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Dependence of RC high-rise buildings response on the earthquake intensity

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Preliminary report

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Dependence of RC high-rise buildings response on the earthquake intensity

The relationship between the ground motion intensity measure (IM) and the engineering demand parameter (EDP) is analysed in the paper so as to identify and define the most efficient EDP-IM relationships for RC high-rise buildings. A 30-story RC high-rise building with the core wall structural system was selected as the reference building. 240 nonlinear time-history analyses were conducted for 60 ground motions on the spatial model of the building. The existing intensity measures were analyzed and the new ones – providing the most efficient relationships for RC buildings - were proposed.

Key words:
high-rise RC buildings, intensity measure, engineering demand parameter, nonlinear time-history analysis

Prethodno priopćenje

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Ovisnost odziva armiranobetonskih visokih zgrada o mjeri intenziteta potresa

U radu je provedena analiza ovisnosti između mjere intenziteta potresa IM i parametra seizmičkog odziva EDP, radi pronalaženja i definiranja najučinkovitijih ovisnosti EDP-IM za armiranobetonske visoke zgrade. Kao referentna konstrukcija izabrana je trideseterokatna AB visoka zgrada konstrukcijskog sustava s AB jezgrom. U okviru analize provedeno je 240 nelinearnih dinamičkih analiza na djelovanje 60 potresnih zapisa na prostornom modelu konstrukcije. Analizirane su postojeće i predložene su nove mjere intenziteta koje daju najučinkovitije ovisnosti za AB visoke zgrade.

Ključne riječi:
visoke AB zgrade, mjera intenziteta, parametar seizmičkog odziva, nelinearna dinamička analiza

Vorherige Mitteilung

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Verhalten von Stahlbetonhochbauten im Bezug zur Erdbebenintensität

In dieser Arbeit wird der Bezug zwischen dem Intensitätsmaß IM und dem Parameter seismischer Antwort EDP analysiert, um das wirksamste Verhältnis EDP-IM für Stahlbetonhochbauten zu erforschen und zu definieren. Als Referenzobjekt wurde ein 30-stöckiges Stahlbetongebäude mit Kernwänden als Tragkonstruktion gewählt. Auf einem räumlichen Modell des Tragwerks beruhend wurden unter der Einwirkung 60 verschiedener Erdbebenaufzeichnungen 240 nichtlineare dynamische Analysen durchgeführt. Bestehende Intensitätsmaße wurden analysiert und neue Vorschläge, die insbesondere für Stahlbetongebäude anwendbar sind, wurden gegeben.

Schlüsselwörter:
Hochbauten, Intensitätsmaß, Parameter seismischer Antwort, nichlineare dynamische Analyse

1. Introduction

Over the past decades, the construction of high-rise buildings in seismically active areas has become an everyday design trend, which is mainly due to growing urbanisation, rapid growth of cities, and concentration of material resources in urban environments. In December 2011, the Council on Tall Buildings [1] asserted that an average height of tallest building will double in only two decades, from the year 2000 to 2020. This is why comprehensive studies of vulnerability of RC high-rise buildings have to be conducted for earthquake-prone areas. The Pacific Earthquake Engineering Research Center (PEER), which is currently conducting a large-scale research project called *Tall Buildings Initiative* [2], has been among the first to recognise the lack of research on this topic with regard to tall buildings.

A similar trend is also emerging in the South-European Mediterranean zone. As the entire Mediterranean belt is a seismically active area, detailed seismic analyses have to be undertaken for this category of buildings. This paper is a part of an extensive research work focusing on the probabilistic seismic analysis and estimation of vulnerability of RC high-rise buildings to seismic excitation typical for the South-European Mediterranean zone. More specifically, the theme of this paper is the analysis of relationship between the ground motion intensity measure, IM, and the engineering demand parameter, EDP, in order to identify and define the most efficient EDP-IM dependencies for RC high-rise buildings that could also be useful in practical terms. The EDP-IM dependency is efficient if it provides the lowest dissipation of EDP results for given IM values. The EDP-IM dependence is practical if the relationship can be established through intensity measures that enable a clear physical interpretation, and that can easily be calculated from seismic records and seismic responses directly resulting from the non-linear time-history analysis [3]. This dependence is indispensable for obtaining the probability of exceeding P [EDP/IM] of an appropriate measure of seismic response EDP, as related to the seismic intensity measure IM, in the process of probabilistic seismic analysis according to the performance-based design.

A thirty-storey RC high-rise building with the core wall structural system was selected as the reference building. In order to determine the most efficient EDP-IM model, 240 nonlinear time-history analyses were 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 analysis and statistical processing of results were performed, and appropriate EDP-IM relationships were derived. The existing intensity measures were analyzed and the new ones - providing the most efficient models for RC high-rise buildings - were proposed.

2. Selection and description of reference RC high–rise building

The reference structure selected in this paper is a thirty-storey RC high-rise building with the core wall structural system that assumes the entire seismic force, and with RC frames along the periphery that assume the gravity load only [4]. The typical plan view of the storey, the ETABS model [5] and PERFORM-3D model [6] of reference RC high-rise building are presented in Figure 1. The core wall structural system is applicable for high-rise buildings with 40 to 50 storeys [4]. This structural system has been selected because it corresponds to the number of storeys selected in this paper as realistically applicable to the South-European Mediterranean zone. On the other hand, structural systems with walls are the systems that are most frequently used for assuming earthquake action in the South-European Mediterranean seismic zone.

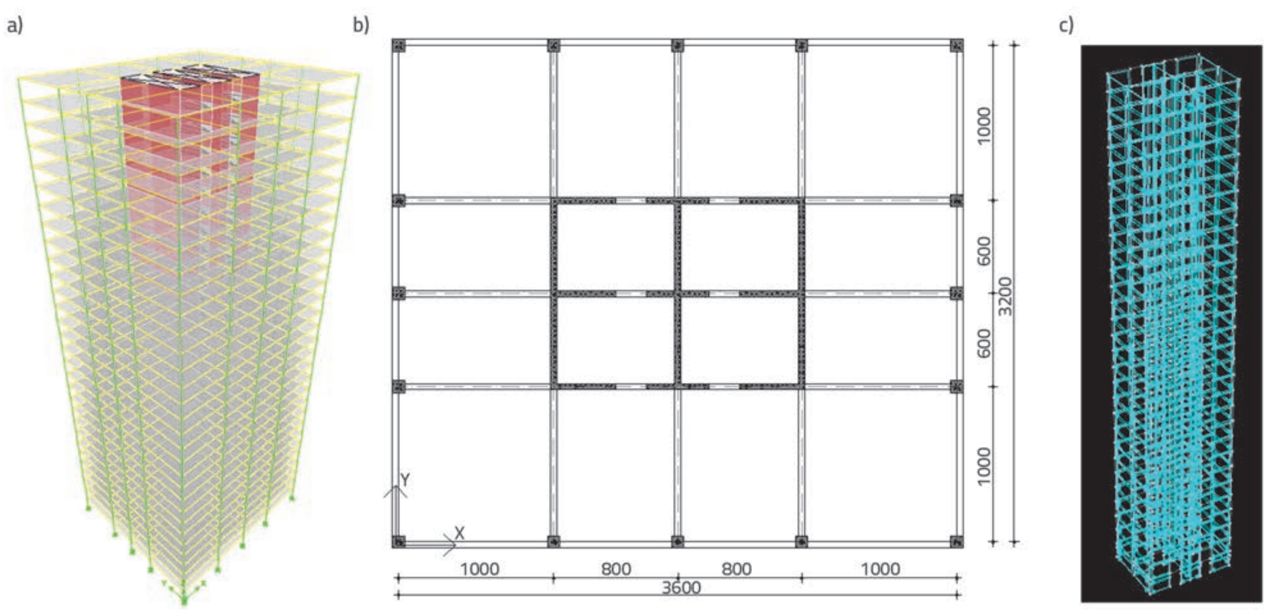

Figure 1. a) Etabs model of a reference building; b) typical plan view of the storey; c) Perform3D model of a reference building

In case of high-rise buildings, the RC core structural systems are very appropriate for architectural reasons, and because they are generally very often used. RC walls are placed in the central part of the building around communication core (lifts, staircases) thus forming a space system of high bearing capacity with regard to horizontal seismic forces in two orthogonal directions. The space between the central RC core and the structure's periphery most often remains empty or is rarely filled with RC columns, connected with the floor structure, in which case these columns represent secondary seismic elements. Basic properties of a reference building are shown in Table 1.

Table 1. Basic properties of reference building

Properties	Description
Total height [m]	90
Number of storeys	30
Storey height [m]	3
Floor structure	solid RC slab 20 cm in thickness
RC beams	40 x 65 cm
RC columns	80 x 80 cm
Core walls	storeys 1 to 5: 40 cm in thickness
	storeys 6 to 30: 30 cm in thickness
Coupling beams in X direction	30 x 80 cm, 40 x 80 cm
Concrete f_{ck} (f_{cm}) [MPa]	45(53)
Reinforcement f_{yk}(f_{ym}) [MPa]	500(575)
Modulus of elasticity of concrete E_{cm} [MPa]	36000

The analysis and design of the reference RC building was conducted according to the Eurocode 2 [7] and Eurocode 8 – Part 1 [8]. 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 reference peak horizontal ground acceleration for the adopted seismic zone amounts to 0.37g. The analysis of the reference building was conducted using the design response spectrum (elastic response spectrum decreased by the behaviour factor q through which it is indirectly assumed that the structure subjected to seismic action will consume energy through plastic behaviour of its elements). The reference building was designed for the medium ductility class (DCM). The structural system of the analysed reference building is a ductile wall system in both horizontal directions according to the classification presented in Eurocode 8 – Part 1. Considering the structural system and the achieved regularity in plan and regularity in elevation, the value of behaviour factor for the ductility class DCM amounts to 3.6. The total seismic force was calculated using the modal response spectrum analysis, which is quite appropriate considering higher-mode effects in high-rise RC buildings. The modal periods of the building and mass participation factors of first four modes, are presented in Table 2. The elastic flexural and shear stiffness properties of structural elements are taken to be equal to one-half of the corresponding stiffness of the uncracked elements, which is in accordance with the Eurocode 8, Part 1 [8]. The ETABS software [5] was used for the linear analysis and seismic design of building. For the purpose of design, the spatial model of building was made.

Table 2. Modal periods and mass participation factors

Mode	1	2	3	4
Period in Y direction [s]	2.880	0.623	0.270	0.164
Period in X direction [s]	2.597	0.702	0.347	0.228
Mass participation factors in Y direction [%]	63.53	19.43	7.05	3.57
Mass participation factors in X direction [%]	67.7	17.4	5.23	2.78
Sum of mass participation factor of listed 4 modes in Y direction [%]				93.58
Sum of mass participation factor of listed 4 modes in X direction [%]				93.11

By the analyses of the calculated seismic forces obtained in the structure, it was 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. At that, the frame action in central axes (peripheral column-beam-wall) is negligible. This is why the RC core was the subject of further detailed analysis and design in accordance with relevant Eurocode provisions, which was followed by nonlinear time-history analyses.
All Eurocode 8 – Part 1 provisions [8] related to the design and reinforcement of DCM ductile walls have been complied with during analysis of reinforced concrete walls. 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 this percentage varies from 0.959 % to 0.893 % for internal walls in Y direction. 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 periphery of boundary elements at no more that 20 cm intervals. The hoops have been adopted at every 10 cm intervals in lower sections (or 15 cm at higher sections) so that the distance between two neighbouring longitudinal bars, supported by hoops, does not exceed 20 cm. 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.

3. Nonlinear model of reference RC high-rise building

The PERFORM-3D software [6] was used for the nonlinear time-history analysis. The nonlinear spatial model of the RC core walls was made. The mathematical model used for elastic analysis is extended to include the strength of structural elements and their post-elastic behaviour. In order to present as realistically

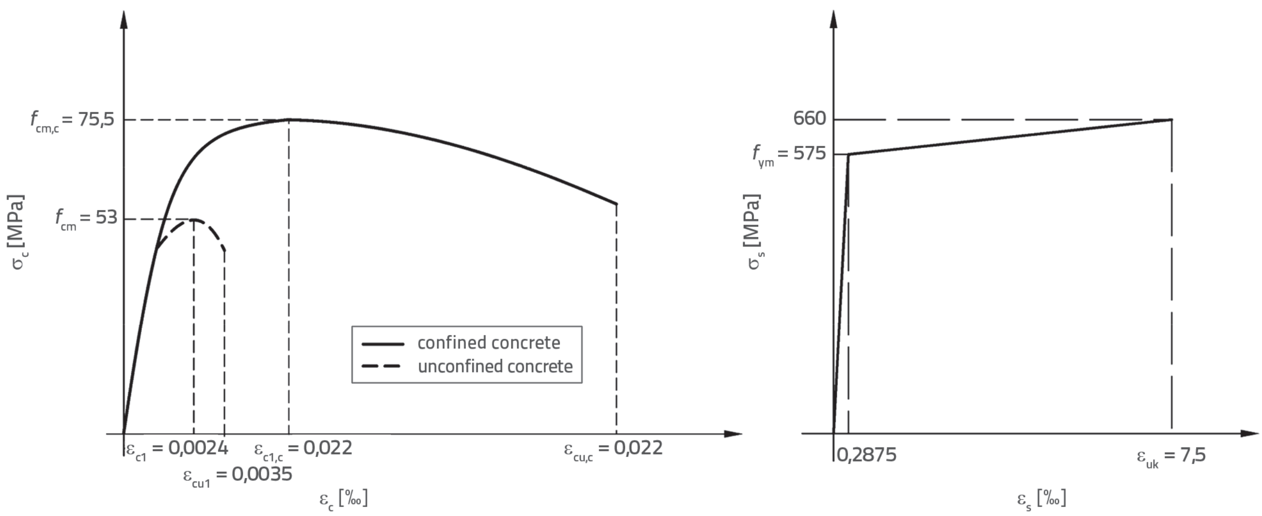

Figure 2. Stress-strain diagrams for: a) the unconfined and confined concrete with concrete mean strength of 53 MPa; b) reinforcing steel with expected yield mean strength of 575 MPa

as possible the real behaviour of the structure during nonlinear analyses, the properties of elements were based on mean values of material properties in accordance with recommendations given in Eurocode 8 – Part 1 [8], which differs from the design analysis phase where typical values of material properties are adopted (values with the fractile of 5 %) so as to remain on the side of safety. Stress-strain relationship for unconfined concrete, confined concrete, and reinforcement, compliant with recommendations given in Eurocode 8 – Part 2 [9], were adopted. The stress-strain diagram for confined concrete defined in Eurocode 8 – Part 2 [9] is based on the proposal given by Mander, Priestley and Park [10]. The stress-strain diagrams for unconfined concrete with the mean compressive strength amounting to 53 MPa, and for the confined concrete with the adopted way of confinement using transverse reinforcement of boundary wall elements, are presented in Figure 2.a. The stress-strain diagram for reinforcing steel is defined in accordance with Eurocode 8 – Part 2 [9], and it represents a bilinear diagram with expected yield mean strength of 575 MPa and ultimate strength of 660 MPa (Figure 2.b).

The core walls are modeled using non-linear vertical fiber elements representing the expected behavior of the concrete and reinforcing steel [11]. The area and location of reinforcement within the cross-section, as well as concrete properties, were defined using individual fibers forming the cross-section of the wall. The shear behavior is modeled as elastic. The coupling beams are defined as elastic beam elements with a nonlinear displacement shear hinge at the mid-span of the beam. The shear hinge behavior is based on test results by J.W. Wallace [12].

4. Ground motion records selection

The South-European Mediterranean seismic zone was selected in this paper as the seismic zone of interest, and so the ground motion selection was done within this zone. The data of the Seismological Institute of Montenegro and the European strong-motion database [13] were used as database of ground motions. In this paper, ground motions were selected as related to the magnitude, *M*, distances to source, *R*, and type of soil. The uncertainties during ground motions selection are taken into account by the use of a greater number of seismic records with a wider range of magnitudes, distances to source, and different types of soil. High-rise buildings are specific, due to their response frequency range is much wider than for low-rise or mid-rise buildings. Accordingly, it is necessary to include a larger number of ground motions, various magnitudes and distances to source. Uncertainties during ground motions selection are usually much higher than other types of uncertainties in the probabilistic analysis of seismic risk.

Sixty seismic records were selected. Out of these records 25 were recorded in rock, which corresponds to the type A soil according to Eurocode 8. The remaining thirty-five records were recorded on stiff soil, which corresponds to the type B soil according to Eurocode 8. Magnitude values for selected records range between 5.1 and 7.0, while distances to source vary from 5 to 70 km. The basic criterion used in this paper for the selection of ground motions is that the mean value of their response spectra be compatible with the corresponding target spectrum in a wider range of periods. The elastic spectrum from Eurocode 8 was selected as the target spectrum for the return period of 475 years, with the design ground acceleration amounting to 0.37 g. Due to the lack of ground motions in the Southern Euro-Meditteranean zone, which may be selected without being previously scaled and with mean spectrum to be in accordance with Eurocode spectrum, it was necessary to scale the ground motions. The mean squared error method (MSE) was chosen as a mode of scaling of ground motions [14]. By this method ground motions are scaled in a way where the mean squared error is minimized over the whole range of periods. The mean square error represents the difference between the spectral acceleration of ground motion records and target spectrum and it is calculated by expression (1).

$$MSE = \frac{\sum_{i=1}^{n}[S_a^{target}(T_i) - f \cdot S_a^{record}(T_i)]^2}{n} \qquad (1)$$

The parameter f in expression (1) is the linear scaling factor. The geometric mean spectrum of the selected ground motions is adopted to be the mean spectrum [14].

Besides earthquakes corresponding to a 475-year return period (earthquake with a 10 % chance of exceedance in 50 years), the reference structure was also tested for seismic action with a 2475-year return period (earthquake with a 2 % chance of exceedance in 50 years). As such a high level of seismic intensity is not defined in Eurocode 8 – Part 1, a more recent literature was consulted in this paper for defining appropriate earthquakes with the 2 %/50 intensity. The data for this earthquake strength were defined in the scope of the project *Seismic hazard harmonization in Europe – SHARE* [15]. 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 [15]. Accordingly, the mean value of seismic record acceleration spectra for the intensity of 2 %/50 is two times greater than the mean value of seismic record acceleration spectra for the intensity of 2 %/50.

Figures 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 certain soil types.

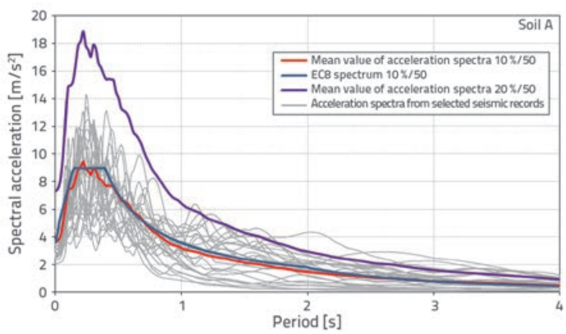

Figure 3. Response spectra of the selected ground motions for soil type A, mean spectra of the selected ground motions for intensity levels 10 %/50 and 2 %/50 and elastic EC8 spectrum for soil type A for intensity level 10 %/50

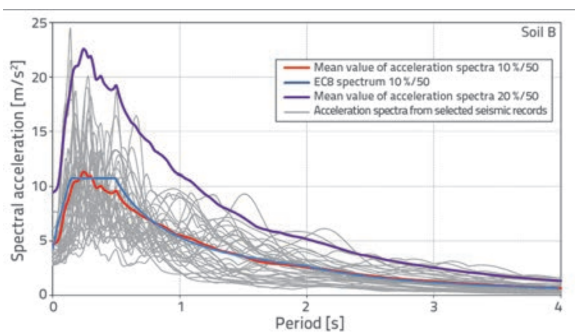

Figure 4. Response spectra of the selected ground motions for soil type B, mean spectra of the selected ground motions for intensity levels 10 %/50 and 2 %/50 and elastic EC8 spectrum for soil type B for intensity level 10 %/50

5. Selection of earthquake intensity measures and engineering demand parameters

The selection of an appropriate intensity measure is a question that has been studied for a long time in earthquake engineering. The intensity measures should be such that they comprise the greatest possible number of earthquake features such as the amplitude, frequency content, duration of strong part of ground motion, etc. Intensity measures representing ground motion amplitudes are: peak ground acceleration (PGA), peak ground velocity (PGV), and peak ground displacement (PGD). The peak ground acceleration (PGA) exerts the greatest influence on the seismic response of structures with higher frequencies (periods of less than 0.5 s), while structures with lower frequencies, i.e. with periods of more than 0.5 s, are more sensitive to peak ground velocity (PGV) and peak ground displacement (PGD) [16]. Most frequent intensity measures characterizing the frequency content are: spectral acceleration $S_a(T_1)$, spectral velocity $S_v(T_1)$, spectral displacement $S_d(T_1)$, and pseudo spectral velocity $PSV(T_1)$. These intensity measures are dependent on the eigen modal period of structure, and they constitute peak responses of systems with one degree of freedom.

High-rise buildings are specific, due to their response frequency range is much wider than for low-rise or mid-rise buildings. Intensity measures such as spectral values $S_a(T_1)$, $S_v(T_1)$, $S_d(T_1)$ and $PSV(T_1)$ represent only specific points in frequency content of the response spectrum. For that reason, intensity measures comprising a wider range of frequency content of response spectra are more appropriate for the case of high-rise buildings. In this paper, the existing intensity measures proposed by individual researchers, which cover a wider range of response spectra, are analysed in detail in the first phase. Then, on the basis of these measures, the authors of this paper propose new intensity measures. The following intensity measures, comprising a wider frequency range of response spectra, are studied in this paper:

Housner's spectrum intensity SI_H

The Housner's mean spectrum intensity SI_H is defined as the area below the elastic spectrum of velocity between the periods of 0.1 s and 2.5 s [17]:

$$SI_H = \frac{1}{2.4} \cdot \int_{0.1}^{2.5} S_v(T)dT \tag{2}$$

Matsumura mean spectrum intensity SI_m

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 period corresponding to yield of structure [18]:

$$SI_m = \frac{1}{T_y} \cdot \int_{T_y}^{2T_y} S_v(T)dT \tag{3}$$

Martinez-Rueda mean spectrum intensity SI_{yh}

The Martinez-Rueda proposed 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 [19]:

$$SI_{yh} = \frac{1}{T_h - T_y} \cdot \int_{T_y}^{T_h} S_v(T)dT \tag{4}$$

The last two intensity measures (expressions (3) and (4)) take into account the increase of the modal period of vibration during the seismic action due to nonlinear stiffness and strenght degradation. The corresponding period intervals $[T_y, 2T_y]$ i $[T_y, T_h]$ were adopted for that reason. In the case of RC high-rise buildings, higher-mode effects can not be neglected, and so the MPF (mass participation factor) weighted average value is adopted in this paper for the period corresponding to yield of structure T_y (expression 5) [16]:

$$T_y = \frac{m_1 \cdot T_1 + m_2 \cdot T_2 + \ldots + m_n \cdot T_n}{m_1 + m_2 + \ldots + m_n} \tag{5}$$

where $m_1,\ldots m_n$ 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 [19], based on the following expression:

$$T_h = T_y \cdot \sqrt{\frac{\mu}{1 + \alpha \cdot \mu - \alpha}} \tag{6}$$

where $\mu = \Delta_h / \Delta_y$ is the displacement ductility factor, Δ_h 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.
Two cases were considered for the reference building in order to analyse the influence of higher-mode effects for the computation of intensity measures and yield periods T_y. Only the structure modes with mass participation factors greater than 5 %, which are the first three modes of the reference building, were taken into account in the first case. In the second case, in which the first four modes of structure were taken into account, the analysis was made so as to consider the need of taking into account the modes whose mass participation factors are smaller than 5 %.
The authors of this paper defined for RC high-rise buildings the corresponding mean spectral values as the intensity measures that take into account the higher-mode effects (three modes in total), namely:

- Mean spectral velocity $S_{v,avg1}$, Eq (7):

$$S_{v,avg1} = \frac{m_1 \cdot S_v(T_1) + m_2 \cdot S_v(T_2) + m_3 \cdot S_v(T_3)}{m_1 + m_2 + m_3} \tag{7}$$

- Mean spectral displacement $S_{d,avg1}$, Eq (8):

$$S_{d,avg1} = \frac{m_1 \cdot S_d(T_1) + m_2 \cdot S_d(T_2) + m_3 \cdot S_d(T_3)}{m_1 + m_2 + m_3} \tag{8}$$

- Mean pseudo spectral velocity PSV_{avg1}, Eq (9):

$$PSV_{avg1} = \frac{m_1 \cdot PSV(T_1) + m_2 \cdot PSV(T_2) + m_3 \cdot PSV(T_3)}{m_1 + m_2 + m_3} \tag{9}$$

In analogy to these intensity measures, the values $S_{v,avg2}$, $S_{d,avg2}$ and PSV_{avg2} were defined, which take into account spectral values for the first four structural modes.
The Matsumura mean spectrum intensity SI_m, and the Martinez-Rueda mean spectrum intensity SI_{yh}, are intensity measures defined through the integral along the velocity spectrum, which is not very practical for rapid calculation of these values. Figure 5 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 with the area of the trapezium defined with points T_y-$2T_y$-B-A, or with rectangles whose areas are defined with spectrum values in point $1.5T_y$ (rectangle T_y-$2T_y$-D-C) or in point T_{GM}, which is 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').

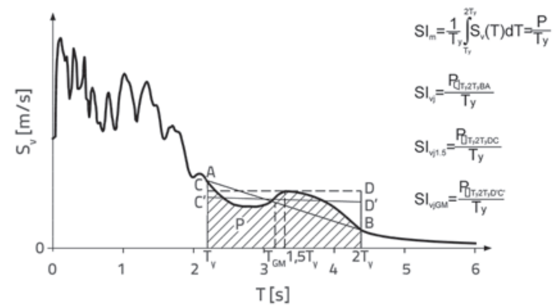

Figure 5. Schematic view of method for obtaining proposed new intensity measures SI_{vj}, $SI_{vj1.5}$ and SI_{vjGM}

Consequently, the authors of this paper defined and proposed new intensity measures as follows:

- Mean velocity spectrum intensity SI_{vj}, Eq (10):

$$SI_{vj} = \frac{S_v(T_y) + S_v(2T_y)}{2} \tag{10}$$

- Mean velocity spectrum intensity $SI_{vj1.5}$, representing the velocity spectrum value for the modal period of $1.5T_y$
- Mean velocity spectrum intensity SI_{vjGM} representing the geometric mean of the velocity spectrum values from the modal period T_y to the modal period $2T_y$.

Proposed new intensity measures can easily and efficiently be calculated from the velocity spectrum, i.e. they are practical intensity measures. The applicability of these measures was tested by the analysis of the obtained dispersion of results.
The interstorey drift (relative storey drift divided with the storey height) was selected in this paper as a engineering demand parameter. In fact it is the most frequently used engineering demand parameter. The interstorey drift can be calculated very easily, and it belongs to the group of practical engineering demand parameters, as it is the direct result of the nonlinear time-history analysis. The following two characteristic interstorey drift values were selected: maximum interstorey drift for the entire structure IDR_{max} and mean value of maximum interstorey drifts IDR_{sr}. The maximum interstorey drift, IDR_{max}, is the engineering demand parameter that is most often used for describing the state of collapse, while the mean value of maximum interstorey drifts IDR_{sr} is used to describe the damage level.

6. Analysis results

In order to define the EDP-IM relationship for RC high-rise buildings, the reference building was exposed to 60 ground motions with two levels of intensity in both directions of the structure. The total of 240 nonlinear time-history analyses were performed. This required approximately 60 hours of runtime on computer Intel® Core™ i5-3470 CPU 3.20 GHz with 8 GB of memory. Only the results obtained for earthquake records in Y direction of the reference building are presented in this paper. The results obtained for the records in X direction are in accordance 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 scatter diagrams with 120 pair points (IM_i, EDP_i) were obtained. The regression analysis was performed for each of these diagrams and, in the scope of these analyses, detailed statistical processing of results was made, and the corresponding EDP-IM relationships were derived. A special program was created to conduct this analysis using the Matlab software [20]. The algebraic models of the EDP and IM relationship were analysed through the regression analysis conducted in this paper, and it was established that the greatest statistical level of connection is obtained using the regression model that is defined by the following expression (11):

$$\widehat{EDP} = a \cdot IM^b \qquad (11)$$

The relationship between the engineering demand parameter EDP and the intensity measure IM shown in expression (11) was assumed. Engineering demand parameter histograms were prepared (for IDR_{max} and IDR_{sr}) and they show that the distribution of seismic response corresponds to the lognormal distribution. The comparison of the obtained distribution with the theoretical lognormal distribution was made using two tests, i.e. the X^2 test and the Kolmogorov test. Both tests confirmed that the seismic response distribution EDP (for IDR_{max} i IDR_{sr}) is lognormal for the corresponding intensity measure IM. Figure 6 shows the maximum interstorey drift histogram IDR_{max} obtained at 60 ground motions for two intensity levels 10 %/50 and 2 %/50, and the corresponding theoretical lognormal distribution. The lognormal distribution describes well the obtained distribution of the maximum interstorey drift IDR_{max}. Distribution of the random variable EDP/IM, i.e. distribution of the seismic response parameter with regard to the intensity measure is lognormal, with the following mean value:

$$\hat{\mu}_{\ln EDP/IM} = \frac{\sum_{i=1}^{n} \ln \widehat{EDP}_i}{N} \qquad (12)$$

and with the standard deviation that is calculated as deviation of the natural logarithms of the residuals IDRmax data obtained (on random sample) from the regression line:

$$\sigma^2_{\ln EDP/IM} = \frac{\sum_{i=1}^{n} \left[\ln EDP_i - \ln \widehat{EDP}_i \right]^2}{N-2} \qquad (13)$$

where N is the size of random sample.

The standard deviation value defined by expression (13) is used in this paper for the estimation of the dispersion of results. For each analysed EDP/IM relationship, the median (defined by expression 11) was derived, as well as the 16 % and 84 % percentiles, representing dependences that correspond to a plus-minus standard deviation from the median.

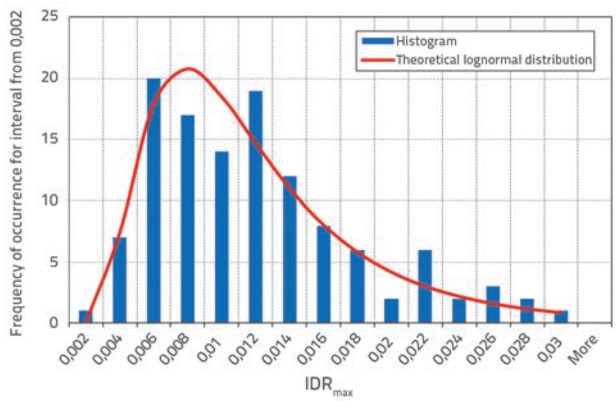

Figure 6. Maximum interstorey drift IDR_{max} histogram and the corresponding theoretical lognormal distribution

Derived regression-model parameters *a* and *b* (expression 11), dispersion results (standard deviations), and variation coefficients, are presented in Table 3. Coefficients of variation are smaller than 0.3 for most EDP-IM relationships considered, which means that a very small variability of results was obtained. This points to a high level of accuracy of calculated EDP-IM relationships, which is due to the great number of selected ground motions, i.e. in statistical term, to a great size of random sample.

As to the ground motion amplitude parameters, the peak ground velocity (PGV) provided less dispersion of results, compared to the peak ground acceleration (PGA), and peak ground displacement (PGD). In general terms, all intensity measures related to velocity provided less dispersion, compared to those related to acceleration and displacement, because the reference building has basic modal periods in the tripartite spectrum area that is sensitive to velocity [21].

With respect to amplitude parameters, spectral values (parameters representing concrete points of frequency content) have proven to be more efficient, i.e. the presented smaller dispersion of results. With regard to these measures, the smallest dispersion of results was provided by spectral velocity $S_v(T_1)$, while other spectral values provided greater dispersion (Table 3).

The derived relationships between the spectral velocity $S_v(T_1)$ and interstorey drifts IDR_{max} and IDR_{sr}, are shown in Figure 7. It can be concluded from these derived relationships that a regression model (expression 11) with a very high correlation coefficients r=0.9094 and r=0.9390 can be established between these parameters, which means that a very high derived mathematical connection exists between these parameters. In addition, very small dispersion values $\sigma_{DRmax/Sv}$ and $\sigma_{IDRsr/Sv}$ were obtained,

Table 3. Derived regression model parameters, standard deviations and coefficients of variation for the analyzed IDR_i-IM_i relationships

Engineering demand parameter EDP	Intensity measure IM – Description	Intensity measure IM – Denotation	Regression model parameters a	Regression model parameters b	Standard deviation σ	Coefficient of variation C.O.V
IDR_{max}	Ground motion amplitude parameters	PGA	0.0039	0.5483	0.5222	0.5599
		PGV	0.0194	1.0960	0.2809	0.2865
		PGD	0.0096	0.0498	0.5719	0.6220
	Parameters representing concrete points of frequency content	$S_a(T_1)$	0.0079	0.6178	0.2838	0.2896
		$S_v(T_1)$	0.0108	0.8450	0.2391	0.2426
		$S_d(T_1)$	0.0204	0.6045	0.2876	0.2937
		$PSV(T_1)$	0.0129	0.6135	0.2848	0.2907
	Parameters comprising a wider range of frequency content	SI_H	0.0050	0.9663	0.2339	0.2371
		SI_m	0.0114	0.9181	0.2134	0.2159
		SI_{yh}	0.0117	0.9342	0.2108	0.2132
	New intensity measures	$S_{v,avg1}$	0.0111	1.0760	0.2033	0.2054
		$S_{d,avg1}$	0.0267	0.6924	0.2611	0.2656
		PSV_{avg1}	0.0133	0.9442	0.2071	0.2093
		$S_{v,avg2}$	0.0114	1.0830	0.2051	0.2073
		$S_{d,avg2}$	0.0274	0.6940	0.2608	0.2653
		PSV_{avg2}	0.0136	0.9582	0.2070	0.2092
		SI_{vj}	0.0113	0.9163	0.2101	0.2124
		$SI_{vj1,5}$	0.0113	0.8643	0.2014	0.2035
		SI_{vjGM}	0.0115	0.9251	0.2001	0.2021
IDR_{sr}	Ground motion amplitude parameters	PGA	0.0030	0.4198	0.5445	0.5875
		PGV	0.0116	1.0489	0.3161	0.3242
		PGD	0.0059	0.0436	0.5725	0.6228
	Parameters representing concrete points of frequency content	$S_a(T_1)$	0.0048	0.6627	0.2066	0.2088
		$S_v(T_1)$	0.0067	0.8722	0.1977	0.1996
		$S_d(T_1)$	0.0134	0.6494	0.2108	0.2132
		$PSV(T_1)$	0.0082	0.6582	0.2081	0.2104
	Parameters comprising a wider range of frequency content	SI_H	0.0031	0.9633	0.1950	0.1969
		SI_m	0.0071	0.9456	0.1798	0.1813
		SI_{yh}	0.0073	0.9600	0.1760	0.1774
	New intensity measures	$S_{v,avg1}$	0.0069	1.0721	0.1680	0.1692
		$S_{d,avg1}$	0.0176	0.7336	0.1895	0.1912
		PSV_{avg1}	0.0083	0.9613	0.1795	0.1810
		$S_{v,avg2}$	0.0071	1.0780	0.1690	0.1702
		$S_{d,avg2}$	0.0181	0.7351	0.1893	0.1910
		PSV_{avg2}	0.0085	0.9739	0.1824	0.1839
		SI_{vj}	0.0070	0.9375	0.1682	0.1694
		$SI_{vj1,5}$	0.0071	0.8910	0.1674	0.1686
		SI_{vjGM}	0.0072	0.9518	0.1632	0.1643

corresponding to coefficients of variation less than 0.3, which also points to a very small variability of the obtained data. The derived regression curve represents the median or mean value of IDR_i-S_v relationships. Curves corresponding to plus or minus one standard deviation from median, i.e. 16 % percentiles and 84 % percentiles, are presented in Figure 7.

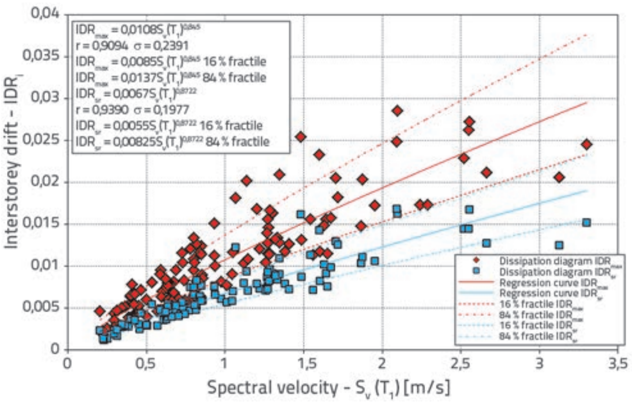

Figure 7. Derived relationships between the spectral velocity $S_v(T_1)$ and interstorey drifts IDR_{max} and IDR_{sr}

Intensity measures comprising a wider range of response spectra SI_m and SI_{yh} provided a smaller dispersion of results (from 15 to 35 %) compared to individual spectral values $S_a(T_1)$, $S_v(T_1)$, $S_d(T_1)$ and $PSV(T_1)$. 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 intensity measures comprising a wider range of response spectra are more efficient. In the case of mean Matsumura intensity SI_m and mean Martinez-Rueda intensity SI_{yh}, the dispersion is practically the same, because the modal period T_h is approximately equal to $2T_y$ for the case of the reference building. It was also observed that intensity measures SI_m and SI_{yh} provided a smaller dispersion of results compared to SI_H although all three covere a wide range of response spectra, the only difference being that SI_m i SI_{yh} comprise response spectra values in the range of greater periods, compared to SI_H.
Mean spectral values as the intensity measures that take into account the influence of higher-mode effects, $S_{v,avg1}$, $S_{d,avg1}$ and PSV_{avg1}, defined by the authors of this paper and proposed as new intensity measures for RC high-rise buildings (expressions 7, 8, and 9), provide approximately (10-40) % smaller dispersion of results compared to spectral values $S_v(T_1)$, $S_d(T_1)$ and $PSV(T_1)$, and hence demonstrate that they are better intensity measures for the case of RC high-rise buildings. Relationships derived between mean spectral velocities $S_{v,avg1}$ and $S_{v,avg2}$ and seismic response parameters IDR_{max} and IDR_{sr} are presented in Figure 8. By comparing two typical cases of mean spectral velocity, the first one in which only the structure modes with mass participation factors greater than 5 % are taken into account, which represents only the first three structural modes $S_{v,avg1}$, and the second one where the first four structural modes $S_{v,avg2}$ are taken into account, it can be observed that differences in dispersion are not great, i.e. that the dispersion is practically the same. It can therefore be concluded that it would be sufficient, during calculation of mean spectral values, to take into account only the modes that dominantly influence the system's response, i.e. in accordance with Eurocode 8, those vibration modes whose mass participation factors are greater than 5 % with the total sum of more than 90 %.

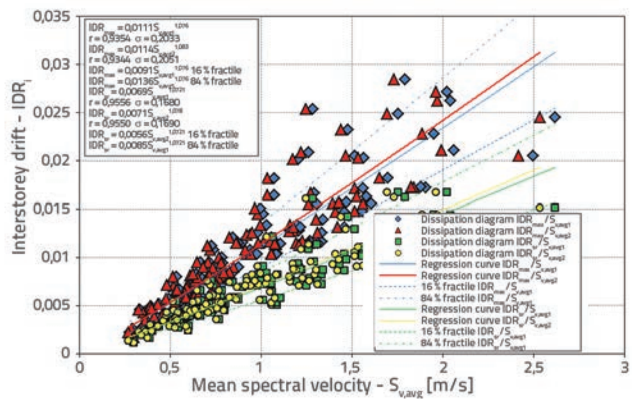

Figure 8. Derived relationships between mean spectral velocities $S_{v,avg1}$ and $S_{v,avg2}$ and interstorey drifts IDR_{max} i IDR_{sr}

Relationships derived between mean spectrum intensities and seismic response parameters IDR_{sr} are presented in Figure 9 as an illustration of the results obtained for the new intensity measures, mean velocity spectrum intensities SI_{vj}, $SI_{vj1.5}$ i SI_{vjGM}, as defined by the authors of this paper. All three new intensity measures provide approximately the same or smaller dispersion of results when compared to the corresponding spectrum intensities SI_m i SI_{yh} (which not meeting the practicality property due to computation via integrals), which qualifies them for use as their replacement, and also as new intensity measures. Here an emphasis can be placed on the values SI_{vj} i $SI_{vj1.5}$ due to simplicity of calculation i.e. their practicality property.
The most efficient of the EDP-IM relationships considered in the paper for RC high-rise buildings are obtained in the case of the proposed new intensity measures SI_{vj}, $SI_{vj1.5}$, SI_{vjGM} (mean velocity spectrum intensities) and $S_{v,avg}$ (mean spectral velocities).

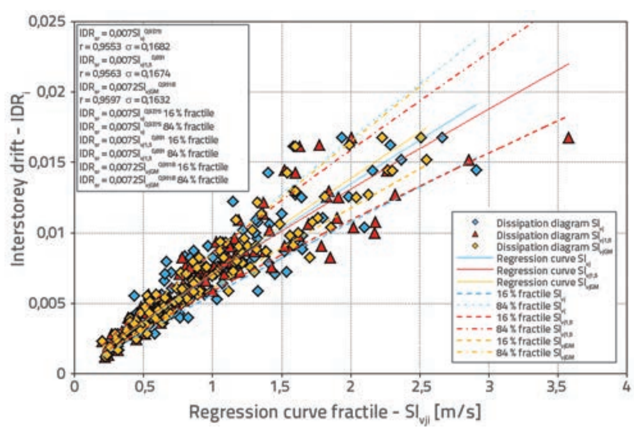

Figure 9. Relationships derived between new proposed intensity measures SI_{vj}, $SI_{vj1.5}$ and SI_{vjGM} and the interstorey drift IDR_{sr}

A smaller dispersion of results was obtained when the seismic response parameter was taken to be the mean value of maximum interstorey drifts IDR_{sr}, as related to the maximum interstorey drift for the entire structure IDR_{max}. It can be concluded that the level of damage can be determined more accurately (via IDR_{sr}) than the possibility of collapse of the structure (via IDR_{max}) in case of RC high-rise buildings.

Considering the quality of results obtained, the derived relationships can be used for determining interstorey drifts (IDR_{max} and IDR_{sr}) of RC high-rise buildings of structural system applicable to the reference building, for the case of design in the South-European Mediterranean zone.

7. Conclusion

In the scope of analysis of relationships between the earthquake intensity measure IM and the engineering demand parameter EDP, as conducted on the example of a selected reference RC high-rise building, appropriate conclusions were made regarding the efficiency of individual intensity measures IM, as related to the considered engineering demand parameters IDR_{max} and IDR_{sr}. Appropriate relationships between IDR_{max}-IM_i and IDR_{sr}-IM_i were derived as a result of a detailed analysis and statistical processing of results. Considering the quality of results obtained, the derived dependencies can be used for defining interstorey drifts (IDR_{max} and IDR_{sr}) for RC high-rise buildings of the structural system corresponding to the reference building and similar systems, for the case of design in the South-European Mediterranean zone.

A smaller dispersion of results was obtained when the seismic response parameter was taken to be the mean value of maximum interstorey drifts IDR_{sr}, as related to the maximum interstorey drift for the entire structure IDR_{max}, which means that the level of damage can be determined more accurately (via IDR_{sr}) than the possibility of collapse of the structure (via IDR_{max}) in case of RC high-rise buildings.

Intensity measures related to velocity provided less dispersion, compared to those related to acceleration and displacement and, by that, they have proven to be more efficient. Intensity measures based on frequency content are more efficient than the measures representing ground motion amplitudes (PGA, PGV, and PGD). Intensity measures comprising a wider range of response spectra are the intensity measures that provide the most efficient relationships between the engineering demand parameter and the intensity measure, in the case of RC high-rise buildings.

Mean spectral values that take into account spectral values of modes with mass participation factors greater than 5 %, $S_{v,avg}$, $S_{d,avg}$ and PSV_{avg}, defined by the authors of this paper as intensity measures, have proven to be more efficient for the case of RC high-rise buildings as related to the spectral values $S_v(T_1)$, $S_d(T_1)$, and $PSV(T_1)$. For that reason, they are proposed as the intensity measures appropriate for RC high-rise buildings. The value of $S_{v,avg}$ can however be singled out as it provides the smallest dispersion of results.

Mean velocity spectrum intensities SI_{vj}, $SI_{vj1.5}$, and SI_{vjGM}, defined by the authors of this paper as intensity measures, provide approximately the same or lower dispersion of results compared to the corresponding intensity measures SI_m and SI_{yh} that are defined via integrals according to the velocity spectrum. For that reasons, the authors of this paper propose, for the case of RC high-rise buildings, new intensity measures SI_{vj} i $SI_{vj1.5}$ that are at the same time quite practical, i.e. they can easily be calculated from the velocity spectrum while also providing the most efficient IDR_{max}-IM and IDR_{sr}-IM relationships. The authors of this paper currently work on verification of the results and conclusions made in this paper for various numbers of storeys of the structural system corresponding to that of the reference building. This will enable creation of EDP-IM relationships for the entire class of RC high-rise buildings of the RC core structural system for the South-European Mediterranean zone.

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