the six-story building using eleven ground-motion time histories

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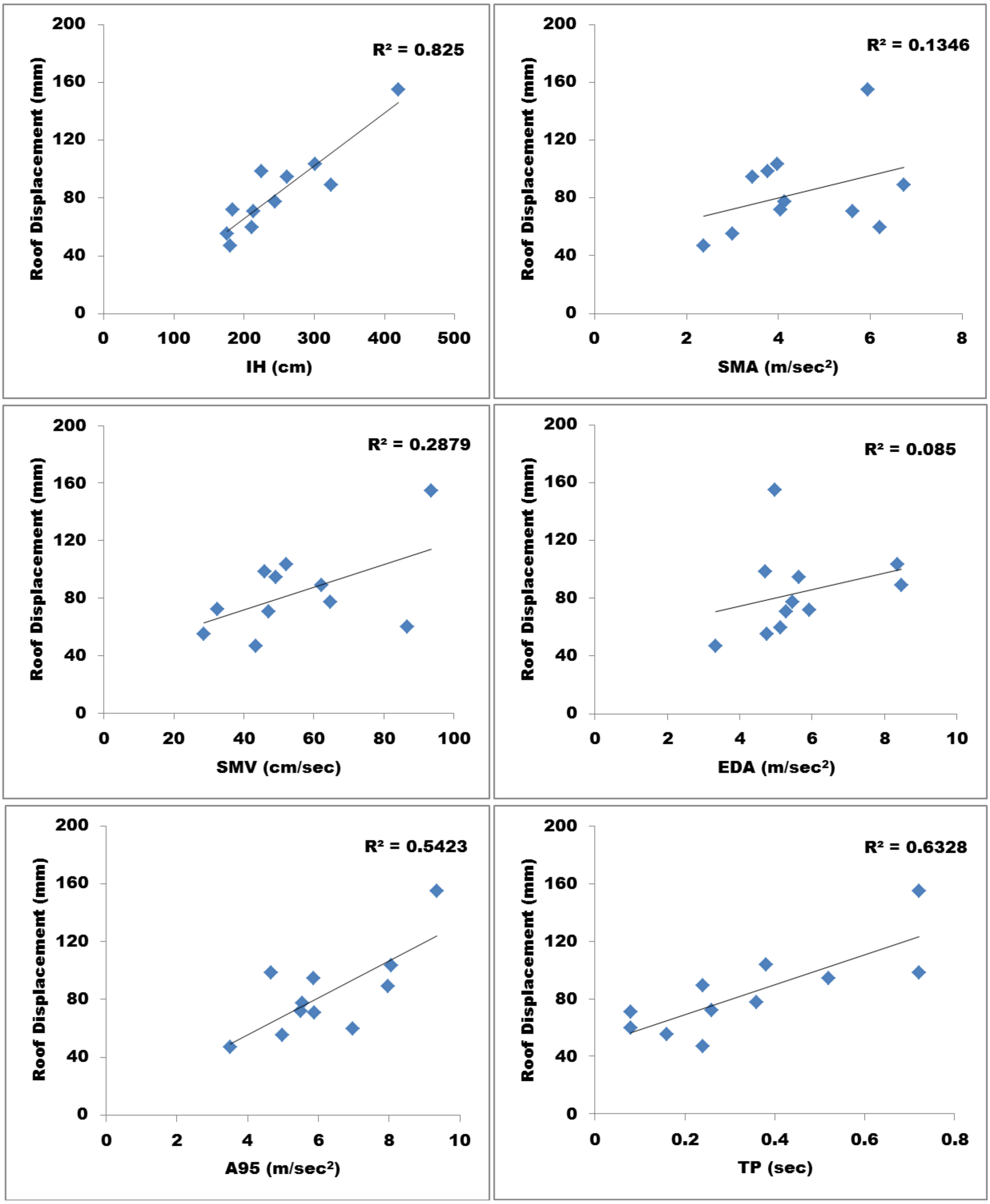

Fig. (7): Graphical representations of the correlation between ground-motion IMs (IH, SMA, SMV, EDA, A95, and TP) and the structural response of the six-story building using eleven ground-motion time histories

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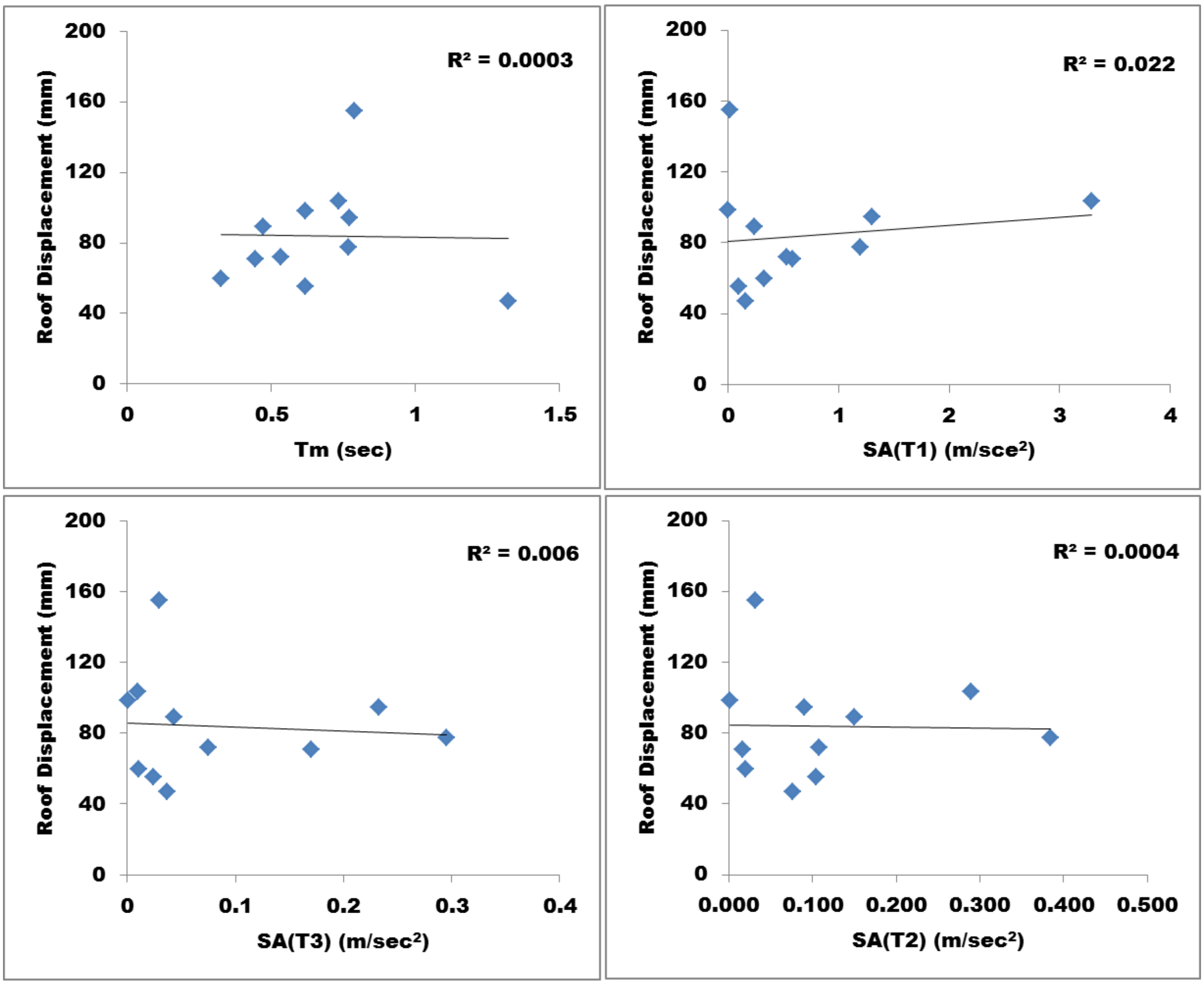

Fig. (8): Graphical representations of the correlation between ground-motion IMs (Tm, SA(T1), SA(T2), and SA(T3)) and the structural response of the six-story building using eleven ground-motion time histories

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