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Selecting Appropriate Intensity Measure in View of Efficiency

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Abstract: This study attempts to answer the question of distinguishing appropriate intensity measure parameter for performance-based design or assessment, taking into account the efficiency aspect. The comprehensive comparative tables proposed in this paper could be an effective support in the decision making procedure for intensity measure selection, comprising most of the frequently utilized intensity measures for low-rise buildings with different fundamental periods. In addition, since some specific intensity measures are commonly applied in codes, the amounts of standard deviation computed in this study could be very beneficial in answering the question of being worthy to consider another intensity measure, to improve the certitude of structural responses, noting expansion in calculationefforts.

Keywords: Efficiency, Interstory Drift Ratios (IDR), Engineering Demand Parameters (EDP), Intensity Measure (IM), Peak Floor Acceleration (PFA), Performance-Based Earthquake Engineering (PBEE).

INTRODUCTION

The unacceptable response of some structures as well as the economic and life losses resulted in recent earthquakes such as the 1999 Loma Prieta,1998 Northridge and 2003 Bam, made the current design philosophy, which has been conventionally based on prevention of overall collapse, questionable and insufficient (Mahdavi, 2012). Performance-based earthquake engineering (PBEE) has received much attention in recent years as the new proficient method that can provide a quantitative basis in assessment of the seismic performance of structures and aims at the design of structures to achieve expected acceptable performance levels which are more relevant to stakeholders, namely, deaths (loss of life), dollars (economic losses) and downtimes (temporary loss of applications) during probable future earthquakes (Gunay and Mosalam, 2013). The proposed fully probabilistic methodology of the Pacific Earthquake Engineering Research (PEER) Center (one of the very frequently used performance assessment procedures) is divided into four basic stages accounting for the following: ground motion hazard of the site, structural response of the building, damage of the building components and repair costs. The first stage utilizes probabilistic seismic hazard analysis to generate a seismic hazard curve, which quantifies the frequency of exceeding a ground motion intensity measure (IM) from a certain value for the specific site.The second stage involves using structural response analysis to estimate engineering demand parameters (EDPs), such as inter-story drift and peak floor

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accelerations, and the collapse capacity of the structure. The third stage produces damage measures (DMs) using fragility functions, which are cumulative distribution functions relating EDPs to the probability of being or exceeding particular levels of damage. The fourth and final stage sets up decision variables (DVs), like economic loses, which stakeholders can use to make more informed design decisions (Ramirez and Miranda, 2009; Zareian and Krawinkler, 2012). The outcomes of each stage serve as input to the next stage. Uncertainty in the loss estimation of the structural system is mainly due to uncertainties in the ground motion, structural and soil properties, can be costly because it is directly related to the repair cost. Thus, it is very important to identify and rank the sources of uncertainty according to their relative influence on the stability of the structure (Lotfollahi-Yaghin et al., 2013).

The first step of the PEER approach is the main area under discussion in this paper. In this step, according to previous history of occurred earthquakes, the rate of return for each earthquake and other seismological conditions of the site, the hazard's curves were figured out through the help of hazard analysis of the site, corresponding to the selected intensity measure for records.

The confidence of PBEE implementation strongly depends on the ability to estimate the probability of incurred EDPs; so as to decouple the seismological and structural uncertainties (stages 1 and 2 of the PEER approach), an intermediate variable, called Intensity Measure (IM), is typically used in the seismic performance assessment of structures (Bazzurro, 1998; Shome, 1999; Luco, 2002). The results of hazard analysis and structural analysis can finally be re-coupled by integration over all levels of the selected IM, in accordance with the total probability theorem (Bozorgnia and Bertero, 2004). By manipulating this approach, the probability of exceeding a specific level of EDP estimate, $\nu(EDP > edp)$ is expressed in the following equation:

$$\nu\left(EDP > edp\right) = \int_0^\infty [1 - p(EDP < edp\,|IM\,]\left|\frac{d\,\nu(IM)}{IM}\right| dIM \qquad (١)$$

where the $p(EDP < edp|IM)$: is the probability that the structural response parameter is smaller than a certain level of EDP at the ground motion intensity measure, *IM*, and *ν(IM)*: denotes the mean annual rate of exceedance of the ground motion intensity measure, *IM*, from a certain value $p(EDP < edp|IM)$ is customarily estimated through incremental dynamic analyses (IDA), under a set of ground motions.

From Eq. (1), it can be concluded that appropriate selection of *IM* parameter plays a significant role in investigating the amounts of EDPs and their mean annual rates, as well as it challenges both researchers and practitioners, since an appropriate *IM* can significantly decrease the runtime of estimating probability parameters and can lead to more reliable evaluations of the seismic performance, as it strongly influences structural responses (Lignos and Krawinkler, 2013).

The responses of structures are greatly more against near-fault records than ordinary or far-fault ones (Tehranizadeh and Movahed, 2011). This fact motivates more comprehensive investigation of *IM* selection for this type of records. In near-fault regions, records are influenced by forward directivity or filing step phenomena and most of the seismic rupture energy appears as a single coherent pulse-type motion. Some vector-type *IMs* have been lately introduced for near-fault records (Shrey and Baker, 2007; Welch et al., 2014). It is obvious from equation 1thatthe purpose of computing the mean annual rate of exceedance of EDP for a certain value, slope of the seismic hazard curve, has to be evaluated at an anticipated level of *IM* and when *IM* is a vector-type parameter, calculating the derivation of *v(IM)* according to this type of *IM* becomes more complicated and time-consuming. Besides, a unique suitable IM has been explored associated with both near and far-fault records to get data for aggregating seismic hazard of several sources in a specific site. Therefore, utilizing scalar *IM* was preferred by PBEE codes like ATC-58 and almost all evaluators and researchers. In this research, some common used scalar *IM* factors were evaluated accompanied by a newly introduced scalar *IM*.

METHODOLOGY

Records

Despite the high variability in ground motions, earthquake engineers would ideally like to select as few representative ground motions as possible for design purposes, having critical ground motion properties are expected to demonstrate a certain response within a given structure. This is mainly because the non-linear modeling and dynamic analysis are computationally expensive, while still being inevitable in earthquake prone areas. It is true that by increasing the number of records, the variability related to record-to-record will be reduced, but each percent of reduction expenses much with respect to non-linear dynamic analysis. In addition, the intent is not to reduce response dispersion by applying great quantities of records; however, the intent is to obtain an unbiased estimate of the structural response with limited error. The bias structural responses to the implemented records could reach the satisfying reliability level, if the served group of records is definite and not too large (Lignos et al., 2015). It is fine to mention that instead of enlarging the group of records, many studies like Iervolino and Cornell (2005), Wang (2010), Baker (2007), Reye and Kalkan (2014) and Haselton (2009) recommended the guidelines for selecting appropriate records for declining the dependency of responses on the number and selection procedures of the utilized records. For instance, preliminary results from the COSMOS 2007 workshop concluded that for a first-mode dominated structure, time histories that closely match the target spectrum conditioned on the period of the first mode of the structure can yield a good estimate of the median response of EDPs (e.g. Maximum inter-story drift ratio) for the scenario of an earthquake (Haselton, 2009).

Regarding the number of ground motions, the typical practice in structural design is to use seven motions according to ASCE05-7 and eleven ground motions as stated by ATC, but the appropriate number of motions is still a topic of desired researches. According to the ASCE/SEI-7 (ASCE, 2010), if at least seven ground motions are analyzed, the design values of engineering demand parameters (EDPs) are taken as the average of the EDPs determined from the analyses. If fewer than seven ground motions are analyzed, the design values of EDPs are taken as the maximum values of the EDPs and by utilizing fewer than seven ground motions the ASCE/SEI-7 scaling procedure is conservative. Pointing out that the ground motions may exhibit significant variability in frequency content and amplitude, small dispersion (variability) of EDPs is desired as it provides an acceptable confidence level (Quiroz-Ramíreza et al., 2014).

A suit of randomly selected eleven pairs of ground motions is the minimum recommended by the ATC-58. Such a suite will provide 75% confidence that the predicted median response will be with ±20% of the true median value of response for an assumed dispersion of 0.5 (ATC-58, 2011). In this respect, by the assumption of normal or lognormal distribution for EDPs, 75% confidence provides a good condition for reaching the median values. Regarding for example, 99% confidence level for a coefficient of variation equivalent to 4 for normal distribution, the coefficient of variation obtained in this study are smaller than this value and could get to the specified level of confidence with less number of specimens.

With respect to the considerable effects of pulse motions on dynamic responses of structures, the database in this study comprises eleven near-fault earthquake records identified as containing distinct velocity pulses and enclosing source-to-site distances less than 10 km and all of them were recorded on free-fault sites classified as site class D (stiff soil, very dense soil and rock) based on NEHRP site classification, equal to Zone 4 of UBC (UBC, 1997) and soil type II according to the Iran Seismic Code (2800 standard, 2005), or adjusted for this class of soil. Moreover, eleven far-fault records were supplemented to comprehend the comparison. All far-fault records have distances above 50 km and do not include any pulse-like wave. Table 1 presents complete specifications of the selected ground motions.

These records have been employed in many previous researches in the PEER and SAC centers and could be applied in many studies in this field (Somerville et al., 1997a; Somerville et al., 1997b). Recorded motions were derived from a bin of recorded motions including PEER Strong Motion Database (PEER, online) and Iran Strong Motion Network Data Bank (BHRC, online). The effects of horizontal shaking were considered serving the east-west components of the records along the 2D models.

Table 1. Specifications of ground motions

Near-fault Ground Motions					
Earthquake	**Year**	**Station**	**Distance (km)**	**M_W**	**Duration (sec)**
Tabas	1978	Tabas	1.2	7.4	32.84
Bam	2003	Bam	1.0	6.8	66.56
Loma Prieta	1989	Los Gate	3.5	7.0	24.96
Mendocino	1992	Petrolia	8.5	7.1	35.98
Erzincan	1992	Erzincan	2.0	6.7	20.78
Landerz	1992	Lucerne	1.1	7.3	48.12
Northridge	1994	Olive View	6.4	6.7	39.98
Kobe	1995	JMA	0.6	6.9	47.98
Chichi	1995	TCU068	1.1	7.6	90.00
Superstition Hill	1987	Parachute T.S	1.0	6.5	10.5
Coalinga	1983	Transmitter Hill	9.5	5.8	3.9
Coyote lake	1979	Gilroy Array	3.1	5.7	3.4
Far-fault Ground Motions					
Earthquake	**Year**	**Station**	**Distance (km)**	**M_W**	**Duration (sec)**
Tabas	1978	Ferdoos	94.4	7.4	40.00
Morgan Hill	2003	Morgan	76.3	6.8	36.00
Landerz	1992	12026 Indio	55.7	7.3	60.00
Whittier Narrows	1987	Downey - Birchdale	56.8	6.0	29.0
Imperial Valley	1979	Victoria	54.1	6.5	86.0
Northridge	1994	Terminal Island	60.0	6.7	40.0
Northridge	1994	Lakewood- Del Amo	59.3	6.7	35.0
Loma Perieta	1989	APEAL 2E	57.4	6.9	25.0
Loma Perieta	1989	Alameda Naval	70.9	6.9	46.0
Chichi	1999	TCU094	54.5	7.6	25.5
Chichi	1999	TCU026	56.1	7.6	22.1

Scaling Ground Motions

Probabilistic seismic demands were obtained through Incremental Dynamic Analyses (IDA) of a building subjected to a suite of ground motions. In IDA, the intensity of each record increased after each inelastic dynamic analysis, using *IM* as the seismic intensity scaling index. One method of scaling is to choose a point or domain as a reference point or domain usually in the case of design spectrums. This requires presumed intensity as the reference intensity and achieves different scaling factors for different records depending on the type of soil, first fundamental period (T_1) and also the number and type of incorporated records. Since the peak acceleration in near-fault ground motions occurred in periods less than T_1 and scaling performed at the T_1 point, this procedure provides big scaling factors for these records in company with the far-fault records, making it very difficult in convergence procedure of analyzing subject of near-fault ground motions. Also, this procedure provides the same intensity level of all earthquakes in reference points, in addition to its requirement so as to distinguish the reference spectrum for scaling. The other method is scaling the ground motions in some corresponding intensity levels. In this research, the *IM* magnitudes of records were scaled in four levels of 0.1, 0.5, 0.8 and 1.0 for the *IM* values. Since utilizing big scale factors incurs unrealistic structural responses contributing to collapse mode and problems in converging the analysis, the scale factors prefer to be less than unity for near-fault ground motions. However, these scaling factors may be inadequate in crossing the models to the threshold of non-linear behavior for the far-fault records. Therefore, the scaling factors expanded by ratios of 1.2 and 1.5 of the *IM* values. The behavior of models in non-linear situations under near and far-fault ground motions have been checked under these scaling factors.

Description of Structural Systems Used For Evaluation

On account of the need for generality of the results, the assumed models were not intended to represent a specific structure. For this purpose, the efficiency of the IMs was considered by conducting Incremental Dynamic Analysis (IDA) of two dimensional generic one-bay frames proposed by Krawinkler and Medina (2004). It is worth noting that their study in company with some others, proved that one-bay generic frames are generally adequate to capture the global behavior of multi-bay frames (Yahyaabadi and Tehranizadeh, 2011).

The generic frames utilized in this study consist of frames with a number of stories, N, equal to 3.In addition, for consideration of period softening and going towards the non-linear behavior, the period of designed models considered are equal to 0.45(s) and 1.062(s); however, for low-rise buildings these effects are not so dominating. The height of each story and the length of each span are deemed equal to 3 and 6m respectively. The frames were modeled by means of the open system for earthquake engineering simulation (Opensees, 2009). Since the numbers of deemed periods are two and the numbers of records are 22 (11 near-fault and 11 far-fault ground motions), then the number of models is equal to 44 (2×22). The designed specifications for the models by different periods are presented in Table 2.

Plastification was modeled using nonlinear material gained from parallel aggregation of some elastoplastic materials. All the non-linear dynamic analyses were conducted as Direct Integration Transient time history analyses using Direct Integration in Hilber, Hughes and Taylor's method by consideration of damping ratio for all modes equal to 5% and P-Δ effects.

Respecting that the efficiency of the EDPs should be evaluated at the collapse level, as well as the other levels of inelastic response, the hysteretic model has to incorporate all the important deterioration sources that contribute to demand prediction as the structure approaches collapse. In this study, the served deterioration model was reconciled acceptably by the model proposed by Ibara et al. (2005), which permits modeling of four major sources of cyclic deterioration (basic strength, post-capping strength, unloading stiffness and accelerated reloading stiffness). This model incorporates the cyclic deterioration controlled by hysteretic energy dissipation, as well as the deterioration of the backbone curve similar to the real-world structures which do not have infinite displacement capacity while such systems are able to explicitly take into consideration the effect of stiffness and strength degradation (Dimakopoulou et al., 2013). The amounts of each point for cyclic deterioration model were derived from the specification of steel A992Fy50 and exhibited in Figure 1.

The selected EDPs used in this study are performance-based assessments such as inter-story drift ratio (IDR) and peak floor acceleration (PFA). The IDR and PFA which account for computing the standard deviations are the ones on the roof story.

Diagrams of the static non-linear behavior of the models are presented in Figure 2, conducted based on FEMA 273.The collapse has been considered as a progressive one because it is a chain reaction of failures propagating throughout a portion of the structure disproportionate to the original local failure occurring when a sudden loss of a critical load-bearing element initiates a structural element failure, eventually resulting in partial or full collapse of the structure (Zahrai and Ezoddin, 2013). The pushover diagrams illustrated acceptable non-linear capacity of the models. In addition, non-linear responses of the node located in the beam to column connection point in story 1, subjected to the four near-fault and two far-fault ground motions based on the scale factor of 1 are presented in Figures 3. The stated diagrams present proceedings of the model's behavior in the non-linear region under both near and far-fault ground motions. Some intensity measures like $S_{di}(T_1)$ (Inelastic Spectral Displacement) that is defined in non-linear situation could be defined for both cases of near and far-fault records in this study.

Table 2. Design specifications for models by different periods

T		Story 1	Story 2	Story 3
0.45	Column Section	Box 35×35×2	Box 35×35×2	Box 30×30×2
	Beam Section	IPE360	IPE360	IPE330
1.062	Column Section	Box 20×20×2	Box 20×20×2	Box 15×15×1.5
	Beam Section	IPE270	IPE270	IPE240

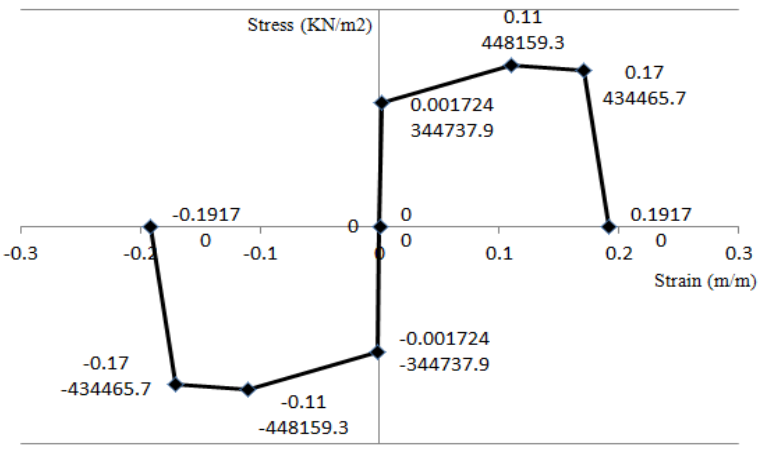

Fig. 1. Non-linear behavior of material used for nonlinear modeling

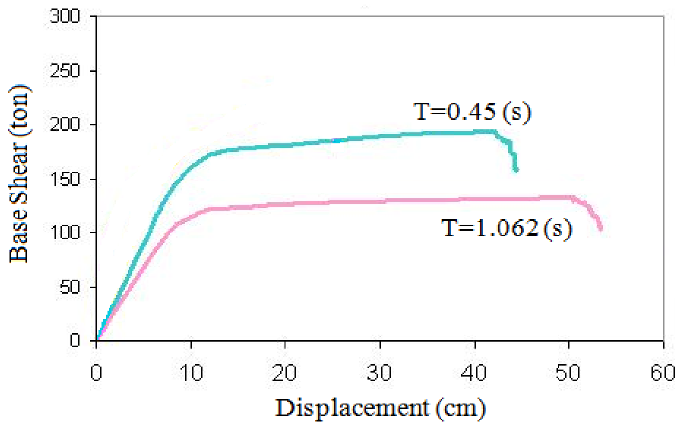

Fig. 2. Static non-linear behavior of the models

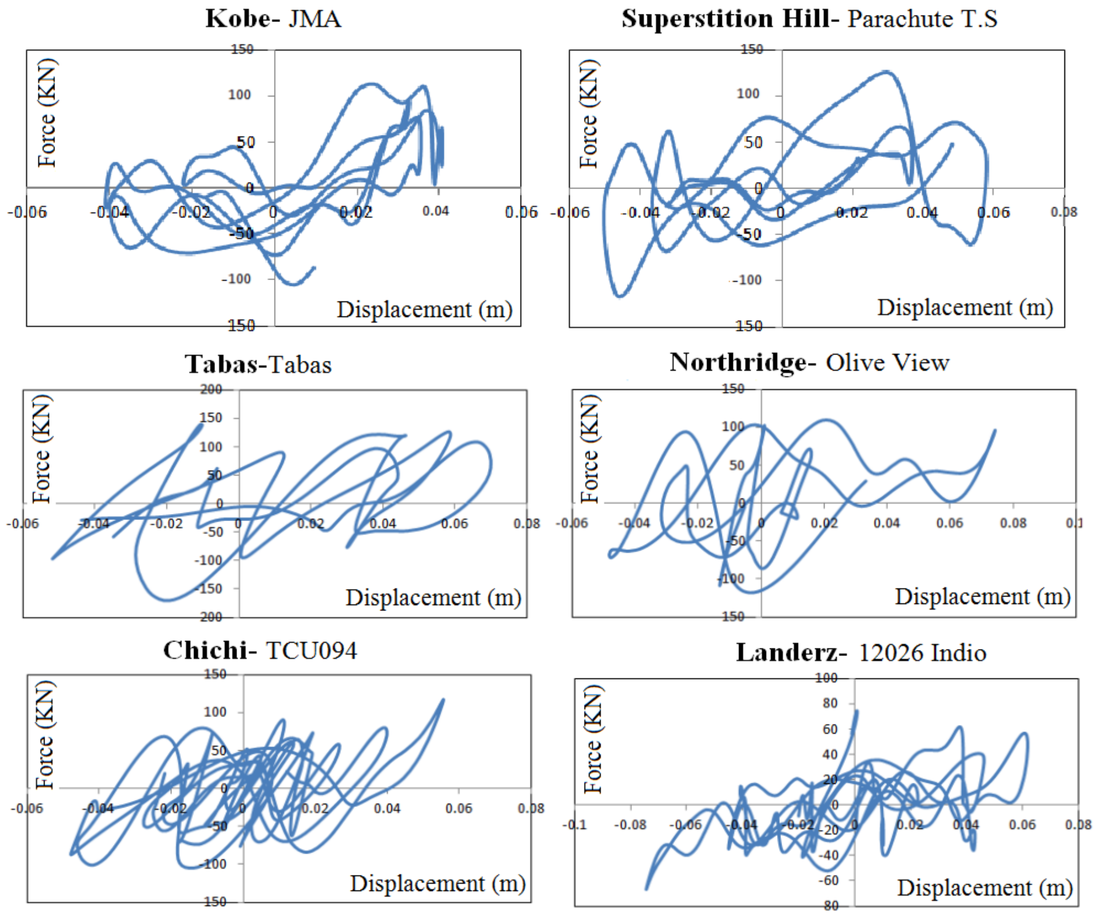

Fig. 3. Hysteresis diagrams for displacement of the node located in the beam to column connection point in story 1, subjected to the four near-fault and two far-fault ground motions based on the scale factor of 1

Considered Ground Motion Intensity Measures

Several alternative *IMs* have been proposed in recent studies with respect to the seismological characteristics of records and structural configurations of models. Some frequently used *IMs* that were recently worked on in many researches are briefly introduced in this section. Comparative evaluative standard deviation analysis could service the designers and evaluators in the procedure of *IM* selection taking into account both near and far-fault ground motions with respect to efficiency aspect.

In the initial investigation, intensity measures were divided into two categories in terms of their definitions; non-structure-specific and structure-specific intensity measures (Mollaioli et al., 2013). The non-structure-specific intensity measures are the ones which are independent from the specifications of the building like period, inelastic specifications, structural responses or mode participation factors. The structure-specific intensity measures are dependent parameters on the specifications of the models and could be computed after calibrating model's characteristics (Bradley, 2012). One of the attempts of this research is to reveal the efficiency of intensity parameters based on this classification to be a guide for evaluators for responding to this question that is it worth switching to the structure-specific intensity measures just for making a slight improvement in efficiency. The evaluation was conducted based on these parameters:

PGA: This is a non-structure-specific *IM* defined as the peak ground acceleration of the ground motion. Since calculation of this *IM* is very straightforward and does not require computation of the structural response, it was manipulated widely in preliminary studies. Non-structure-specific *IMs* are preferred for near-fault ground motions from a seismology standpoint. However, they do not incorporate spectral characteristics of the structures.

PGV: This is a non-structure-specific *IM* defined as the peak velocity of the earthquake's ground motion.

$S_a(T_1)$: This is the elastic acceleration spectral ordinate evaluated at the model's fundamental period of vibration, T_1. This intensity measure mainly facilitated *IM* both in practice and research. In part, this *IM* choice was driven by convenience, as seismic hazard curves in terms of spectral acceleration at the fundamental period of structure are either readily available (e.g., from the U.S. Geological Survey at http:/geohazards.cr.usgs.gov/eq/) or commonly computed.

The maximum demand of EDP in structures under near-fault records is affected by the ratio of near-fault pulse period to the fundamental period of the structure (Campbell and Bozorgnia, 2014; Zhong et al., 2013); and as such $S_a(T_1)$ could not adequately predict the seismic demands of structure under near-fault pulse-like records. Another important shortcoming of the $S_a(T_1)$ is its inability in describing the effective frequency content of earthquakes at a period not equal to the fundamental period of the structure. This dominates higher mode period elongation effects due to non-linearity (Bozorgnia and Bertero, 2004). This weakness is more pronounced when pulse motions dominate the structural responses. These inadequacies could be approximately improved through the use of vector-type IMs (Baker and Cornell, 2005; Baker and Cornell, 2006). Nevertheless, pulse like motions cannot be adequately characterized by the means of $S_a(T_1)$, since their response spectra usually exhibit a sharp conversion, making it difficult to simply estimate spectral shape by this type of IM (Tothong and Luco, 2007). It is worth mentioning that the displacement spectral ordinate $S_d(T_1)$ could be considered instead of acceleration spectral ordinate by the modification factor of $(\frac{T_1}{2\pi})^2$.

$S_v(T_1)$: This can be defined as the elastic velocity spectral ordinate evaluated at the fundamental period of vibration for the structural model, T_1. Velocity response spectrums in the fault-normal component of the near-fault records contain at least one predominate peak, which provides a good estimation of the period of the pulse contained in the near-fault records (Krawinkler and Medina, 2004). In some cases, the period of these pulses and the structural predominate period match each other considering the velocity pulse effects through implementation of $S_v(T_1)$ as the *IM* parameter. However, in most cases, this matching does not take place and this *IM* includes some deficiencies for accounting pulse effects. This feature is one of the points that motivated us to introduce a new *IM* factor based on velocity characteristics which are unrestricted from deficiencies of the previous velocity-based *IMs* (Bradley, 2012).

S_{di} (T_1): This can be defined as the inelastic spectral displacement considered in some studies, in order to reflect the period shift effect in near-fault ground motions (Tothong and Luco, 2007; Luco and Cornell, 2007). This *IM* was calculated through the SDOF system with an elastic perfectly plastic hysteresis behavior evaluated at T_1, and with a yield displacement of Δ_{ySDOF} calculated as:

$$\Delta_{ySDOF} = \frac{\Delta_{yr}}{\Gamma_1 \phi_{1,r}} \tag{2}$$

where Δ_{yr}: is the roof displacement for MDOF model at yielding, estimated from static pushover analysis applying the first mode lateral load pattern. Γ_1: is the modal participation factor of the first mode and $\Phi_{1,r}$: is the amplitude of the first mode at the roof level (Aslani, and Miranda, 2005; ATC-58, 2011). While this *IM* is generally accurate and has the ability to describe the period elongation effects, one drawback of the non-linear spectral values is that they imply a coupling between the earthquake hazard definition and the inelastic structural properties that it requires inelastic SDOF time history analyses and complicates development of seismic hazard maps for general practice.

Δ_{cdc}: This implies the combination of the spectral displacement evaluated at two periods of vibration incorporating both period softening and higher mode effects and thereby reducing record-to-record variability (Cordova and Deierlein, 2000). This intensity parameter could be calculated as:

$$\Delta_{cdc} = S_d(T_1)\left(\frac{S_d(cT_1)}{S_d(T_1)}\right)^{\alpha} \tag{3}$$

where $S_d(T_1)$: is the displacement spectral ordinate evaluated on the structure's first fundamental period of vibration. c and α are constant parameters that can be tailored to achieve a certain level of preciseness for a specific structural model. Δ_{cdc}: is equal to the geometric mean of $\Delta_e(T_1)$ and $\Delta_e(2T_1)$ through application of these suggested amounts of the pair of $c = 2$ and $\alpha = 0.5$ as stated in this research and that reported by Cordova et al. (2000).

$\Delta_{v_e v_{\max}}$: This *IM* merges the amount of maximum velocity that is correlated strongly to the pulse intense and the amount of velocity spectrum at the structural fundamental period which implicitly represents the distance of the pulse by the amplitude of spectral velocity in the fundamental period of the model.

Distinguishing the magnitude of velocity pulses and corresponding period of pulse occurrence are concepts under discussion as well as they are very computationally expensive (Campbell and Bozorgnia, 2012). Hence, to describe the peculiar spectral shape of pulse-like records that has been observed chiefly in near-fault records through applying a simple index, this paper assessed in company with some frequently served *IM*, a new introduced *IM* factor that aggregates both non-structure-specific and structure-specific terms which is defined as the geometric mean of spectral velocity evaluated at the structure's first period of vibration and maximum amount of velocity record. This *IM* proposed by Najafi and Tehranizadeh (2015) and could be calculated as:

$$\Delta_{v_e v_{\max}} = (\Delta_{ev}(T_1) V_{\max})^{0.5} \tag{4}$$

where $S_v(T_1)$: is the elastic velocity spectral ordinate evaluated at the fundamental period of vibration and V_{max}: is the maximum amount of velocity record.

a_{rms}: It is an *IM* proposed by Trifunac and Bradly (1975) according to the radical square means of accelerations in the domain of 5 to 95% of the record duration.

$$a_{rms} = \sqrt{P_a} \quad (5)$$

$$P_a = \frac{1}{t_2 - t_1} \int_{t_1}^{t_2} a^2(t)dt \quad (6)$$

where $t_1=t_{0.05}$, $t_2=t_{0.95}$ and t: is record duration.

a_{Sq}: Square of acceleration values.

$$a_{Sq} = \int_0^{t_f} a^2(t)dt \quad (7)$$

where t_f: is total time duration of record.

a_{rs}: Radical of square values of acceleration given as:

$$a_{rs} = \sqrt{a_{Sq}} \quad (8)$$

I_c: Proposed by Park et al. (1984) given as:

$$I_c = a_{rms}^{1.5} t_d^{0.5} \quad (9)$$

$$t_d = t_{0.95} - t_{0.5} \quad (10)$$

I_a: Proposed by Riddel and Garcia (2001) given as:

$$I_a = PGA.t_d^{\frac{1}{3}} \quad (11)$$

EPA: Effective Peak Acceleration; The mean of acceleration spectral values between T=0.1(s) and T=0.5 (s) divided by 2.5. (Applied in ATC-58, 2011) given as:

$$EPA = S_a(0.1s, 0.5s)/2.5 \quad (12)$$

IM_{1eff}: This *IM* considers the period softening effects proposed by Cordova et al. (2000) given as:

$$IM_{1eff} = \sqrt{\frac{S_d(2T_1,\zeta_1)}{2S_d(T_1,\zeta_1)}} \left|\Gamma_1^{[1]}\right| S_d(T_1,\zeta_1) \quad (13)$$

where $\left|\Gamma_1^{[1]}\right|$: is the first-mode participation factor in maximum drift and S_d: is the elastic acceleration spectral ordinate evaluated at the model's fundamental period of vibration, T_1 with damping ratio of ζ_1.

Requirements for Selected Intensity Measures

The goal of most studies of improved intensity measures is to characterize ground motion hazards in a statistically meaningful way for predicting structural performance. This implies that the best intensity measures are those that contribute to the least record-to-record variability, measured with respect to a common intensity index when evaluating structural performance to multiple earthquake sets. Obviously, even with the best ground motion characterization, uncertainties will persist in characterizing the geologic earthquake hazard and in simulating inelastic structural performance. Desirably, the point estimators for EDPs evaluated by the certain intensity measure should have three properties: consistency, efficiency and sufficiency.

A point estimator is consistent if its error asymptotically decreases with the enlargement in the sample size. On the basis of the law of large numbers, it could be shown that for different intensity measures the point estimators of various types of structural response, EDPs, are consistent. Hence, the consistency of EDPs is not going to be discussed further in this study (Aslani and Miranda, 2004; Benjamin and Cornell, 1970; Aslani and Miranda, 2004).

A point estimator was considered more efficient if it leads to a smaller dispersion in comparison to the other point estimators of the same seismic performance parameter. In this study, the standard deviation of natural logarithm of EDP parameters was utilized to compare dispersion around the median values for each EDP parameter and have been assessed for each of the models subjected to a suite of far-fault and near-fault earthquake records.

In favor of improved understanding about the dispersion of results around the mean value, and also to restrict the values of standard deviations from the united EDP measurement, the coefficient of variation parameter (COV) substitute the standard deviation parameter, where it could be calculated by Eq. (14) as:

$$\mathrm{COV} = \frac{\sigma}{\mu} \tag{14}$$

where *COV*: is coefficient of variation parameter, σ: is standard deviation and μ: is mean value.

Another important aspect in evaluation of structure-specific *IM* is the dependency of structural response parameters on the other seismological aspects, such as its magnitude and source-to-site distance. An estimator was considered sufficient if it utilizes all the information in the sample that is relevant to the estimation of the seismic performance parameter (Aslani and Miranda, 2005). This feature can significantly affect the level of complexity of the structural response estimations and eventually impacts the runtime, though this aspect is out of the field of this study.

EVALUATION AND DISCUSSION OF DISPERSION RESULTS

The amounts of *COV* for the *IM* factors in each of the scaling level are reported in Tables 3 to 6 for the models with two different periods. Also, the amounts of *COV* for scaling level of unite could be displayed schematically by the help of the diagrams of Figures 4 and 5 for far-fault and near-fault ground motions presenting wavering inherent of dispersion for structural responses in case of the near-fault records subjected to different intensity measures.

Table 3. COV values of inter-story drift ratios according to different scaling levels subjected to near and far-fault records for the model by fundamental period of 0.45 (s)

IM Factors	COV Values For IDR											
	Near-Fault Records						Far-Fault Records					
	0.1 IM	0.5 IM	0.8 IM	1.0 IM	1.2 IM	1.5 IM	0.1 IM	0.5 IM	0.8 IM	1.0 IM	1.2 IM	1.5 IM
PGA	0.399	0.395	0.399	0.408	0.415	0.426	0.225	0.276	0.275	0.276	0.281	0.292
PGV	0.958	0.897	0.956	0.962	0.978	0.983	0.252	0.257	0.252	0.253	0.252	0.254
$S_a(T_1)$	0.334	0.338	0.334	0.333	0.356	0.381	0.261	0.261	0.261	0.260	0.263	0.273
$S_v(T_1)$	0.522	0.540	0.528	0.526	0.541	0.567	0.161	0.162	0.162	0.161	0.172	0.182
$S_{di}(T_1)$	0.508	0.503	0.505	0.507	0.512	0.516	0.282	0.281	0.281	0.282	0.289	0.291
Δ_{cdc}	0.126	0.129	0.129	0.126	0.135	0.143	0.287	0.287	0.288	0.289	0.291	0.298
$\Delta_{v_e v_{max}}$	0.200	0.206	0.209	0.207	0.209	0.240	0.279	0.240	0.241	0.242	0.267	0.278
a_{rms}	2.375	1.964	2.829	2.348	2.634	2.768	0.266	0.267	0.266	0.265	0.273	0.283
a_{Sq}	2.749	2.746	2.741	2.750	2.879	2.984	0.273	0.274	0.275	0.273	0.289	0.302
a_{rs}	1.548	1.542	1.544	1.545	1.567	1.592	0.129	0.126	0.127	0.127	0.138	0.147
I_c	2.563	2.560	2.555	2.564	2.674	2.634	0.224	0.228	0.228	0.227	0.234	0.278
I_a	0.449	0.462	0.439	0.476	0.489	0.494	0.274	0.273	0.273	0.274	0.294	0.314
EPA	0.240	0.248	0.238	0.238	0.278	0.293	0.278	0.277	0.277	0.273	0.274	0.286
IM_{1eff}	0.45	0.301	0.300	0.299	0.430	0.426	0.288	0.286	0.282	0.289	0.296	0.308

Table 4. COV values of peak floor acceleration ratios according to different scaling levels subjected to near and far-fault records for the model by fundamental period of 0.45 (s)

IM Factors	**COV Values For IDR**											
	Near-Fault Records						**Far-Fault Records**					
	0.1 IM	**0.5 IM**	**0.8 IM**	**1.0 IM**	**1.2 IM**	**1.5 IM**	**0.1 IM**	**0.5 IM**	**0.8 IM**	**1.0 IM**	**1.2 IM**	**1.5 IM**
PGA	0.163	0.167	0.165	0.169	0.172	0.175	0.278	0.277	0.276	0.277	0.279	0.305
PGV	0.224	0.226	0.227	0.229	0.234	0.236	0.254	0.228	0.241	0.231	0.237	0.239
$S_a (T_1)$	0.299	0.302	0.310	0.308	0.321	0.324	0.262	0.263	0.262	0.263	0.264	0.274
$S_v (T_1)$	0.435	0.472	0.463	0.467	0.487	0.498	0.275	0.265	0.259	0.250	0.268	0.269
$S_{di} (T_1)$	0.501	0.509	0.508	0.523	0.534	0.529	0.282	0.282	0.282	0.283	0.289	0.292
Δ_{cdc}	0.179	0.184	0.186	0.198	0.214	0.219	0.289	0.286	0.285	0.289	0.284	0.292
$\Delta_{v_e v_{max}}$	0.188	0.187	0.195	0.196	0.204	0.206	0.225	0.242	0.242	0.242	0.245	0.246
a_{rms}	1.890	1.925	1.967	1.923	1.941	1.982	0.203	0.216	0.196	0.203	0.199	0.202
a_{Sq}	1.05	1.092	1.106	1.115	1.207	1.243	0.204	0.218	0.198	0.204	0.210	0.212
a_{rs}	0.194	0.208	0.210	0.214	0.218	0.224	0.256	0.287	0.289	0.257	0.267	0.274
I_c	0.540	0.578	0.584	0.587	0.594	0.624	0.234	0.236	0.232	0.234	0.239	0.241
I_a	0.261	0.267	0.269	0.273	0.275	0.280	0.274	0.274	0.274	0.274	0.275	0.279
EPA	0.312	0.324	0.325	0.337	0.332	0.345	0.280	0.278	0.278	0.279	0.281	0.284
IM_{1eff}	0.248	0.246	0.238	0.253	0.276	0.284	0.289	0.291	0.293	0.294	0.299	0.302

Table 5. COV values of inter-story drift ratios according to different scaling levels subjected to near and far-fault records for the model by fundamental period of 1.062 (s)

IM Factors	**COV Values For IDR**											
	Near-Fault Records						**Far-Fault Records**					
	0.1 IM	**0.5 IM**	**0.8 IM**	**1.0 IM**	**1.2 IM**	**1.5 IM**	**0.1 IM**	**0.5 IM**	**0.8 IM**	**1.0 IM**	**1.2 IM**	**1.5 IM**
PGA	0.939	0.935	0.929	1.028	1.045	1.156	0.375	0.396	0.407	0.403	0.438	0.433
PGV	0.758	0.697	0.756	0.762	0.778	0.783	0.206	0.201	0.210	0.211	0.212	0.221
$S_a (T_1)$	0.434	0.428	0.437	0.435	0.454	0.428	0.336	0.326	0.325	0.326	0.324	0.337
$S_v (T_1)$	0.592	0.590	0.598	0.596	0.601	0.617	0.231	0.232	0.242	0.245	0.242	0.252
$S_{di} (T_1)$	0.548	0.541	0.533	0.535	0.532	0.536	0.378	0.378	0.375	0.367	0.363	0.365
Δ_{cdc}	0.156	0.159	0.159	0.156	0.165	0.173	0.187	0.187	0.188	0.189	0.196	0.198
$\Delta_{v_e v_{max}}$	0.304	0.312	0.306	0.304	0.313	0.320	0.179	0.175	0.179	0.184	0.183	0.189
a_{rms}	3.627	3.629	3.608	3.634	3.623	3.637	0.316	0.317	0.316	0.315	0.323	0.323
a_{Sq}	3.045	3.046	3.041	3.047	3.086	3.098	0.343	0.344	0.345	0.343	0.349	0.352
a_{rs}	2.059	2.053	2.056	2.054	2.066	2.066	0.269	0.256	0.257	0.267	0.258	0.247
I_c	1.956	1.957	1.955	1.956	1.967	1.963	0.202	0.203	0.208	0.207	0.214	0.218
I_a	1.045	1.046	1.044	1.048	1.049	1.056	0.456	0.472	0.473	0.474	0.498	0.512
EPA	0.804	0.848	0.838	0.835	0.878	0.893	0.503	0.509	0.514	0.526	0.527	0.538
IM_{1eff}	0.959	0.936	0.930	0.998	0.954	0.943	0.365	0.362	0.345	0.362	0.378	0.394

Table 6. COV values of peak floor acceleration ratios according to different scaling levels subjected to near and far-fault records for the model by fundamental period of 1.062 (s)

IM Factors	COV Values For IDR											
	Near-Fault Records						Far-Fault Records					
	0.1 IM	**0.5 IM**	**0.8 IM**	**1.0 IM**	**1.2 IM**	**1.5 IM**	**0.1 IM**	**0.5 IM**	**0.8 IM**	**1.0 IM**	**1.2 IM**	**1.5 IM**
PGA	0.357	0.354	0.365	0.358	0.364	0.375	0.282	0.283	0.286	0.287	0.299	0.315
PGV	0.344	0.346	0.347	0.349	0.356	0.366	0.263	0.254	0.267	0.271	0.277	0.289
$S_a (T_1)$	0.389	0.408	0.415	0.418	0.421	0.424	0.269	0.272	0.278	0.286	0.284	0.294
$S_v (T_1)$	0.534	0.573	0.584	0.592	0.583	0.598	0.293	0.302	0.308	0.310	0.312	0.314
$S_{di} (T_1)$	0.621	0.612	0.628	0.633	0.624	0.629	0.302	0.314	0.318	0.320	0.322	0.325
Δ_{cdc}	0.203	0.206	0.216	0.218	0.226	0.239	0.339	0.336	0.335	0.339	0.344	0.354
$\Delta_{v_e v_{max}}$	0.254	0.264	0.257	0.253	0.250	0.246	0.325	0.322	0.322	0.322	0.335	0.346
a_{rms}	2.067	2.079	2.074	2.089	2.094	2.098	0.269	0.268	0.264	0.265	0.254	0.272
a_{Sq}	1.578	1.583	1.575	1.564	1.572	1.575	0.253	0.241	0.259	0.254	0.250	0.252
a_{rs}	0.348	0.354	0.359	0.363	0.369	0.354	0.318	0.314	0.319	0.315	0.327	0.326
I_c	0.603	0.610	0.615	0.612	0.613	0.624	0.334	0.338	0.339	0.346	0.357	0.358
I_a	0.342	0.348	0.359	0.373	0.375	0.376	0.328	0.327	0.339	0.349	0.345	0.346
EPA	0.424	0.428	0.431	0.437	0.439	0.449	0.310	0.317	0.319	0.326	0.327	0.334
IM_{1eff}	0.332	0.338	0.337	0.342	0.359	0.367	0.365	0.364	0.372	0.374	0.389	0.405

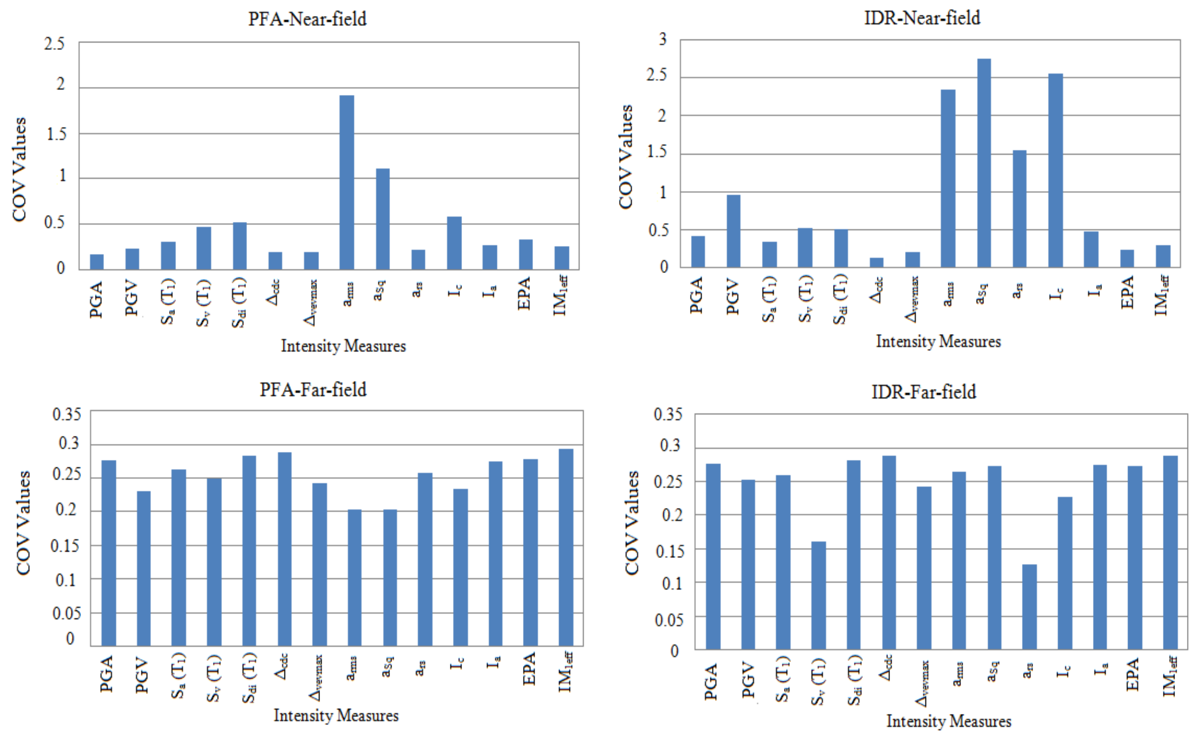

Fig. 4. The amounts of dispersion of structural responses for scaling level of unite subjected to near and far-fault ground motions for a model by period of 0.45 (s)

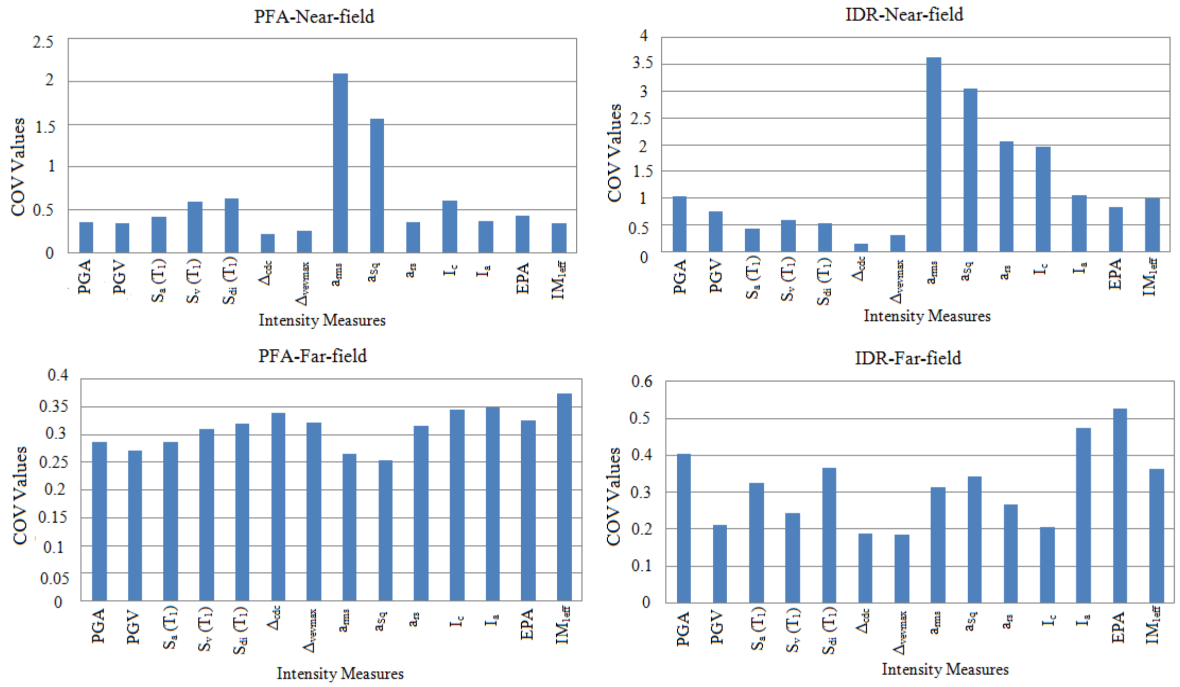

Fig. 5. The amounts of dispersion of structural responses for scaling level of unite subjected to near and far-fault ground motions for a model by period of 1.062 (s)

It could be inferred from these tables and figures that the *COV* values for far-fault records are almost close to each other subjected to both IDR and PFA. However, these values differ significantly from one *IM* to the other for near-fault records, presenting the prominence of choosing appropriate *IM* factor under near-fault ground motions. In addition, increasing the period of models also leads to increase in the amounts of *COV* values and the importance of distinguishing appropriate intensity measure becomes more obvious.

The scaling level of *IMs* do not play significant role in the amounts of the coefficient of variation values as the coefficient of variation are almost constant under different levels of scaling for both near and far-fault records. Although, the amounts of standard deviations of the structural responses are amplified by amplification in the scaling factors, as the mean values increase and the amounts of coefficient of variation remain approximately constant. In other words, scaling robustness of the incorporated intensity measures has been preserved by assuming coefficient of variation for evaluating dispersion amounts.

The non-structure-specific *IMs* of *PGA* has diminutive amounts of *COV* values. The obtained values of coefficient of variation are smaller for acceleration sensitive EDPs (PFA) than the drift sensitive ones (IDR) and also for short period models than the long period ones.

For short period building subjected to PFA, the intensity measure of *PGA* has the least amounts of *COVs* representing the efficiency of *IM*. For other situations, this *IM* could also be categorized in the group of intensity measures with small COVs; taking into account that calculating the amounts of *IMs* based on PGA is very straight forward, it emphasizes the fact of not complicating time-consuming *IM* factors, as the simple ones could bring about adequate certitude in EDP parameters in terms of efficiency. It should be noted at this point that the other important parameter of decision making about the *IM* parameters are consistency and sufficiency which have been studied in some of the other researches and is beyond the field of study in this paper. The intensity measure parameter should satisfy the consistency and sufficiency requirements. For a short noting, the study of Aslani and Miranda (2005) could be referred presenting very unsatisfying condition for *PGA* subjected to sufficiency view. That is the main purpose for switching from *IM* to some other *IMs* for example $S_a(T_1)$. In addition, considering the *PGA* amounts of the common earthquakes in a narrow range, the conclusion could be expanded that *PGA* covers more limited domain of IM than the other *IMs* (Najafi and Tehranizadeh, 2015). Theefficiency of intensity measures is the central issue of this paper.

The *IMs* of Δ_{cdc}, and the new proposed *IM* by the authors, $\Delta_{v_e v_{\max}}$ have moderately small amounts of dispersions around the mean values, illustrating the efficiency of these parameters, specially subjected to near-fault ground motions. The efficiency and sufficiency of Δ_{cdc} and $\Delta_{v_e v_{\max}}$ have been comprehensively assessed in the study of Najafi and Tehranizadeh (2015).

$S_a(T_1)$ is a very frequently applied *IM* in recent activities and codes also have small amounts of *COV* increase by converting from a short period model to the long period one. The differences of the *COV's* values under far-fault and near-fault ground motion was calculated based on this intensity measure and are little among the intensity measures with small amounts of *COVs* supporting to reach reasonable amounts of dispersion in a specific site, taking into account both near and far-fault ground motions. In addition, the sufficiency studies of $S_a(T_1)$ exhibiting outstanding satisfying sufficiency in comparison with some frequently used IMs. These considerations in addition to the very simple and straight forward required computations of this *IM*, that easily adopts spectral evaluations by non-linear characteristics, motivate researchers to utilize this *IM* without excessive assessments. However, this study confirms that in efficiency view some other IM could be substituted $S_a(T_1)$ with less dispersion around the mean and less computational efforts. For a more comprehensive study, the dispersion values around mean values for six frequently employed *IMs* from the list of applied *IM* in company with the new proposed IM by the authors, $\Delta_{v_e v_{\max}}$ (Najafi and Tehranizadeh, 2014), are presented in Figures 6 and 7.

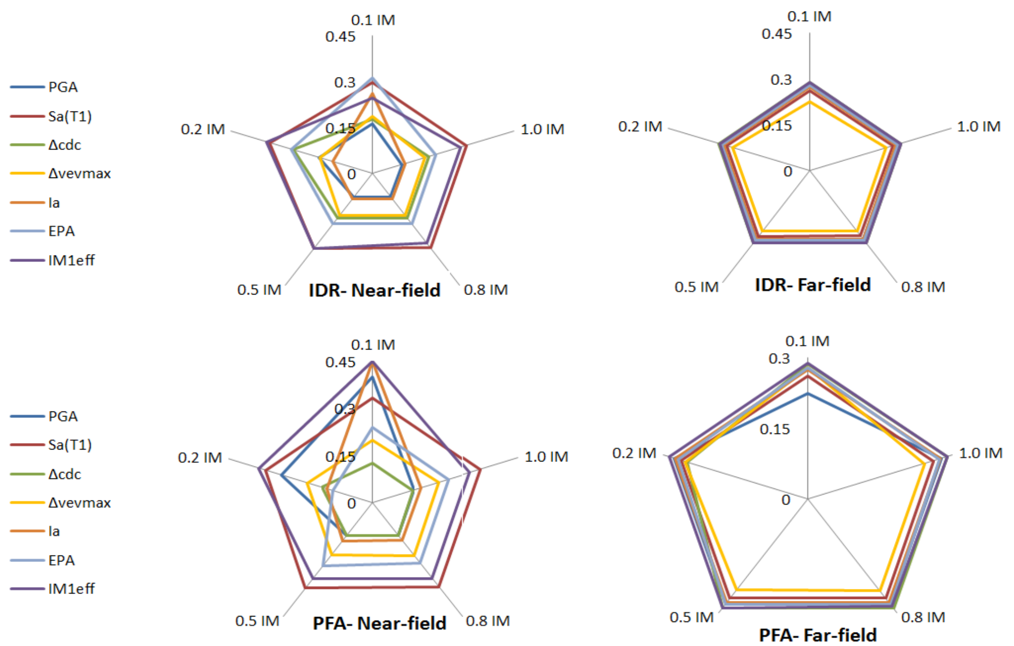

Fig. 6. The coefficient of variation of IDR and PFA subjected to near and far-fault ground motions for a model by the period of 0.45 (s)

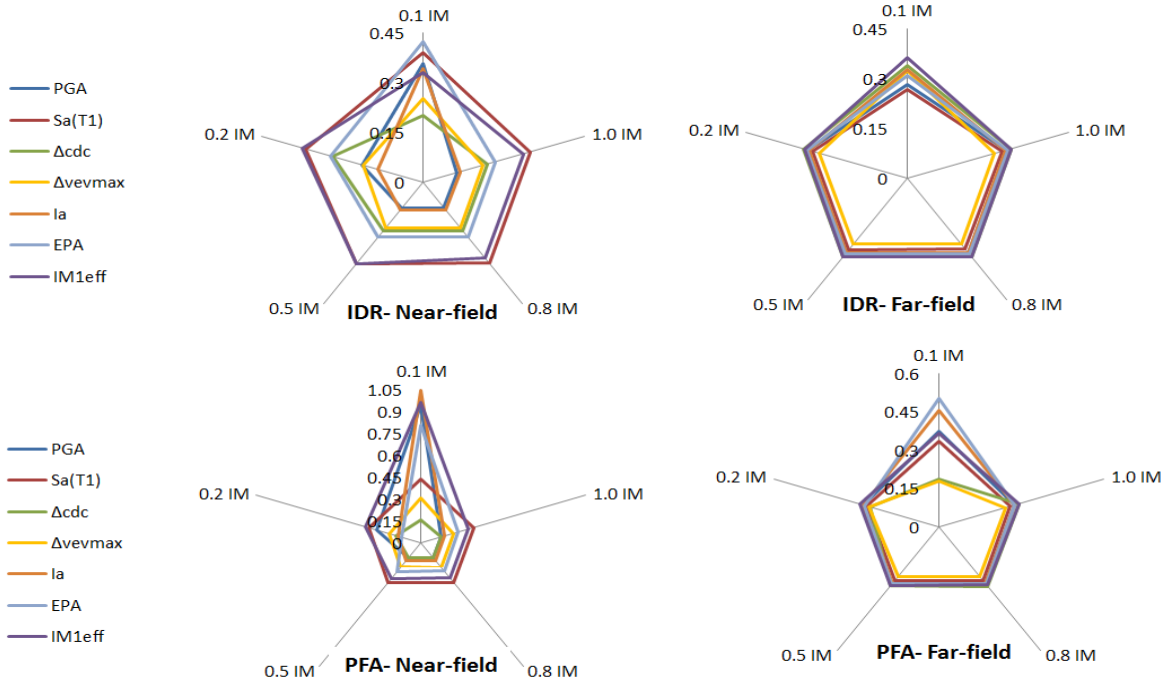

Fig. 7. The coefficient of variation of IDR and PFA subjected to near and far-fault ground motions for a model by the period of 1.062 (s)

As could be perceived under far-fault ground motions all the IMs concludes to the close amounts of dispersion; however, under near-fault records efficient IM could settle much less amounts of dispersion in structural responses and utilizing the new proposed *IM*, $\Delta_{v_e v_{\max}}$, could decline the dispersion of results both in far-fault and predominately in near-fault ground motions.

Selecting the appropriate *IM* was according to the aims of assessment, the amount of required certitudes, acceptable complicate computations and the amounts of time in calculations in company with bringing out to more accurate results of structural assessments and more reliable decision making parameters. Three characteristics of consistency, sufficiency and efficiency should be checked along with the predictability for a distinguished IM. For assessing efficiency for some frequently utilized intensity measures Tables 3 to 6 are very supportive, presenting very wavering amounts for near-fault ground motions.

CONCLUSIONS

The comprehensive comparative tables proposed in this paper could be an effective support in decision making procedure for intensity measure selection comprising most of the frequently utilized intensity measures.

The prominence of choosing an appropriate *IM* factor under near-fault ground motions was apparently presented, noting significantly different dispersion values of structural responses around the mean values switching from one *IM* to the other, despite the far-fault records that the dispersion values of structural responses for both IDR and PFA are almost close to each other under this type of ground motions. In addition, increasing the period of models the amounts of coefficient of variationalso increases and the importance of selecting appropriate intensity measure becomes more obvious.

The scaling level of IMs do not play a significant role in the amounts of coefficient of variation, as thecoefficient of variationvalues are almost constant under different levels of scaling for both near and far-fault records.

REFERENCES

ASCE. (2010). *Minimum design loads for buildings and other structures, ASCE/SEI 7-10*. American Society of Civil Engineers, Reston, Virginia.

Aslani, H. and Miranda, E. (2004). "Optimization of response simulation for loss estimation using PEER's methodology", *Proceeding of the 13th World Conference on Earthquake Engineering*, Vancouver, Canada.

Aslani, H. and Miranda, E. (2005). "Probabilistic earthquake loss estimation and loss disaggregation in buildings", Report 157, Ph.D. Dissertation, John A. Blume Earthquake Engineering Center, Stanford University, United State.

ATC-58. (2011). *Guidelines for seismic performance assessment of buildings*, Applied Technology Council, Washington D.C. Retrieved October 13, 2014, from https://www.atccouncil.org/pdfs/ATC-58-50 persent Draft.pdf

Baker, J.W. (2007). "Measuring bias in structural response caused by grand motion scaling", *Proceedings of the 8th Pasific Conference on Earthquake engineering*, Nangyang Technological University, Singapore.

Baker, J.W. and Cornell, C.A. (2006). ''Spectral shape, epsilon and record selection'', *Earthquake Engineering and Structural Dynamics*, 35(9), 1077-1095.

Baker, J.W. and Cornell, C.A. (2005). ''A vector-valued ground motion intensity measure consisting of spectral acceleration and epsilon'', *Earthquake Engineering and Structural Dynamics*, 34(10), 1193-1217.

Bazzurro, P. (1998). ''Probabilistic seismic demand analysis'', Ph.D. Thesis, Department of Civil Engineering, Stanford University, United State.

Benjamin, J. and Cornell, C.A. (1970). *Probability, statistics and decision for civil engineers*, McGraw-Hill, 1st ed., New York.

BHRC, Iran's Building and House Research Center, (2014). Retrieved October 13, from http://www.bhr.gov.ir.

BHRC. (2005). *Iranian code of practice for seismic resistant design of buildings*, Standard No. 2800, 3rd ed., Building and Housing Research Center, Tehran, Iran.

Bozorgnia, Y. and Bertero, V.V. (2004). *Earthquake engineering from engineering seismology to performance-based engineering, CRC Press*, Washigton DC.

Bradley, B.A. (2012). "Empirical correlations between peak ground velocity and spectrum-based intensity measures", *Earthquake Spectra*, 28(1), 17-35.

Campbell, K.W. and Bozorgnia, Y. (2014), "NGA-West2 ground motion model for the average horizontal components of PGA, PGV, and 5% damped linear acceleration response spectra", *Earthquake Spectra*, 30(3), 1087-1115.

Campbell, K.W. and Bozorgnia, Y. (2012), "Cumulative Absolute Velocity (CAV) and seismic intensity based on the PEER-NGA database", *Earthquake Spectra*, 28(2), 457-485.

Cordova, P.P., Deierlein G.G. (2000). "Development of a two parameter seismic intensity measure and probabilistic assessment procedure", *Proceedings of the 2nd US-Japan Workshop on Performance Based Earthquake Engineering Methodology for Reinforced Concrete Building Structures*, Hokkaido, Japan.

Cordova, P.P., Deierlein, G.G., Mehanny, S.F. and Cornell, C.A. (2000). "Development of a two-parameter seismic intensity measure and probabilistic assessment procedure", *Proceeding of the 2nd U.S.-Japan Workshop on Performance-Based Earthquake Engineering of Reinforced Concrete Building Structures*, Hokkaido, Japan.

Dimakopoulou, V., Fragiadakis, M. and Spyrakos, C. (2013), "Influence of modeling parameters on the response of degrading systems to near-field ground motions", *Engineering Structures*, 53, 10-24.

FEMA. (1997). *NEHRP guideline for seismic rehabilitation of buildings, building seismic safety council for the Federal Emergency Management Agency*, Report FEMA 273, Federal Emergencies Management Agencies, Washington D.C.

Gunay S. and Mosalam, K.M. (2013), "PEER performance-based earthquake engineering methodology, revisited", *Journal of Earthquake Engineering*, 17(6), 829-858.

Haselton, C.B. (2009). *Evaluation of ground motion selection and modification methods: predicting median interstory drift response of buildings*. PEER Report 2009/01, Pacific Earthquake Engineering Research Center, University of California, Berkeley.

Ibarra, L.F., Medina, R.A. and Krawinkler, H. (2005). "Hysteretic models that incorporate strength and stiffness deterioration", *Earthquake Engineering and Structural Dynamic*, 34(12), 1489–1511.

Iervolino, I. and Cornell, C.A. (2005). "Record selection for nonlinear seismic analysis of structures", *Earthquake Spectra*, 21(3), 685-713.

Krawinkler, H. and Medina, R. (2004). "Seismic demands for nondeteriorating frame structures and their dependence on ground motions'', Report PEER 2003/15, Pacific Earthquake Engineering Research Center, University of California at Berkeley, Berkeley, CA.

Lignos, D.G. and Krawinkler, H. (2013). "Development and utilization of structural component databases for performance-based earthquake engineering", *Journal of Structural Engineering*, 139 (Special Issue: NEES 2: Advances in Earthquake Engineering), 1382-1394.

Lignos, D.G., Putman, C. and Krawinkler, H. (2015). "Application of simplified analysis procedures for performance-based earthquake evaluation of steel special moment frames", *Earthquake Spectra* (in Press).

Lotfollahi-Yaghin, Gholipour Salimi, M. and Ahmadi, H. (2013), "Probabilistic assessment of pseudo-static design of gravity-type quay walls", *Civil Engineering Infrastructures Journal*, 46(2), 209-219.

Luco, N. (2002). "Probabilistic seismic demand analysis, SMRF connection fractures, and near source effect'', Ph.D. Dissertation, Department of Civil and Environmental Engineering, Stanford University, United State.

Luco, N. and Cornell, C.A. (2007). "Structure-specific scalar intensity measures for near-source and ordinary earthquake ground motions", *Earthquake Spectra*, 23(2), 357-392.

Mahdavi Adeli, M., Banazadeh, M., Deylami, A. and Alinia M.M. (2012). "Introducing a new spectral intensity measure parameter to estimate the seismic demand of steel moment-resisting frames using Bayesian statistics", *Advances in Structural Engineering*, 15(2), 231-245.

Mollaioli, F., Lucchini A., Cheng Y. and Monti, G. (2013). "Intensity measures for the seismic response prediction of base-isolated buildings", Bulletin of Earthquake Engineering, 11(1), 1841-1866.

Najafi, L.H. and Tehranizadeh, M. (2015). "New intensity measure parameter based on record's velocity characteristics", *Scientia Iranica, Transaction A: Civil Engineering*, 22(5), 1674-1691.

OpenSees (2009). "Open system for earthquake engineering simulation", Pacific Earthquake Engineering Research Center, Berkeley, CA.

Park, Y.J., Ang, A.H.S., and Wen, Y.K. (1984). "Seismic damage analysis and damage-limiting design of R/C buildings", *Civil Engineering Studies*, Technical Report SRS 516, University of Illinois, Urbana.

PEER, Pacific Earthquake Engineering Research Center (PEER), (2014). PEER strong motion database, Retrieved October 13, 2014, from http://peer.berkeley.edu/smcat.

Quiroz-Ramíreza, A., Arroyob, D., Terán-Gilmoreb, A. and Ordazc, M. (2014). "Evaluation of the intensity measure approach in performance-based earthquake engineering with simulated ground motions", *Bulletin of the Seismological Society of America*, 104(2), 669-683.

Ramirez, C. and Miranda, E. (2009). "Building-specific loss estimation methods and tools for simplified performance-based earthquake engineering'', Report No. 171, Ph.D. Dissertation, John A. Blume Earthquake Engineering Center, Stanford University, United State.

Reye, J.C. and Kalkan, E. (2014). "How many records should be used in ASCE/SEI-7 ground motion scaling procedure", USGS Report, Retrieved October 13, from http://nsmp.wr.usgs.gov/ekalkan/PDFs/Papers/J39_Reyes_Kalkan.pdf/

Shome, N. (1999). "Probabilistic seismic demand analysis of nonlinear structures", Ph.D. Dissertation, Department of Civil and Environmental Engineering, Stanford University, United State.

Shrey, S.K. and Baker, J.W. (2007). "Quantitative classification of near-fault ground motions using wavelet analysis", *Bulletin of the Seismological Society of America*. 97(5), 1486–1501.

Somerville, P., Smith, N., Graves, R.W. and Abrahamson, N.A. (1997). "Modification of empirical strong ground motion attenuation relations to include the amplitude and duration effects of rupture directivity", *Seismological Research Letters*, 68(1), 199–222.

Somerville, P., Smith, N., S. Punyamurthula, S. and Sun, J. (1997). "Development of ground motion time histories for phase 2 of the FEMA/SAC project", Report SAC/BD-97-04, Retrieved October 13, 2014, from http://www.sacsteel.org.

Tehranizadeh, M. and Movahed, H., (2011). "Evaluation of steel moment-resisting frames performance in tall buildings in near fault areas", *Civil Engineering Infrastructures Journal*, 44(5), 621-633.

Tothong, P. and Luco, N. (2007). ''Probabilistic seismic demand analysis using advanced ground motion intensity measures'', *Earthquake Engineering and Structural Dynamics*, 36(13), 1837–1860.

Trifunac, M.D. and Brady, A.G. (1975). "A study on duration of strong ground motions", *Bulletin of Seismological Society of America*, 65, 581-626.

UBC97. (1997). "Uniform building code", Vol. 2, *International Conference of Building Officials*, Whittier, CA.

Wang, G. (2010). "A ground motion selection and modification method preserving characteristics and aleatory variability of scenario earthquakes", *Proceeding of the 9th US national and 10th Canadian Conference on Earthquake Engineering*, Toronto, Canada.

Welch, D.P., Sullivan T.J. and Calvi, G.M. (2014). "Developing direct displacement-based procedures for simplified loss assessment in performance-based earthquake engineering", *Journal of Earthquake Engineering*, 18(2), 290-322.

Yahyaabadi, A. and Tehranizadeh, M. (2011). "New scalar intensity measure for near-fault ground motions based on the optimal combination of spectral responses", *Scientia Iranica, Transactions A: Civil Engineering*, 18(6), 1149-1158.

Zahrai S.M. and Ezoddin A.R. (2013). "Numerical study of progressive collapse in intermediate moment resisting reinforced concrete frame due to column removal", *Civil Engineering Infrastructures Journal*, 47(1), 71-88.

Zareian, F. and Krawinkler, H. (2012), "Conceptual performance-based seismic design using building-level and story-level decision support system", *Earthquake Engineering and Structural Dynamics*, 41(11), 1439–1453.

Zhong, J.F., Zhang, L.W. and Liang, J.W. (2013), "An improved source model for simulation near-field strong ground motion acceleration time history", *Applied Mechanics and Materials*, 438(1), 1474-1480.