ANALYSIS OF DAMAGE POTENTIAL OF MONTENEGRO 1979 EARTHQUAKE

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SUMMARY

Civil engineering structure has an adequate earthquake resistance if its supply limit capacity exceeds, within certain margin of safety, seismic demands in the case of severe earthquakes. For the rational aseismic design of building structures, a procedure is needed which would yield an adequate estimate of seismic demands in terms of structural stiffness, strength, ductility and energy dissipation. The response spectrum approach is the most commonly used method in earthquake engineering. In seismic codes, design spectra based on the application of constant reduction factors are used for determination of seismic demand. The paper presents the results of parametric studies of inelastic response for strong ground motions representing design earthquake (Life Safety Performance Level). As the input data in the dynamic analysis the total of 20 accelerograms of earthquakes recorded in Montenegro and USA were used. The basic structural parameters contributing to the inelastic seismic response of building structures are identified and their influences are demonstrated.

1. INTRODUCTION

Present seismic design codes allow structures to undergo inelastic deformations in the event of strong earthquake ground motions. In current design procedures an inelastic behaviour is accounted for through the use of strength reductions factors that allow structures to be designed for lateral forces that are smaller than those required by the structure to remain elastic during severe

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earthquakes. The design spectrum of seismic action is determined on the basis of elastic response spectrum by application of the force reduction (or behaviour) factor R. The adoption of this factor in the analysis depends on the structural system, materials used and the assumed global behaviour of the system [6]. The strength reduction factor R is defined as the ratio of the elastic strength demand F_e to the inelastic strength demand:

$$R = \frac{F_e}{F_y(\mu = \mu_t)} \tag{1}$$

where F_e is the lateral strength of the structure required to avoid yielding in the system under a given ground motion and $F_y(\mu = \mu_t)$ is the lateral yield strength required to maintain the displacement ductility ratio demand μ equal to a predetermined target ductility ratio μ_t, when the system is subjected to the same ground motion.

According to the concept of the current world regulations [3], the determination of reduction factor is based on the principle of equal displacements of elastic and inelastic systems. The "equal-displacement principle" implies that the ductility μ achieved by the inelastic system is equal to the force reduction factor R ($u_n = u_e \Rightarrow R = \mu$). However, the relation between elastic (u_e) and inelastic (u_n) displacements is considerably more complex than the assumed one, because during an earthquake they only rarely happen to be equal. This relation depends on the excitation of a system and on structural characteristics, i.e. a number of factors among which the most important ones are stiffness, strength and ductility [2].

2. PARAMETRIC STUDIES OF INELASTIC RESPONSE

The structural system is simulated by an equivalent single-degree-of-freedom (SDOF) model with a bilinear force-deformation envelope. Two hysteretic models have applied, bilinear (BL) and stiffness degrading (SD), which simulates flexural behaviour of the structure. In both cases hysteretic model is used with or without strain hardening. The basic structural parameters contributing to the response of inelastic system are: mass (m), fundamental period (T), damping (ξ), yield strength (F_y) and displacement ductility ($\mu = u / u_y$, where u is the actual displacement and u_y is the yield displacement). In the paper the non-dimensional strength parameter N_y was used ($N_y = F_y / ma_g$, where a_g is the peak ground acceleration). Alongside the influence of various input motion to seismic response, the following parameters of the structure have been varied: the fundamental vibration period T, reduction factor R, yield strength N_y, ductility ratio μ, hysteretic behaviour and strain-hardening ratio κ.

Inelastic spectra are obtained through statistical studies of the non-linear response of SDOF system subjected to ground motions representing a design earthquake. As the input data in the dynamic analysis we used the total of 20 accelerograms of earthquakes recorded in Montenegro (5 different records during the 1979 earthquake: Herceg Novi – "D. Pavičić", Ulcinj – "Albatros", Ulcinj – "Olimpik", Petrovac – "Oliva" and Bar) and in USA (5 different earthquakes which occurred in California: Imperial Valley 1940 – El Centro, Western Washington 1949 – Olympia, Kern County 1952 – Taft, San Fernando 1971 – Castaic and Hector Mine 1999 – Amboy). Earthquakes are divided into two major groups of 10 accelerograms (USA and Montenegro), and both groups are labelled with the identification mark Standard EQ.

2.1. The influence of structural parameters

The period range from 0.1 to 3.0 s was considered. All results were presented for bilinear hysteretic models without hardening ($\kappa = 0$). Five per cent mass-proportional damping (a time-independent damping coefficient based on elastic properties) was assumed in all cases ($\xi = 5\%$). The influence of the applied reduction factors on the strength demand is highly significant, which can be observed from the given diagrams (Fig. 1). As the required strength for the elastic response ($R = 1$) is strongly dependent on the frequency domain, the stiffness has significant influence on the strength demand in the non-linear response.

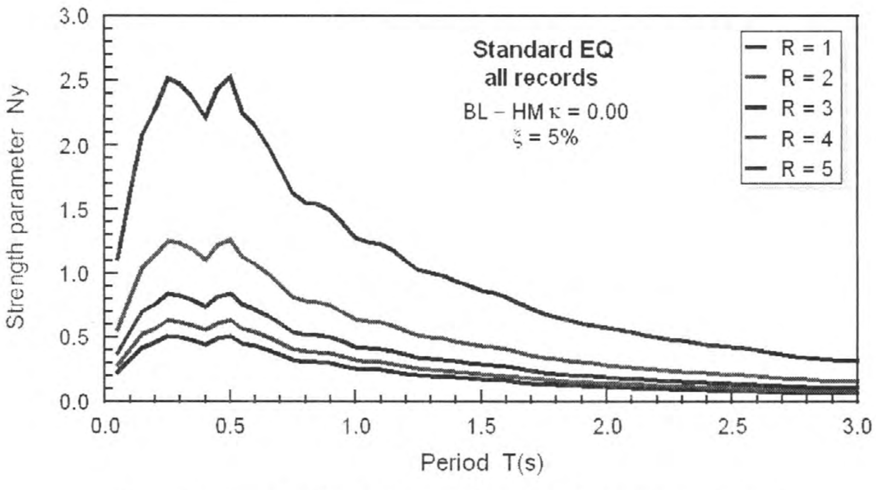

Fig. 1 *Influence of the reduction factor on the strength demand*

Beside the influence of reduction factor, the influence of the supplied yield strength on structural response is analysed. For stiffer structures and for non-linear systems whose strength is equal to or smaller than usual strength capacity ($N_y \leq 0.6 - 0.8$), the displacement ductility demand is extremely large (Fig. 2). As this amount of required ductility cannot be realized in reality, this points to the possibility of the collapse of commonly designed structures if they do not have considerable overstrength. For weaker systems ($N_y = 0.2 - 0.4$), even in mediumfrequency spectral region, relatively large ductility demand is obtained ($\mu > 5$), which points to a higher possible degree of damage to these structures. In order to limit the amount of ductility demand for the "standard" ground motion, for stiffer structures a strength larger than average one must be supplied, at least $N_y \geq 0.8$.

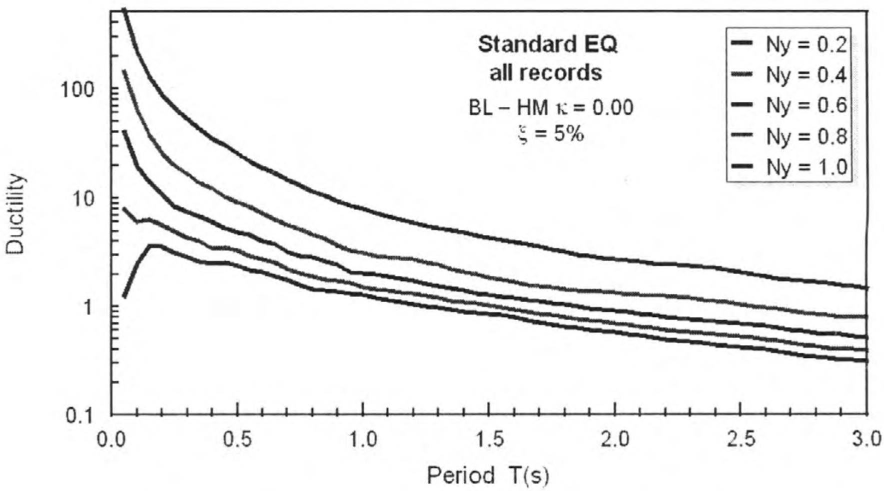

Fig. 2 *Influence of the yield strength on the ductility demand*

In seismic design one of the most important goals is the supply of a sufficient capacity of structure deformations. Therefore, it is very important to know to what extent the seismic action can be reduced in order to make sure that, on the basis of the seismic demand which will be determined in such a way, the sufficient strength capacity can be determined, so that the structure should not be heavy damaged or collapsed during severe earthquakes. In other words, if the supplied ductility is known, the question arises to what extent the strength of a linear elastic system can be reduced for the ductility demand to be equal to the target one during a real earthquake action. The answer to this question can be seen in the diagram (Fig. 3), which shows the strength demands depending on target ductility and stiffness.

Constant ductility response spectra, i.e. the functions $N_y(\mu, T)$, differ considerably in their shapes and in their magnitudes from the functions $N_y(R, T)$, especially for larger values of reduction factors, i.e. for the higher values of target ductility. These differences are more prominent for extremely high-frequency spectral region, and the largest certainly is the one of infinite stiff structures (i.e. for $T = 0 \Rightarrow N_y(\mu, 0) = 1$, $N_y(R, 0) = 1/R$). This proves that the concept of "equal displacements" of linear and non-linear systems, for which the relation $R = \mu$ generally holds, although very attractive and convenient in practice, is not true especially for stiff structures.

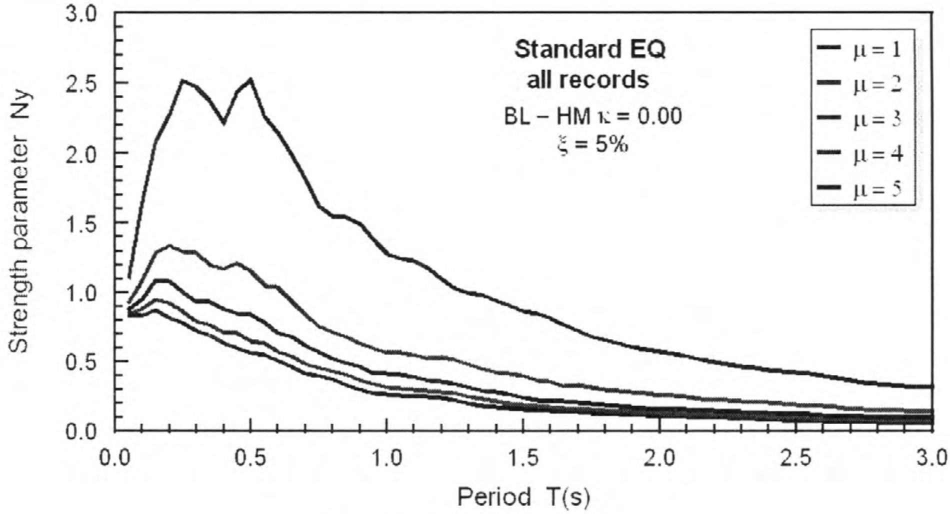

Fig. 3 *Constant ductility response spectra*

The influence of the target ductility on reduction factor is shown in corresponding diagram (Fig. 4). On the basis of the shapes of the functions $R(\mu, T)$, obtained by averaging all the responses, it can be concluded that stiffer structures yield lower values of reduction factors than the assumed magnitude of the global ductility ($R < \mu$).

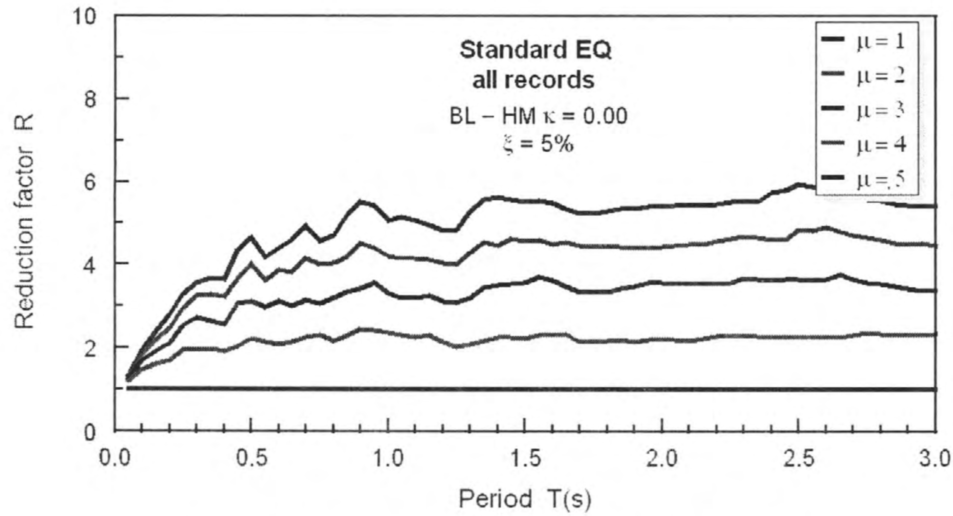

Fig. 4 *Dependence of the reduction factor from ductility*

Based on statistical studies of inelastic response a simplified expression is proposed to estimate strength reduction factor:

$$R(\mu,T) = R_{\mu} = 1 + (\mu - 1)\left[1 - \exp\left(-\frac{15 + 2\mu}{3\mu} \cdot \frac{T}{T_g}\right)\right] \quad (2)$$

Proposed reduction factor depends on structural system parameters (structural period T and permitted level of inelastic deformations represented by target displacement ductility μ) and on characteristic of earthquake (through the predominant period of vibration of ground motion T_g). Proposed reduction factor intend to account for energy dissipation capacity. It is different in comparison to the reduction factors prescribed in seismic codes, which are based on observation of the performance of different structural systems in previous earthquakes.

Fig. 5 shows a comparative diagram of required strength, which is determined by the application of the constant reduction factor and the strength demand determined by using the constant ductility concept. The application of constant reduction factors for stiffer structures yields insufficient lateral strength. As a result, during strong earthquakes, the damage that is considerably greater than the anticipated one could occur. In some cases the structures, designed in this way, could even be collapsed.

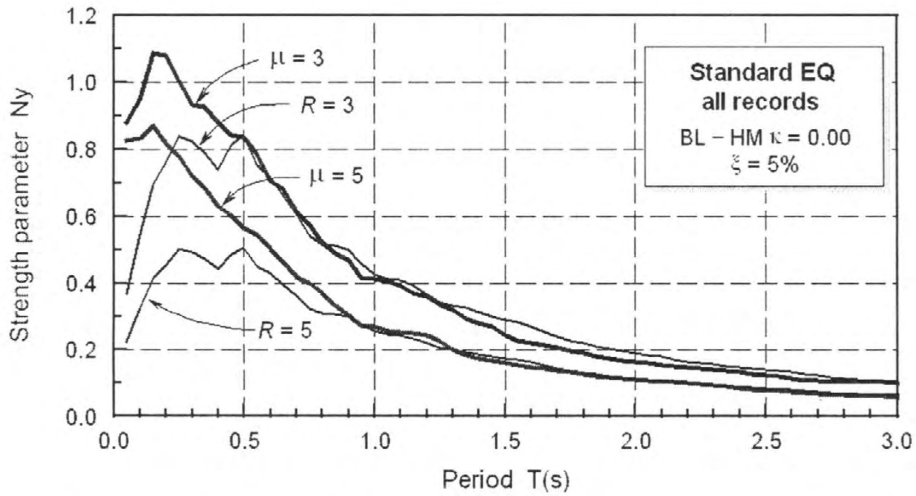

Fig. 5 *Strength demands for constant ductility and constant reduction factor*

The inelastic displacement ratio or displacement amplification factor δ is defined as the ratio of inelastic to elastic displacement demand ($\delta = u_n / u_e$, where u_n is the maximum inelastic and u_e is the maximum elastic displacement). At common amounts of displacement ductility (up to $\mu = 5$), even with very stiff structures, the displacement demand ratio is not larger than the value of target ductility (Fig. 6). According to the obtained results a simplified expression for displacement amplification factor δ is proposed:

$$\delta = \delta(\mu, T) = \left[1 + \left(\frac{1}{\mu} - 1 \right) \exp\left(-\frac{15 + 2\mu}{3\mu} \cdot \frac{T}{T_g} \right) \right]^{-1} \tag{3}$$

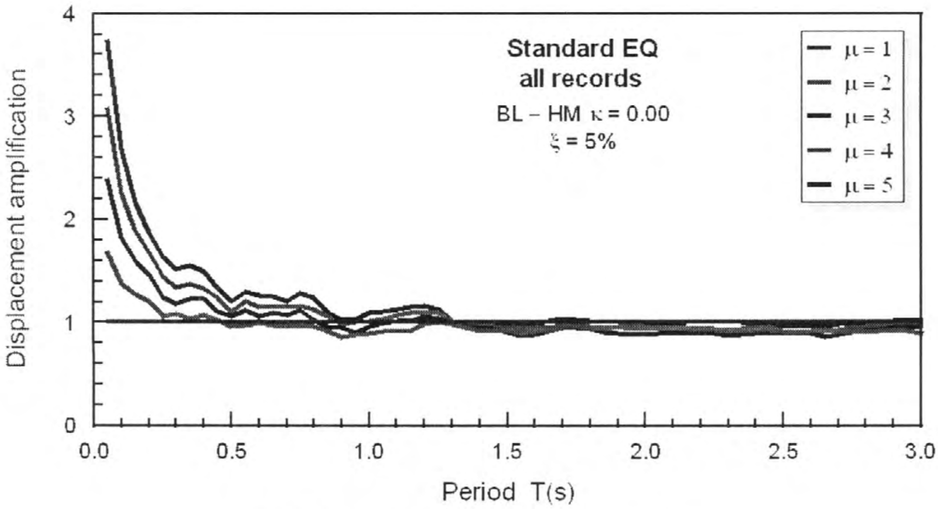

Fig. 6 Mean inelastic displacement ratio

Displacement amplification factor δ depends on structural system parameters (structural period T and target displacement ductility ratio μ) and on the predominant period of vibration of ground motion T_g.

2.2. The influence of hysteretic model

The seismic response of structures to strong earthquakes depends on the hysteretic behaviour, i.e. the load-displacement relationship between induced forces and displacements. Similarly, for different strain-hardening ratio at the same hysteretic model of behaviour, the obtained seismic demands can differ significantly (Fig. 7).

The results of numerical investigations point to the fact that at different hysteretic models of behaviour, for structures of certain characteristics (stiffness, strength and strain-hardening ratio), different seismic demands can be obtained during an earthquake. The influence of hysteretic model on the seismic response is analysed in function of the ductility, which represents the most important parameter of a structure. In the numerical analyses, three characteristic values of target ductility ratio μ_t, amounting 1.5, 3.0 and 5.0, are taken (Fig. 8). A bilinear (BL) and stiffness degrading (SD) hysteretic model without hardening were used. The required strength N_y, which a structure must have in order to achieve ductility demand μ equals to the target ductility μ_t, is shown as comparative diagram of the functions $N_y(\mu, T)$ for both hysteretic models. The results of analysis show that the application of these two hysteretic models yields similar results. If a structure in an earthquake behaves according to SD hysteretic model, the lateral strength demand N_y is, as a rule, somewhat smaller compared to the structures with BL hysteretic model. The only exception is the case of very stiff structures, but only for great amount of the ductility.

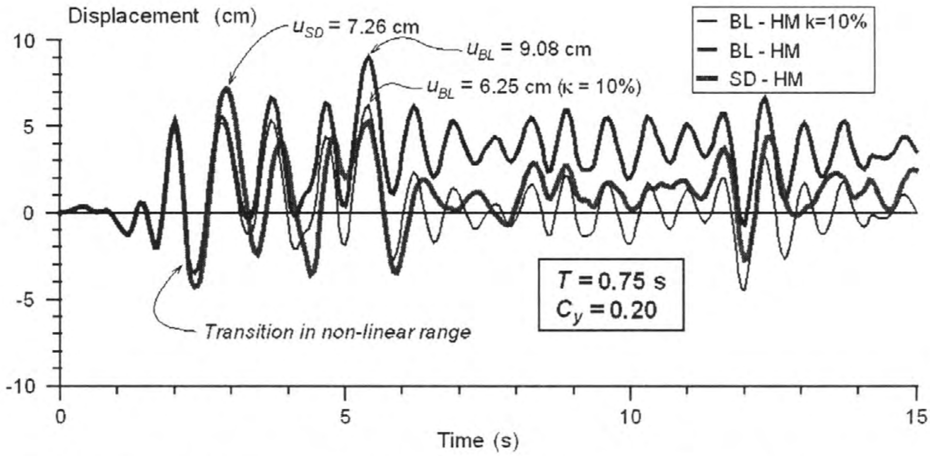

Fig. 7 *Effects of the hysteretic model and strain-hardening ratio on inelastic response*

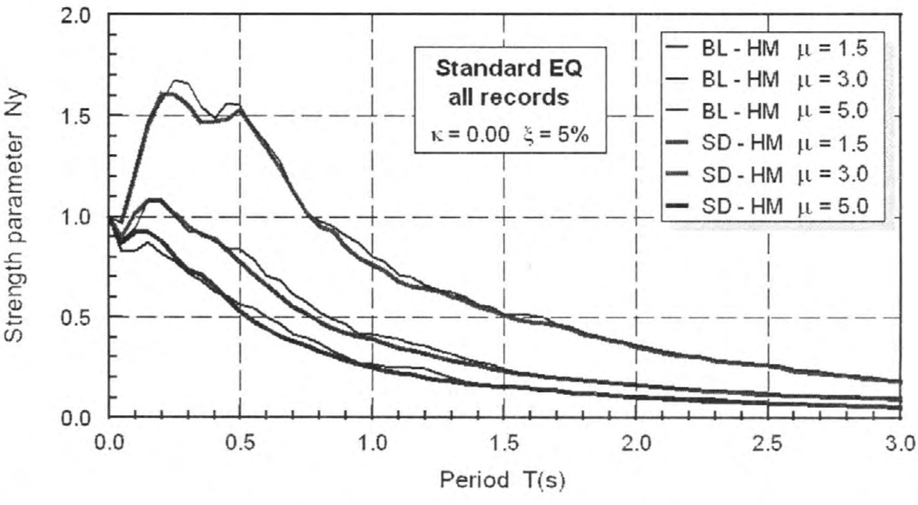

Fig. 8 *Effect of the hysteretic model on the strength demand for various target ductility*

The comparative diagram of the functions $R(\mu, T)$, which shows the extent to which the strength capacity of a linear elastic structure can be reduced to maintain the ductility demand less than or equal to target ductility ratio, is given in Fig. 9. That the application of BL hysteretic models yields conservative results can be seen from the comparison of numerical values of the functions $R(\mu, T)$. With SD models, for ordinary earthquakes (Standard EQ) and for a certain value of period T, greater reduction of the strength capacity of a linear elastic structure is permitted.

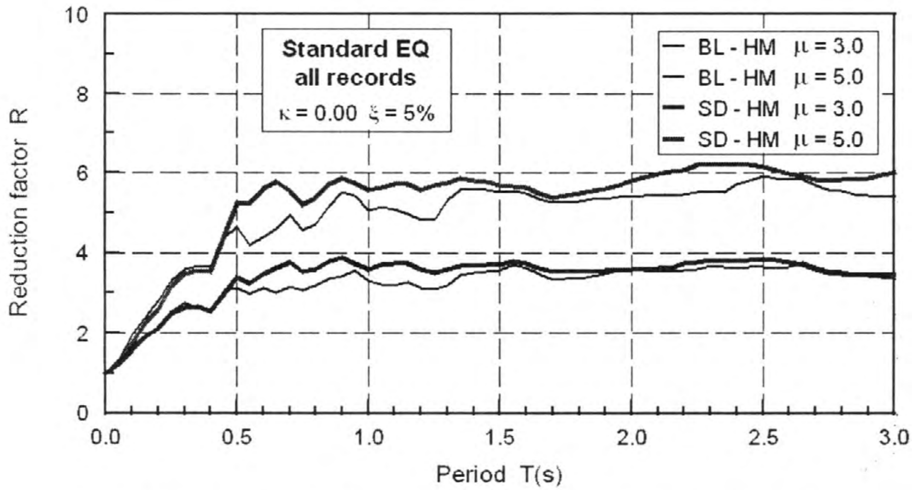

Fig. 9 *Dependence of the reduction factor from the hysteretic model*

The influence of the strain-hardening ratio κ on the behaviour of non-linear structures and on seismic demands has been considered for different value of target ductility [2]. As a rule, a larger amount of the strain-hardening ratio has a beneficial effect on structure response. At a greater amount of hardening, a higher average reduction of strength demand is obtained. The strain-hardening ratio also has a relatively significant influence on the amount of non-linear displacements. When the strain-hardening ratio is increased by 2, 5 and 10%, there is an average decrease of displacement at structures by 6.6, 12.6 and 16.9%, while the maximum values of decrease are 13.9, 21.0 and 25.1%, respectively. The results of the analysis show that the effects of hardening have a rather similar influence for both analysed hysteretic models (BL and SD).

3. ESTIMATION OF STRUCTURAL DAMAGE

The intensity of seismic action can be estimated through displacement ductility demand μ. Ductility demand, besides the characteristics of ground motions, depends on the structural properties (stiffness, strength capacity, structural system, materials, etc.). Therefore, displacement ductility represents an important parameter of non-linear behaviour of the structure, and consequently a specific measure for the damage assessment of the structure. Although ductility demand is important parameter of the non-linear response, it by itself does not give information on the level of damage. In order to assess the structural damage it is necessary to know the available deformation capacity of the structure. Level of structural damage can be estimated by damage index DI, through comparison of specific structural response parameters demanded by the earthquake with available structural deformation capacity. Damage index is a normalized quantity, whose numeric value is by definition between 0 and 1. Value of $DI = 0$ denotes the non-damaged structure, $DI \leq 0.5$ implies repairable damage, while $DI = 1$ denotes the failure of the structure.

There are several definitions of damage index – while some consider maximum deformation demands, the others take into account the cumulative plastic deformation demands. Park-Ang model of damage assessment during an earthquake [7] is one of the most frequently used. Park and Ang proposed the damage index as a combination of maximum deformation μ and hysteretic energy dissipation E_h. The improved damage index was obtained through the modification of Park-Ang index to correct its deficiencies connected with the physical meaning [5]. It is given by the following expression:

$$DI_m = \frac{\mu_p}{\mu_u - 1}\left(1 + \alpha\,\beta \frac{\varepsilon}{\mu_p}\right) = \frac{\mu_p}{\mu_u - 1} F(\varepsilon, \mu) \tag{2}$$

where μ_p is the plastic ductility ($\mu_p = \mu - 1$), μ_u the monotonic ductility capacity, α coefficient by which the influence of hysteretic energy E_h under monotonically increasing deformations is eliminated, and ε the normalized hysteretic energy ($\varepsilon = E_h / (F_y u_y)$). This damage index depends on the maximum plastic deformations during the earthquake μ_p, the available deformation capacity μ_u and function $F(\varepsilon, \mu)$ that includes cumulative effects of plastic deformations. Coefficient α ($\alpha = 1 - \mu_c / \mu_{ac}$) depends on cyclic ductility μ_c and accumulative ductility μ_{ac}. Cyclic ductility depends on maximum plastic excursion $u_{c,max}$ and accumulative ductility depends on the sum of inelastic displacements (both positive and negative) during all of the plastic excursions. Accumulative ductility is associated with the history of cyclic deformations during earthquake and it depends on the number of plastic excursions.

To provide enough seismic resistance for the structure whose strength capacity is known, it is necessary that its available deformation capacity μ_u be higher than ductility demand μ_r for which the allowed level of damage has been obtained during the given ground motion ($\mu_u \geq \mu_r$). An investigation has been carried out in which it is analysed whether the structures with monotonic ductility capacity $\mu_u = 8$ and strength determined according to Eurocode 8 (EC8) provisions [1], have deformation capacity sufficient enough to withstand given ground motions (Olimpik, Rinaldi, El Centro, Bar and Mexico City) without collapse. Strength capacity of the considered structures is determined according to EC8 for peak ground acceleration $a_g = 0.40g$, soil category B and behaviour factor $q = 3$. On the basis of the obtained results (Fig. 10), it can be concluded that the structures with given properties (provided strength capacity according to EC8 and monotonic ductility capacity $\mu_u = 8$) have sufficient ultimate deformation capacity to resist all considered ground motions, except accelerogram Rinaldi RS 228. But even for this very strong excitation (with $a_g = 0.841g$), structures with $T \geq 1.0$ sec will not undergo failure.

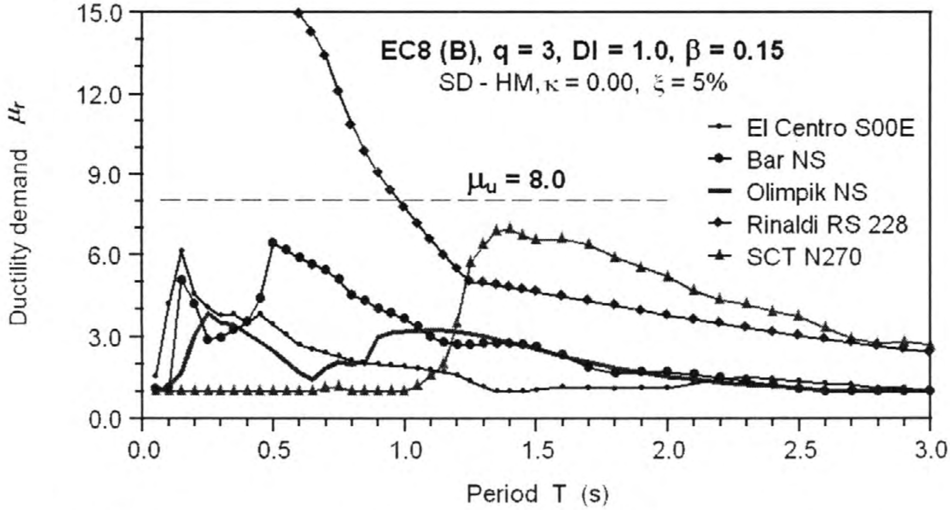

Fig. 10 *Ductility demand for ultimate limit state during different ground motions*

Deformation demand significantly depends on the intensity of the seismic action and type of ground motions, but also on the stiffness of the structure. However, it is necessary to point out that required ductility capacity μ_r for a given ground motion significantly depends not only on stiffness, but also on available strength capacity and allowed level of damage [4].

It is possible to determine the minimum lateral strength capacity C_y ($C_y = F_y / W$, where F_y is the yield strength and W is the weight of the structure) that will result in an adequate control of inelastic deformations (i.e. damage) during strong ground motion [5]. The required strength (lateral load capacity) C_y can be determined for a given seismic action. Here, four characteristic accelerograms (El Centro, Mexico City, Petrovac and Bar) were used in the analysis. Thus determined strength is compared with strength demand according to the provisions of EC8 for the area of high seismic activities ($a_g = 0.40g$), B soil class and two values of behaviour factor: $q = 3$ and $q = 5$ (Fig 4).

In the analysis the adopted value of damage index is $DI = 0.5$, which refers to the level of damage corresponding to the design seismic action (i.e. the return period of $T_r = 475$ years). The results of analysis indicate that although EC8 is a contemporary concept of design of earthquake resistant structure, it does not always provide sufficient resistance against occurrence of damage within acceptable limits ($DI \leq 0.5$).

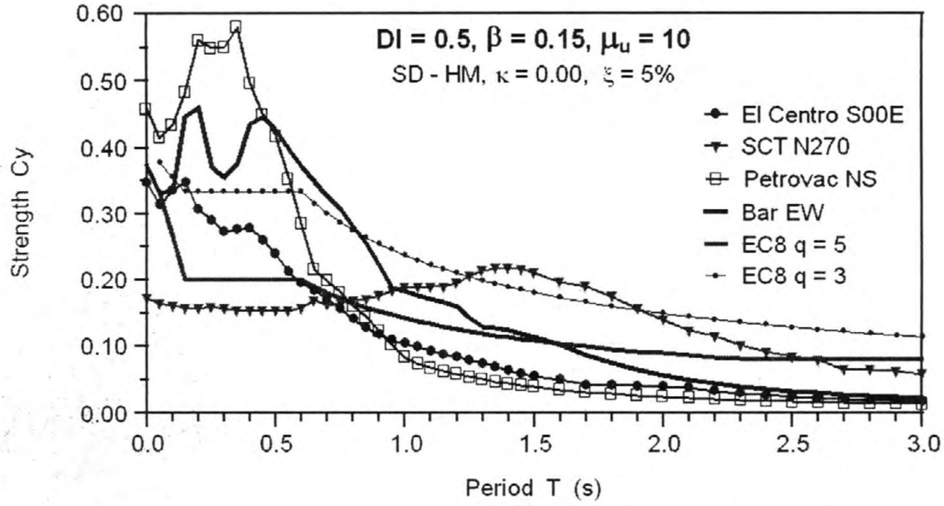

Fig. 11 Strength demand according to EC8 and for equal damage level

4. CONCLUSIONS

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. The current concept of seismic protection anticipated by the technical codes is not quite satisfying and does not provide the 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 necessary structure safety, but also reduce damage of the structure and non-structural elements to acceptable level.

The great majority of the investigation performed in this paper deal with seismic demands. In this investigation the strength reduction factor, which permits estimation of inelastic demands from elastic strength demands, is evaluated. Proposed reduction factor intends to account for energy dissipation capacity. The evaluation of the results indicates that strength reductions are primarily influenced by the maximum tolerable displacement ductility, the period of the system and the predominant period of vibration of ground motions. On the basis of obtained results it can be concluded that the procedure based on the constant force reduction factors, prescribed in existing seismic codes, can yield non-conservative values of strength and ductility demand, especially for stiffer structures. The consequence of this is the occurrence of larger inelastic deformations than anticipated, i.e. a higher level of damage for the structures designed according to the current seismic regulations.

It is necessary to point out that the actual concept of seismic design does not offer an insight into the possible damage of the structure exposed to ground motion whose characteristics correspond to design seismic action. Besides, the existing seismic design procedures cannot provide an adequate inspection of damage level of structures in quantitative terms. Design based on force-reduction factor results in non-uniform risk, thus different buildings designed to the same code and with the same force-reduction factor may experience different levels of damage under a given earthquake. The uniform risk is possible to achieve only through the application of new design procedures based on the assessment of seismic performance, including the effect of low-cycle fatigue. It is shown that estimation of the damage potential of the earthquakes and real structural response required the use of procedures that take into account the level of acceptable damage and cumulative effects of inelastic deformations.

5. REFERENCES

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