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Optimal intensity measures for probabilistic seismic demand models of RC high-rise buildings

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Abstract. One of the important phases of probabilistic performance-based methodology is establishing appropriate probabilistic seismic demand models (PSDMs). These demand models relate ground motion intensity measures (IMs) to demand measures (DMs). The objective of this paper is selection of the optimal IMs in probabilistic seismic demand analysis (PSDA) of the RC high-rise buildings. In selection process features such as: efficiency, practically, proficiency and sufficiency are considered. RC high-rise buildings with core wall structural system are selected as a case study building class with the three characteristic heights: 20-storey, 30-storey and 40-storey. In order to determine the most optimal IMs, 720 nonlinear time-history analyses are conducted for 60 ground motion records with a wide range of magnitudes and distances to source, and for various soil types, thus taking into account uncertainties during ground motion selection. The non-linear 3D models of the case study buildings are constructed. A detailed regression analysis and statistical processing of results are performed and appropriate PSDMs for the RC high-rise building are derived. Analyzing a large number of results it are adopted conclusions on the optimality of individual ground motion IMs for the RC high-rise building.

Keywords: RC high-rise building; probabilistic seismic demand model; intensity measure; optimality; nonlinear time-history analysis, regression analysis

1. Introduction

In the last decades, the construction of high-rise buildings in seismic active areas has become an everyday design trend, that is mainly due to rapid growth of cities, and concentration of material resources in urban environments. In December 2011, the Council on Tall Buildings (CTBUH 2011) asserted that an average height of tallest building will double in only two decades, from the year 2000 to 2020. For that reason comprehensive probabilistic seismic analyses of RC high-rise buildings have to be conducted for seismic active areas. The Pacific Earthquake Engineering Research Center (PEER), which is currently conducting a large-scale research project called Tall Buildings Initiative (PEER TBI 2014), has been among the first to recognize the lack of research on this topic with regard to high-rise buildings.

Regarding to the specificity of the RC high-rise buildings behavior under seismic excitation current provisions and the regulations for the seismic design (etc. EN1998-1 2004) most often are not sufficient for their seismic design and analysis. Limitations of traditional design approach based on the forces have been recognized and probabilistic performance-based seismic design methodology becomes the basic approach of seismic analysis and design of the high-rise buildings (PEER TBI 2014, Ji *et al*. 2007). The one of the most important phase of probabilistic performance-based seismic design methodology is PSDA. In the process of PSDA, it is necessary to establish appropriate PSDM: the relationship between the ground motion IM and the DM. This relationship is necessary to obtain the conditional probability of exceeding a certain level of demand DM for a given IM in PSDA (Eq. (1)) as well as for generating analytical fragility curves.

$$P[DM/IM] = P[DM \geq dm/IM] \tag{1}$$

The selection of an optimal IM significantly affects the establishment of appropriate PSDM through reducing uncertainties associated with PSDM (Stewart *et al*. 2002). This is a question that has been studied for a long time in earthquake engineering. A number of studies have investigated the issue of IM selection, and a range of different IMs have been proposed for PSDA of buildings in general (Shome *et al*. 1998, Tothong and Luco, 2007, Luco and Cornell 2007, Baker and Cornell 2005, Kostinakis and Athanatopoulou 2015).

The peak ground acceleration (PGA), one of the ground motion amplitude IMs, was widely used in the past. Afterwards, spectral acceleration at the fundamental period $S_a(T_1)$ became widely adopted in the seismic analysis. The studies (Vamvatsikos and Cornell 2002, Ye *et al*. 2011) indicate that $S_a(T_1)$ is an efficient IM for the first mode-dominated structures. However, for high-rise buildings, application of $S_a(T_1)$ could be limited because does not account for contribution of period elongation caused by structural nonlinearity (Cordova *et al*. 2001), higher modes (Vamvatsikos and Cornell 2005) or spectral shape (Baker and Cornell 2005).

Cordova *et al*. (2001) proposed a combined IM to consider the period elongation effects, based on the spectral acceleration at the fundamental period and the spectral acceleration at the

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inelastic period. Baker and Cornell (2005) proposed a two-parameter vector IM consisting of S_a and epsilon. They founded that epsilon (defined as a normalized residual, the ratio of the difference between the logarithm of an actual record spectral acceleration and the mean of the logarithm given by ground motion prediction equation (GMPE) to the standard deviation of the GMPE) have significant ability to predict structural response.

Hereafter, Tothong and Luco (2007) proposed the inelastic spectral displacement S_{di} as the more advantageous relative to the conventionally used elastic spectral acceleration Sa and the vector IM consisting of S_a and epsilon ε for structural demands that are dominated by a first mode of vibration. Also, they indicated that for structural demands with significant higher-mode contributions S_{di} (alone) is not sufficient, so an advanced scalar IM that additionally incorporates higher modes is proposed.

In addition, for obtaining the fragility curves, several IMs are mostly used by the researchers: PGA (Mosalem *et al*. 1997), spectral acceleration at some periods $S_a(T_1)$ (Singhal and Kiremidjian 1997, Bayat *et al*. 2015a, Bayat *et al*. 2017, Bavaghar and Bayat 2017) and spectral displacement at selected periods $S_d(T_i)$ (Rossetto and Elnashai 2003, Nagashree *et al*. 2016). HAZUS MR4 (2003) have used spectral displacement $S_d(T_i)$, spectral acceleration $S_a(T_1)$ and PGA for defining fragility curves.

Today, in literature, there is a lack of information regarding to the most optimal IM for use in establishing PSDMs for RC high-rise buildings. Some studies (Lu *et al*. 2013, Zhang *et al*. 2017, Su *et al*. 2017) proposed new improved IMs considering the higher mode effects and the fact that high-rise buildings response frequency range is much wider than for low-rise or mid-rise buildings. Lu *et al*. (2013) proposed an improved ground motion IM based on spectral acceleration accounting higher modes. Zhang *et al*. 2017 developed a spectral-acceleration-based linear combination-type earthquake IM for high-rise buildings. Also, Martinez-Rueda (1998) conducted an analytical study for using a large number of earthquake records in order to identify spectrum IMs that optimizes the correlation with displacement ductility demand. They proposed the appropriate spectrum type IMs defined as the area under the elastic velocity spectrum for different range of modal periods of structures. Accordingly, the selection of an optimal IM has to be studied more in order to establish appropriate PSDMs for RC high-rise buildings.

The main objective of this paper is to identify the optimal IMs for establishing PSDMs for RC high-rise buildings. RC high-rise buildings with core wall structural system are selected as a case study building class with the three characteristic heights: 20-storey, 30-storey and 40-storey. In order to determine the most optimal IMs, 720 nonlinear time-history analyses are conducted for 60 ground motion records with a wide range of magnitudes and distances to source, and for various soil types, thus taking into account uncertainties during ground motion selection. A detailed regression analysis and statistical processing of results are performed and appropriate probabilistic seismic demand models (PSDMs) for the RC high-rise building are derived. The obtained results lead to conclusions on the optimality of IMs for the RC high-rise buildings and the direction of future research.

2. Review of probabilistic seismic demand analysis (PSDA)

PSDA defines appropriate PSDM for the considered building class: the relationship between the IM and DM. The procedure of defining relationship between the IM and DM consists in the following: 1) performing non-linear time history analyses of considered building class for selected ground motions, 2) plotting DMs against IMs and obtaining the scatter diagrams with points (IM_i,DM_i), 3) conducting the regression analysis on obtained scatter diagrams in order to define the regression parameters and DM/IM relationship. The different DM/IM relationship models could be analyzed through the regression analysis. It is founded that the DM/IM relationship can be established by power model with the strongest correlation between two variables (with the correlation coefficient greater than 0.80) (e.g., Pejovic and Jankovic 2015)

$$\widehat{DM} = a \cdot IM^b \tag{2}$$

where a and b are regression coefficients.

Furthermore, it is observed by many authors that distribution of seismic DMs corresponds to the lognormal distribution (e.g., Cornell *et al*. 2002, Pejovic and Jankovic 2015). Therefore, distribution of the random variable DM/IM, i.e. distribution of the seismic DM with regard to the IM, is lognormal with the following mean value

$$\mu_{DM/IM} = \frac{\sum_{i=1}^{n} ln\widehat{DM_i}}{N} \tag{3}$$

and with the standard deviation that is calculated as deviation of the natural logarithms of the residuals DM data obtained (on random sample) from the regression line, used to estimate dispersion

$$\sigma^2_{DM/IM} = \frac{\sum_{i=1}^{n}[lnDM_i - ln\widehat{DM_i}]^2}{N-d_f} \tag{4}$$

where N is the size of random sample and *df* =2 is the number of parameters being estimated in a regression on the DM data (parameters a and b).

The Eq. (1) can be rewritten in terms of lognormal distribution using two main parameters (mean value and standard deviation) in the following form

$$P[DM \geq dm/IM] = 1 - \Phi\left(\frac{\ln(dm) - \mu_{DM/IM}}{\sigma_{DM/IM}}\right) \tag{5}$$

where is the standard normal cumulative distribution function.

For the random variable DM/IM, the median is defined by Eq. (2), whereas the 16th and 84th percentiles, are defined as relationships that correspond to plus-minus standard deviation from the median.

3. IMs and DMs for PSDM

3.1 Considered IMs and DMs

The appropriate IMs for constituting PSDMs should comprise the greatest possible number of earthquake features such as the amplitude, frequency content, duration of strong part of ground motion etc. High-rise buildings are specific, due to their response frequency range that much wider than for low-rise or mid-rise buildings. For that reason, IMs comprising a wider range of frequency content of response spectra should be more appropriate for the high-rise buildings. The IMs and their abbreviations, studied in this paper, are listed in

Table 1. The most used IMs in seismic analysis (PGA and $S_a(T_1)$) do not comprise wider range of response spectra frequency content. PGA, peak ground velocity (PGV) and peak ground displacement (PGD) are representing ground motion amplitudes. PGA exerts the greatest influence on the seismic response of structures with periods less than 0.5s, while structures with periods higher than 0.5s, are more sensitive to PGV and PGD (Ji *et al.* 2007). $S_a(T_1)$, spectral velocity at the fundamental period $S_v(T_1)$ and spectral displacement at the fundamental period $S_d(T_1)$ are IMs which represent the specific points in response spectrum frequency content.

The Housner's mean spectrum intensity SI_H is defined as the area below the elastic spectrum of velocity between the periods of 0.1s and 2.5s (Housner 1952)

$$SI_H = \frac{1}{2.4}\int_{0.1}^{2.5} S_v\,(T)dT \tag{6}$$

The Matsumura mean spectrum intensity SI_m is defined as the area below the velocity spectrum between the periods T_y and $2T_y$, where T_y is the yield period of the structure (Matsumura 1992)

$$SI_m = \frac{1}{T_y}\int_{T_y}^{2T_y} S_v\,(T)dT \tag{7}$$

The Martinez-Rueda defined mean spectrum intensity SI_{yh} by proposing that the second integration limit in the integral of the Matsumura mean spectrum intensity SI_m be replaced with the period T_h which represents the new vibration period of the structure in the hardening range after yielding (Martinez-Rueda 1998)

$$SI_{yh} = \frac{1}{T_h - T_y}\int_{T_y}^{T_h} S_v\,(T)dT \tag{8}$$

The last two IMs (Eqs. (7) and (8)) take into account the increase of the modal period of vibration during the seismic action due to nonlinear stiffness and strength degradation. The corresponding period intervals (T_y, $2T_y$) and (T_y, T_h) are adopted for that reason. For the yield period of the structure T_y, MPF (mass participation factor) weighted average value at the first three modes is adopted (Eq. (9)) (Ji *et al*. 2007)

$$T_y = \frac{m_1 \cdot T_1 + m_2 \cdot T_2 + \cdots + m_n \cdot T_n}{m_1 + m_2 + \cdots m_n} \tag{9}$$

Table 1 IMs considered

Description	Intensity measures IM	Units
Ground motion amplitude parameters	PGA	m/s^2
	PGV	m/s
	PGD	m
Parameters representing concrete points of frequency content	$S_a(T_1)$	m/s^2
	$S_v(T_1)$	m/s
	$S_d(T_1)$	m
Parameters comprising a wider range of frequency content	SI_H	m/s
	SI_m	m/s
	SI_{yh}	m/s
	$S_{a,avg}$	m/s^2
	$S_{v,avg}$	m/s
	$S_{d,avg}$	m

where $m_1, \cdots m_n$ are mass participation factor of structural modes.

The value of the period T_h in the hardening range after yielding is determined using the nonlinear static pushover method, as proposed by Martinez-Rueda (1998), based on the following expression

$$T_h = T_y \cdot \sqrt{\frac{\mu}{1 + \alpha \cdot \mu - \alpha}} \tag{10}$$

where $\mu = \Delta_u / \Delta_y$ is the displacement ductility factor, Δ_u is the maximum displacement at the top of the structure, Δ_y is the yield displacement at the top of the structure, and α is the post-yield stiffness ratio.

The mean spectral values (mean spectral acceleration $S_{a,avg}$, mean spectral velocity $S_{v,avg}$ and mean spectral displacement $S_{d,avg}$) are the IMs that take into account the higher-mode effects and are derived as the combination of spectral values at first three modes (Pejović 2016)

$$S_{i,avg} = \frac{m_1 \cdot S_i(T_1) + m_2 \cdot S_i(T_2) + m_3 \cdot S_i(T_3)}{m_1 + m_2 + m_3} \tag{11}$$

In this paper, the interstorey drift (relative storey drift divided with the storey height) is selected as a seismic DM. It is the most frequently used DM for the buildings. The interstorey drift can be calculated very easily, as it is the direct result of the nonlinear time-history analysis. The two characteristic interstorey drift values are selected: maximum interstorey drift for the entire structure IDR_{max} and mean value of maximum interstorey drifts IDR_{sr}.

3.2 Characteristic of an optimal IM

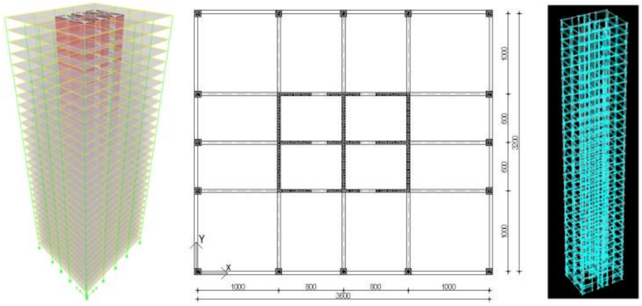
The selection of an optimal IM for constitution PSDM is not a trivial matter and has been the focus of numerous studies. An optimal seismic IM has to possess different features as it has been presented in literature (Luco and Cornell 2007, Giovenale *et al*. 2004, Mackie and Stojadinovic, 2001, Padgett *et al*. 2008, Bayat and Daneshjoo 2015, Bayat *et al*. 2015b). In this paper, features such as: efficiency, practically, proficiency and sufficiency are analyzed.

3.2.1 Efficiency of IM

The most used feature that has been examined in characterizing an optimal IM is efficiency. Efficiency of IM is measured by the degree of scatter, i.e. by the dispersion of the obtained DMs with respect to the regression fit line for the given value IM. Less dispersion of the results means more efficient IM and is represented in this study by lower $\sigma_{DM/IM}$ (Eq. (4)).

3.2.2 Practicality of IM

Practicality refers to whether or not there is any direct correlation between an IM and DM (Padgett *et al*. 2008). In the case of the not practical IM there is a little or no dependence of the demand level DM to the level of the IM. Practicality is measured by the regression model parameter *b* (Eq. (2)). The lower values of parameter *b* mean a less practical IM. When this parameter approaches to zero value, the IM contributes negligibly to the demand estimate. The IM with the larger regression model parameter *b* (the higher slope of regression line) is more practical.



(a) (b) (c)

Fig. 1 30-story case study building ETABS2013 model (left), plan view of the story (middle) and 30-story case study building PERFORM3D model (right)

3.2.3 Proficiency of IM

The proficiency is feature that represents the composite characteristic of practicality and efficiency defined by value of modified dispersion (Padgett *et al*. 2008)

$$\xi = \frac{\sigma_{DM/IM}}{b} \tag{12}$$

Padgett (2008) defined the measure of proficiency, or modified dispersion, on a simple way, substituting the power regression model (Eq. (2)) in the form of the PSDM (Eq. (5)). The composite measure of practicality and efficiency, as noted by Padgett (2008), could overcome the difficulties in balancing selection between these two features.

3.2.4 Sufficiency of IM

IM is sufficient if the seismic DM, for the given IM, is independent of earthquake magnitude, M and distance to source, R. For an accurate estimate of P(DM/IM) (Eq. (1)), it is necessary that the DM, for the given IM, should be independent of M and R. In the case that IM is not sufficient, it is necessary to change the Eq. (1) in the sense of addition of the new variables: M and R (Shome 1999).

The sufficiency of an IM is evaluated by performing the regression analysis on the residuals, ε of DMs, from the PSDM, to the ground motion characteristic, M and R. Residuals ε of DMs are "horizontal" distances between observed value of DMi and its estimate (Eq. (2)). Sufficiency is quantified by the p-value for the c estimate. C is slope of regression line of residuals of DMs on M or R (Luco and Cornell 2007). Hence, a small p-value (e.g., less than about 0.05) suggests that the estimated coefficient c is significantly different from 0, and therefore that IM is insufficient.

4. Description of RC high-rise case study buildings for PSDA

RC high-rise buildings with core wall structural system are selected as a case study building class. The three characteristic heights are considered: 20-storey, 30-storey and 40-storey. The plan view of the storey for all case study buildings, ETABS model (ETABS 2013) and PERFORM-3D model (PERFORM 2006) of the 30-storey case study RC high-rise building are shown on Fig.1. RC core wall structural system is a system that is applicable for RC high-rise buildings up to the 50 storey (Taranath 2010). The central core wall system assumes the entire seismic force, and RC frames along the perimeter assume the gravity load only (Taranath 2010). The main properties of the considered case study buildings are shown in Table 2.

Seismic analysis and design of the case study RC high-rise buildings are done according to Eurocode 2 (EN1992-1-1 2004) and Eurocode 8-1 (EN1998-1 2004). Seismic linear analysis of buildings is done using a multi-modal response spectrum analysis, considering higher mode effects. For linear analysis and seismic design of buildings, ETABS spatial buildings models (ETABS 2013) are constructed. The seismic load was defined using the elastic response spectrum, type 1 (with the magnitude of surface wave amounting to M_S>5.5). The adopted design peak horizontal ground acceleration is 0.37g. The modal periods of case study buildings and mass participation factors of first four modes are shown in the Table 3. By the analyses of the calculated seismic forces, it is noted, that the total seismic force is dominantly assumed by RC core walls (95 % of the total seismic force), while the columns at peripheral frames assume only 5% of the total seismic force. Therefore, the RC core is the subject of further detailed design and nonlinear time-history analyses.

The percentage of longitudinal reinforcement in the boundary elements in external core walls in Y direction varies from 1.879% in ground floor sections, to 0.893% in higher sections, while for internal walls this percentage varies from 0.959% to 0.893%. The percentage of longitudinal reinforcement in the boundary elements of walls in X direction varies from 3.587% to 1.340%. Longitudinal bars are uniformly distributed along the perimeter of boundary elements at no more than 20 cm intervals. The hoops have been adopted at every 10 cm intervals in lower sections (or 15 cm at higher sections). The percentage of vertical web reinforcement ranges from 0.536% at ground floor sections to 0.211% at higher sections, while the percentage of horizontal web reinforcement ranges from 0.785% at ground floor sections to 0.263% at higher sections. The vertical and horizontal web reinforcement is uniformly distributed along the length and height.

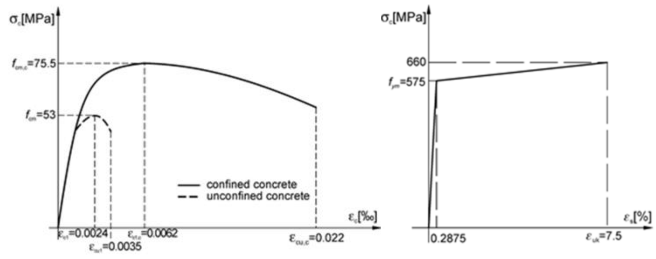
Table 2 Main properties of the RC high-rise case study buildings

Properties	20-storey	30-storey	40-storey
Total height (m)	60	90	120
Storey height (m)	3	3	3
Floor RC slab thickness (cm)	20	20	20
RC beams (cm)	40x65	40x65	40x65
RC columns (cm)	80x80	80x80	90x90
Core walls thickness (cm)	1-5 storey: 30 6-20 storey: 20	1-5 storey: 40 6-30 storey: 30	1-10 storey: 55 11-40 storey: 45
Coupling beams in X direction (cm)	20x80 30x80	30x80 40x80	45x80 55x80
Concrete f_{ck} (f_{cm})* (MPa)	35(43)	45(53)	55(63)
Reinforcement f_{yk} (f_{ym})* (MPa)	500(575)	500(575)	500(575)
Modulus of elasticity E_{cm} (MPa)	34000	36000	38000

* k and m are related to characteristic and mean values of concrete and yield reinforcing steel strength

Table 3 Modal periods and mass participation factors for RC high-rise case study buildings

Case study buildings			20-storey	30-storey	40-storey
Period in Y direction (sec)	Mode	1	1.652	2.880	4.097
		2	0.389	0.623	0.858
		3	0.181	0.270	0.355
		4	0.117	0.164	0.207
Period in X direction (sec)	Mode	1	1.641	2.597	3.511
		2	0.480	0.702	0.880
		3	0.250	0.347	0.423
		4	0.164	0.228	0.275
Mass participation factors in Y direction (%)	Mode	1	64.26	63.53	63.24
		2	20.32	19.43	18.94
		3	7.04	7.05	7.05
		4	3.23	3.57	3.65
Sum of mass part.factors in Y direction (%)			94.85	93.58	92.88
Mass participation factors in X direction (%)	Mode	1	69.36	67.70	66.08
		2	15.96	17.40	18.78
		3	5.49	5.23	5.68
		4	2.83	2.78	2.64
Sum of mass part.factors in X direction (%)			93.64	93.11	93.18



(a) (b)

Fig. 2 Stress-strain diagrams (a) unconfined and confined concrete with concrete mean strength of 53 MPa and (b) reinforcing steel with expected yield mean strength of 575MPa

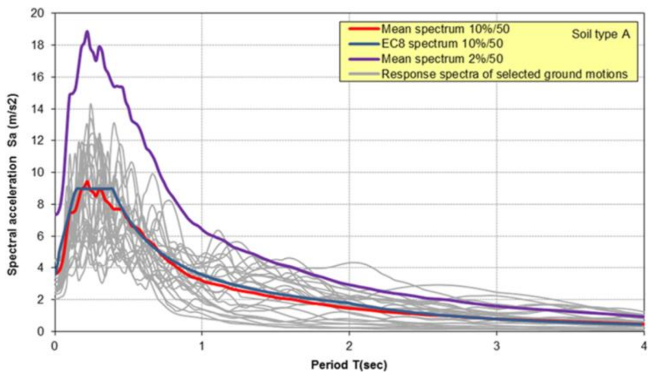


Fig. 3 Response spectra of the selected ground motions for soil type A, mean spectra of the selected ground motions for intensity levels 10%/50 and 2%/50 and elastic EC8 spectrum for soil type A for intensity level 10%/50

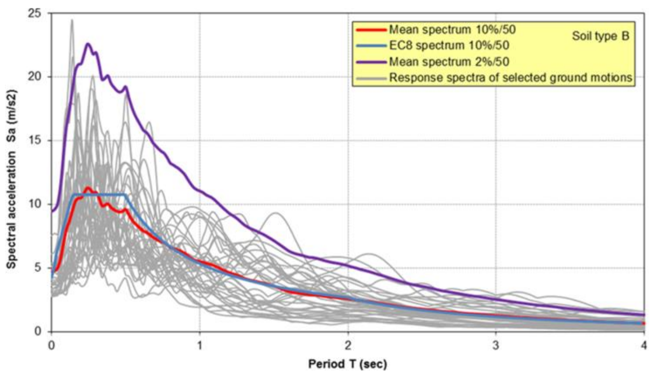


Fig. 4 Response spectra of the selected ground motions for soil type B, mean spectra of the selected ground motions for intensity levels 10%/50 and 2%/50 and elastic EC8 spectrum for soil type B for intensity level 10%/50

5. Nonlinear time-history analysis of RC high-rise case study buildings for PSDA

For the nonlinear time-history analysis, the PERFORM-3D software (PERFORM 2006) is used. The nonlinear spatial models of the RC core wall structural system are made. In order to present the real behaviour of the structure during nonlinear analyses, the properties of structural elements are based on mean values of material properties (EN1998-1 2004). The stress-strain diagram for confined concrete based on the Mander *et al*. (1988) model is adopted. The stress-strain diagrams for unconfined concrete with the mean compressive strength of 53 MPa and for the confined concrete are presented in Fig. 2(a). The adopted bilinear stress-strain diagram for reinforcing steel with expected yield mean strength of 575MPa and ultimate strength of 660MPa is presented on Fig. 2(b).

The core walls are modeled using non-linear vertical fiber elements (Powell 2007). The area and location of reinforcement within the cross-section, as well as concrete properties, are defined using individual fibers forming the cross-section of the wall. The shear behavior is modeled as elastic.

The selection of ground motions is done using data of the Seismological Institute of Montenegro and the European strong-motion database (Ambraseys *et al*. 2002). From the large number of available records 60 ground motions are selected: 25 ground motions are recorded on the rock which correspond to soil type A and 35 ground motions recorded on stiff soil which correspond to soil type B, according to Eurocode 8-1 (EN1998-1 2004). Magnitude values range between 5.1 and 7.0, while distances to source vary from 5 to 70 km. By selecting larger number of ground motions with wider range of magnitudes, distance to source and different site conditions, uncertainties during ground motions selection are being included. High-rise buildings are specific, because their response frequency range is much wider than for low-rise or mid-rise buildings. According to this, it is necessary to include a larger number of ground motions, with various magnitudes and distances to source. Uncertainties during ground motions selection are usually much higher than other types of uncertainties in the probabilistic seismic analysis (Ji *et al*. 2009).

The basic criterion used in this paper for the ground motions selection is that the mean value of selected ground motion response spectra be compatible with the corresponding target spectrum in a wider range of periods. The elastic Eurocode 8 spectrum for the return period of 475 years (10% probability of exceedance in 50 years, 10%/50) with the design ground acceleration of 0.37g is selected as the target spectrum. The mean squared error method (MSE) is chosen as a scaling method of ground motions (PEER 2010). Using MSE method ground motions are scaled that the mean squared error is minimized over the whole range of periods (T=[0;4s]). The considered buildings are also exposed to seismic intensity level with a 2% probability of exceedance in 50 years, 2%/50, (i.e., 2475-year return period (EN1998-3 2005). The more recent literature was consulted in this paper for defining appropriate earthquakes with the 2%/50 intensity. The data for this earthquake level were defined in the scope of the project Seismic hazard harmonization in Europe – SHARE (Giardini *et al*. 2013). This project resulted in preparation of seismic hazard maps for the South-European Mediterranean seismic zone for different levels of seismic intensity. The seismic intensity corresponding to a 2475-year return period is two times greater than the seismic intensity corresponding to a 475-year return period (Giardini *et al*. 2013). Fig. 3 and 4 show: response spectra of selected ground motions scaled by MSE method for the intensity level of 10%/50, the mean spectrum and relevant target spectra (Eurocodes 8 elastic spectra) for the intensity level of 10%/50 and the mean spectrum for the intensity level of 2%/50, for the considered soil types.

Table 4 PSDM parameters for the analyzed IMs

DM	Description	IM	Regression model parameters a	b	Dispersion $\sigma_{DM/IM}$	Proficiency parameter ξ	Coefficient of variation COV
IDR_{max}	Ground motion amplitude parameters	PGA	0.0039	0.528	0.5371	1.0173	0.5783
		PGV	0.0191	**1.1254**	**0.2855**	**0.2537**	0.2914
		PGD	0.0093	0.0459	0.5820	12.6790	0.6349
	Parameters representing concrete points of frequency content	$S_a(T_1)$	0.008	0.4136	0.3992	0.9651	0.4156
		$S_v(T_1)$	0.0106	0.832	0.2801	0.3367	0.2857
		$S_d(T_1)$	0.0202	0.5671	0.3377	0.5955	0.3476
	Parameters comprising a wider range of frequency content	SI_H	0.0103	0.9510	0.2834	0.2980	0.2892
		SI_m	0.011	**0.905**	**0.2468**	**0.2727**	0.2506
		SI_{yh}	0.0114	**0.923**	**0.2449**	**0.2653**	0.2486
		$S_{a,avg}$	0.003	0.8108	0.4017	0.4955	0.4185
		$S_{v,avg}$	0.0111	**1.0468**	**0.2525**	**0.2412**	0.2566
		$S_{d,avg}$	0.0255	0.635	0.3283	0.5171	0.3373
IDR_{sr}	Ground motion amplitude parameters	PGA	0.0028	0.4299	0.5713	1.3289	0.6212
		PGV	0.0115	1.0814	0.3485	0.3223	0.3594
		PGD	0.0058	0.0463	0.5989	12.9352	0.6568
	Parameters representing concrete points of frequency content	$S_a(T_1)$	0.0048	0.4951	0.3171	0.6405	0.3252
		$S_v(T_1)$	0.0066	0.8863	0.2510	0.2832	0.2550
		$S_d(T_1)$	0.0131	0.5987	0.3287	0.5490	0.3378
	Parameters comprising a wider range of frequency content	SI_H	0.0064	1.0776	0.3098	0.2875	0.3174
		SI_m	0.0069	**0.9661**	**0.2042**	**0.2114**	0.2063
		SI_{yh}	0.0072	**0.9869**	**0.1926**	**0.1952**	0.1944
		$S_{a,avg}$	0.0017	0.8737	0.3903	0.4467	0.4056
		$S_{v,avg}$	0.0071	**1.0742**	**0.2413**	**0.2246**	0.2449
		$S_{d,avg}$	0.0163	0.6557	0.3351	0.5111	0.3447

Note: Bold value indicates the most efficient, practical and proficient IMs

6. PSDA results and comparison of different IMs

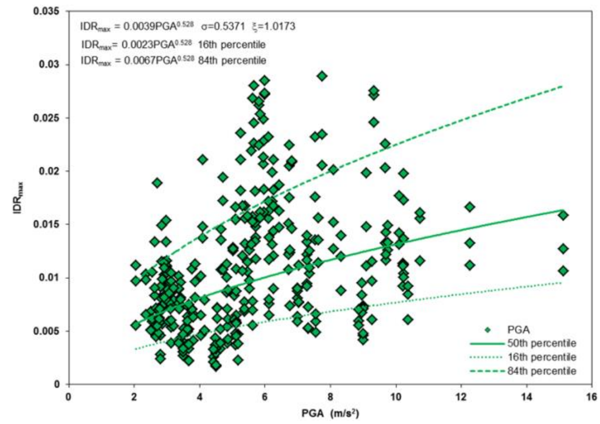
In order to determine the most optimal IMs, from the considered ones, the case study RC high-rise buildings are exposed to 60 ground motions with two intensity levels (10%/50 and 2%/50) in both directions of the buildings. A total of 720 nonlinear time-history analyses are performed. Only the results obtained for ground motion records in Y direction of the buildings are presented in this paper. The results obtained for the ground motion records in X direction are in compliance with the results for the Y direction, and they confirm conclusions made in this paper. Performing nonlinear time-history analyses for the selected ground motions the scatter diagrams are obtained. The regression analysis is performed for each of these diagrams, detailed statistical processing of results is made, and the corresponding PSDMs are derived. The exponential relationship between the DMs and IMs (Eq. (2)) is adopted. For each analyzed PSDM, the median (50th percentile), defined by Eq. (2) is derived, as well as the 16th and 84th percentiles, representing relationships that correspond to a plus-minus standard deviation from the median.

6.1 Efficiency, practicality and proficiency comparison

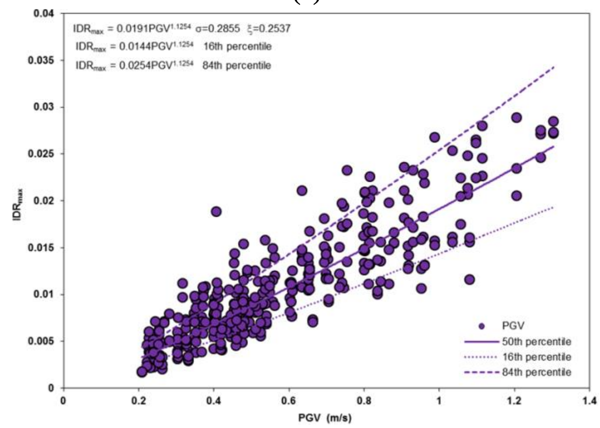
As indicated in Section 3, the efficiency of different IMs can be assessed by comparing the standard deviation $\sigma_{DM/IM}$, the practicality is indicated by the slope, *b*, of the PSDM, and proficiency is measured by the proficiency parameter, ξ. Table 4 lists the derived regression-model parameters *a* and *b*, dispersion $\sigma_{DM/IM}$, the proficiency parameter ξ and variation coefficients *COV* for the considered IMs. From the derived smaller values of the dispersion σ and the coefficients of variation (smaller than 0.3 for most considered IMs) it could be noticed that small variability of results is obtained. This points to a high level of accuracy of derived PSDMs, that is due to the great number of selected ground motions, i.e., to a great size of random sample in statistical term.

Amongst the ground motion amplitude parameters, PGV is the most efficient, practical and proficient. PGV provided significantly less values of dispersion DM/IM and especially, modified dispersion compared to PGA and PGD (Table 4). Also, the regression parameter b has the largest value in the case of PGV. Fig. 5 shows the PSDMs for the IDRmax conditioned upon PGA and PGV. As it is shown, PGV becomes a strong contender for an optimal IM with respect to PGA. The superior practicality of PGV for IDRmax is illustrated by the larger regression parameter, b, of 1.1254 vs 0.5280, and the enhanced efficiency of PGV is indicated by the lower standard deviation of 0.2855 vs 0.5371. The composite measure termed proficiency indicates that PGV is the preferred IM over PGA and PGD, as shown in Table 4. Some studies (Lu *et al*. 2012, Guan *et al*. 2015) have come to similar conclusions regarding to the PGV and PGA as appropriate and inappropriate IM for high-rise buildings.

Comparing the results for analyzed spectral measures $S_a(T_1)$, $S_v(T_1)$ and $S_d(T_1)$, the most efficient, practical and proficient is spectral velocity $S_v(T_1)$ (Table 4). In comparison with amplitude parameters PGA, PGV and PGD, spectral values $S_a(T_1)$, $S_v(T_1)$ and $S_d(T_1)$ have proven to be more efficient, but not in every case more practical, that affects their proficiency. The derived results show that PGV proved to be more proficient then $S_v(T_1)$ for IDR_{max}, with proficient parameter of 0.2537 vs 0.3367, while for IDR_{sr} is vice versa.

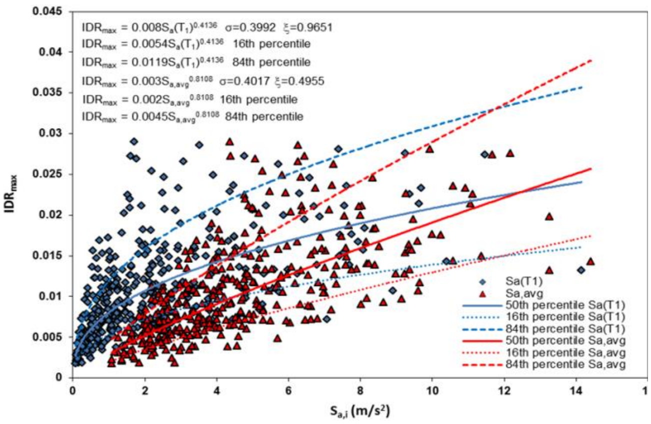


(a)

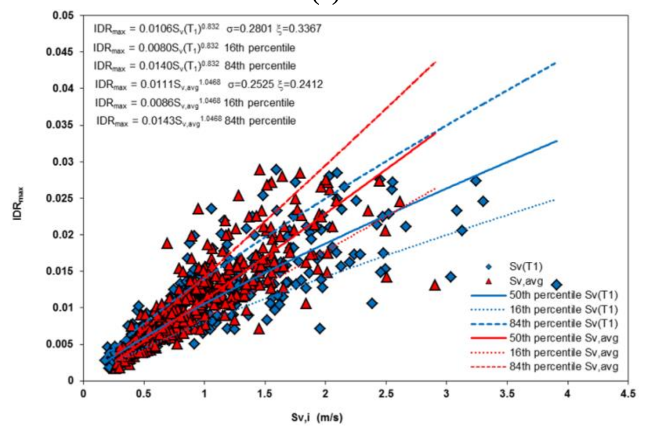


(b)

Fig. 5 PSDMs for IDR_{max} conditioned upon (a) PGA and (b) PGV



(a)



(b)

Fig. 6 PSDMs for IDR_{max} conditioned upon (a) S_a(T1) and $S_{a,avg}$ and (b) S_v(T1) and $S_{v,avg}$

Mean spectral velocity $S_{v,avg}$, that takes into account the higher-mode effects, has proven to be more efficient, practical and sufficient then S_v(T1), and hence demonstrate that is better IM for the RC high-rise buildings (Fig. 6(b)). For the case of $S_{a,avg}$ and $S_{d,avg}$, it is noticed that are not more efficient then S_a(T1) and S_d(T1) respectively, but are more proficient that result from higher value of parameter *b* (i.e. higher practicality) (Fig. 6a). In this case is evident that the proficiency, taking into account composite effect of efficiency and practicality could overcome the difficulties in selection process of optimal IM (Padgett *et al*. 2008). Comparing mean spectral measures among each other $S_{a,avg}$, $S_{v,avg}$ and $S_{d,avg}$, it is evident that the most optimal is $S_{v,avg}$.

Fig. 7 shows the PSDMs for IDR_{sr} conditioned upon the SI_H, SI_m and SI_{yh}. The considered IMs comprising a wider range of frequency content (SI_m and SI_{yh}) are the most efficient IMs, that is indicated by the the lowest standard deviation $\sigma_{DM/IM}$ of 0.2042 and 0.1926, respectively for the IDR_{sr} and 0.2468 and 0.2449 for IDR_{max}.

This is due to the fact that the range of frequency response of high-rise buildings is much wider compared to lower buildings, and hence the IMs comprising a wider range of response spectra are more appropriate. In the case of SI_m and SI_{yh}, the values of derived PSDM parameters are practically the same, because the modal period T_h is approximately equal to $2T_y$ for the case of the considered case study buildings.

It is also observed that intensity measures SI_m and SI_{yh} provided a smaller dispersion of results $\sigma_{DM/IM}$ compared to SI_H although all three cover a wide range of response spectra, but with the only difference that SI_m and SI_{yh} comprise response spectra values in the range of greater periods, compared to SIH that refers on fixed lower range [0.1s;2.5s]. It is for the reason that the first modal periods of 30-storey and 40-storey case study high-rise buildings are greater then 2.5s. Similar results are obtained by Martinez-Rueda (1998), that suggest use of SIH for the medium-period structures with first modal period range from 0.6s to 1.6s, and SI_m for long-period structures with

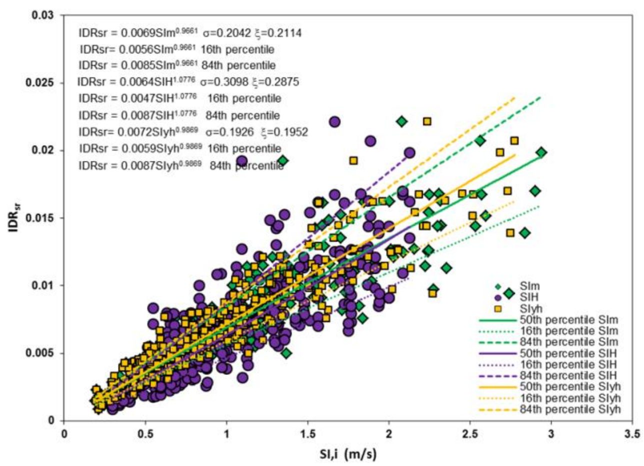


Fig. 7 PSDMs for IDR_{sr} conditioned upon SI_H, SI_m and SI_{yh}

periods more then 1.6s.

When it comes to proficiency, which takes into account composite effects of the efficiency and practicality, the most proficient measures for IDR_{sr} are SI_m and SI_{yh}, following with $S_{v,avg}$, while for IDR_{max} are PGV, Sv,avg , SI_m and SI_{yh}.

In general, all IMs related to velocity are proved to be more efficient, practical and proficient, compared to those related to acceleration and displacement, because the considered case study buildings have fundamental modal periods in the tripartite spectrum area that is sensitive to velocity (Chopra 1995). IMs as PGA, PGD, S_a(T1) and S_d(T1), the most used IMs in literature for obtaining the fragility curves (etc. HAZUS 2003, Mosalem *et al*. 1997, Rossetto and Elnashai 2003), are not proved to be appropriate for the RC high-rise buildings with considered range of first modal periods.

Furthermore, contrary to former studies that have selected as the most appropriate IMs for high-rise buildings, improved IMs based on spectral acceleration (Lu *et al*. 2013, Zhang *et al*. 2017) and spectral displacement quantities (Tothong and Luco 2007), the results of this study reveal that on the basis of efficiency, practicality and proficiency IMs based on velocity spectrum rather then acceleration and displacement ones, are the most optimal. Further research on IMs optimality for RC high-rise buildings should consider different spectral velocity quantities.

6.2 Sufficiency comparison

The considered IMs are studied to check their independence from *M*, and *R*.

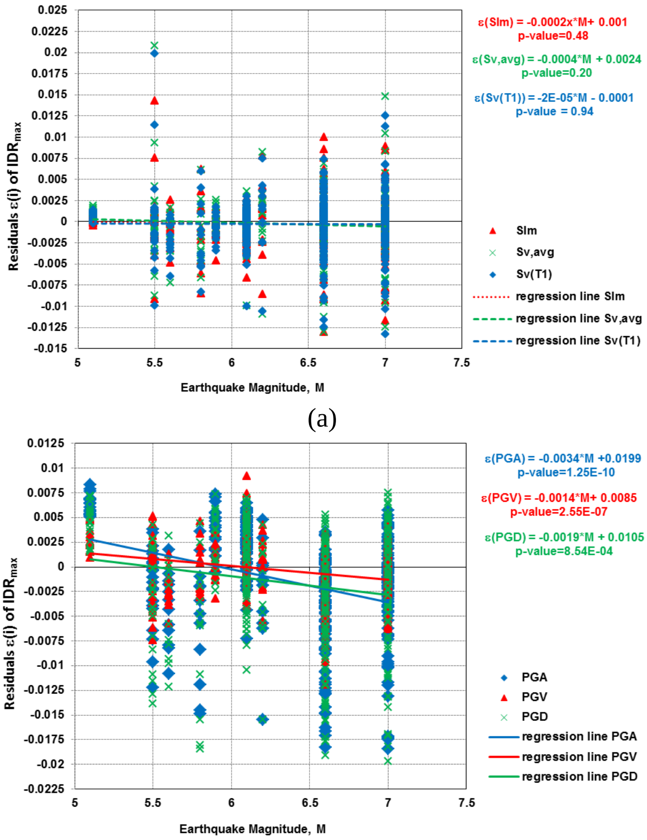
Results evaluated by performing the regression analysis on the normalized residuals, *ε*, from the PSDM, to the ground motion characteristic, *M* and *R* and *p*-values are used to assess the sufficiency, where smaller *p*-values indicate an insufficient IM. The limit value for an insufficient IM is assumed to be a *p*-value of 0.05. Table 5 presents the *p*-values for the considered IMs and demand measure IDR_{max}.

The derived results show that all of considered IMs are independent of R with p-values in range from 0.08 to 0.71. It is

Table 5 Sufficiency comparison of IMs using p-values for IDRmax

Description	IM	p-value Magnitude (M)	p-value Distance (R)
Ground motion amplitude parameters	PGA	**1.25E-10**	0.71
	PGV	**2.55E-07**	0.08
	PGD	**8.54E-04**	0.52
Parameters representing concrete points of frequency content	$S_a(T_1)$	0.15	0.31
	$S_v(T_1)$	0.94	0.44
	$S_d(T_1)$	0.67	0.13
Parameters comprising a wider range of frequency content	SI_H	0.13	0.31
	SI_m	0.48	0.47
	SI_{yh}	0.38	0.59
	$S_{a,avg}$	**2.92E-06**	0.15
	$S_{v,avg}$	0.20	0.26
	$S_{d,avg}$	0.55	0.11

Note: Bold value indicates insufficient IMs



(b)

Fig. 8 Comparison of the IMs sufficiency from *M* for IDR_{max}

obtained that IMs are less sufficient in the case of independence of M. The ground motion amplitude values PGA, PGV, PGD, and mean spectral acceleration $S_{a,avg}$ are insufficient with respect to M, while other considered IMs have proved to be sufficient. Fig. 8 shows the comparison of the regression for the IDRmax to compare the sufficiency of SI_m, $S_{v,avg}$ and $S_v(T_1)$ (Fig. 8(a)), to insufficiency of PGA, PGV and PGD (Fig. 8(b)) relative to M. The slope of regression line c for PGA, PGV and PGD differed from 0 significantly comparing with the slope of SI_m, $S_{v,avg}$ and $S_v(T_1)$. The p-values are 0.48, 0.20 and 0.94 for SI_m, $S_{v,avg}$ and $S_v(T_1)$ respectively, unlike the small values obtained for PGA, PGV and PGD, indicating that SI_m, $S_{v,avg}$ and $S_v(T_1)$ are much more sufficient IM for conditioning the PSDM.

7. Conclusions

The objective of this study is to identify the optimal IMs for establishing PSDMs for RC high-rise buildings. The features of optimal IMs such as: efficiency, practically, proficiency and sufficiency are analyzed in detail for the considered IMs. 20-storey, 30-storey and 40-storey RC high-rise buildings with core wall structural system are selected as the case study buildings. After conducting detailed analysis of results, the appropriate conclusions are made regarding the optimality of the considered IMs for the RC high-rise buildings.

The results of this study indicate that on the basis of efficiency, practically, proficiency and sufficiency, the IMs comprising a wider range of frequency content SI_m, SI_{yh} and $S_{v,avg}$, are the most optimal IMs for the RC high-rise buildings. Generally, the IMs based on velocity are proven to be more optimal compared to those related to acceleration and displacement because the considered case study buildings have fundamental modal periods in the tripartite spectrum area that is sensitive to velocity.

Contrary to former studies that have considered mostly improved spectral acceleration and displacement quantities (in this study $S_{a,avg}$ and $S_{d,avg}$ are analyzed), the results of this study reveal that IMs based on velocity spectrum rather than acceleration and displacement ones, are the most optimal for the RC high-rise buildings.

Considering that enable a clear physical interpretation, that can easily be calculated from seismic records and with respect to its efficiency, practically and proficiency, PGV, following with S_v(T1), are proved to be quite satisfactory IMs for the RC high-rise buildings. The only disadvantage of PGV is insufficiency with respect to M. IMs such as PGA, PGD, S_a(T1) and S_d(T1), the most used IMs in literature, especially in obtaining the fragility curves, are not proved to be appropriate for the RC high-rise buildings.

Further research for RC high-rise buildings should consider analysis of IMs based on velocity spectrum.

References

Ambraseys, N., Smit, P., Sigbjornsson, R., Suhadolc, P. and Margaris, B. (2002), *Internet-site for European strong-motion data, European Commission*, Directorate-General XII, Environmental and Climate Programme; Brussels, Belgium. http://www.isesd.cv.ic.ac.uk.

Baker, J.W. and Cornell C.A. (2005), "A vectored-valued ground motion intensity measure consisting of spectral acceleration and epsilon", *Earthq. Eng. Struct. Dyn.*, **34**(10), 1193-1217.

Bavaghar, Y. and Bayat, M. (2017), "Seismic fragility curves for highly skewed highway bridges", *J. Vibroeng.*, **19**(4), 2749-2758.

Bayat M. and Daneshjoo F. (2015), "Seismic performance of skewed highway bridges using analytical fragility function methodology", *Comput. Concr.*, **16**(5), 723-740.

Bayat, M., Daneshjoo, F. and Nisticò N. (2015a), "Probabilistic sensitivity analysis of multi-span highway bridges", *Steel. Compos. Struct.*, **19**(1), 237-262.

Bayat, M., Daneshjoo, F. and Nisticò N. (2015b), "A novel proficient and sufficient intensity measure for probabilistic analysis of skewed highway bridges", *Struct. Eng. Mech.*, **55**(6), 1177-1202.

Bayat, M., Daneshjoo, F. and Nisticò N. (2017), "The effect of different intensity measures and earthquake directions on the seismic assessment of skewed highway bridges", *Earthq. Eng. Eng. Vib.*, **16**(1), 165-179.

Chopra, A.K. (1995), *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, First edition, Prentice Hall, Inc., New Jersey, USA.

Cordova, P.P., Deierlein, G.G., Mehanny, S.S., and Cornell, C.A. (2001), "Development of a two-parameter seismic intensity measure and probabilistic assessment procedure", *The Second U.S.-Japan Workshop on Performance based Earthquake Engineering Methodology for Reinforced Concrete Building Structures*, Sapporo, Japan.

Cornell, A.C., Jalayer, F., Hamburger, R.O. and Foutch, D.A. (2002), "Probabilistic basis for 2000 SAC federal emergency management agency steel moment frame guidelines", *J. Struct. Eng.*, **128**(4), 526-532.

CTBUH (2011), *The Tallest 20 in 2020: Entering the Era of the Megatall; The Council on Tall Buildings and Urban Habitat Illinois Institute of Technology*, Chicago, USA.

EN1992-1-1 (2004), *Design of Concrete Structures. Part 1: General Rules and Rules for Buildings*, European Committee for Standardization, Brussels, Belgium.

EN1998-1 (2004), *Design of Structures for Earthquake Resistance. Part 1: General Rules, Seismic Actions and Rules for Buildings*, European Committee for Standardization, Brussels, Belgium.

EN1998-3 (2005), *Design of Structures for Earthquake Resistance. Part 3: Assessment and Retrofitting of Buildings*, European Committee for Standardization, Brussels, Belgium.

ETABS (2013), *ETABS 2013 Integrated Analysis, Design and Drafting of Buildings Systems*, CSI Computers & Structures Inc., Berkeley, USA.

Giardini, D., Woessner, J., Danciu, L., Crowley, H., Cotton, F., Grünthal, G., Pinho, R., Valensise, G. and the SHARE consortium (2013), *SHARE European Seismic Hazard Map*, European Commission. http://www.share-eu.org.

Giovenale, P., Cornell, A.C. and Esteva, L. (2004), "Comparing the adequacy of alternative ground motion intensity measures for the estimation of structural responses", *Earthq. Eng. Struct. Dyn.*, **33**(8), 951-979.

Guan, M., Du, H., Cui, J., Zeng, Q. and Jiang H. (2015), "Optimal ground motion intensity measure for long-period structures", *Meas. Sci. Technol.*, **26**(10), 1-12.

HAZUS MR4 (2003), *Technical manual, Multi-hazard Loss Estimation Methodology - Earthquake Model, National Institute of Building Sciences*, Federal Emergency Management Agency, Ishington DC.

Housner, G.W. (1952), "Spectrum intensities of strong motion earthquakes", *Proceedings of Symposium on Earthquake and Blast Effects on Structures*, EERI.

http://www.ctbuh.org/TallBuildings/HeightStatistics/BuildingsinNumbers/TheTallest20in2020/tabid/2926/language/en-US/Default.aspx

Ji, J., Elnashai, A.S and Kuchma, D.A. (2009), "Seismic fragility relationships for reinforced concrete high-rise buildings", *Struct. Des. Tall Spec. Build.*, **18**(3), 259-277.

Ji, J., Elnashai, A.S. and Kuchma, D.A. (2007), "Seismic fragility assessment for reinforced concrete high-rise buildings", *Research Report No. 07-14; Mid-America Earthquake Center*, University of Illinois at Urbana-Champaign.

Kostinakis, K.G. and Athanatopoulou A.M. (2015), "Evaluation of scalar structure-specific ground motion intensity measures for seismic response prediction of earthquake resistant 3D buildings", *Earthq. Struct.*, **9**(5), 1091-1114.

Lu, X., Lu, X.Z. and Ye, L.P. (2012), "Discussion on the ground motion intensity measures for super high-rise buildings", *China Civil. Eng. J.*, **45**(1), 292-296.

Lu, X., Ye, L.P., Lu, X.Z., MengKe, L.I. and XiaoWei M.A. (2013), "An improved ground motion intensity measure for super high-rise buildings", *Sci. China Tech. Sci.*, **56**(6), 1525-1533.

Luco, N. and Cornell, C.A. (2007), "Structure-specific scalar intensity measures for near-source and ordinary earthquake motions", *Earthq. Spectra*, **23**(2), 357-391.

Mackie, K. and Stojadinovic, B. (2001), "Probabilistic seismic demand model for California bridges", *J. Bridge Eng.*, **6**(6), 468-480.

Martinez-Rueda, J.E. (1998), "Scaling procedure for natural accelerograms based on a system of spectrum intensity scales", *Earthq. Spectra*, **14**(1), 135-152.

Matsumura, K. (1992), "On the intensity measure of string motions related to structural failures", *Proceedings of the 10WCEE*, Madrid, July.

Mosalem, K.M., Ayala, G., White, R.N. and Roth, C. (1997), "Seismic fragility of LRC frames with and without masonry infill walls", *J.*

Earthq. Eng., **1**(4), 693-719.

Nagashree, B.K., Kumar R.C.M. and Reddy V.D. (2016), “A parametric study on seismic fragility analysis of RC buildings”, *Earthq. Spectra*, **10**(3), 629-643.

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Padgett, J.E., Nielson, B.G. and DesRoches, R. (2008), “Selection of optimal intensity measures in probabilistic seismic demand models of highway bridge portfolios”, *Earthq. Eng. Struct. Dyn.*, **37**(5), 711-726.

PEER (2010), *Technical Report for the PEER Ground Motion Database Web Application*, Pacific Earthquake Engineering Research Center, University of California, Berkeley, USA.

PEER TBI (2014), *Tall Buildings Initiative; Pacific Earthquake Engineering Research Center*, University of California, Berkeley. http://peer.berkeley.edu/tbi/

Pejović, J. and Janković, S. (2015), “Dependence of RC high-rise buildings response on the earthquake intensity”, *J. Croatian Assoc. Civil Eng.*, **67**(8), 749-759.

Pejović, J.R. (2016), “Seismic analysis of reinforced concrete high-rise building”, Ph.D. Dissertation, Faculty of Civil Engineering Podgorica, University of Montenegro, Podgorica.

PERFORM (2006), *PERFORM 3D Nonlinear Analysis and Performance Assessment for 3D Structures*, CSI Computers & Structures Inc., Berkeley, USA.

Powell, G.H. (2007), *PERFORM 3D Detailed example of a tall shear wall building - Nonlinear Modeling, Analysis and Performance Assessment for Earthquake Loads*, Computers & Structures Inc, Berkeley, USA.

Rossetto, T. and Elnashai, A. (2003), “Derivation of vulnerability functions for European-type RC structures based on observational data”, *Eng. Struct.*, **25**(10), 1241-1263.

Shome, N. (1999), “Probabilistic Seismic Demand Analysis of Nonlinear structures”, Ph.D. Dissertation, Stanford University, Stanford.

Shome, N., Cornell, C.A., Bazzurro, P. and Carballo, J.E. (1998) “Earthquakes, records and nonlinear responses”, *Earthq. Spectra*, **14**(3), 469-500.

Singhal, A. and Kiremidjian, A.S., (1997), “A Method for Earthquake Motion-Damage Relationships with Application to Reinforced Concrete Frames”, *NCEER-97-0008; National Center for Earthquake Engineering Research*, State Univ. of New York at Buffalo.

Stewart, J.P., Chiou, S.J., Bray, J.D., Graves, R.W., Somerville, P.G. and Abrahamson, N.A. (2002) “Ground motion evaluation procedures for performance-based design”, *Soil Dyn. Earthq. Eng.*, **22**(9), 765-772.

Su, N., Lu, X., Zhou, Y. and Yang, T.Y. (2017) “Estimating the peak structural response of high- rise structures using spectral value-based intensity measures”, *Struct. Des. Tall Spec. Build.*, **26**(8), 1-8.

Taranath, B.S. (2010), Reinforced Concrete Design of Tall Buildings, International Code Council, Concrete Reinforcing Steel Institute, CRC Press Taylor & Francis Group, Boca Raton, USA.

Tothong, P. and Luco, N. (2007), “Probabilistic seismic demand analysis using advanced intensity measures”, *Earthq. Eng. Struct. Dyn.*, **36**(13),1837-1860.

Vamvatsikos, D. and Cornell, C.A. (2002), “Incremental dynamic analysis”, *Earthq. Eng. Struct. Dyn.*, **31**(3), 491-514.

Vamvatsikos, D. and Cornell, C.A. (2005), “Developing efficient scalar and vector intensity measures for IDA capacity estimation by incorporating elastic spectral shape information”, *Earthq. Eng. Struct. Dyn.*, **34**(13), 1573-1600.

Ye, L.P., Ma, Q.L., Miao, Z., Guan, H. and Zhuge, Y. (2011), “Numerical and comparative study of earthquake intensity indices in seismic analysis”, *Struct. Des. Tall Spec. Build.*, **22**(4), 362-381.

Zhang, Y., He, Z., Lu, W. and Yang, Y. (2017), “A spectral-acceleration-based linear combination-type earthquake intensity measure for high-rise buildings”, *J. Earthq. Eng.*, DOI: 10.1080/13632469.2017.1286624.