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DEMAND HAZARD ANALYSIS: A TOOL FOR ASSESSING THE ADEQUACY OF CAPACITY DESIGN APPROACH FOR DEFINITION OF THE DESIGN SHEAR FORCES IN THE FRAME BUILDINGS

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Abstract. *The risk-based design of buildings using nonlinear analysis is a challenging task. A question how to design components for non-simulated failure still remains unsolved. The simplified possibility is to use the capacity design which prevents shear failure of the components according to deterministic approach. In this paper an attempt has been made to obtain additional insight regarding this issue. For this purpose, the demand hazard analysis was performed for an example of an eight-storey reinforced concrete building. The ground motions were selected to match different target spectra. In addition to the uniform hazard spectrum, the conditional spectra conditioned to the first, second and third vibration periods were used. It was found that the return period of the design shear force according to capacity design vary significantly depending on location of the column as well as on the set of ground motions. However, for the investigated building it can be concluded that conditional spectrum with consideration of the first period of structure can be sufficient for determination of design shear forces.*

1 INTRODUCTION

No-collapse requirement is a fundamental performance objective of current standards for earthquake-resistant design of ordinary buildings which has to be met with an adequate reliability. Although the significant damage is allowed in the case of this fundamental performance objective, a structure is almost always designed by the linear elastic response analysis. Consequently the dissipation of energy due to damage of a structure is only implicitly accounted in the design. Several assumptions have to be introduced when a structure is designed by utilizing linear elastic response analysis. For example, the global plastic mechanism is assumed on the basis of capacity design approach, which, in the case of frame buildings, involves the concept of strong column – weak beam. Additionally, the objective of the capacity design is to prevent shear failure of the columns and beams. However, it is well known that many different global plastic mechanisms can occur in frame buildings due to earthquakes although they are designed according to the capacity design approach. Some of them can be similar to that considered by the capacity design, whereas some other can be significantly different. Therefore it can be argued that the capacity design does not always guaranty global plastic mechanism which is considered in design. To overcome this issue the alternative methods of designing the structures can be used. For example, Lazar Sinković et. all [1] and Lazar and Dolšek [2] recently proposed an iterative design procedure which makes it possible to use different type of analyses (including non-linear), and to control the adjustment of structure during an iterative design. The iterative procedure is currently used for checking the adequacy of collapse risk, whereas other performance objectives are considered according to building codes. Although the proposed procedure solves a wide range of issues (e.g. incompleteness of capacity design procedure, the design of structure to target collapse risk), the problem of design against shear failure is still unsolved if the non-linear model is not capable of simulation of shear failure of the structural components.

The aim of the study presented in this paper was to investigate if the design shear forces in the columns, which are obtained by capacity design, can be justified in terms of uniformity of mean annual frequency of exceedance. For this purpose demand hazard analysis [3–5] was performed for an example of an 8-storey building, which was analyzed for one direction of seismic action. Different sets of ground motions were selected using conditional spectrum (CS) approach [6] in order to study whether the conditioning period has an impact on the return period of shear forces. The three conditioning periods corresponded to the first three periods of the investigated 8-storey building. For comparison, the uniform hazard spectrum was also used as a target spectrum for the selection of a set of ground motions. The results are presented in terms of so-called demand hazard curves for the shear forces in selected columns. A discussion regarding the variability of return periods corresponding to design shear forces in the columns as well as the discussion about the relationship between so-called maximum credible design shear force and design shear force based on capacity design is also given. Finally the impact of the conditioning period of CS on the demand hazard curves for shear forces in the columns is presented.

2 THE SEISMIC DEMAND HAZARD

A conventional risk integral can be used to determine the so-called seismic demand hazard, which provides relationship between mean annual frequency of exceedance of certain engineering demand parameter:

$$\lambda_{EDP}\left(edp\right)=\int_{0}^{\infty} P\left[EDP>edp \mid IM=im\right]\left|\frac{d\lambda_{IM}\left(im\right)}{dIM}\right|dIM \tag{1}$$

where $P\left[EDP > edp \mid IM = im\right]$ is the probability that *EDP* is greater than *edp* with given *IM=im*, and the $\lambda_{IM}(im)$ is the seismic hazard curve which provides the rate of exceedance of *IM>im*. Integrating the equation for intensities [0, ∞) mean annual frequency of exceedance is computed for only one *edp*. The entire seismic demand hazard curve is therefore obtained by evaluated the Eq. (1) for entire range of *edp*. More details about demand hazard and the corresponding limitations are given elsewhere (e.g. [7,8]).

The demand hazard curve is obtained by numerical integration of the Eq.(1). In the case if it is assumed that the probability P_i, which represent the probability that *EDP* is greater than *edp* at given intensity $IM=im_i$, is constant in the range $[im_i-\Delta im/2 \;\; im_i+\Delta im/2]$ and the trapezoidal rule on integration of derivate of hazard function is used, the Eq.(1) can be rewritten as follows:

$$\lambda_{EDP}(edp) \approx \sum_{i=1}^{n} P_i\left[EDP > edp \mid IM = im_i\right] \cdot \left| \frac{\lambda_{IM}(im_i + \Delta im) - \lambda_{IM}(im_i - \Delta im)}{2} \right| \quad (2)$$

The Δim is defined as difference between intensities im_{i+1} and im_i. The probability P_i could be calculated directly from complementary cumulative distribution function obtained on the basis of results of dynamic analysis. In such a case the type of distribution (usually lognormal) has to be assumed and the distribution parameters μ and σ which are obtained from *EDP*s at intensity $IM=im_i$ has to be calculated. However, in case of this study the probability P_i was estimated as the fraction of ground motions for which it was observed that at given intensity $IM=im_i$ the *EDP>edp*. For example, if 30 ground motions were used for calculation of *EDP*s and 4 of it exceed *edp*, then the probability P_i was equal to 4/30.

In this study, the engineering demand parameter (*EDP*) is the shear force in columns *Q*, which is presented as a function of the return period ($T_R = 1/\lambda_Q$) and not as a function of mean annual frequency of exceedance of shear force.

3 DESCRIPTION OF THE INVESTIGATED STRUCTURE

The geometry of the 8-storey building is presented in Figure 1. The building was designed according to provisions of Eurocode 8 [9] for ductility class M, peak ground acceleration at rock outcrop equal to 0.25 g (Ljubljana, Slovenia) and soil type C (soil factor 1.15). The behavior factor was assumed 3.9, the quality of reinforcing steel was prescribed S500B whereas the concrete C30/37 was used. The slab depth was 20 cm. The total mass of structure amounted to 2338 t. The first three periods in X direction were, respectively, 1.23 s, 0.40 s and 0.23 s. The ratio between the design base shear and the weight was estimated to 7.6% and 7.3% for X and Y direction, respectively. The typical reinforcement of columns and beams are presented in Figure 1.

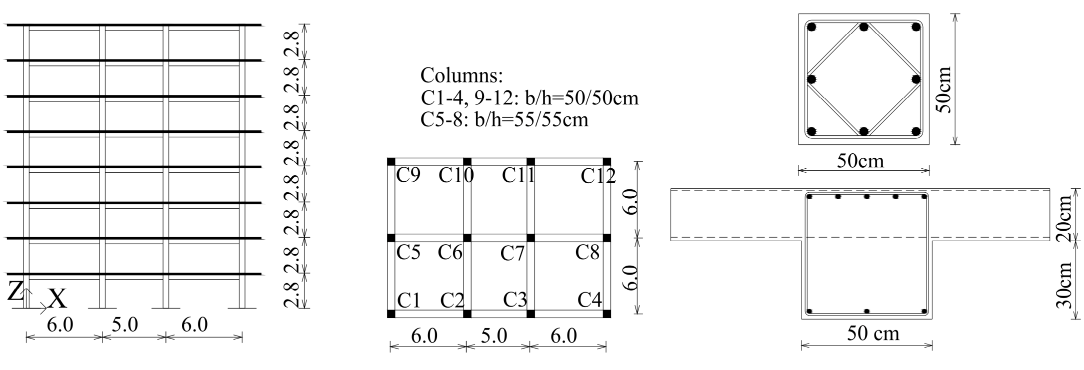

Figure 1: The elevation, plan view and reinforcement in typical columns and beams of 8-storey building

4 SEISMIC HAZARD AND THE SETS OF GROUND MOTIONS FOR DYNAMIC ANALYSIS

The seismic hazard curves for spectral acceleration at the first three vibration periods of the investigating building when analysed only in X direction (Figure 2) were obtained on the basis of probabilistic seismic hazard analysis (PSHA) which was performed for the purpose of seismic hazard maps used in Slovenia [10,11]. The PSHA based on so-called distributed (background) seismicity since the seismicity could not be assigned to specific geologic structures due to inaccurate and insufficient seismologic and geologic data. Five different seismicity models which were appropriately weighted were considered in the analysis in order to adequately take into account distribution of hypocentres and energy released during earthquakes.

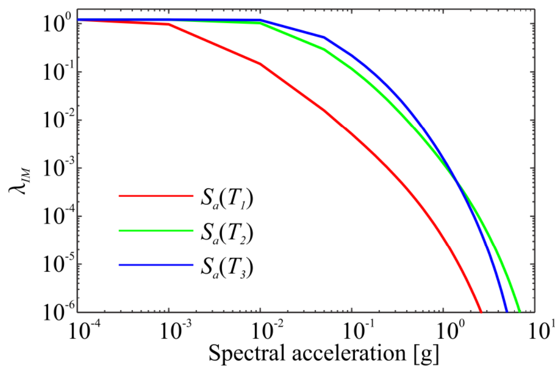

Figure 2: The hazard curves for Ljubljana for the spectral acceleration corresponding to the first, second and third period of the investigated building.

The relationship between engineering demand parameters and seismic intensity, which is needed to calculate seismic demand hazard, was simulated by incremental dynamical analysis [12]. Four sets of 30 ground motions (Figure 3) were selected from PEER Next Generation Attenuation (NGA) strong-motion database [13,14]. Ground motions of the first three sets were selected according to conditional spectrum approach [6]. The conditioning periods corresponded to the first three periods (hereinafter referred as set CS T_1, CS T_2 and CS T_3). For comparison, the fourth set of ground motions was also selected. It is consistent with the uniform hazard spectrum (hereinafter referred as set UHS). In this case the spectral acceleration corresponding to the first vibration period was selected for the intensity measure. All target spectra corresponded to return period of 475 years which defined in Eurocode 8 for design of ordinary buildings. The target conditional mean spectra were defined using Sabetta and Pugliese [15] attenuation relationship which was also used in the PSHA. The mean magnitude (M), mean distance (R) and the epsilon (ε) are presented in Table 1. All selected ground motions were recorded on soil type C (Eurocode 8, shear-wave velocity in the upper 30 m from 180 to 360 m/s), and corresponded to events with magnitudes between 4.0 and 7 and source-to-site distances in the range from 4.5 to 50 km.

	CS T_1	CS T_2	CS T_3
R [km]	10.2	8.4	6.9
M	6.0	5.8	5.6
ε	0.78	0.89	0.94

Table 1: The mean distance, mean magnitude and epsilon used for definition of conditional spectra.

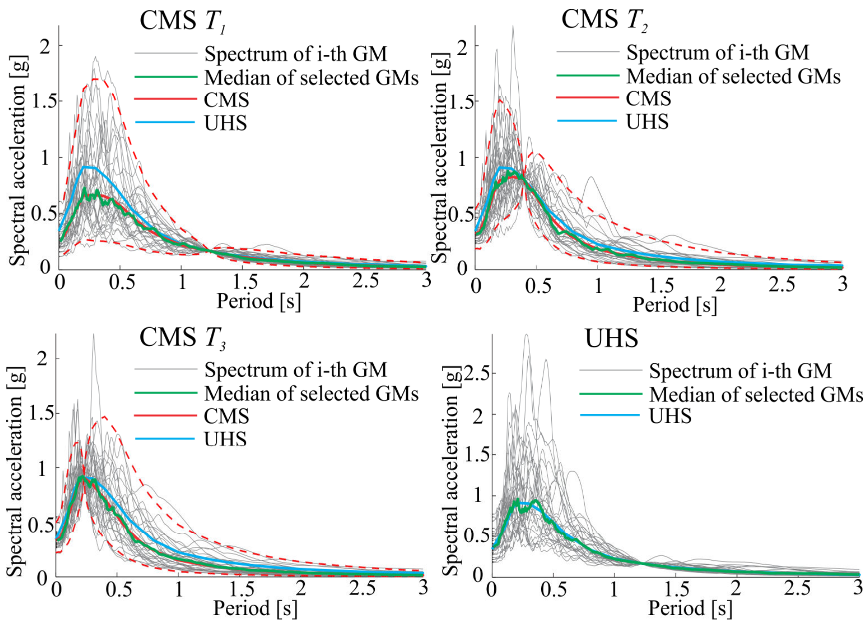

Figure 3: Spectra of selected ground motions (GM) corresponding median spectrum and the of target spectrum for the four sets of ground motions.

5 STRUCTURAL MODEL

The structural model for nonlinear analysis is based on the Eurocode 8 requirements as discussed elsewhere [16]. The beam and column flexural behavior is therefore modelled by one-component lumped plasticity elements, composed of an elastic beam and two inelastic rotational hinges (defined by a moment-rotation relationship). The element formulation is based on the assumption of an inflexion point at the midpoint of the element. The gravity load is represented by the uniformly distributed load on the beams, and by the concentrated loads at the top of the columns. For the beams, the plastic hinge is used for major axis bending only. For the columns, two independent plastic hinges for bending about the two principal axes are used. The moment-rotation relationship before strength deterioration is modelled by a bilinear relationship. A linear negative post-capping stiffness is assumed after the maximum moment has been achieved. The axial force due to gravity loads is taken into account when determining the moment-rotation relationship for the columns, while in the case of the beams zero axial force and the T cross-sections were assumed. The ultimate rotation Θ_u in the columns and beams at the near collapse (NC) limit state, which corresponds to a 20% reduction in the maximum moment, is estimated by means the EC8-3 formulas [17]. The parameter γ_{el} was assumed to be equal to 1.0, since mean values of near collapse rotation were used. Mean (actual) concrete (38 MPa) and steel (570 MPa) strength were assumed. Post-capping negative stiffness is calculated by assuming the ratio between the rotation at zero strength and the rotation corresponding to the maximum moment equal to 3.5.

All the analyses were performed with OpenSees [18], using the PBEE toolbox [16], which is a simple yet effective tool for the seismic performance assessment of reinforced concrete.

6 THE DEMAND HAZARD CURVES AND SHEAR DEMAND IN COLUMN ACCORDING TO CAPACITY DESIGN

The return period of design shear forces (Q_{CD}) in columns, which were estimated according to capacity design, are presented on the basis of demand hazard analysis for four sets of ground motion (Table 2). Furthermore the influence of different target spectrum on the demand hazard curves for the shear forces in the columns are also presented. For simplicity, the results are presented only for columns C5 and C6 (Figure 4 and 5).

In the most cases the lowest return periods for Q_{CD} corresponded to the ground motions consistent with the UHS, follows the set CS T_1, CS T_2 and CS T_3. It is interesting to note, that the return periods for inner column C6 are greater than that observed for the column C5, which is positioned in the peripheral frame. The exception is the first storey. The return periods for Q_{CD} are the lowest in the 6-th and 7-th storey (271-480 years and 363-834 years, respectively, in the case of columns C5 and C6) and the greatest in last 8-th storey (3310-10100 years in case of column C5 and 4910-24400 for column C6). Note also, that the return period for Q_{CD} in storey one is greater than the return period for Q_{CD} in storey two in case of both columns.

			T_R [years]			
	Storey	Q_{CD} [kN]	*CS* T_1	*CS* T_2	*CS* T_3	*UHS*
C5	1	371	3600	5430	8410	2120
	2	237	532	605	512	396
	3	247	1060	983	1130	804
	4	236	885	898	856	782
	5	216	609	585	667	442
	6	206	426	375	338	271
	7	193	480	371	336	271
	8	234	6900	8240	10100	3110
C6	1	451	1240	1410	1560	681
	2	369	987	1390	1460	624
	3	379	1910	2330	2450	1300
	4	327	1230	1220	1200	849
	5	307	1,020	832	1,390	797
	6	299	718	763	834	431
	7	268	602	546	537	363
	8	293	9110	10100	24400	4910

Table 2: The return periods for the design shear forces in columns C5 and C6 over stories for different sets of ground motions.

However, it was realized that it may be misleading to judge the adequacy of the design shear forces on the basis of uniform return period of demand. For example, let’s compare the results for column C6 in 3-th and 6-th storey. The return period of design shear force in 3-rd storey amounted from 1300 to 2450 years whereas in the 6-th storey the corresponding return periods are around three times lower (431-834 years). Although the design shear force is more frequent in the 6-th storey it cannot be concluded that the column in the 6-th storey is more sensitive to shear failure than the column in the 3-rd storey.

The return period of the design forces in the columns may not be sufficient parameter in order to decide whether the occurrence rate of shear failure of the columns is acceptable or not. Several other issues may affect the decision. Firstly, the return period presented in Table 2 corresponds to the design shear forces which are significantly smaller in comparison to the median shear resistance. Thus the return period of shear resistance of the columns is certainly significantly larger. Secondly, the return period is very sensitive to the variation of the shear force and the shape of demand hazard curve is slightly different for each column. Therefore, the ratio between actual shear strength and design shear force can be different in comparison to the ratio between the corresponding return periods. It is also worth to note that the nonlinear model, which is used in dynamic analysis, is based on the median strength of the material, whereas the design shear forces are used to estimate the amount of shear reinforcement taking into account the design value of the material strength. Consequently the decision regarding an acceptable occurrence rate of shear failure cannot be solely based on the occurrence rate of design forces.

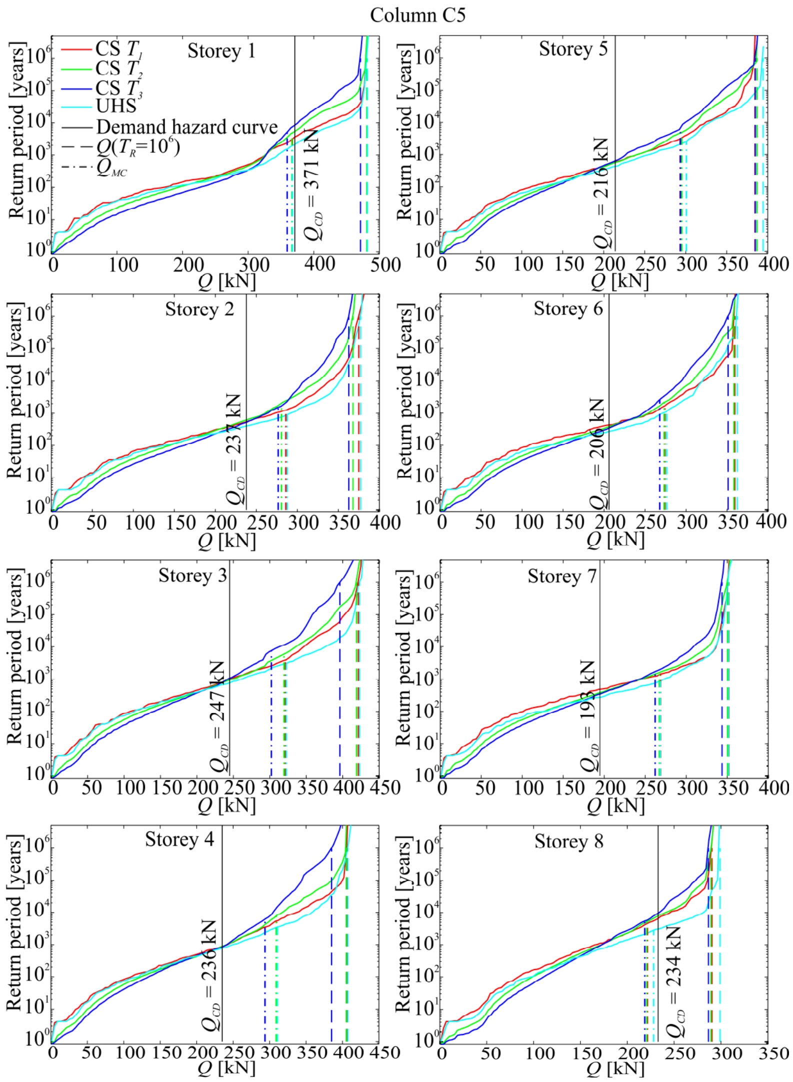

Figure 4: The demand hazard curves for column C5

In order to provide more insight into the adequacy of design shear forces in structural components it is quite reasonable to estimate the so-called maximum credible design shear force (Q_{MC}). Different definitions can be used for Q_{MC}. In this study it was simply assumed that Q_{MC} (Figures 4 and 5) is the shear force in the component which has very long return period, e.g. $Q(T_R=10^6)$, divided by the overstrength factor, which takes into account the difference between the median value of shear capacity and design value of shear capacity of a structural component. If it is assumed that the contribution of the shear reinforcement is a dominant part to the shear resistance of the structural components (e.g. Eurocode 2), then the overstrength factor depends on the material safety factor used for the strength of steel (γ_S=1.15) and the ratio between median strength and the characteristic strength of steel, which was estimate from experiments (1.14). The total overstrength factor was thus estimated to 1.31 (1.14·1.15). The actual overstrength is even higher since the contribution of concrete and diagonal strut to the shear strength, was neglected. The minimum requirement of standards Eurocode 2 and 8 [17,19] can additionally increase the overstrength. The Q_{MC} was then estimates as $Q(T_R=10^6)/1.31$. In this procedure, it is assumed that when Q_{MC} is based on $Q(T_R=10^6)$ the shear failure of the components, if simulated in the nonlinear model, do not have an impact on the probability of the failure of the system. Note that such an assumption may be even too conservative since the $Q(T_R=10^6)$ is almost always close to the maximum possible shear force observed from dynamic analysis. However, such hypothesis still has to be proven.

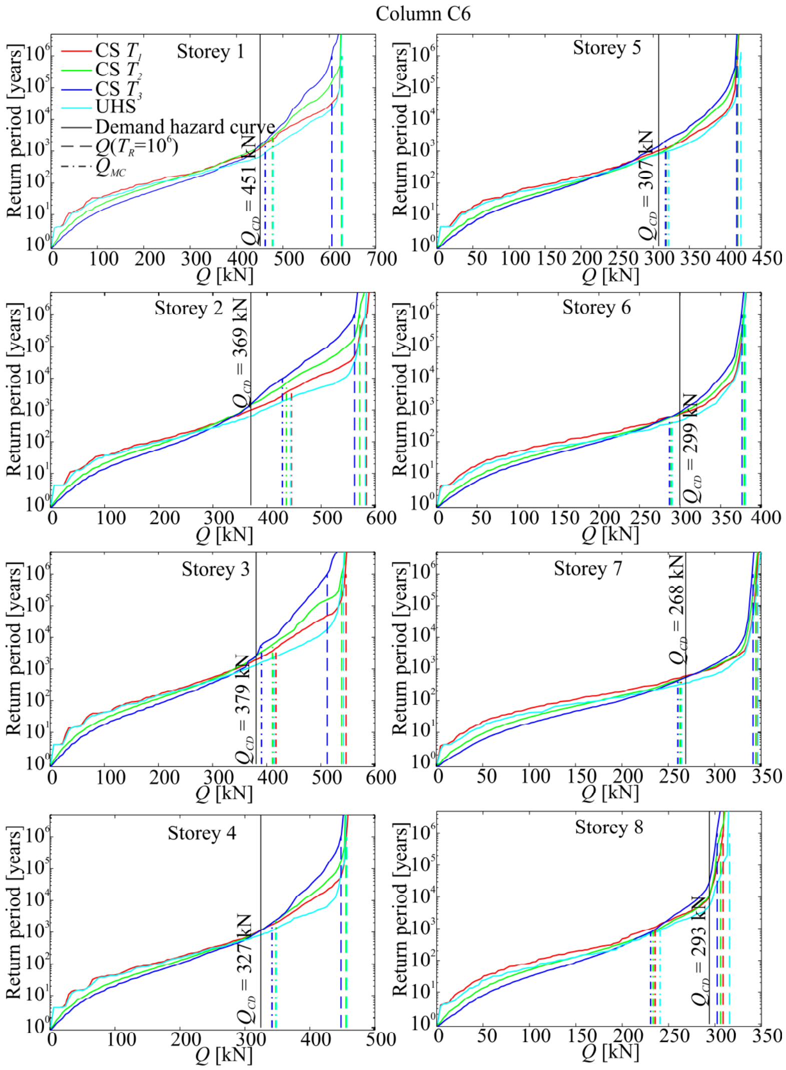

Figure 5: The demand hazard curves for column C6

The maximum credible design shear forces (Q_{MC}) were then compared to the design shear forces according to Eurocode 8 (Q_{CD}) (see Figure 4 and 5). Firstly, it can be observed that Q_{MC} does not vary significantly depending on the shape of the target spectrum. The smallest (3 kN) and the greatest (27 kN) difference are observed for column C6 respectively in 6-th and 3-th storey. Note that this is only 1-5% of total mean $Q(T_R=10^6)$. However, the variation between Q_{MC} and Q_{CD} is quite significant. For column C6 the Q_{MC} and Q_{CD} were quite similar for most of stories, but for the top storey it was observed that $Q_{CD} > Q_{MC}$. This means that the design shear force Q_{CD} is overestimated in the case of the column C6 at the top. The opposite conclusion can be made for the exterior column C5, where $Q_{CD} < Q_{MC}$ in most of the stories.

In the worst case the Q_{CD} is only around 70% of maximum credible design shear force. Additional studies are needed in order to provide more insight regarding such observations.

An interesting results of the study presented in this papers are the demand hazard curves for the shear forces in the columns, which were estimated using different sets of ground motions. Several trends can be observed. If someone would be interested to estimate the maximum shear force for given the return period it can be concluded that this cannot be achieved by a single set of ground motions. For example, if the long return periods are of the interest, then the maximum shear forces are obtained on the basis of the UHS spectrum using $S_a(T_1)$ for the intensity measure. However, this is not the case for the shear forces having high occurrence rate (short return period). In this case the shear forces are underestimated if assessed on the basis of UHS spectrum.

If it is assumed that the UHS spectrum is not appropriate target spectrum for dynamic analysis, then it can be concluded that shear forces estimated on the basis of CS conditioned to the first vibration period can be considered appropriate to use in design since in the case of long return periods so-determined shear forces are the greatest. However, this statement may not be general since it is based on the results obtained only for the investigated building. For this particular example, the envelope of the demand hazard curve is obtained by using CSs conditioned to the first and third vibration period.

7 CONCLUSIONS

In the presented study the return periods of capacity design shear forces in the columns of 8-storey building were calculated for different sets of ground motions. It was found that the return period of the design shear force depends on position, storey and set of ground motion. The smallest return periods of Q_{CD} were observed for the exterior column (except in the top storey) when the demand was estimated on the basis of the ground motions consistent with target UHS.

The adequacy of the design shear force in the columns according to capacity design was estimated by the ratio between Q_{CD} and the maximum credible design shear force. The results of the demand hazard analysis point to the following conclusions: (i) it is expected that the safety in design using capacity design procedure is greater for inner columns and (ii) the sets of ground motions do not have great impact on maximum credible design shear force.

It was also observed that if the long return periods are of the interest, then the maximum shear forces for given return period are obtained on the basis of the UHS spectrum using $S_a(T_1)$ for the intensity measure, whereas in case of short return period, the maximum shear force are obtained for CS conditioned to the third vibration period (CS T_3).

Since only one example was used for this study, it is important to note that the conclusions are not general. Several issues have to be addressed in future research in order to develop guidelines for design of shear forces using demand hazard analysis.

8 ACKNOWLEDGMENTS

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