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ESTIMATION OF STRENGTH DEMAND FOR PERFORMANCE BASED DESIGN

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SUMMARY

The paper presents the application of methodology for design of earthquake resistant structures based on the estimation of seismic performance. It describes the procedure to determine necessary strength capacity in order to provide the permitted damage level during earthquake with different probability of exceedance. Three design levels are considered to which different levels of structural behaviour during earthquake correspond. Depending on the chosen protection degree, the seismic hazard level is associated with the corresponding performance level (permitted damage level). Inelastic spectra have been obtained through statistical studies of the spectra determined by the non-linear analysis of SDOF system subjected to ground motions representing different hazard levels. As part of the SAC project, several sets of 20 ground motions representative of different probabilities of exceedance in different locations. Each set consists of recorded and simulated ground motions in order to include a balance of earthquake faulting mechanisms (strike-slip, oblique and dip-slip). The obtained results point out that, for overall seismic behaviour of the structure, several limit states are to be observed, i.e. it is necessary to check the available deformation capacity for seismic actions with different probability of exceedance. The advantages of this approach are in possibility to assess seismic performance of the buildings, obtain the necessary resistance to design earthquake actions and also prevent the collapse of the structure in the maximal possible earthquake. Besides, limited damage of the structure is achieved for minor earthquakes. It is shown that the necessary seismic performance of the structure can be determined by satisfying multi-level design criteria (in displacements, deformations and damage), provided they are checked for seismic actions with different probability of exceedance.

INTRODUCTION

The current concept of seismic protection is still based on the design of structure for so-called design seismic action with referent return period of seismic event of $T_r \approx 500$ years. Adequate design of the structure against failure is conducted for this seismic event. The necessary strength capacity of the structure is determined for influences due to seismic forces corresponding to the given design level. The magnitude of design seismic forces is determined through the force-reduction factor, which is adopted

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depending on the anticipated deformation capacity of the structure. The structure, which is designed in such way, can withstand the seismic action without the local or total collapse, if the characteristics of the occurring earthquake ground motion (peak ground acceleration, frequency content and duration of ground oscillations) correspond to the adopted seismic hazard. For the design seismic action whose probability of exceedance in service life of building is relatively small (about 10%), the codes prescribe a partial break in occupancy of the building and the occurrence of damage. For this seismic event, even the partial destruction of non-structural elements can be allowed, however, the local or total collapse of the building must be avoided. The deficiency of such concept is reflected in the fact that the force-based design does not provide insight into the damage degree of the structure, while the assessment of damage of partition walls and building equipment is carried out in unsatisfying manner. The experiences from the past earthquakes show that such design of structures does not provide uniform risk because different structures, which are designed according to the same codes, can behave differently and have a very different degree of damage during the same earthquake ground motion [12]. The proclaimed aim of aseismic design and construction is the request to prevent human injuries and limit the damage. According to the current approach to seismic protection, in design process is not possible to estimate the damage level of the structure or whether the structure retains its structural integrity and a residual load bearing capacity after seismic event.

The current concept of seismic protection anticipated by the technical codes is not completely satisfying and does not provide adequate protection of people and economic resources. Catastrophic consequences from recent earthquakes signify the need of developing new analysis methods and establishing new design criteria, which would ensure the needed structure safety, but also reduce damage of structure and non-structural elements to acceptable level. The analysis of the consequences of recent earthquakes show that, although many structures behave according to the design philosophy and withstand the earthquake without collapse, they are so severely damaged that their rehabilitation is not economical. Because of all specified deficiencies, when designing structures it is necessary to perform safety control based on several design criteria [2], [4], [8]. Besides the determination of strength and deformation demands (the top building displacement, inter-story drift, $P - \Delta$ effects), it is necessary to check the damage of the structure and non-structural elements. Within that, it is necessary to satisfy design criteria for several limit states, where every performance level and degree of damage corresponds to seismic action with different probability of exceedance in service life of the structure [3].

Considering the specified deficiencies of the current design concept, this paper considers the design methodology based on evaluation of the seismic performance of structures. The evaluating procedures suitable for practical application are proposed, which can be used to assess the seismic performance of structures in non-linear behaviour. Differently to the traditional approach, which is force-based, the proposed approach is based on the evaluation of damage of the building structure.

STRUCTURE SAFETY AT DIFFERENT SEISMIC HAZARD LEVELS

Besides the fact that the current seismic protection approach does not provide equal degree of structure damage during the design earthquake ($T_r \approx 500$ years), it does not enable the adequate safety during the earthquake having a different probability of occurrence than the design one. Namely, besides the life protection and limitation of damages due to design seismic action, the design must also provide the adequate resistance against occurrence of non-structural damage or limitations in building operation, in case of earthquakes that can happen several times in service life of the building. At the same time, the design must provide and adequate protection against structure collapse due to seismic action having a lower probability of occurrence than the design one. However, because the earthquake-resistant design is

performed only for the design seismic action, the behaviour of structures cannot be evaluated well enough due to seismic action having different probability of occurrence than the design one.

In most of the current codes, besides determining the strength demands for the design seismic action, only the obligation of serviceability control is prescribed. The check of structure safety against collapse due to seismic action that is stronger than the design one usually is not performed. Building functionality is controlled by checking structure deformations under design seismic forces using linear elastic structural model. In addition to that, the adequate deformation criteria for displacement of the building top and maximum relative displacements (inter-story drift) are given. The deficiency of such procedure is reflected mainly in the fact that in such a way it does not consider amount of non-linear displacements and the frequency of earthquake occurrence. Even recent codes, such as Eurocode 8 (EC8), have a similar concept [6]. The approach in EC8 is comprehensive than usual, since it estimates that the expected displacements during the earthquake are significantly larger than the design ones, because of inelastic deformation of structure. For ultimate limit state (ULS) design displacements are determined through multiplication of elastic ones by behaviour factor, depending on the ductility class and the building importance category. The check of deformation is performed for serviceability limit state (SLS), where the value of non-linear displacements is reduced by the coefficient ν (Fig. 1). The coefficient ν depends on importance category of the structure, and for ordinary buildings its value is $\nu = 2$ [6]. The design criterion for building deformation is given by maximum permitted inter-storey drift. Its value depends on the applied materials of partition walls and their connection with the bearing structure. This criterion practically defines the minimum stiffness of building structure. In this way, for the serviceability limit state is allowed that the structure behaves inelastically and endures some degree of damage, which depends on adopted value of force-reduction (behaviour) factor R.

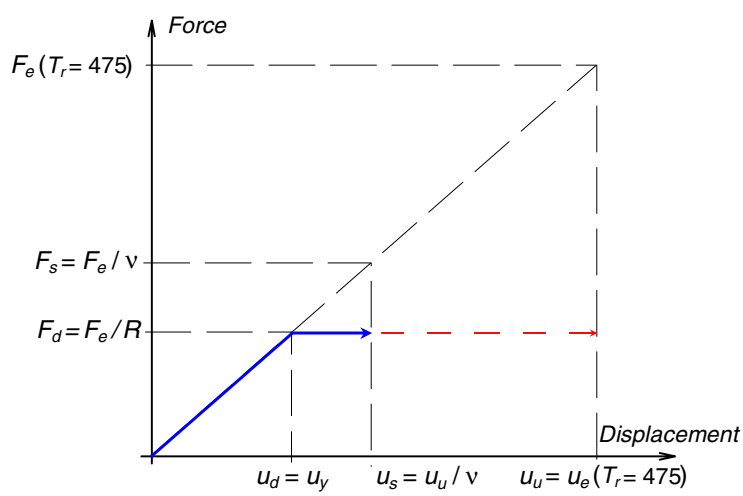

Figure 1. Allowed displacements according to EC8

Displacement that is equal to half of a design displacement (due to elastic response for an earthquake with a return period of $T_r = 475$ years) will be caused by an earthquake with two times smaller peak ground acceleration. In other words, it will be an earthquake having larger probability of occurrence than the design one, but its return period stays unknown. The relationship between maximum ground acceleration and the return period (probability of an earthquake occurrence) is neither linear nor the same for all areas inside one observed region. Different areas have different seismic hazard, so the dependence of maximum acceleration on the return period is also different for specific locations (Fig. 2). It is usually a function of seismicity at the site. Decrease or increase of design ground acceleration for a constant factor leads to different increase or decrease of structure safety in the areas with different seismicity (Fig. 3). From the

given example, which is taken from [1], it can be seen that the hazard curves for Vancouver and Montreal have a different slope, so the reduction of design acceleration for double gives a different return period. Because of that, this procedure leads to different seismic hazard for different locations, which gives non-uniform risk.

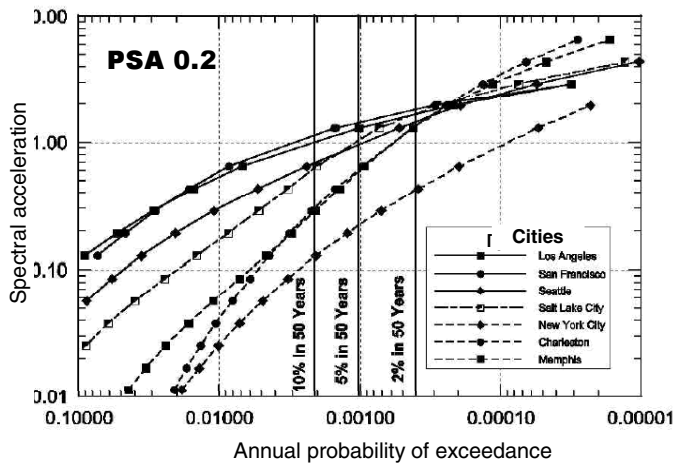

Figure 2. Seismic hazard curves of different locations in the United States

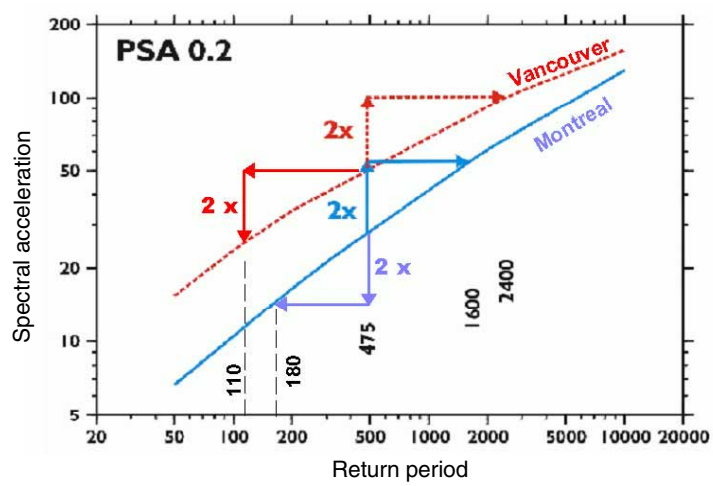

Figure 3. PSA 0.2 curves for Vancouver and Montreal

Based on this example it can be concluded that reduction of PGA for the same value (for factor $\nu = 2$), as EC8 prescribes for the serviceability limit state, produces a different safety against damage of structures built in different seismic areas. There are certainly no special reasons for that, because it is usual that all structures of same importance category have the equal degree of damage for considered level of seismic hazard. In order to provide the same level of protection of buildings with same importance category, they must be designed for seismic action with the same return period. Actually, for check of the serviceability limit state in these two areas, different reduction factors ν must to be applied. In addition to that, based on the given seismic hazard curves and the condition that PGA is determined for the same return period, it can be concluded that the value of the reduction factor ν for the area of higher seismicity (Vancouver) should be notably smaller than for the area of lower seismicity (Montreal). This indirectly shows that the use of current design concept leads to the fact that structures, which are built in higher seismic zones, often do not have enough resistance against occurrence of damages due to seismic action having larger probability of occurrence than design one. In addition to all this, diagram given in Fig. 3 shows another,

potentially even larger deficiency of current design concept. It is known that the design level of seismic action (T_r = 475 years) is not the largest possible seismic action and that there is a certain probability that a stronger earthquake will occur during the service life of the structure. In that case, the structure can be severe damaged but it must be capable to withstand this earthquake without collapse. However, the structures that are designed only for the design seismic action, and which are located in the areas of low seismicity, often do not have adequate safety against failure for a seismic action having lower probability of occurrence than the design one.

SELECTION OF SEISMIC HAZARD LEVEL AND EXPECTED STRUCTURE BEHAVIOUR

In accordance with basic objectives of aseismic design, it is necessary to provide adequate structure behaviour during an earthquake. Because of the random nature of the earthquake, realization of these goals is still only partially possible and can be quantified only by means of probability. Besides prevention of serious damages and human injury during a design earthquake, an adequate protection must be ensured against occurrence of non-structural damage or limitations in the functioning of the structure. Design must also provide an adequate safety for the seismic action that is stronger than the design one. During that, it is allowed to appear the serious damage whose would be irreparable or economically non-profitable. In this way, the aspect of seismic protection has been significantly widened – besides human safety and control of damage, it is also needed to provide an adequate safety against collapse. In order to satisfy these requirements, a multi-level seismic design must be conducted, where each limit state corresponds to a different level of seismic hazard.

Depending on the chosen protection degree, a certain level of acceptable risk is connected to a corresponding level of seismic performances (structural behaviour), i.e. to the permitted damage level (Fig. 4). The expected building behaviour is a function of seismic demands, where each level of behaviour describes a post-earthquake state of the building. Each level of seismic hazard corresponds to a different frequency of the seismic action – frequent, occasional, rare, and very rare. For each performance level of the structure, a check of satisfaction of certain design criteria must be conducted. The design criteria, besides the control of strength and displacement, must be based on the control of inelastic deformations, i.e. damages.

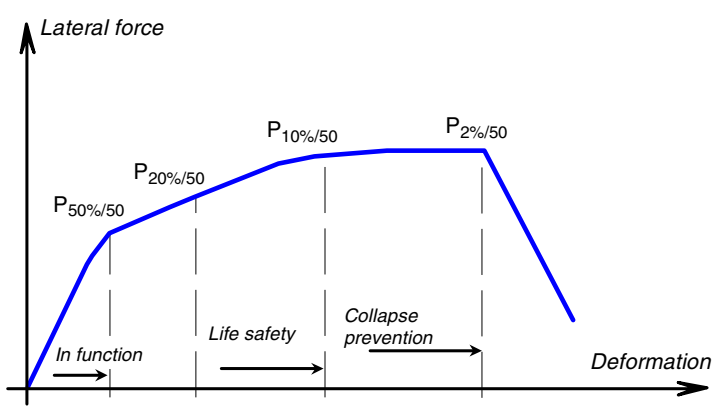

Figure 4. Seismic hazard level and the level of permitted damage

The criteria that are given by means of displacements allow the check of structure collision with adjacent ones, or with different parts of the same structure that are separated by the seismic expansion joints. Because the displacements are the greatest during the strongest earthquake, displacement control should

be conducted for the ultimate limit states. In the most of current codes, this control is conducted for design seismic action (life safety performance level), since the strongest seismic action corresponds to that limit state.

The deformations amount during the earthquake is usually evaluated through the relative storey displacement, i.e. the inter-storey drift index d_i:

$$d_i = \frac{\Delta_i}{h_i} \tag{1}$$

where Δ_i is relative storey displacement of storey *i,* and h_i is the storey height. This index is used for checking the given criteria by deformations, i.e. for satisfaction of the requirements for limitation of structural and non-structural damage. It is common for the value of inter-storey drifts to be limited depending on the used materials of the partition walls.

Control of deformations according to the current design concept is conducted only for the seismic actions with return period smaller than the design one. On the other hand, the check of deformations should be conducted for all considered limit states, where the value of permitted deformations depend on the expected level of the structure behaviour during the earthquake. This approach is prescribed in the documents such as FEMA 273 [7] and SEAOC Vision 2000 [14], in which the Performance-Based Seismic Design is represented. The description of expected damages according to the document Vision 2000, depending on the level of behaviour, is given as an illustration (Table 1). These criteria practically define the minimum stiffness of the structure. Which of these conditions will be relevant, depends on the seismicity of the area for the considered location, i.e. on the slope of the seismic hazard curve.

Table 1. Design levels and expected damage degree of structure

Design level	Structural performance	Degree of damage	Deformation d_i
1	Operational	Minor	≤ 0.2 %
2	Immediate occupancy	Moderate	≤ 0.5 %
3	Life safety	Severe	≤ 1.5 %
4	Collapse prevention	Collapse	≤ 2.5 %

Relative storey displacement or the inter-storey drift index d_i, also is an important factor for structure capability to withstand the unfavourable $P-\Delta$ effects. The evaluation of the $P-\Delta$ effects is usually conducted using the sensitivity coefficient θ. Because of the fact that the occurrence of negative stiffness leads to extremely unfavourable structure behaviour in non-linear response, it is necessary to limit the value of the sensitivity coefficient. For the life safety performance level, the sensitivity coefficient should be limited to the maximum value of $\theta_m = 0.20$, where as for collapse prevention performance level it should be $\theta_m = 0.30$ [6]. The criterion for the limitation of story drift can be expressed as:

$$d_{is} = \frac{\Delta_{is}}{h_i} \leq C_{ys} \cdot \theta_{ms} \tag{2}$$

where θ_{ms} is the permitted value of the sensitivity coefficient for the considered limit state *s*, and C_{ys} the normalized strength:

$$C_{ys} = \frac{F_{ys}}{W} \tag{3}$$

where F_{ys} is the yield strength which is necessary to satisfy the damage criteria for the considered limit state *s*, and *W* the weight of the structure. Besides satisfying the requirements for the limitation of the

structure damage (Table 1), condition (2) needs to be checked to prevent the loss of stability and structure collapse due to additional influences of $P - \Delta$ effects.

Besides checking the displacements and deformations, which are usually requested by the relevant technical codes, for comprehensive seismic protection of humans and structures, it is necessary to satisfy the requests for limitation of structural damage. In that case, three design levels will be observed with a different level of structure behaviour during the earthquake (Table 2), to which correspond serviceability limit state, limit state of damage control, and failure limit state, respectively. For each design level, a permitted structure damage level is defined by the damage index [11]. The acceptable seismic hazard level (probability of exceedance in service life of the structure) is also defined and that determines the value of PGA and other relevant characteristics of earthquake ground motion.

Table 2. Interpretation of design level and seismic hazard level

	Design Level – DL		
	1	2	3
Performance level	In function	Life safety	Collapse prevention
Damage index *DI*	≤ 0.05	≤ 0.50	≤ 1.00
Probability of exceedance	50% / 50 years	10% / 50 years	2% / 50
Return period (years)	72	475	2475

For the first design level (DL=1), i.e. for frequent earthquakes ($T_r = 72$ years), the expected behaviour of the structure is close to linearly elastic one, where the damage of the partition walls is very small. Because of this, there are recommendations [5] that the dynamic analysis for this design level is performed with the coefficient of viscous damping $\xi = 2\%$. Considering that the prescribed value of damage index for this design level is very small ($DI = 0.05$), the value of the coefficient of viscous damping $\xi = 5\%$ was accepted for all performance levels.

When designing the structure it is necessary to provide a satisfying reserve of capability for plastic deformation between "life safety" and "collapse prevention" performance levels. In the FEMA-274 [7] there are recommendations that the ductility demand corresponding to life safety performance level should be 2/3 of the available ductility capacity, in other words:

$$\mu_3 \geq 1.5\,\mu_2 \tag{4}$$

where μ_2 and μ_3 are the ductility demands obtained for the second (DL=2) and the third (DL=3) design level, respectively.

By fulfilling all of design criteria (in displacements, deformations and damages), would all of the goals of seismic protection (safety, functionality and economy) be satisfied. This provides safety of humans and structures, and prevents the occurrence of such damage or break in the building usage, whose price would be too high.

EVALUATION OF THE STRENGTH DURING AN EARTHQUAKE HAVING A DIFFERENT PROBABILITY OF OCCURRENCE

In this section is shown the evaluation methodology of the necessary strength of the structure, which is enough to provide the permitted damage level due to the seismic action having different probability of exceedance. In order to define the necessary strength that will satisfy the damage criteria for all seismic performance levels (Table 2), it is necessary to have adequate input data, or in other words, records of the

ground acceleration that represent the seismic actions for a certain seismic hazard level. Than, using the proposed procedure, an adequate strength C_{yDL} can be determined for each of the considered DL design levels, marked as 1, 2 and 3. The necessary strength capacity, by which the expected structure behaviour for all levels of seismic hazard can be achieved, is determined as a maximum value of strength demands of each individual design level:

$$C_y = \max \begin{cases} C_{y1} \\ C_{y2} \\ C_{y3} \end{cases} \tag{5}$$

In that way it will be provided that the degree of structure damage will be less than the permitted values under any frequency of an earthquake.

The analysis procedure, which enables the selection of the damage degree during the design process, is based on determining the necessary strength that will limit the structure's damage. Although any definition of the damage index can be used for the application of such design procedure, all further calculations will relate to the modified damage index [10], given by:

$$DI = \frac{\mu - 1}{\mu_u - 1} F(\varepsilon, \mu) \tag{6}$$

where μ is ductility demand, μ_u the monotonic ductility capacity, and $F(\varepsilon,\mu)$ function:

$$F(\varepsilon, \mu) = \left(1 + \alpha \beta \frac{\varepsilon}{\mu_p} \right) \tag{7}$$

This function depends on the structural parameter β ($\beta > 0$), maximum plastic deformations during an earthquake μ_p ($\mu_p = \mu - 1$), normalized hysteretic energy ε ($\varepsilon = E_h / (F_y u_y)$), and the coefficient α that includes cumulative effects of plastic deformations [9]:

$$\alpha = 1 - \frac{\mu_c}{\mu_{ac}} \tag{8}$$

where μ_c is cyclic ductility and μ_{ac} is accumulative ductility. Function $F(\varepsilon,\mu)$ depends not only on the structure's properties, but also on earthquake characteristics and it includes effects of strong-motion duration. Because of that, it is a good indicator to identify the type of earthquakes [9].

The necessary strength is determined based on the condition that the damage index (DI) for the considered excitation needs to be equal to the adopted or prescribed value (DI_p):

$$DI \le DI_p \tag{9}$$

This task cannot be solved directly, but only in iterative way. That is to say, the value of the damage index (6), besides the available ductility capacity μ_u, also depends on the ductility demand, hysteretic energy dissipation and accumulation of the inelastic deformations. Because these parameters directly depend on the strength of the system, equation (9) can be solved only in the iterative way, through a larger number of non-linear dynamic analyses.

The structure will have the adequate safety if the ductility demand during an earthquake is smaller than ductility satisfying prescribed damage criterion. This condition can be presented as:

$$\mu \le \mu_m \tag{10}$$

where μ is the ductility demand that is realized during an earthquake, and μ_m the ductility satisfying prescribed damage criterion (9). Ductility μ_m can be defined based on the equation (6), by setting the condition that $\mu_m = \mu$, which leads:

$$\mu_m = 1 + \frac{\mu_u - 1}{F(\varepsilon, \mu)} DI \tag{11}$$

In order to illustrate this procedure, necessary strength is determined for the three structures with different periods: $T = 0.3$ sec, $T = 1.0$ sec and $T = 2.5$ sec (stiff, moderately stiff and flexible structures). The analysis is conducted with three different records of ground acceleration for the area of Los Angeles, which are marked as LA-58, LA-16 and LA-25 [13]. These accelerograms represent an earthquake with 50, 10 and 2% probability of exceedance in 50 years, respectively. The analysis is performed with damping of $\xi = 5\%$ and stiffness degrading hysteretic model (SD – HM) without hardening ($\kappa = 0$). The same value of the parameter $\beta = 0.15$, and the same monotonic ductility capacity $\mu_u = 10$ for all structures is adopted. For comparison, the analysis results are given for all three-design levels (Table 3):

Table 3. Response parameters for different design levels

T	DL	μ	μ_c	μ_{ac}	ε	$F(\varepsilon,\mu)$	DI	C_y
0.3	1	1.220	1.387	7.663	1.878	2.051	0.05	0.4157
	2	3.947	4.921	37.150	12.335	1.545	0.50	0.4915
	3	7.984	8.593	52.240	16.084	1.289	1.00	**0.7153**
1.0	1	1.262	1.522	6.338	1.639	1.712	0.05	0.1895
	2	4.493	5.535	20.959	9.121	1.288	0.50	**0.2806**
	3	7.798	10.418	43.129	19.354	1.324	1.00	0.1597
2.5	1	1.380	1.400	3.383	0.803	1.186	0.05	**0.0876**
	2	4.435	6.213	20.720	10.129	1.310	0.50	0.0433
	3	8.690	8.690	27.535	12.758	1.170	1.00	0.0484

For the first design level (DL=1), that is under LA-58 ground motion with $a_g = 0.231g$, which represents the seismic action with 50% probability of exceedance in 50 years, the permitted degree of damage is very small ($DI = 0.05$). Because the structure behaviour for this design level must be close to linear elastic one, values of the ductility demand (μ) and cyclic ductility (μ_c), i.e. the structure response parameters that describe the maximum deformation of the system, are limited to a relatively small value in comparison to the monotonic ductility capacity. In that case, the value of the parameters that comprise the cumulative effects of inelastic deformations during an earthquake (accumulative ductility μ_{ac} and normalized hysteretic energy ε) is considerably smaller than in a case when a greater degree of damage is permitted.

For the second design level (DL = 2) significantly stronger seismic actions are anticipated (an earthquake with the return period of $T_r = 475$ years), with the degree of damage ($DI = 0.5$) that is respectably larger than in the previous case. During an LA-16 ground motion with $a_g = 0.580g$, which represents the seismic effect with 10% probability of exceedance in 50 years, non-linear behaviour of the structure is significantly expressed (column 3 in the Table 3). In addition, a considerable increase of accumulative ductility and normalized hysteretic energy in comparison to the previous design level is noticeable. That is a consequence of more explicit non-linear behaviour under which the structure is subjected to a larger number of repeated cycles of moderately inelastic deformations ($\mu_{max} < 4.5$).

For the third design level (DL = 3), that is for the collapse prevention performance level ($DI = 1.0$), the structure must withstand the seismic action with the return period of $T_r = 2475$ years without collapse. With that seismic hazard (2% probability of exceedance in 50 years), the intensity of the expected seismic

action is by rule significantly larger than for the usual design level – maximum ground acceleration of LA-25 record is $a_g = 0.868g$, which is 49.8% more than for the previous design level (DL = 2). For this design level, the non-linear deformations are very expressed ($\mu_{max} = 8.69$), and the accumulation of plastic deformations is significantly larger than in the two previous cases.

From the obtained results, it is evident that for all structures is obtained the sufficient minimum reserve of capability for plastic deformation between the collapse prevention and life safety performance level. The smallest value of this reserve is for the structure with a vibration period of $T = 1.0$ sec, and it amounts to 73.6% ($\mu_3 = 1.736\,\mu_2$), so it can be stated that the condition (4) is fulfilled.

The results of this analysis show that the available capacity of structure deformation under the monotonically increasing deformations ($\mu_u = 10$) cannot be completely used during the strong ground motion. The value of ductility demand μ, under which the considered structures fails, is from 78 to 87% of the available monotonic deformation capacity. Necessary limitation of maximum ductility demand of the considered examples was, in average, about 20%. In general case, the necessary limitation of ductility demand depends on the characteristics of ground motion (especially the frequency content and duration), stiffness of the structure and its hysteretic behaviour model.

When the yield strength demands for each design level are known, the necessary strength capacity is obtained on the basis of (5), which can serve to perceive what design level is relevant to the structure of the considered stiffness. The obtained results show that for structures with vibration periods of $T = 0.3$, $T = 1.0$ and $T = 2.5$ sec, are competent seismic actions with 2, 10, and 50% probability of exceedance in 50 years, respectively. A similar procedure is conducted for all structural periods that are of interest for the usual structures, and the results are given as a comparative diagram (Fig. 5).

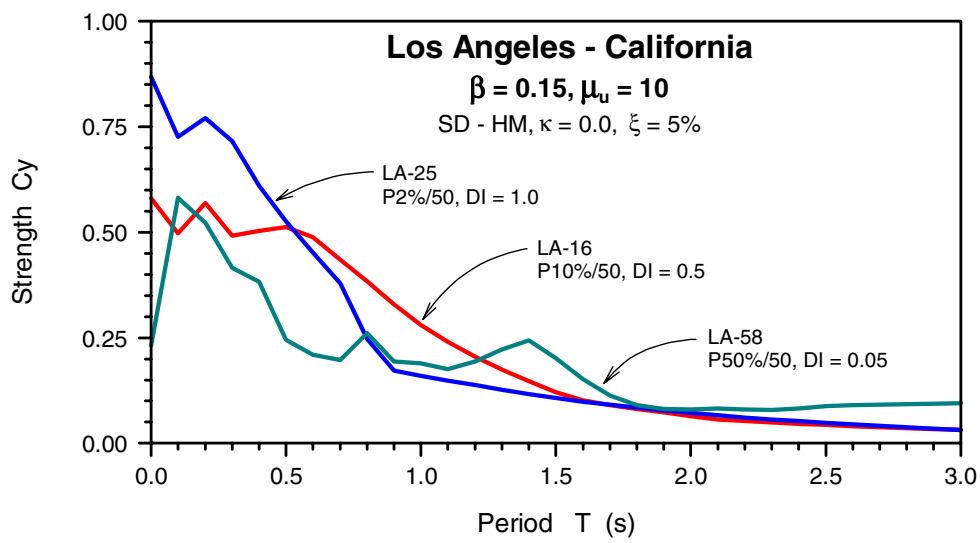

Figure 5. Strength demands for earthquakes with different probability of exceedance

Based on the results of the performed numerical analysis it can be concluded that an earthquake with the return period of $T_r = 2475$ years is relevant for stiff structures. The collapse prevention performance level is critical for this frequency range. The second design level (seismic action with return period of $T_r = 475$ years), that is, the damage control limit state, is relevant for moderately stiff structures. Finally, it is

obtained that the serviceability limit state, i.e. frequent earthquakes (with the return period of $T_r = 72$) determine the required strength for flexible structures.

It is necessary to state that these results should not be generalized, because they are valid only for the considered seismic actions. In addition, as the main deficiency of this analysis can be taken the fact that it was performed with only one excitation for each considered design level. It is known that numerical analysis with the seismic action given as ground motion record must be done using several accelerograms with similar characteristics. For example, EC8 prescribes that the analysis must be conducted with at least three different accelerograms. The FEMA-273 document recommends at least five accelerograms, allowing using statistical analysis of results if more than seven accelerograms are used.

Because of the insufficient number of accelerograms, the results of this research are incomplete and cannot be used for formulating the general conclusions. On the other hand, they indicate that the seismic performances of the structure cannot be evaluated adequately if their earthquake resistance is determined for only one level of seismic action.

ESTIMATION OF THE RELEVANT INFLUENCES FOR THE AREAS OF DIFFERENT SEISMICITY

An analysis is conducted in with an objective to establish whether the usually design level of seismic action (return period of $T_r = 475$ years) provides enough safety for all limit states of structures which are built in the areas of different seismicity. Based on the performed analysis it can be estimated which limit state is relevant for the global seismic safety of the structure.

The analysis was performed using several accelerograms for each design level, for two locations with different seismicity. First of them (Los Angeles) belongs to the extremely high seismicity zone, for which a very high value of the peak ground acceleration is prescribed ($a_g = 0.6g$) for return period of $T_r = 475$ years. Design accelerations that are so high are unusual for European conditions, where the strongest seismic action is estimated to the level of $a_g = 0.4g$. The second location (Boston), with the expected maximum ground acceleration of $a_g = 0.2g$ for return period of $T_r = 475$ years, belongs to the zone that is usually considered as moderate seismicity area.

The accelerograms that represent the seismic action with 50, 10 and 2% probability of exceedance in 50 years are used as input data in the dynamic analysis. These records are given in the SAC project [13], in which the earthquake ground motions with the return periods of 475 and 2475 years, are defined for two locations in the United States (Boston and Seattle). For the third location (Los Angeles), the accelerograms are given including also the return period of 72 years. For each location and for each level of the seismic hazard, 20 different records of ground acceleration, as sets, are selected. They include the ground motions of recorded and artificial earthquakes, which gave the adequate balance of different focal mechanisms (strike-slip, oblique and dip-slip).

For Los Angeles, the non-linear dynamic analysis was conducted with 3×20 different accelerograms. Since no records, which represent the seismic effects with 50% probability of exceedance in 50 years, are given for Boston, for this level of seismic hazard were used records representing an earthquake with $T_r = 475$ years. In this case particular accelerograms were scaled in such a manner that the average value of maximum ground acceleration equals $a_g = 0.049g$, which is the expected value of ground acceleration for seismic action with the return period of 72 years for this location. Dynamic analysis was conducted with damping of $\xi = 5\%$ and stiffness degrading hysteretic model (SD – HM) without hardening ($\kappa = 0$). The same value of the parameter $\beta = 0.15$ and the same monotonic ductility capacity $\mu_u = 10$ for all structures

was adopted. The results of the numeric analyses are given in the form of diagrams that compare the average values of the strength demands for the considered locations.

For high seismicity area (Los Angeles), the analysis results (Fig. 6) indicate that the serviceability limit state is relevant for all stiffness, except for extremely rigid structures ($T < 0.15$ s). The requirement that the damage value due to frequent earthquakes needs to stay in the prescribe limits, determines the total strength demands for this location. If the structures are designed only for seismic action with 10% probability of exceedance in 50 years, greater damages than the permitted ones will appear during a ground motion of frequent earthquakes (50%/50 years). In that case, greater damage repair costs are needed, and even great break in the building service can be caused. Thus, the requirements for service and economic aspect of the structures, which are basic goals of seismic protection, will not be satisfied. On the other hand, it is noticeable that for periods $T < 1.2$ sec greater strength demands is obtained for an earthquake ground motions with probability of exceedance of 2% than 10% in 50 years. That means that structures which are designed only for one level of seismic action (with $T_r = 475$ years), besides of inadequate resistance to the occurrence of damages due to frequent earthquakes, also do not have the adequate strength against collapse during earthquakes having smaller probability of occurrence.

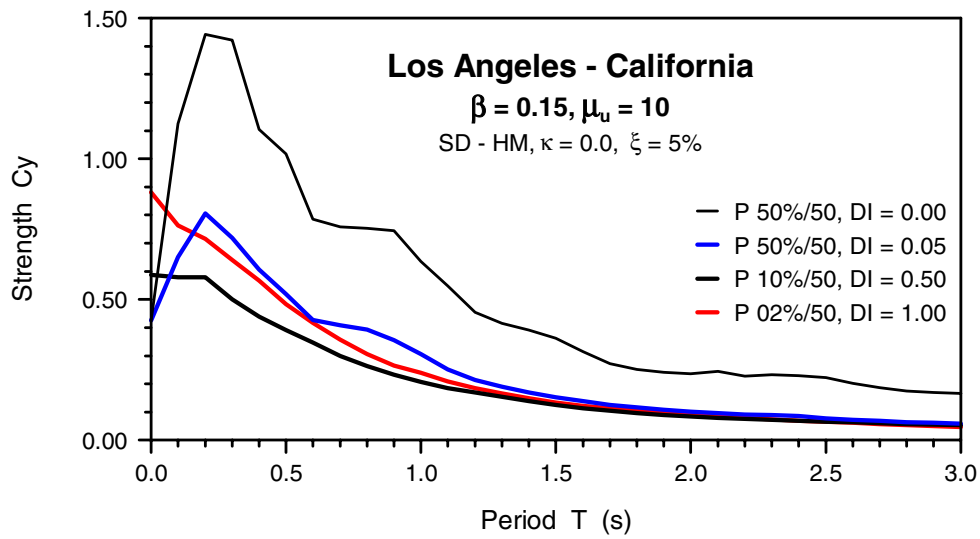

Figure 6. Strength demands for earthquakes with different probability of exceedance (Los Angeles)

Such results of the analysis, the ones that confirm that the serviceability limit state is relevant, are unexpected at the first view, considering that it is a region with extremely high seismicity where it is usually expected that the collapse prevention determine the minimum lateral strength. However, because of a very small slope of the seismic hazard curve for this location, the intensity of seismic action having greater probability of occurrence is respectable – the average value of maximum ground acceleration of all 20 accelerograms with the return period of 72 years equals $a_g = 0.43g$. Such high peak ground accelerations are not noticeably different from the design acceleration ($a_g = 0.60g$, $T_r = 475$ years) or from the average value of the maximum ground acceleration $a_g = 0.88g$ for collapse prevention performance level ($T_r = 2475$ years). Because of these facts, and because a very small degree of damage is permitted for this limit state, the seismic action for the first design level (DL = 1) is relevant.

On the same diagram (Fig. 6) the strength demand for linear elastic structural behaviour ($DI = 0$, $\xi = 2\%$) due to seismic action with 50% probability of exceedance in 50 years is given. In this case it is required to have such structure characteristics that would prevent the occurrence of any damage, their extremely high

strength would have to be provided, especially for the structures with vibration period of $T < 0.9$ sec. Providing such extreme strength would be very expensive, so it would be more rational to use some other forms of seismic protection – for example, the base isolation or energy dissipation devices. In addition to that, it is extremely important that all requirements to be recognized in the initial design phase. It can only be accomplished by the use of performance based seismic design concept.

Slightly different results are obtained for Boston than in the previous case – collapse prevention performance level is relevant (Fig. 7). If the strength demands of structures in this location are determined only for seismic action with 10% probability of exceedance in 50 years, there is a real possibility of its collapse under very rare earthquakes (2% probability of exceedance in 50 years). The expected maximum acceleration according to 2475 years of return period for this location equals $a_g = 0.48\,g$, which is 2.36 times larger value than for the return period of 475 years ($a_g = 0.202\,g$). This high increase of intensity of seismic action, with a slightly changed frequency content and strong-motion duration, leads to the fact that the third design level, or the collapse prevention, is relevant for this location.

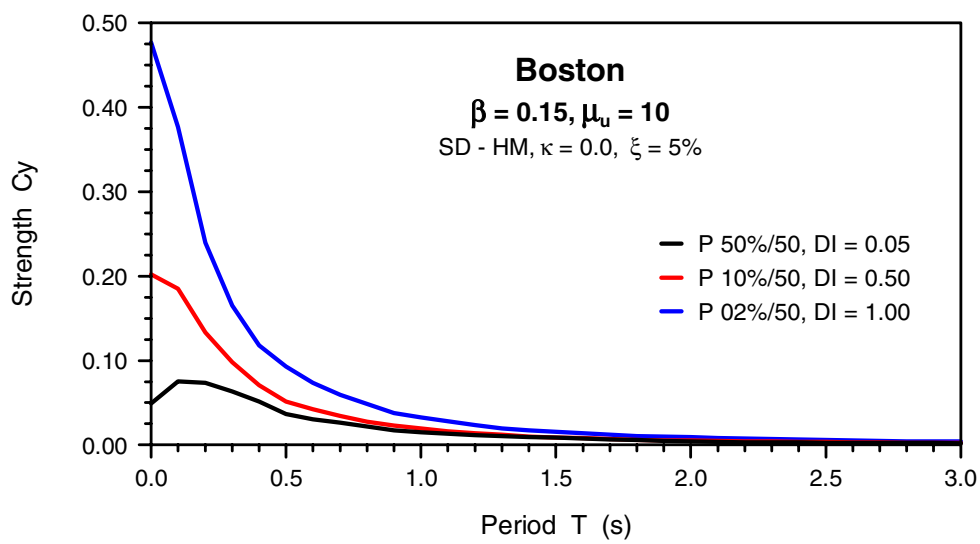

Figure 7. Strength demands for earthquakes with different probability of exceedance (Boston)

Based on the carried out analyses, it is evident that one level seismic design does not provide a complete insight into the structure behaviour during earthquakes with the different probability of exceedance. The analysis of the relevant effects for the area of different seismicity showed that the strength demands obtained in this way do not provide their favourable behaviour during frequent earthquakes (the area of Los Angeles). Furthermore, such strengths are often not enough to prevent the structure collapse due to stronger earthquakes than the design one (both considered areas). It is also evident that, for the areas of low or moderate seismicity, the current design concept does not provide the adequate safety against collapse.

CONCLUSIONS

For evaluation of seismic resistance, it is necessary to control the damage degree for several performance levels, in accordance to the chosen seismic hazard level and the expected structure behaviour. In this paper, three design levels are observed, to which the different performance levels of the structure (different limit states of structure) correspond – serviceability, damage control and collapse prevention. For each design level, a permitted level of structure damage is defined, as well as the acceptable level of seismic

hazard (expected value of maximum ground acceleration and other relevant earthquake characteristics, depending on the considered location).

The evaluation methodology of the necessary strength of the structure, which is enough to ensure the permitted damage level during an earthquake with a different probability of exceedance, is also presented. A part of this research is conducted with a goal to establish whether the current design concept, which is based on one level of seismic action (for $T_r = 475$ years), provides the adequate resistance against severe damages or collapse. The results show that the seismic performances of the structures cannot be adequately evaluated if strength demands are determined for only one level of the seismic action.

Current design concept, which is based on only one level of seismic action, does not provide favourable structure behaviour during seismic action with different probability than the design one. Because of a small number of locations (only two areas with different seismicity were observed), general conclusions cannot be formulated. On the other hand, considering the shapes of the seismic hazard curves in the areas with different seismicity, the results of this research can be regarded as relevant. They clearly show that the structure behaviour, during an earthquake with the different probability of occurrence, cannot be adequately evaluated if the strength demand is determined only for so-called design seismic action. It is especially unfavourable that the current design concept does not provide the adequate safety against the collapse of the structures, whereby this particularly relates to the areas of low and moderate seismicity.

At the end, it can be concluded that the required seismic performances of the structures can be defined only by satisfying multi-level design criteria (in displacements, deformations and damages), whereby their check must be conducted for seismic actions with different probability of exceedance. Only in this way can be provided the satisfaction of all goals and requirements of the seismic protection (safety, functionality and economy).

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