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Novel optimal intensity measures for probabilistic seismic analysis of RC high-rise buildings with core

Jelena R. Pejovic*, Nina N. Serdar[a] and Radenko R. Pejovic[b]

The Faculty of Civil Engineering, University of Montenegro, Podgorica, Montenegro

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Abstract. In this paper the new intensity measures (IMs) for probabilistic seismic analysis of RC high-rise buildings with core wall structural system are proposed. The existing IMs are analysed and the new optimal ones are presented. The newly proposed IMs are based on the existing ones which: 1) comprise a wider range of frequency velocity spectrum content and 2) are defined as the integral along the velocity spectrum. In analysis characteristics of optimal IMs such as: efficiency, practicality, proficiency and sufficiency are considered. As prototype buildings, RC high-rise buildings with core wall structural system and with characteristic heights: 20-storey, 30-storey and 40-storey, are selected. The non-linear 3D models of the prototype buildings are constructed. 720 non-linear time-history analyses are conducted for 60 ground motion records with a wide range of magnitudes, distances to source and various soil types. Statistical processing of results and detailed regression analysis are performed and appropriate demand models which relate IMs to demand measures (DMs), are obtained. The conducted analysis has shown that the newly proposed IMs can efficiently predict the DMs with minimum dispersion and satisfactory practicality as compared to the other commonly used IMs (e.g., PGA and $S_a(T_1)$). The newly proposed IMs overcome difficulties in calculating of integral along the velocity spectrum and present adequate replacement for IMs which comprise a wider range of frequency velocity spectrum content.

Keywords: RC high-rise building; intensity measure; demand measure; optimality; non-linear time-history analysis; velocity spectrum; regression analysis

1. Introduction

The construction of high-rise buildings in seismic active zones has become an everyday design trend, mainly due to rapid growth of cities and concentration of material resources in urban environments. The Council on Tall Buildings (CTBUH 2011) asserted that an average height of the 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. The Pacific Earthquake Engineering Research Center (PEER) which conducted a large-scale research project called Tall Buildings Initiative (PEER TBI 2014), has been among the first to recognise the lack of research on this topic with regard to high-rise buildings.

The current provisions and the regulations for the seismic design (etc. EN1998-1 2004) most often are not sufficient for seismic analysis of RC high-rise buildings. Limitations of traditional design approaches have been recognized and probabilistic performance-based seismic design methodology has become 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 probabilistic seismic demand analysis (PSDA). In the process of PSDA, it is necessary to establish an appropriate probabilistic seismic demand model (PSDM): the relationship between IM and 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.

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

where Φ is the standard for normal cumulative distribution function, μ and σ are log-normal distribution parameters (mean value and standard deviation).

The selection of optimal IM significantly affects the establishment of appropriate PSDM through reducing uncertainties associated with PSDM (Stewart *et al*. 2002). Many 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).

Several IMs are mostly used by the researchers in application of probabilistic performance-based seismic methodology: PGA (Mosalem *et al*. 1997, HAZUS MR4 2003), spectral acceleration at some periods $S_a(T_i)$ (Singhal and Kiremidjian 1997, Bayat *et al*. 2015a, Bayat *et al*. 2017, Bavaghar and Bayat 2017, HAZUS MR4 2003) and spectral displacement at selected periods $S_d(T_i)$ (Rossetto and Elnashai 2003, Nagashree *et al*. 2016, HAZUS MR4 2003).

*Corresponding author, Assistant Lecturer
E-mail: jelenar@t-com.me
[a]Assistant Lecturer
[b]Professor

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Today, in literature, there is a lack of information regarding the most optimal IMs for use in establishing PSDMs for RC high-rise buildings. In general, the most used IMs in literature (e.g., PGA, $S_a(T_1)$) have not proved to be quite appropriate for high-rise buildings because they do not account for contribution of higher modes (Vamvatsikos and Cornell 2005) and spectral shape (Baker and Cornell 2005). The most studies today (Lu *et al*. 2013, Zhang *et al*. 2017, Su *et al*. 2017, Lu *et al*. 2012) are focused on improving existing IMs based on spectral and displacement quantities, 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 that accounts for higher modes. Zhang *et al*. (2017) developed a spectral-acceleration-based linear combination-type earthquake IM for high-rise buildings. Su *et al*. (2017) proposed a novel spectral acceleration value-based IM for high-rise buildings that accounts for the higher mode effects using the modal participation of masses.

Contrary to former studies that have considered improved spectral acceleration and displacement quantities, the results of the study (Pejovic *et al*. 2017) revealed that IMs based on velocity spectrum rather than acceleration and displacement ones, are more optimal for the RC high-rise buildings and that future research for high-rise buildings should consider analysis of IMs, based on velocity spectrum. Also, mentioned study showed that on the basis of efficiency, practicality, proficiency and sufficiency, the IMs which comprise a wider range of frequency velocity spectrum content are the most optimal IMs for the high-rise buildings. In general, IMs based on wider range of frequency content (e.g., Matsumura mean spectrum intensity SI_m and Martinez-Rueda mean spectrum intensity SI_{yh}) are defined as integral of velocity spectrum that is not very practical for calculation.

According to conclusions obtained in previous study (Pejovic *et al*. 2017), the idea in this paper was to analyse overcoming the difficulties in calculating the integral of velocity spectrum by proposing the new IMs that can be easily and efficiently calculated from the velocity spectrum. For this purpose, 720 non-linear time-history analyses are conducted for 60 ground motion records with a wide range of magnitudes, distances to source and various soil types, taking into account uncertainties during ground motion selection. RC high-rise buildings with core wall structural system are selected as prototype building class with the three characteristic heights: 20-storey, 30-storey and 40-storey. 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. On the basis of analysis: efficiency, practicality, proficiency and sufficiency of considered IMs, appropriate conclusions regarding the newly proposed IMs are made.

2. The considered IMs

2.1 Selection of the existing IMs

Table 1 Considered existing IMs

Intensity measures IMs	Abbreviations	Units
Peak ground acceleration	PGA	m/s^2
Peak ground velocity	PGV	m/s
Spectral acceleration	$S_a(T_1)$	m/s^2
Spectral velocity	$S_v(T_1)$	m/s
Matsumura mean spectrum intensity	SI_m	m/s
Martinez-Rueda mean spectrum intensity	SI_{yh}	m/s
Mean spectral velocity	$S_{v,avg}$	m/s

The existing IMs, analysed in this paper, are listed in Table 1. The obtained results in previous study (Pejovic *et al*. 2017) revealed that IMs based on velocity spectrum rather than acceleration and displacement ones, are more optimal for the RC high-rise buildings. Also, conducted study (Pejovic *et al*. 2017) showed that IMs which comprise a wider range of frequency response spectra content are more appropriate for the high-rise buildings because of their much wider response frequency range. For that reason, in this study, the existing IMs based on velocity (PGV and $S_v(T_1)$) are selected for analysis wherein the emphasis is placed on IMs which comprise a wider range of frequency response (SI_m, SI_{yh}, $S_{v,avg}$). In addition, the most used IMs in seismic analysis of structures PGA and $S_a(T_1)$) are selected for comparative analysis.

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)

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

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 is replaced with the period T_h which represents the new vibration period of the structure in the hardening range after yielding (Martinez-Rueda 1998)

$$SI_{yh} = \frac{1}{T_h - T_y}\int_{T_y}^{T_h} S_v\,(T)dT \tag{3}$$

For the yield period of the structure T_y, MPF (mass participation factor) weighted average value at the first three modes is adopted (Eq. (4)) (Ji *et al*. 2007)

$$T_y = \frac{m_1\cdot T_1 + m_2\cdot T_2 + \cdots + m_n\cdot T_n}{m_1 + m_2 + \cdots m_n} \tag{4}$$

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

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

$$T_h = T_y\cdot\sqrt{\frac{\mu}{1+\alpha\cdot\mu-\alpha}} \tag{5}$$

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

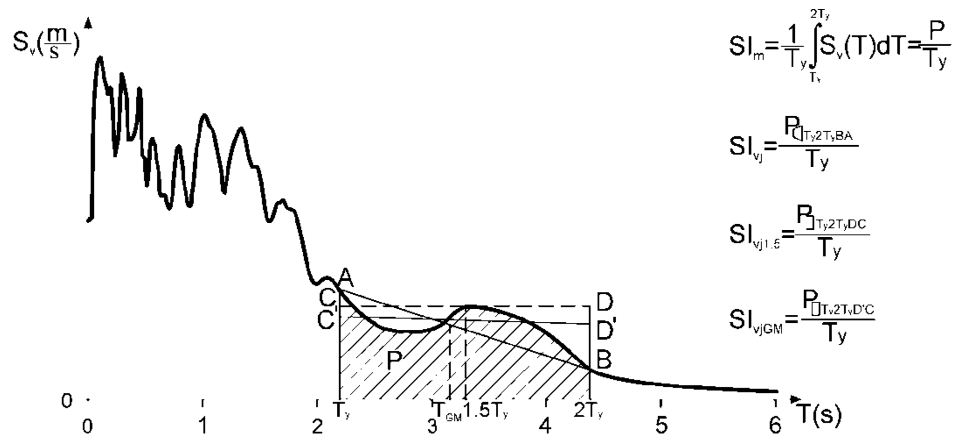


Fig. 1 Schematic presentation of method for obtaining newly proposed IMs

post-yield stiffness ratio.

The mean spectral velocity $S_{v,avg}$ takes into account the higher-mode effects and is derived as the combination of spectral values at first three modes (Pejovic 2016, Pejovic and Jankovic 2015)

$$S_{v,avg} = \frac{m_1 \cdot S_v(T_1) + m_2 \cdot S_v(T_2) + m_3 \cdot S_v(T_3)}{m_1 + m_2 + m_3} \tag{6}$$

2.2 Novel IMs for the RC high-rise buildings

Based on the existing IMs (Matsumura mean spectrum intensity SI_m and Martinez-Rueda mean spectrum intensity SI_{yh}) the new IMs are proposed. The Matsumura mean spectrum intensity SI_m and the Martinez-Rueda mean spectrum intensity SI_{yh}, are defined as the integral, along the velocity spectrum, which is not very practical for calculation. Fig. 1 shows that the mean spectrum intensity SI_m represents the area below the velocity spectrum diagram from point T_y to point $2T_y$ divided with T_y. This area can be adequately and approximately replaced: 1) with the area of the trapezium defined with points T_y-$2T_y$-B-A; 2) with rectangle which area is defined with spectrum value in point $1.5T_y$ (rectangle T_y-$2T_y$-D-C); or 3) with rectangle which area is defined with spectrum value in point T_{GM}, obtained as a geometric mean of the velocity spectrum from the period T_y to the period $2T_y$ (rectangle T_y-$2T_y$-D'-C').

Consequently, newly proposed IMs are:

• Mean velocity spectrum intensity SI_{vj}

$$SI_{vj} = \frac{S_v\left(T_y\right) + S_v\left(2T_y\right)}{2} \tag{7}$$

• Mean velocity of spectrum intensity $SI_{vj1.5}$, representing the velocity spectrum value for the modal period of $1.5T_y$

• Mean velocity of spectrum intensity SI_{vjGM} representing the geometric mean of the velocity spectrum values from the modal period T_y to the modal period $2T_y$.

The newly proposed IMs can be easily and efficiently calculated from the velocity spectrum. The optimality of these IMs was tested and in this paper, the comparison with the optimality of existing IMs was done by the conducted analysis.

2.3 Optimality of IM

An optimal seismic IM requires possession of 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, practicality, proficiency and sufficiency are analysed.

Efficiency of IM is measured by the degree of scatter, i.e., by the dispersion of the obtained DMs with respect to the regression of fit line for the given value IM (Eq. (8))

$$\widehat{DM} = a \cdot IM^b \tag{8}$$

where a and b are regression coefficients.

Less dispersion of the results means more efficient IM and is represented in this study by lower $\sigma_{DM/IM}$ (Eq. (9)).

$$\sigma^2_{DM/IM} = \frac{\sum_{i=1}^{n}[lnDM_i - ln\widehat{DM_i}]^2}{N-d_f} \tag{9}$$

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

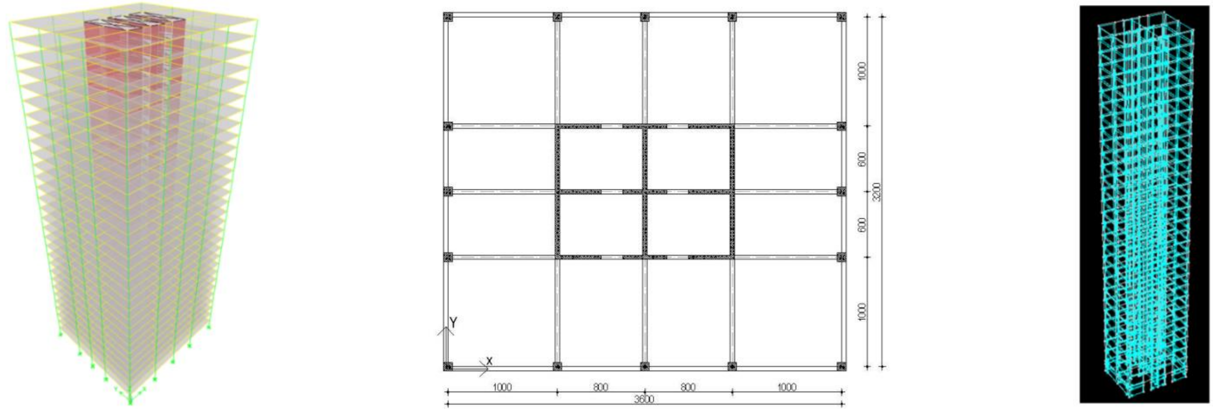
Practicality refers to whether or not any direct correlation exist between an IM and DM (Padgett *et al*. 2008). In the case of the non- practical IM there is a little or no dependence of the demand level DM to the level of the IM. Practicality is measured by the regression of model parameter b (Eq. (8)). 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 of model parameter b (the higher slope of regression line) is more practical.

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

$$\xi = \frac{\sigma_{DM/IM}}{b} \tag{10}$$

The composite measure of practicality and efficiency, as noted by Padgett (2008), could overcome the difficulties in balancing selection between these two features.

IM is sufficient if DM, for the given IM, is independent



(a) (b) (c)

Fig. 2 30-storey prototype building ETABS2013 model (left), plan view of the storey (middle) and 30-storey prototype building PERFORM3D model (right)

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 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 DM_i and its estimate (Eq. (8)). 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 IM is insufficient.

2.4 Selection of the demand measure DM

In this paper, the interstorey drift (relative storey drift divided with the storey height) is selected as a seismic demand measure (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 non-linear 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_{mv}.

3. RC high-rise prototype buildings

RC high-rise buildings with core wall structural system are selected as a prototype building class. The three characteristic heights are considered: 20-storey, 30-storey and 40-storey. The plan view of the storey for all prototype buildings, ETABS model (ETABS 2013) and PERFORM-3D model (PERFORM 2006) of the 30-storey prototype RC high-rise building are shown on Fig. 2. The central core wall system assumes the entire seismic force, and RC frames along the perimeter assume the gravity load only

Table 2 Main properties of the RC high-rise prototype 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)	40×65	40×65	40×65
RC columns (cm)	80×80	80×80	90×90
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)	20×80 30×80	30×80 40×80	45×80 55×80
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

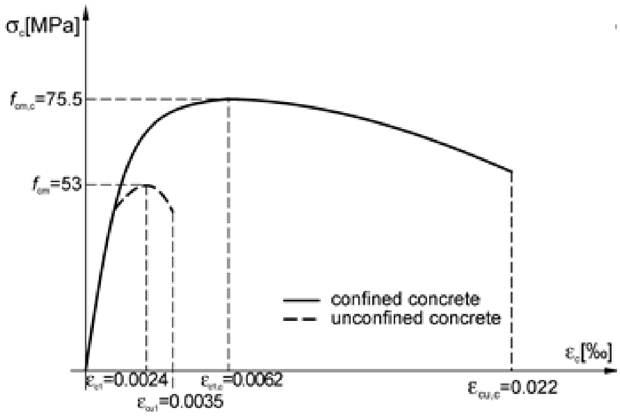
**k* and m are related to characteristic and mean values of concrete and yield reinforcing of steel strength

(Taranath 2010). The main properties of the considered prototype buildings are shown in Table 2.

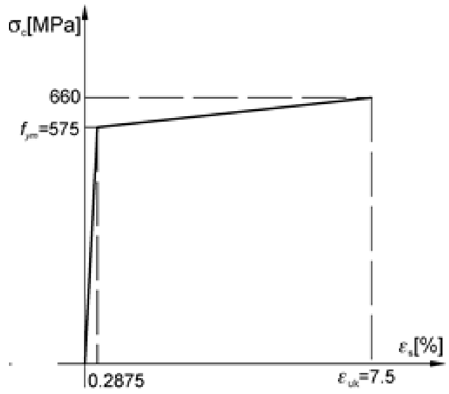
Seismic analysis and design of the prototype RC high-rise buildings are done according to Eurocode 2 (EN1992-1-1 2004) and Eurocode 8-1 (EN1998-1 2004). Seismic linear analysis of buildings is done using a multi-modal response spectrum analysis, considering higher mode effects. For linear analysis, ETABS spatial buildings models (ETABS 2013) are constructed. The seismic load is 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 0.37 g. The modal periods of prototype buildings and mass participation factors of first three 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 non-linear time-history analysis.

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

Prototype 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
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
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
Sum of mass part.factors in Y direction (%)			91.62	90.01	89.23
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
Sum of mass part.factors in X direction (%)			90.81	90.33	90.54



(a)



(b)

Fig. 3 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 575 MPa

4. Non-linear models of RC high-rise prototype buildings

For the non-linear time-history analysis, the PERFORM-3D software (PERFORM 2006) is used. The non-linear spatial models of the RC core wall structural system are made. In order to present the real behaviour of the structure during non-linear 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. 3(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. 3(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.

5. Ground motion records selection

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). 60 ground motions are selected from a large number of available records : 25 ground motions are recorded on the rock which corresponds to soil type A and 35 ground motions recorded on stiff soil which corresponds 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 motion 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 motion selection is that the mean value of the selected ground motion response spectra is 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 0.37 g 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 so 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

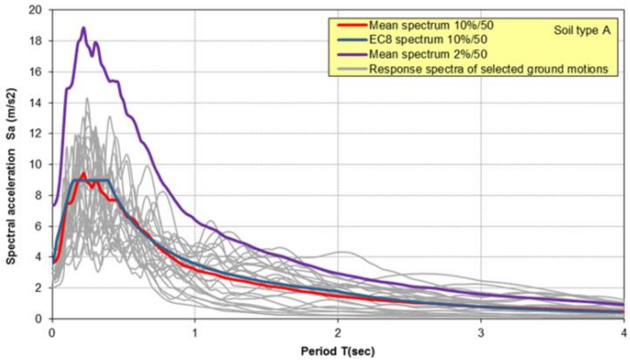


Fig. 4 Response spectra of the selected ground motions for soil type A, mean spectra of the selected ground motions for intensity levels 10%/50 and 2%/50 and elastic EC8 spectrum for soil type A for intensity level 10%/50

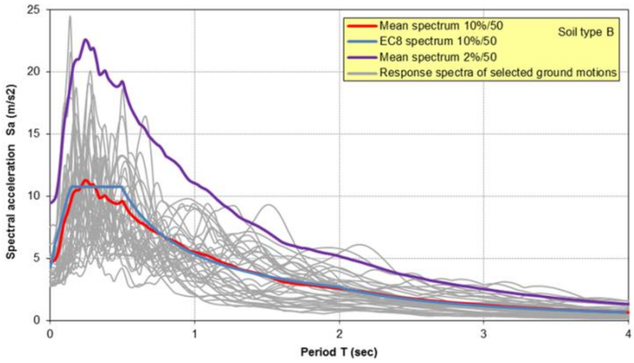


Fig. 5 Response spectra of the selected ground motions for soil type B, mean spectra of the selected ground motions for intensity levels 10%/50 and 2%/50 and elastic EC8 spectrum for soil type B for intensity level 10%/50

for this earthquake level were defined in the scope of the project of 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). Figs. 4 and 5 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.

6. Analysis results and verification of the proposed IMs

In order to examine and compare newly proposed IMs with existing ones, prototype 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 non-linear 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.

Table 4 PSDM parameters for the analysed IMs

DM	IM	Regression model parameters		Dispersion	Proficiency parameter	Error (%)
		a	b			
IDR_{max}	PGA	0.0039	0.5280	0.5371	1.0173	135.7
	PGV	0.0191	1.1254	0.2855	0.2537	25.3
	$S_a(T_1)$	0.0080	0.4136	0.3992	0.9651	75.1
	$S_v(T_1)$	0.0106	0.8320	0.2801	0.3367	22.9
	SI_m	0.0110	0.9050	0.2468	0.2727	8.3
	SI_{yh}	0.0114	0.9230	0.2449	0.2653	7.5
	$S_{v,avg}$	0.0111	1.0468	0.2525	0.2412	10.8
	SI_{vj}	0.0110	0.9147	0.2401	0.2625	5.4
	$SI_{vj1.5}$	0.0111	0.8659	0.2312	0.2670	1.4
	$SI_{vj\,GM}$	0.0112	0.9129	0.2279	0.2496	0.0
IDR_{mv}	PGA	0.0028	0.4299	0.5713	1.3289	215.3
	PGV	0.0115	1.0814	0.3485	0.3223	92.3
	$S_a(T_1)$	0.0048	0.4951	0.3171	0.6405	75.0
	$S_v(T_1)$	0.0066	0.8863	0.2510	0.2832	38.5
	SI_m	0.0069	0.9661	0.2042	0.2114	12.7
	SI_{yh}	0.0072	0.9869	0.1926	0.1952	6.3
	$S_{v,avg}$	0.0071	1.0742	0.2413	0.2246	33.2
	SI_{vj}	0.0069	0.9746	0.1915	0.1965	5.7
	$SI_{vj1.5}$	0.0070	0.9501	0.1854	0.1951	2.3
	$SI_{vj\,GM}$	0.0070	0.9785	0.1812	0.1852	0.0

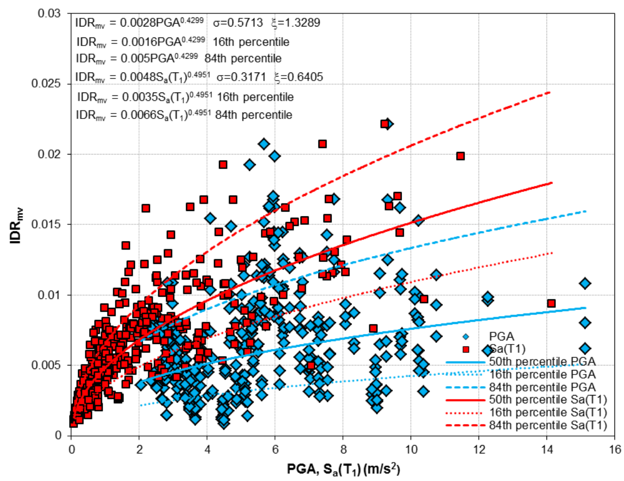


Fig. 6 PSDMs for IDR_{mv} conditioned upon PGA and $S_a(T_1)$

Appropriate PSDMs (the relationship between the selected IMs and DMs) are obtained. In the procedure of defining relationship between selected IMs and DMs, the regression analysis is performed and detailed statistical processing of results is made. The exponential relationship between the DMs and IMs (Eq. (8)) is adopted. For each analysed relationship between IMs and DMs, the median (50th percentile), defined by Eq. (8) is derived, as well as the 16th and 84th percentiles, representing relationships which correspond to a plus-minus standard deviation from the median.

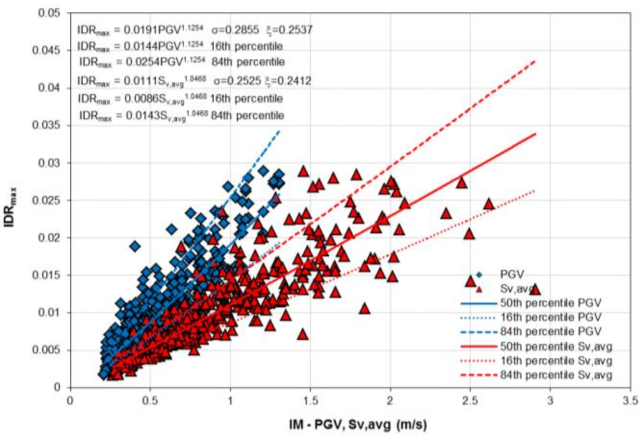


Fig. 7 PSDMs for $IDR_{\max}$ conditioned upon PGV and $S_{v,avg}$

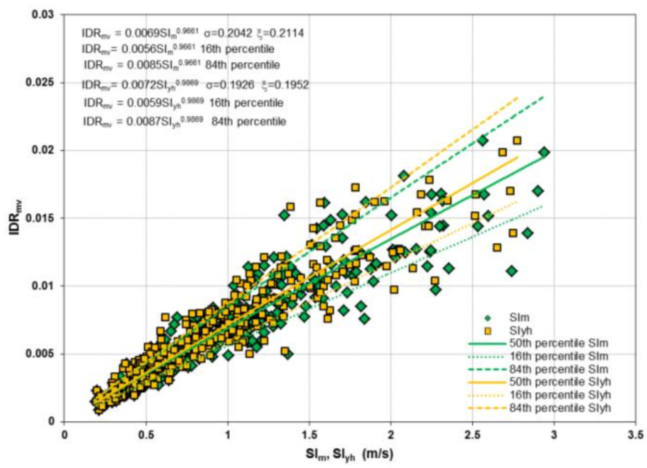


Fig. 8 PSDMs for IDR_{mv} conditioned upon SI_m and SI_{yh}

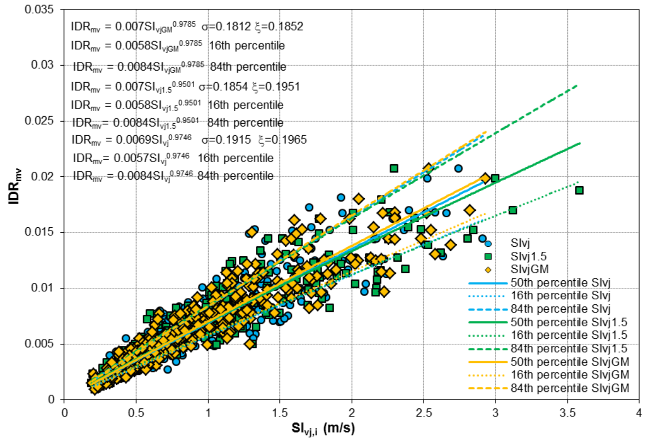


Fig. 9 PSDMs for IDR_{mv} conditioned upon SI_{vj} , $SI_{vj1.5}$ and SI_{vjGM}

6.1 Comparison and verification related to efficiency, practicality and proficiency

The results of the analysis are reported in Table 4 in terms of: 1) standard deviation $\sigma_{DM/IM}$ as measure of the efficiency, 2) the slope, *b*, of the PSDM as measure of the practicality and 3) the proficiency parameter, ξ as measure of the proficiency. The derived PSDMs for the selected DMs ($IDR_{\max}$ and IDR_{mv}) conditioned upon: 1) PGA and $S_a(T_1)$, 2) PGV and $S_{v,avg}$, 3) SI_m and SI_{yh} and 4) SI_{vj}, $SI_{vj1.5}$, SI_{vjGM} are showed on Figs. 6, 7, 8 and 9 respectively.

Different IMs have been compared against the newly proposed ones and correspondent values of errors are calculated (Table 4 and Fig. 10). As comparative value, dispersion for SI_{vjGM} is adopted.

The data (Table 4 and Fig. 6) confirm the results obtained by the authors (Pejovic *et al*. 2017): PGA and $S_a(T_1)$, the most used IMs in literature, especially in obtaining the fragility curves, are not proved to be appropriate for the RC high-rise buildings. The error of PGA and $S_a(T_1)$ with respect of the newly proposed IMs are: 1) the PGA error ranges ≈135.7 and 215.3% and 2) the $S_a(T_1)$ error is ≈75% .

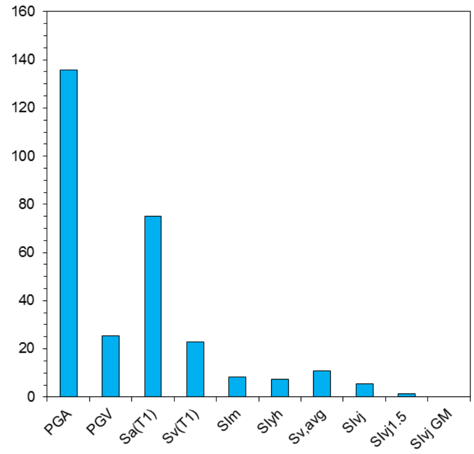
Unlike PGA and $S_a(T_1)$, PGV following with $S_{v,avg}$ and $S_v(T_1)$ are proved to be more optimal IMs for the RC high-rise buildings. Although PGV and $S_{v,avg}$ are the most practical (illustrated by the largest regression parameter, *b*, ranges from 1.0468 to 1.1254), their efficiency (indicated by the higher standard deviation) is lower then for the newly proposed IMs (SI_{vj}, $SI_{vj1.5}$ and $SI_{vj,GM}$). The errors with respect of the newly proposed IMs are: 1) the PGV error ranges ≈25.3 and 92.3%, 2) the $S_{v,avg}$ error ranges ≈10.8 and 33.2% and 3) the $S_v(T_1)$ error ranges ≈22.9 and 38.5%.

Amongst the all considered existing IMs, 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_{mv} 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 which comprise a wider range of response spectra are more appropriate. The values of derived PSDM parameters for SI_m and SI_{yh} are approximately the same, because the modal period T_h is approximately equal to $2T_y$ for the considered prototype buildings. Also, regarding the proficiency, SI_m and SI_{yh} are the most proficient amongst considered existing IMs, following with $S_{v,avg}$. Further on, SI_m and SI_{yh} have approximately the same or higher standard deviations and proficient parameters compared to the newly proposed IMs (SI_{vj}, $SI_{vj1.5}$ and $SI_{vj,GM}$). The errors with respect of the newly proposed IMs are: 1) the SI_m error ranges ≈8.3 and 12.7% and 2) the SI_{yh} error ranges ≈6.3 and 7.5%.

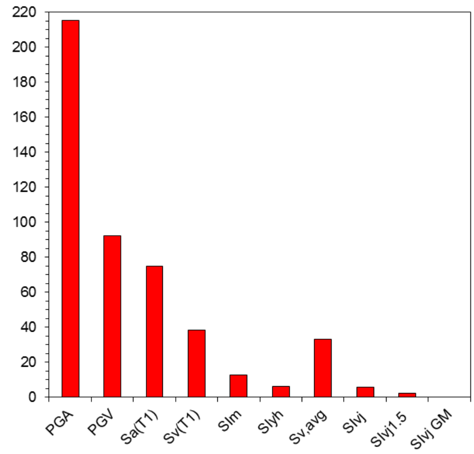
Based on the previous observations it can be stated that the newly proposed IMs (SI_{vj}, $SI_{vj1.5}$ and $SI_{vj,GM}$) can be considered as appropriate IMs since: 1) analysis conducted on the large sample of ground motions confirmed approximately the same or higher optimality compared to SI_m and SI_{yh} and 2) they are characterised by easily and efficiently calculation from the velocity spectrum. In general, the basic idea of SI_{vj}, $SI_{vj1.5}$ and $SI_{vj,GM}$ to find adequate replacement for the SI_m and SI_{yh} and overcome difficulties in calculating integral along the velocity spectrum is achieved.

6.2 Sufficiency comparison

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



(a)



(b)

Fig. 10 IM error percentage: (a) for PSDM with $IDR_{\max}$ and (b) for PSDM with IDR_{mv}

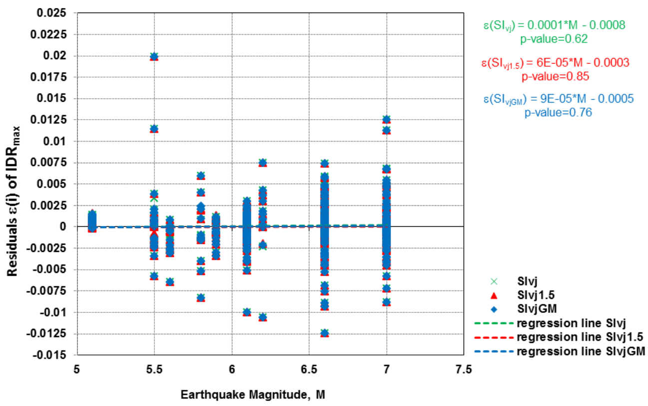
Table 5 Sufficiency comparison of IMs using p-values for $IDR_{\max}$

IM	p-value	
	Magnitude (M)	Distance (R)
PGA	**1.25E-10**	0.71
PGV	**2.55E-07**	0.08
$S_a(T_1)$	0.15	0.31
$S_v(T_1)$	0.94	0.44
SI_m	0.48	0.47
SI_{yh}	0.38	0.59
$S_{v,avg}$	0.20	0.26
SI_{vj}	0.62	0.80
$SI_{vj1.5}$	0.85	0.81
SI_{vjGM}	0.76	0.76

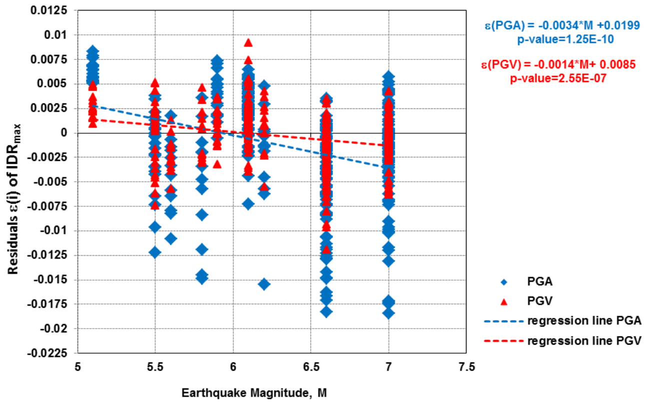
Note: Bold value indicates insufficient IMs

reported in Table 5: the p-values for the considered IMs and demand measure $IDR_{\max}$.

Results evaluated by performing the regression analysis on the normalised 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.



(a)



(b)

Fig. 11 Comparison of the IMs sufficiency from M for $IDR_{\max}$

The derived results show: 1) all of considered IMs are independent of R with p-values in range from 0.08 to 0.81 and 2) PGA and PGV are insufficient with respect to M, while other considered IMs have proved to be sufficient.

Fig. 11 shows the comparison of the regression for the $IDR_{\max}$ to compare the sufficiency of SI_{vj}, $SI_{vj1.5}$ and $S_{vj,GM}$ (Fig. 11(a)) to insufficiency of PGA and PGV (Fig. 11(b)) relation to M. The slope of regression line c for PGA and PGV differed from 0 significantly comparing with the slope of SI_{vj}, $SI_{vj1.5}$ and $S_{vj,GM}$. The p-values are 0.62, 0.85 and 0.76 for SI_{vj}, $SI_{vj1.5}$ and $S_{vj,GM}$ respectively, unlike the small values obtained for PGA and PGV, indicating that SI_{vj}, $SI_{vj1.5}$ and $S_{vj,GM}$ are much more sufficient IMs for conditioning of the PSDM.

7. Conclusions

In this paper, the novel IMs which are more appropriate for RC high-rise buildings than other existing IMs, are proposed. The proposed IMs are based on existing IMs (SI_m and SI_{yh}) that accounts for wider range of frequency velocity spectrum content. Comparison of the newly proposed IMs with the selected existing ones was done by analysing the features of optimal IMs such as: efficiency, practicality, proficiency and sufficiency. 20-storey, 30-storey and 40-storey RC high-rise buildings with core wall structural system are selected for conducted analysis. A detailed regression analysis and statistical processing of results are performed and appropriate demand models which relate IMs to DMs are derived. On the basis of the derived PSDMs appropriate conclusions regarding newly proposed IMs are made.

The results show that the newly proposed IMs (SI_{vj}, $SI_{vj1.5}$ and $SI_{vj,GM}$) can be considered as appropriate IMs since: 1) efficiently predict the DMs with minimum dispersion and satisfactory practicality as compared to the other commonly used IMs for high-rise buildings (e.g., PGA and $S_a(T_1)$), 2) analysis conducted on the large sample of ground motions confirmed approximately the same or higher optimality compared to SI_m and SI_{yh} and 3), they are characterised by easily and efficiently calculation from the velocity spectrum and thus overcome the difficulties in calculating integral, along the velocity spectrum.

The proposed IMs exhibit relatively high efficiency, practicality, proficiency and sufficiency based on the analysis conducted on the large number of selected ground motions where wide range of magnitudes, distances to source and various soil types are included. The high optimality of the proposed IMs is confirmed in analysis conducted on the large size of ground motions random sample, thus they pretend to be adequate replacement for the SI_m and SI_{yh} and in general, they are quite appropriate for RC high-rise buildings.

In future, research conclusions in this paper could be validated by conducting analysis on more ground motions with different characteristics.

References

Ambraseys, N., Smit, P., Sigbjornsson, R., Suhadolc, P. and Margaris, B. (2002), Internet-site for European Strong-motion Ddata, European Commission, Directorate-General XII, Environmental and Climate Programme, Brussels, Belgium.

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. Concrete*, **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.

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., ... and Valensise, G. (2013), SHARE European Seismic Hazard Map for Peak Ground Acceleration, 10% Exceedance Probabilities in 50 years, Erdbebengefahrenkarte, Zürich, ETH Zürich.

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.

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

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., Lu, X., Li, M. and Ma, X. (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 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. Struct.*, **10**(3), 629-643.

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.

Pejovic, 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.

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

Pejovic, J.R., Serdar, N.N. and Pejovic, R.R. (2017), "Optimal intensity measures for probabilistic seismic demand models of RC high-rise buildings", *Earthq. Struct.*, **13**(3), 221-230.

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. (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.

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.*, 1-30.

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