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PROBABILISTIC PERFORMANCE-BASED SEISMIC DEMAND MODEL FOR R/C FRAME BUILDINGS

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SUMMARY

In order to obtain a probabilistic assessment of seismic structural performance of a given structure for a given seismic hazard, it is necessary to know the relationships between ground motion intensity measures, *IMs*, and engineering demand parameters, *EDPs*. These relationships, denoted as the *probabilistic seismic demand models, PSDMs*, are investigated in this paper. This investigation is performed on four reinforced concrete frame buildings with deferent number of story (4, 6, 8 and 12) designed according to Eurocode 8, for the seismic hazard exposure of Southern Europe. *PSDM* "efficient" and "sufficient" features are examined and the most appropriate *PSDMs* are recommended.

INTRODUCTION

The performance-based engineering concept has been developing in the USA for the past two decades (SEAOC Vision 2000 [1]). Performance objectives (which couple performance level with ground motion recurrence periods) defined in SEAOC Vision 2000 [1] are given explicitly. While location-specific seismic hazard is defined probabilistically (in terms of the mean return period or, equivalently, in terms of the probability that a more severe earthquake will be experienced in a 50-year period), performance levels are expressed in a deterministic manner. Acceptable seismic performance is defined through the comparisons of mean levels of seismic demand and structural capacity using conventional code provisions (FEMA 273 [2]).

An important step forward is the probabilistic performance-based design and evaluation methodology proposed by the Pacific Earthquake Engineering Research Centre, *PEER* (Cornell and Krawinkler [3]). This methodology yields one of the first performance-based evaluation procedures that explicitly accounts for the *randomness* (measure of our inability to precisely understand the factors that effect phenomena such as seismic loadings and capacity of structures) and *uncertainty* (measure of the error introduced into calculations as a result of our inability to precisely characterise reality, e.g. seismic methods, structural models and etc.) inherent in performance prediction. Acceptable seismic performance is in this way defined by an explicit quantification of the confidence level at which the performance objective has been achieved.

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PEER probabilistic performance-based evaluation methodology can be mathematically expressed as:

$$\lambda[LS] = \iint_{EDP,IM} P[LS|EDP] \left| dP[EDP|IM] \right| \left| d\lambda[IM] \right| \tag{1}$$

In equation (1), $\lambda[LS]$ is the mean annual frequency of exceeding the limit state *LS*, i.e. annual probability that seismic response will be larger than the capacity; $P[LS|EDP]$ is the conditional probability of exceeding *LS* given the value of engineering demand parameter, *EDP*; $P[EDP|IM]$ is the conditional probability of exceeding each value of *EDP* given the value of ground motion intensity measure, *IM*; and $\lambda[IM]$ is the mean annual frequency of exceeding each value of *IM* i.e. ground motion hazard.

To obtain $P[EDP|IM]$, it is necessary to define pairs of *IM* and *EDP*, i.e. to define *probabilistic seismic demand models - PSDMs*. Relationships between *IM* and *EDP* are obtained by statically analysing the results of nonlinear time-history analyses of structure responses under earthquakes of different intensity. In this paper different *PSDMs*, which can be used in the probabilistic seismic performance assessment of R/C frame structures, are examined. Four reinforced concrete frame structures with different number of stories (4, 6, 8 and 12), all designed according to the Eurocode 8 [4], are used as the prototypes for the analysis. All R/C frames are exposed to the ''ordinary'' ground motions (recorded at source-to-site distance between 17 and 30 km in the region of the South Europe). On the given structures, "efficiency" and "sufficiency" of the chosen *PSDMs* are investigated.

Efficiency implies a smaller dispersion of *EDP* results for a given *IM*, making fewer non-linear dynamic analyses necessary to evaluate a *PSDM* with the same confidence. It is easy to show that for the same confidence level, decreasing dispersion by *n* times results in the decrease of the number of dynamic analysis by n^2 times (Shome [5]). *PSDM* is sufficient when *EDP* is conditionally independent, given *IM*, of earthquake magnitude, *M*, and of the source-to-site distance, *R*. The case when *PSDM* is not sufficient requires finding the relationship between *IM* and *M* and (or) *R*, and its inclusion in the total probability equation (1). Similar researches are performed in the steel frame structures (Shome, [5]) and R/C bridges (Mackie and Stojadinovic [6]).

DESCRIPTION OF THE R/C FRAME PROTOTYPES

Basic data about R/C frames

The four-story, six-story, eight-story and twelve-story reinforced concrete frames are the parts of the buildings whose floor plan (shown on the figure 1) is identical for all stories. The analyzed frames are marked RY2 on the floor plan. The building is located on a site with the effective peak ground acceleration *EPA*=0.36g for a return period of 475 years and firm and stable soil conditions.

The R/C frames have dimensions of the beams and columns as follows:

Four-story frame: The dimensions of the all beams and columns are 25/50 cm and 40/40 cm, respectively.
Six-story frame: The dimensions of all beams are 30/55cm, and of all exterior columns 45/45 cm, and of the interior columns 50/50 cm.
Eight-story frame: The dimensions of interior columns of the first five stories are 60/60 cm, while the exterior columns and all columns of the last three stories have 50/50 cm dimensions. Dimensions of the beams are 30/50 cm on the last three stories and 40/60 cm on the first five stories.

Twelve-story frame: The dimensions of interior columns of the first eight stories are 75/75 cm, while exterior columns and all columns of the last four stories have 60/60 cm dimensions. Dimensions of the beams are 40/65 cm on the first eight stories and 30/55 cm on the last four stories.

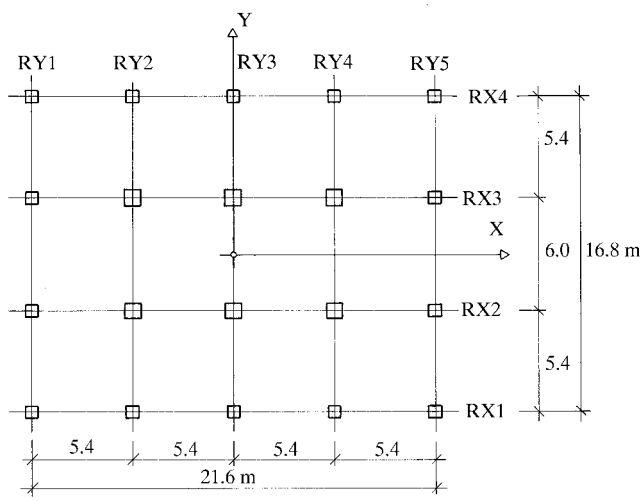

Figure 1. Floor plan of the building

The above cross-sections were adopted for the beams and columns to satisfy the minimum and maximum limits on reinforcement content imposed by Eurocode 8 [4]. All floor diaphragms are 15 cm thick R/C slabs. Story height is 3.2 m and it is the same for all stories. The compression strength of concrete is adopted f_c=25 MPa (C25/30 class according to Eurocode 2) for all elements. Yield strength of longitudinal reinforced steel is f_y=400 MPa and f_y=240 MPa for transverse reinforcement.

Some fundamental characteristics of the considered frames are given in table 1: the fundamental period, the total weight of the frame, design base shear, yield base shear and ultimate base shear. The last two characteristics of the frames are obtained by nonlinear static (Pushover) analyses of the considered frames, (Jankovic [7]). From the table 1, the largest ultimate base shear of the eight-story frame is obvious (as consequence of the largest design base shear). This is the only frame whose model included the effective width of the slab, thus increasing its stiffness.

Table 1. Characteristics of the considered frames

Frame	4-story	6-story	8-story	12-story
Period T_1 (sec)	1.024	1.208	1.702	2.579
Weight G (kN)	2852	4561	6397	10189
Design base shear (kN)	322	472	845	684
Yield base shear (kN)	396	530	853	657
Ultimate base shear (kN)	420	554	957	737

Modeling of R/C frames

Two-dimensional nonlinear dynamic analyses of four R/C frames prototypes, which were designed and dimensioned in details previously, are carried out with DRAIN-2DX model (Prakash [8]). Elements of the type 02 are used (Powell [9]), where all plastic deformations are concentrated at the element ends.

The following values are adopted for the effective flexural stiffness of the cross-sections: $I_{ef} = 0.45\ I_g$ for beams and $I_{ef} = 0.70\ I_g$ for columns (I_g is the moment of inertia of the gross concrete section). The effect of stiffness reduction due to concrete cracking and bar yielding is thus taken in consideration. More detailed analysis of the stiffness modelling issues associated with seismic response of the R/C frames can be found in a Doctoral dissertation of the first author (Jankovic [7]).

The viscous damping matrix for the frames was modelled as a linear combination of the frame stiffness and mass matrices with the coefficients chosen to give 5% of critical damping in first and third response

modes. Initial bending moments due to gravity load and P-delta effects were accounted for in the analyses. It was assumed that nonstructural components were separated from the structure and hence did not affect the stiffness and strength of the frames.

EARTHQUAKE CHOICE

A total of 20 ground motions were selected from the European strong-motion database (Ambraseys et al. [10]) to analyse the frames. These motions were characterized by surface-wave magnitudes, *M*, in the range between 6.16 and 7.04 and closest distances to the rupture surface, *R*, between 17 and 30 km. Earthquake ground motions were recorded on stiff soil or rock with shear-wave travel-time velocity V_s greater than 360 m/sec. To avoid considerations of near field effects, all recorded ground motions were examined and near-field ground motions were deliberately excluded, as these analyses focused on ordinary ground motions.

In order to mach the EC8 design spectrum, these records were scaled as a set (i.e. all earthquake records scaled by the same constant) by a factor of six. The EC8 design spectrum was constructed by adopting a 475-year return period effective peak ground acceleration of 0.36g. Without this scaling, the studied frames would respond in the elastic range under practically all of the chosen ground motions. Except these earthquakes, which have a return period of 475 years, (or analogously the earthquake which has 10% probability that it would be exceeded in the time period of 50 years – denoted with 10%/50), frame structures are tested for a seismic hazard with return period of 2475 years, that is, an earthquake which has 2% probability to be exceeded in the time period of 50 years (denoted with 2%/50). Regarding the fact that such large seismic intensity is not defined in EC8, US literature is used during the definition of adequate earthquakes: (FEMA 273 [2]) and (Cornell and Luko [11]). Accordingly, all earthquakes are scaled by the factor 12, so the final 2%/50 earthquake median is doubled in relation to the 10%/50 earthquakes median.

Medians of the spectra of 20 scaled ground motions for two levels of seismic risk (2%/50 and 10%/50) are shown in fig. 2, together with the EC8 design spectrum scaled to the 10%/50 intensity. Spectra of the scaled 10%/50 ground motions are drawn using thin grey lines. It is evident that, although earthquakes were chosen with narrow limits for *M* and *R*, their spectra are very different.

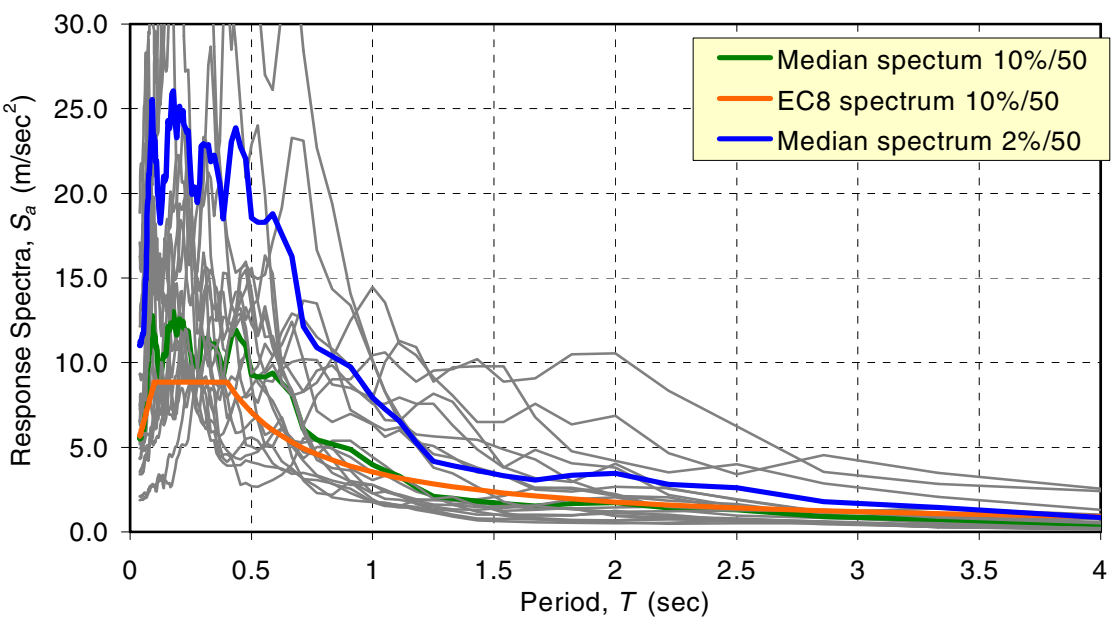

Figure 2. Response spectra of the selected ground motions

SELECTION OF GROUND MOTION INTENSITY MEASURES AND ENGINEERING DEMAND PARAMETERS

Ground motion intensity measures

As it was previously mentioned, a very important question in probabilistic evaluation of the seismic response of the structure is the selection of ground motion intensity measure *IM*. Ideally, an appropriate *IM* should be capable of capturing the amplitude, frequency content and duration properties of ground motions that significantly affect the elastic and inelastic response of the structure. Unfortunately, at this time there is no single parameter suitable for this purpose. A choice of *IM* depends on: performance objectives, engineering demand parameter, structural system and existence of the attenuation relation between the intensity measure and seismological-geological parameters (magnitude, source-to-site distance, faulting style and soil type).

The following *IMs* are analysed in this paper:

- Peak ground acceleration, *PGA*
- Peak ground velocity, *PGV*
- Effective peak acceleration, *EPA*
- Cumulative absolute velocity, *CAV*
- Acceleration response spectrum, S_a
- Velocity response spectrum, S_v
- Cordova's measure, *Cord*

The most common measure of seismic intensity today is **peak ground acceleration *PGA***. This measure has a natural connection with inertial forces and for specific types of structures (very stiff structures) maximum dynamic force, which occurs in the structure, is directly related to *PGA*. **Peak ground velocity *PGV*** is a useful *IM* because it measures the energy input to the structure, which can be related to damage and energy dissipation. For the structures which are in the velocity-sensitive period range this measure may be better related to structural damage than *PGA*. Maximum amplitude parameters, such as *PGA* and *PGV*, do not contain any information about the frequent contents and earthquake duration, which also influence the structure behaviour. Thus, it is necessary to consider other measures to more accurately describe the earthquake intensity.

Applied Technology Council proposed in 1978 the **effective peak acceleration *EPA*** as the intensity measure. *EPA* is defined as the average spectral acceleration (for 5% damping) over the period range from 0.1 to 0.5 seconds divided by 2.5. *EPA* has tendency to be approximately equal to *PGA* for earthquakes with moderate-to-high magnitude of medium-to-long source-to-site distances, while for near field earthquakes with low magnitude, *EPA* is smaller than *PGA*. In this way, influences of local peaks in the spectrum are minimized. *EPA* is used as the seismic intensity measure in numerous seismic codes, including EC8 [4].

The measure **cumulative absolute velocity *CAV***, which includes yet more earthquake characteristic, was also considered in this paper. *CAV* is defined with:

$$CAV = \int_0^{T_d} |a(t)| dt \tag{2}$$

where T_d is the duration of the strong motion and $a(t)$ is the acceleration time history.

Spectral acceleration S_a and **the spectral velocity S_v** describe the maximum response of a single-degree-of-freedom (*SDOF*) system to a particular input motion as a function of the natural period and damping

ratio of the *SDOF* system. Spectrum values indirectly, i.e. over the response of the SDOF system, reflect strong ground motion characteristics. Spectral values depend on amplitude, frequency content, and damping, and to a lesser extent, on earthquake duration. S_a is the intensity measure used in new design documents (FEMA [2]). For all analysed R/C frame structures in this paper, the spectra were computed using a viscous damping ratio of 5%.

Finally, an intensity measure proposed by **Cordova** (Cordova et al. [13]), which was proved to be very appropriate for steel and composite moment frames with moderate first mode periods, was taken into account. This measure is noted as *Cord* and is defined as follows:

$$Cord = S_a(T_1) \cdot \left[S_a(2.0T_1)/S_a(T_1)\right]^{0.5} \tag{3}$$

Chosen *IMs* can be divided in two groups: intensity measures which only depend on the characteristics of the earthquake (*PGA*, *PGV*, *EPA* and *CAV*) and intensity measures which also depend on the characteristics of the structure (S_a, S_v and *Cord*).

Engineering demand parameters

To investigate the *PSDM*, seismic responses of R/C frames are shown using four engineering demand parameters, which are grouped in two categories:

- Parameters at the story level: maximum of all interstory drift ratios (largest story drift divided by story height) (IDR_{max}), parameter, which is used in the literature as an indicator for collapse; average of the maximum interstory drift ratios in each story (IDR_{ave}), which may be related to nonstructural damage; and average of the maximum story ductilities in each story (μ_{ave}), an indicator of structural damage.
- Parameters at the global (frame) level: maximum frame drift ratio (largest frame displacement divided by frame height) (DR_F)

Local parameters at the element level such as plastic hinge rotation, cumulative plastic hinge rotation and damage index, which were analysed in (Jankovic [7]), are not presented here. In general, these parameters produced less efficient *PSDMs*.

PROBABILSTIC SEISMIC DEMAND MODELS

Efficiency of *PSDM*

Probabilistic seismic demand model is assumed in the following form:

$$EDP = a \cdot IM^{\,b} \tag{4}$$

This kind of model makes it possible to compute the total probability integral (equation (1)) in closed form and to rearrange the entire probabilistic framework into "load and resistance factor" format, which is more suitable for engineering practice. Otherwise, the integral defined by equation (1) has to be solved by numerical integration. Secondly, such demand model in log-log plot is a straight line providing easier regression analyses of the results of nonlinear dynamic analyses.

In this paper *PSDMs* are obtained by the regression analyses of the results of nonlinear dynamic analysis of the considered frames under the 40 earthquakes (20 earthquakes of the intensity 10%/50 and 20 earthquakes of the intensity 2%/50). Results from nonlinear dynamic analyses for R/C frames with different number of stories are shown in the figures 3 and 4. These *PSDMs* are presented using S_a (as intensity measure) and using two engineering demand parameters: IDR_{max} and IDR_{ave}. *PSDMs* with *EPA* (as measure intensity) and μ_{ave} and DR_F as EDP's are shown in figures 5 and 6.

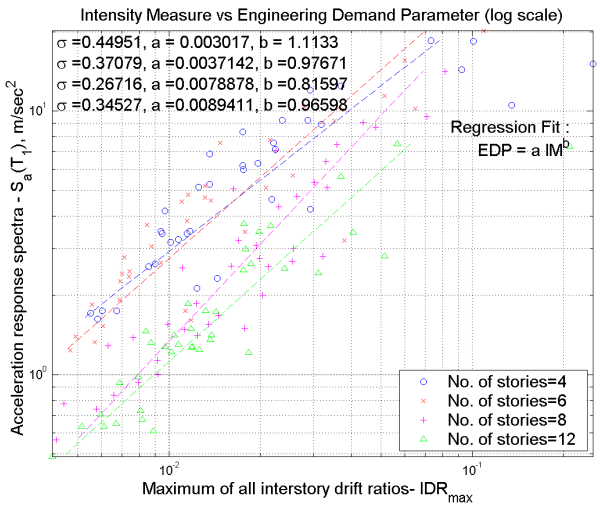

Figure 3. *PSDM* pairing IDR_{max} with S_a

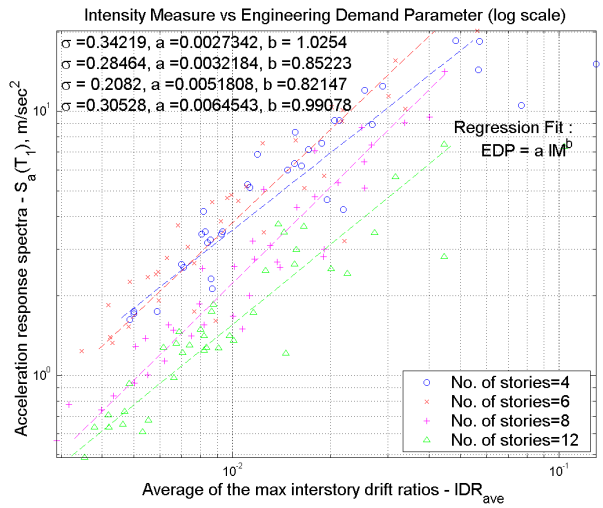

Figure 4. *PSDM* pairing IDR_{ave} with S_a

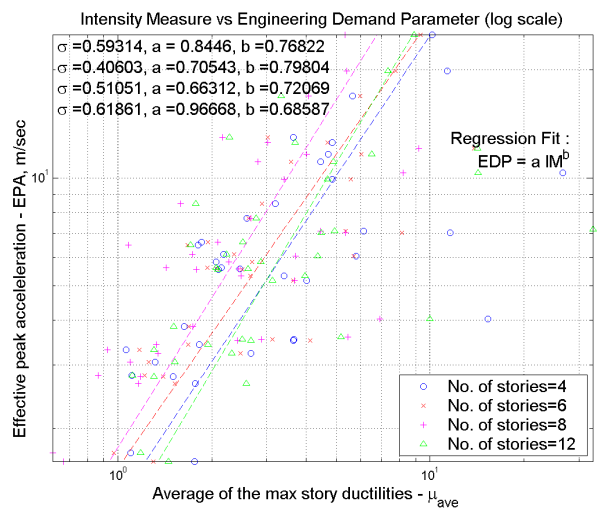

Figure 5. *PSDM* pairing μ_{ave} with *EPA*

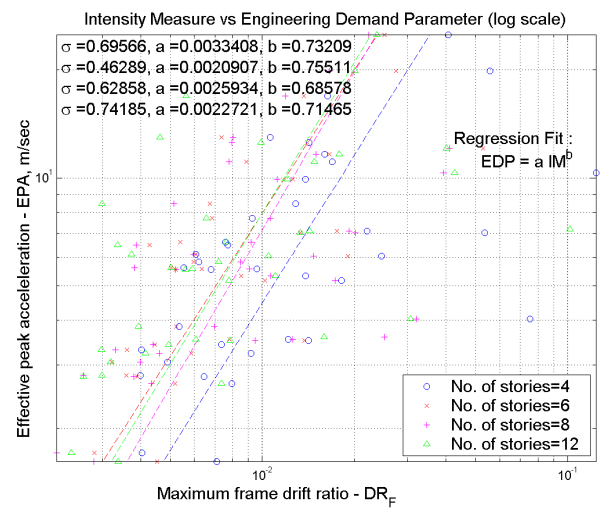

Figure 6. *PSDM* pairing DR_F with *EPA*

Efficiency of *PSDM* is measured by the degree of scatter, i.e. by the dispersion of the obtained *EDPs* with respect to the regression fit line for the given value *IM*. Less dispersion of the results means less number of nonlinear dynamic analyses (fewer numbers of earthquakes) are necessary to estimate the seismic response of the structure with adequate precision. It is easy to show that for the given acceptable response confidence band width, e.g. ±10%, reducing dispersion by n times reduces the necessary number of earthquakes by n^2 times (Shome [5]). In this paper, standard deviation of the natural logarithms of data is used as a measure of dispersion with respect to the *PSDM* given by equation (4). Such dispersion is calculated as the square root of the mean squared deviation of the data points from the fitted line. If the data is lognormally distributed (the assumption of lognormal structural demand distribution for a given intensity measure was confirmed in (Shome [5]) σ is the natural measure of dispersion. For values less than 0.3, it is approximately equal to the coefficient of variation, *COV* (i.e. standard deviation divided by the mean). Dispersion σ as well as the values of coefficients a and b which figure in equation (4) are given in all above-mentioned figures.

Dispersion for all chosen demand parameters *EDP* and intensity measures *IM* are presented in the table 2.

Table 2. Dispersion for all chosen demand parameters *EDP* and intensity measure *IM*

Engineering demand parameters, *EDP*	Frame	Intensity measure, *IM*						
		Ground motion specific *IM*				Structure specific *IM*		
		PGA	*PGV*	*EPA*	*CAV*	S_a	S_v	*Cord*
IDR_{max}	4 story	0.773 mr	0.512 mr	0.716 mr	0.469 r	0.450	0.435	0.379
	6 story	0.636 mr	0.321 r	0.544 mr	0.384 r	0.371	0.286	0.312
	8 story	0.508 mr	0.324	0.529 mr	0.308	0.267	0.239	0.275
	12 story	0.677 mr	0.421	0.638 mr	0.451	0.345	0.321	0.323
IDR_{ave}	4 story	0.679 mr	0.428 mr	0.924 mr	0.388 r	0.342	0.336	0.290
	6 story	0.525 mr	0.242 mr	0.443 mr	0.288 r	0.285	0.201	0.243
	8 story	0.576 mr	0.295	0.524 mr	0.285	0.208	0.187	0.204
	12 story	0.662 mr	0.388	0.620 mr	0.419	0.305	0.275	0.270
μ_{ave}	4 story	0.649 mr	0.398 mr	0.593 mr	0.364 r	0.331	0.317	0.276
	6 story	0.488 mr	0.215 mr	0.406 mr	0.262 r	0.291	0.201	0.249
	8 story	0.562 mr	0.293	0.511 mr	0.280	0.207	0.184	0.204
	12 story	0.662 mr	0.386	0.619 mr	0.417	0.314	0.277	0.279
DR_F	4 story	0.746 mr	0.515 mr	0.696 mr	0.452 r	0.394	0.408	0.332
	6 story	0.539 mr	0.295 mr	0.463 mr	0.298 r	0.299	0.241	0.245
	8 story	0.673 mr	0.407	0.629 mr	0.364	0.237	0.273	0.245
	12 story	0.786 mr	0.495	0.742 mr	0.526	0.351	0.344	0.289

On the basis of the values of dispersions, given in the table 2, the following conclusions can be drawn:

- Dispersion of the results was larger for intensity measures which do not depend on characteristics of the structure (ground motion specific *IM*).
- Today, the most frequently used intensity measures *PGA* and *EPA* had, for all demand parameters, the largest dispersion, which disqualifies them as *IMs* for this type of structure. *EPA* shows somewhat better efficiency then *PGA*.
- Dispersion for *PGV* was approximately two times smaller than dispersion for *PGA*, which means that for a given confidence bandwidth use of *PGV* as the intensity measure the necessary number of earthquakes would be four times smaller.
- *IMs*, which depend on characteristics of the structure, were more successful. A complex intensity measure *Cord* gave better results than S_a, but this increase of efficiency was not worth the effort. Intensity measure S_v was also more efficient than S_a.
- Intensity measures related to velocity (*PGV*, *CAV* and S_v) had noticeable less dispersion than those related to acceleration (*PGA*, *EPA* and S_a). This can be explained by the fact that all analysed R/C frames had periods in the velocity-sensitive domain of the spectrum (Chopra [14]).
- From the table 2, it is also noticeable that the average values of *EDPs* (IDR_{ave}, μ_{ave}) had less dispersion then the maximum values (IDR_{max}, DR_F). This means that the estimation of the structural and nonstructural damage in R/C frame structures (with the help of parameters IDR_{ave}, μ_{ave}) will be carried out with larger degree of confidence than the estimation of collapse and structural damaged (parameters IDR_{max}, DR_F).
- Dispersion for seismic responses of the medium-rise R/C frames (6 and 8 stories) was less than the dispersion for low–rise R/C frames (4 stories) and for high–rise R/C frames (12 stories) for all intensity measures.

Concluding the consideration of *PSDM* efficiency, it should be said that the *PSDM*s presented in this paper and given by equation (4), are correlations between the chosen *EDP* and only one *IM*. It is possible to investigate how seismic response depends on two or more *IMs* simultaneously. In that case, one should extend the equation (4) by including both *IMs*:

$$EDP = a \cdot IM_1^{b_1} \cdot IM_2^{b_2} \tag{5}$$

In this way, it is possible to obtain a more efficient *PSDM* i.e. less dispersion of the seismic response. After carrying out the linear regression in log-log space coefficients a, b_1 and b_2 can be computed together with the dispersion of the seismic responses. Preliminary calculation confirmed that such *PSDM* have smaller dispersions. The problem in using such *PSDMs* in probabilistic analysis of seismic behaviour is that attenuation dependence on earthquake characteristics (*M*, *R*, faulting style and soil type) must be established for all *IMs* used in analyses.

Sufficiency of *IM*

As it was previously mentioned, a *PSDM* is sufficient if the seismic response *EDP*, for the given *IM*, is independent of earthquake magnitude *M* and source-to-site distance *R*. Sufficiency depends on *IM*, structure type, the chosen *EDP* and seismic characteristics of earthquakes (*M* and *R*) and it has fundamental significance for scaling of the ground motion records. Namely, suppose that there are defined *M* and *R* of the "maximum probable" earthquake, which presents the most severe threat for the given structure and suppose that *IM* is defined according to that earthquake by attenuation relation. There is a concern that, if ground motion records with smaller *IM* (i.e. with smaller *M* and/or larger *R*) scaled to match *IM* of the "maximum probable" earthquake will produce different structural responses than that of "maximum probable" earthquake. In the case of *PSDM* sufficiency, structure responses under the scaled and unscaled records are approximately the same and it could be said that the process of scaling is justified.

Also, in the case of sufficient *PSDM*, the probability $\lambda[LS]$, which is given by the equation (1), will be correct no matter which earthquakes are used to estimate $P[EDP|IM]$. In other words, in that case the precise selection of the earthquakes records is not so important, i.e. the structure response will be well estimated, regardless of *M* and *R* characteristics of the chosen earthquakes.

Whereas the efficiency of *PSDM* is gauged by the scatter about the regression (of *EDP* on *IM*) fit, sufficiency is evaluated by the extent to which, after regressing on *IM*, the residuals of *EDPs* are statically independent of *M* and *R*. Residuals of *EDPs* are “horizontal” distances between observed value of EDP_i and its estimate $EDP_i = a\ IM_i^b$ (see figures 3 to 6). Sufficiency is quantified by the *p*-value for the *c* estimate. *C* is slope of regression line of residuals of *EDPs* on *M* or *R* (Luco and Cornell [12]). *P*-value is defined as the probability of finding an estimate of *c* at least as large (in absolute value) as that observed if, in fact, the true value of *c* is 0. 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. For all *EDPs* and *IMs* sufficiency was investigated and in Table 2 all insufficient cases were denoted with *m* and *r*, referring to dependence of *M* or *R*.

For illustration, the regressions of residuals for the (IDR_{max}, *Cord*) *PSDM*, with respect to *M* and *R* are shown in figures 7 and 8 respectively, while the regressions of residuals for (IDR_{max}, *PGA*) *PSDM* with respect to *M* and *R* are shown in figures 9 and 10, respectively. The analysis is done on the R/C frames with different number of stories. It is obvious from figures 7 and 8 that for the *Cord* IM, the slope of regression line *c* is not statistically significantly different from 0 ($p > 0.05$) in relation to *M* and *R*, which means that intensity measure *Cord* is sufficient. On the other hand *PGA* as intensity measure was insufficient at all frames, because the value *p* was less than 0.05 (slope of regression line *c* differed significantly from 0). This means that usage of, for example, an *M*=5 record scaled to match the *PGA* of

"maximum probable" earthquake with a magnitude *M*=7, will produce a different seismic structural response in relation to the "maximum probable" earthquake.

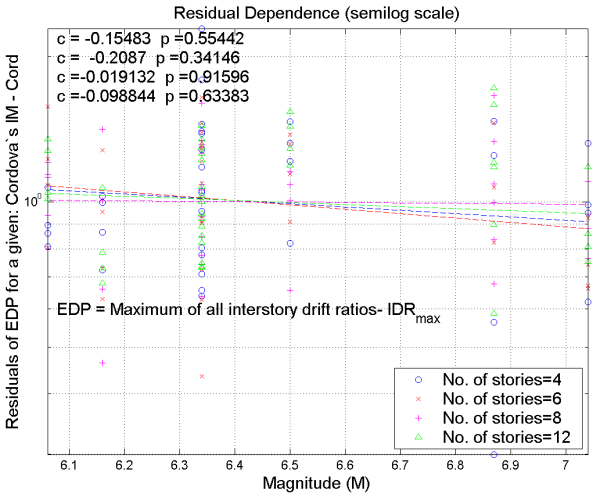

Figure 7. Regression of residuals IDR_{max}, given *Cord*, with respect to *M*

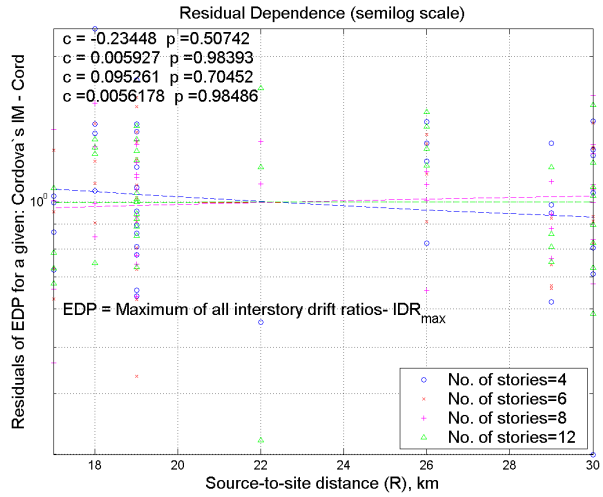

Figure 8. Regression of residuals IDR_{max}, given *Cord*, with respect to *R*

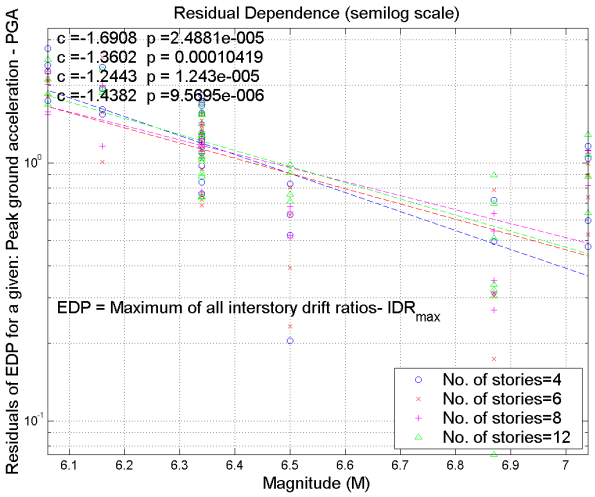

Figure 9. Regression of residuals IDR_{max}, given *PGA*, with respect to *M*

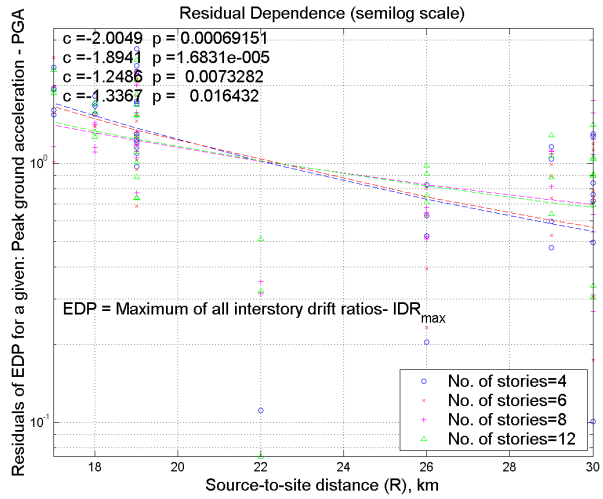

Figure 10. Regression of residuals IDR_{max}, given *PGA*, with respect to *R*

Regarding the definition of sufficient *PSDM*, it should be noted that the conditional independence (given *IM*) of *EDP* should be investigated not just from *M* and *R* (as it is done in this paper) but also from all influential factors. For example, instead of proving conditional independence of seismic response with respect to the soil type, earthquake records can be chosen such that they are recorded on the same soil type as the soil type is on the designated site. This principle is respected in this paper: the R/C frames, whose behaviour is analysed, were founded on rock or on firm ground.

On the basis of the obtained results, which are presented in the table 2 and in the figures 7-10, the following conclusions about the sufficiency of *PSDM*, can be drawn:

- For all considered R/C frames and for all *EDPs*, intensity measures *PGA* and *EPA* were insufficient with respect to magnitude *M* and source-to-site distance *R*. Intensity measures *PGV* and *CAV*, which also do not depend on the structure characteristics, were sufficient for high–rise R/C frames (8 and 12 stories). It is interesting to note that seismic response of low-rise R/C frames (4 and 6 stories), given *CAV*, was dependent only on source-to-site distance *R*, but not on the magnitude *M*.
- All *IMs* which depend on the characteristics of structure (S_a, S_v, *Cord*), were sufficient with respect to both *M* and *R*.

CONCLUSIONS

Probabilistic seismic demand models for *R/C* frames with different number of story designed according to the Eurocode 8 [4] were presented in this paper. The *sufficiency* i.e. whether the *EDP*, conditioned on *IM*, is independent of earthquake magnitude, *M* and source-to-site distance, *R* and *efficiency* i.e. smaller dispersion of *EDP* given *IM*, of several probabilistic seismic demand models were considered. The use of sufficient and efficient *PSDM*s improves the accuracy of probabilistic seismic assessment methods and reduces the number of records necessary to reach the given confidence level in the assessment conclusion.

Peak ground acceleration, *PGA* and effective peak acceleration EPA (probably the most common intensity measures today), were both insufficient and inefficient with respect to all considered *EDPs*. Thus, *PGA* and *EPA* should not be used as an intensity measures for R/C frame type of structures. Among all considered intensity measures, which do not depend on the characteristics of structure and which can be obtained simply from the earthquake records, the most efficient one for R/C frame structures is *PGV*. Its usage reduces the number of earthquakes in relation to the measures *PGA* and *EPA* by approximately four times. However, if *PGV* is used, one should pay attention to the fact that this measure is not sufficient for low-rise R/C frames (4 and 6 stories).

In distinction from measures that do not depend on structure characteristics (*PGA*, *EPA*, *PGV* and *CAV*), the measures which depend on the characteristics of structure (S_a, S_v, *Cord*) were all efficient. These *IMs* (S_a, S_v, *Cord*) were, also, sufficient. Therefore, to get reliable seismic response estimates for R/C frames, it is imperative to use these intensity measures to form probabilistic seismic demand models.

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