13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2135

APPLICATION OF IMPROVED DAMAGE INDEX FOR DESIGNING OF EARTHQUAKE RESISTANT STRUCTURES

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SUMMARY

The paper presents the application of methodology for designing of earthquake resistant structures based on damage assessment. The necessary balance among strength capacity, stiffness and ductility is determined to keep the damage level within prescribed limits. This procedure can be applied to determine necessary deformation or strength capacity for which damage degree of the structure after earthquake is within acceptable (prescribed) limits. The improved damage index proposed in the paper is based on plastic deformation and hysteretic energy dissipation. It is obtained by modifying the well-known Park-Ang model through eliminating some of its deficiencies connected with physical meaning of damage index. With the introduction of corrective coefficient the influence of hysteretic energy under monotonically increasing lateral deformation is eliminated. Through this is also included the influence of accumulation of inelastic deformations connected with the history of cyclic deformations during earthquake. The proposed damage index is presented in generalized form, in the function of maximum amplitudes of plastic deformations, available ductility capacity and function including cumulative damage effects due to the dissipation of hysteretic energy. This function, besides the parameter including influence of deterioration, depends on achieved plastic deformations during earthquake and normalized hysteretic energy, on cyclic ductility (maximum plastic excursion) and also on accumulated ductility. By introducing this function it is possible to include not only the structural properties, but also the characteristics of the earthquake and the effects of duration of ground oscillations. The application of this design method is given through the analysis of a larger number of examples. The analysis of results shows that current design concept in Eurocode and other seismic codes do not provide uniform risk.

INTRODUCTION

Seismic design usually is made for a design seismic action using linear elastic structural model. The current concept of seismic protection is still based on the design of the structure for so-called design earthquake action, to which refers the referent return period of the seismic event of $T_r \approx 500$ years. In codes of different countries, the design seismic action is presented in different form, but they all have in common the fact that it is given in function of adopted seismic hazard level, seismicity of the area, subsoil

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class, dynamical characteristics of the structure, importance category of the buildings and available ductility. Primary goal of earthquake-resistant design is to provide high structure's ductility. Nevertheless, the basic dilemma is how much the design lateral strength can be lower than strength required to maintain a structure in the elastic range in the event of severe ground motions. The magnitude of design seismic forces is determined through the force-reduction factor, which is adopted depending on the anticipated deformation capacity of the structure. The structure designed in this way as a rule can withstand the earthquake ground motion without a collapse, if real earthquake characteristics (peak ground acceleration, frequency content and duration of strong ground motion) correspond to the adopted seismic hazard [8]. The deficiency of this concept is, above all, in the fact that on the basis of the performed calculation there is no adequate insight into the level of damage. Failure of the structure occurs when ultimate deformations are reached, i.e. seismic resistance primarily depends on capacity of deformation, and not on intensity of seismic forces that are induced during earthquake.

During an earthquake ground motion, a structure is as a rule exposed to a number of repeated cycles of inelastic deformations, which depends on earthquake intensity and frequency content, but also on duration of strong-motion duration. Collapse of the structure can occur even at deformations of the system that are considerably smaller in relation to the available ductility, i.e. in case of displacements which are considerably smaller than the nominal monotonic ductility capacity [15]. Because of that, the unfavourable effects of accumulation of inelastic deformations must also be taken into consideration when estimating the overall safety of structures subjected to the seismic action.

ESTIMATION OF STRUCTURE'S DAMAGE

The current concept of seismic design is based on the reduction of design seismic action. The minimum lateral strength is estimated on the basis of influences due to design seismic forces, which is usually determined by reduction of the elastic response spectrum given in relevant code. Thus, for the expected seismic action, a non-linear response of the structure is allowed and according to this, a certain damage of the structure. However, the deficiency of this design concept is that in this way it is not possible to control fully the degree of structure's damage.

Simplified procedures of the analysis and inadequate input data are often used to assess the level of damage after earthquake. In practice the intensity of seismic action is usually estimated through analysis of recorded ground motions, on the basis of peak ground acceleration (PGA), velocity (PGV) and displacement (PGD), or on the basis of the elastic response spectra. However, the intensity of seismic action cannot be estimated only through data of the recorded ground motion. Hence, in the engineering practice various measures have been introduced to assess damage potential of the ground motions – elastic response spectra, spectrum intensity *SI* [9], Arias intensity *AI* [2], drift spectrum [10], RMS acceleration a_{rms}, characteristic intensity I_c [1], etc. Some other parameters could also be important for the estimation of earthquake ground motion effects, such as frequency content and predominant period of ground oscillations, mean period T_m [16], duration of strong ground motion t_D [3], [17], etc.

These variables are based on the analysis of the accelerograms or use linear-elastic analysis of structure's response. Through the analysis of accelerograms it is not possible to describe well enough the damage potential of seismic action, because it does not only depend on ground motions, but also on the mechanic characteristics of the structure. Some of the mentioned parameters (for instance spectrum intensity *SI*) take into account the response of the structure during an earthquake motion, but on the basis of the elastic behaviour. However, usual structures behave nonlinearly during strong earthquakes, so the real effects of seismic actions and damage assessment of structures cannot be seen on the basis of linear analysis.

Damage on structures is associated with non-linear behaviour; hence the destructive potential of the earthquake must be estimated through the parameters of non-linear structural response.

The intensity of seismic action can be estimated by displacement ductility demand μ. It depends on maximum inelastic deformation that is realized during the earthquake. Ductility demand, besides the characteristics of ground motions, also depends on the response of the structure, i.e. on the structural properties (stiffness, strength capacity, 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. The non-linear behaviour of the structure during real earthquakes could also be perceived through energy balance. Hysteretic energy E_h is a measure of inelastic energy dissipation caused by the earthquake action [18]. Hysteretic energy dissipation depends on maximum inelastic deformations, accumulation of inelastic deformations, and on duration of strong ground motion. During the elastic response of the structure, hysteretic energy equals zero by definition.

Although ductility demand and amount of dissipated hysteretic energy are significant parameters of the non-linear response, they by themselves do 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 through 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 value is by definition between 0 and 1. Value of $DI = 0$ denotes the non-damaged structure, i.e. linear elastic behaviour of the structure during earthquake, while $DI = 1$ denotes the failure of the structure, i.e. local or general collapse of the structure. The dependence of damage degree of the structure from damage index was initiated by Park and Ang. On the basis of data on damage in RC buildings that were moderately or severely damaged during several earthquakes in USA and Japan, they defined the relation between degree of damage and damage index (Table 1).

Table 1. Interpretation of damage index

Degree of damage	Damage index	State of structure
Minor	0.0 – 0.2	Serviceable
Moderate	0.2 – 0.5	Repairable
Severe	0.5 – 1.0	Irreparable
Collapse	> 1.0	Loss of storey or buildings

DAMAGE SPECTRUM

There are several definitions of the damage indices, and it is common to all of them that they compare the response parameters demanded by the earthquake with structural capacity. Structural capacity refers to an ultimate value of the response parameter, which is usually defined in sense of its maximum value under monotonically increasing lateral deformation. For example, a deformation Δ_u under which an abrupt loss of the strength occurs, and which represents a fraction of the ultimate deformation capacity of the system under monotonically increasing deformation Δ_{mon}, has been used as the available deformation capacity during the earthquake motion. Damage index usually considers a measure of the deformation demands in the structures. There are several definitions of damage index – while some consider maximum deformation demand, others take into account the cumulative plastic deformation demands. Damage index based on plastic deformation was proposed by Powell and Allahabadi [14]:

$$DI_{\mu} = \frac{u - u_y}{u_u - u_y} = \frac{\mu - 1}{\mu_u - 1} \tag{1}$$

where u is the maximum inelastic displacement during a ground motion, u_y the yield displacement, and u_u an ultimate displacement capacity of the system under a monotonically increasing lateral deformation. In equation (1) μ is the maximum ductility demand during an earthquake ($\mu = u / u_y$), while μ_u ($\mu_u = u_u / u_y$) is the monotonic ductility capacity. Damage index DI_μ is based on displacement ductility, i.e. on the maximum inelastic deformation during an earthquake. It gives an accurate value of damage due to static unidirectional load, but displacement ductility itself does not reveal information on the repeated cycles of inelastic deformation and energy dissipation demand.

During an earthquake ground motion, a number of the repeated cycles of inelastic deformations occur, so other structural response parameters is also used for estimation of the structural damage. Regarding that the hysteretic energy includes cumulative effects of inelastic response and is associated with the structural damage, Mahin and Bertero [12] defined normalized hysteretic energy ductility μ_h:

$$\mu_h = 1 + \frac{E_h}{F_y\, u_y} \tag{2}$$

where E_h, F_y and u_y are hysteretic energy, yield strength of the structure and yield displacement, respectively. The second addend in this equation, i.e. value:

$$\varepsilon = \frac{E_h}{F_y\, u_y} \tag{3}$$

is often called the normalized hysteretic energy. Numerical value of hysteretic ductility μ_h is equal to the displacement ductility of an elastic-perfectly-plastic (EPP) system under a monotonically increasing lateral deformation which dissipates the same hysteretic energy as the actual system. For the EPP system the damage index, which depends of the hysteretic energy [5], can be defined:

$$DI_h = \frac{E_h}{F_y\,(u_u - u_y)} = \frac{\mu_h - 1}{\mu_u - 1} \tag{4}$$

For a general force-displacement relationship, this damage index can be presented in the following form:

$$DI_h = \frac{E_h}{E_{hu}} \tag{5}$$

where E_{hu} is hysteretic energy capacity of the system under monotonically increasing lateral deformation.

Park and Ang [13] were proposed the damage index as a linear combination of damage caused by maximum inelastic deformation and the cumulative damage resulting from repeated cyclic response:

$$DI_{PA} = \frac{u}{u_u} + \beta \frac{E_h}{F_y\, u_u} = \frac{\mu}{\mu_u} + \beta \frac{E_h}{F_y\, u_y\, \mu_u} \tag{6}$$

where β is a dimensionless constant which depends on the structural properties ($\beta > 0$), while all other marks are explained earlier. Coefficient β represents a parameter that depends on system deterioration. For usual RC structures its value vary from 0.10 to 0.25, with an average value of about $\beta = 0.15$ [5].

Park-Ang model of damage assessment during earthquake is one of the most frequently used damage index. Its widespread usage is the consequence of its foundation on extensive experimental results, and its verification on the basis of studies during which control examinations of structures damaged in several earthquakes were carried out. It should be pointed out that Park-Ang damage index has two deficiencies, which are associated with the physical meaning of damage index. In the elastic response, when $E_h = 0$ and damage index is supposed to be zero, according to expression (6) it follows that the value of DI_{PA} is greater than zero. When the system is exposed to monotonically increasing deformations and when maximum deformation capacity u_u is reached, the value of damage index is supposed to be $DI = 1$. However, according to expression (6) the value that is obtained is greater than one. Although the aberrations in these two cases are relatively small, there still exists a disagreement with the definition of damage index. In that, this index gives insufficiently real estimation of the damage for limit values, especially in its lower limit value.

Recently a few additional damage indexes have been defined. Bozorgina and Bertero [4] have proposed two damage indices in the following form:

$$DI_1 = (1-\alpha_1)\frac{\mu-\mu_e}{\mu-\mu_u} + \alpha_1 \frac{E_h}{E_{hu}} \tag{7}$$

$$DI_2 = (1-\alpha_2)\frac{\mu-\mu_e}{\mu-\mu_u} + \alpha_2 \sqrt{\frac{E_h}{E_{hu}}} \tag{8}$$

where E_{hu} is hysteretic energy under monotonically increasing deformation, α_1 ($0 \le \alpha_1 \le 1$) and α_2 ($0 \le \alpha_2 \le 1$) are constants, and μ_e is the ratio of the maximum elastic portion of deformation (u_e) over yield displacement (u_y):

$$\mu_e = \frac{u_e}{u_y} \tag{9}$$

It is noted that μ_e is unity ($DI = 1$) for inelastic behaviour, and is equal to μ if the response remains elastic. Value of the coefficient α_1 (or α_2) could be determined through regression analyses, i.e. by comparing values of DI_1 (or DI_2) with those of DI_{PA} in the intermediate range of damage index ($0.2 < DI_{PA} < 0.8$) for certain set of ground motions [4]. Damage indexes DI_1 and DI_2 satisfy lower and upper bound conditions, but there are certain difficulties in their practical application since coefficient values (α_1 in DI_1 and α_2 in DI_2) can be defined only for specific ground motions. In addition, a many of non-linear analyses previously need to be performed in order to define values of coefficients α_1 and α_2.

The improved damage index, proposed by Ladjinovic [11], is obtained by modifying the well-known Park-Ang model through eliminating its deficiencies connected with physical meaning of damage index. This damage index is given as a function of its displacement history, hysteretic energy E_h and plastic deformation, as:

$$DI_m = \frac{u-u_y}{u_u-u_y} + \alpha\beta\frac{E_h}{F_y\,(u_u-u_y)} \tag{10}$$

where α is the coefficient used to annul the influence of the hysteretic energy under monotonically increasing deformations. This coefficient is given by the expression:

$$\alpha = 1 - \frac{\mu_c}{\mu_{ac}} \tag{11}$$

where μ_c and μ_{ac} are cyclic and accumulative ductility, respectively. Cyclic ductility μ_c ($\mu_c = u_{c,max} / u_y$) depends on the maximum cyclic displacement demand $u_{c\ max}$ during a ground motion [12]. Accumulative ductility depends on the absolute sum of inelastic displacements $u_{p,i}$ (both positive and negative) during all of the plastic excursions. Accumulative ductility is associated with the history of the repeated cycles of inelastic deformations during earthquake and it depends on the number of plastic excursions. Thus defined damage index depends on ultimate deformation capacity under monotonically increasing deformations, maximum inelastic deformations realized during an earthquake and cumulative effects of the repeated cycles of inelastic deformations.

In case of a large number of the repeated cycles of inelastic deformations, such as in case when the structure is subjected to a ground motion with a long duration, accumulative ductility becomes much greater than the cyclic one, hence the value of coefficient α tends to be one. Then the contribution of hysteretic energy to the total damage is taken in its full amount. In case of small number of repeated cycles of inelastic deformation, such in case of near-fault ground motions, the value of coefficient α can be considerably smaller than one. Due to a small number of repeated cycles, cumulative deformations are relatively small. Therefore, in such case it is reasonable to diminish the influence of hysteretic energy, because the structural damage primarily depends on maximum amplitude of inelastic deformation.

Modified damage index DI_m is formulated in such a way that the limit cases are satisfied. In case of the elastic response of the structure, when $E_h = 0$, the condition $DI_m = 0$ is fulfilled. When the system is exposed to monotonically increasing deformations, and there are no cyclic deformations, the cyclic and accumulative ductility are equal ($\mu_c = \mu_{ac}$) and the coefficient α is zero. In such case, proposed damage index depends only on maximum inelastic deformation and not on hysteretic energy dissipation. When the maximum deformation capacity has been reached ($u = u_u$), the value of damage index becomes $DI_m = 1$. In fact, by comparing expressions (1) and (10), the damage index DI_m results in DI_μ for linear elastic response of the structure and in case of monotonically increasing deformations.

Damage index DI_m can also be shown in a slightly more convenient form, in the function of plastic deformations and normalized hysteretic energy ε, as:

$$DI_m = \frac{\mu_p}{\mu_u - 1}\left(1 + \alpha\beta\frac{\varepsilon}{\mu_p}\right) \tag{12}$$

where μ_p is the plastic ductility ($\mu_p = \mu - 1$). It depends only on plastic deformations u_p ($u_p = u - u_y$), and for linear elastic response it is equal to zero.

DETERMINATION OF DAMAGE SPECTRUM

Damage spectrum represents a variation of a damage index versus structural period for a single-degree-of-freedom (SDOF) system subjected to an earthquake ground motion. The damage spectra can be used for seismic performance evaluations of structures with given properties and can provide information of damage potential of the recorded ground motions.

If the definition of the damage index is given, the damage spectrum can be determined based on the non-linear dynamic response according to the following procedure:

1. Series of SDOF systems is defined with structural period T, damping ratio ξ, strength capacity C_y ($C_y = F_y / W$) and a force-deformation relationship. Yield strength is determined according to seismic codes, depending on the seismic zone and site subsoil condition. The hysteretic model of the structural non-linear behaviour, as well as monotonic ductility capacity μ_u and the value of constant β must also be known.
2. For each period, SDOF system is subjected to an earthquake ground acceleration record and its dynamic response is determined. The structural response can be linear or non-linear depending on the structural period and yield strength capacity. In the non-linear response it is necessary to

calculate the maximum displacement demand, the maximum plastic excursion, the sum of all inelastic displacements reached in all cycles, as well as total hysteretic energy dissipated during ground motion.

3. On the basis of the obtained parameters of structural response, the value of damage index can be calculated using equation (12).

According to the given algorithm, the special FORTRAN program has been made, by which it is possible to determine a damage spectrum. For the illustration of damage spectrum application, a research has been conducted for E1 Centro S00E record and various strength of structures (Fig. 1). The yield strength of the structures is determined according to the elastic spectrum of Eurocode 8 [6] for high seismicity zones (peak ground acceleration $a_g = 0.40g$), subsoil class B and different values of behaviour factor q. For all structures the same structural parameters are used: monotonic ductility capacity $\mu_u = 10$, parameter $\beta = 0.15$, damping $\xi = 5\%$ and stiffness degrading hysteretic model (SD – HM) without hardening ($\kappa = 0$).

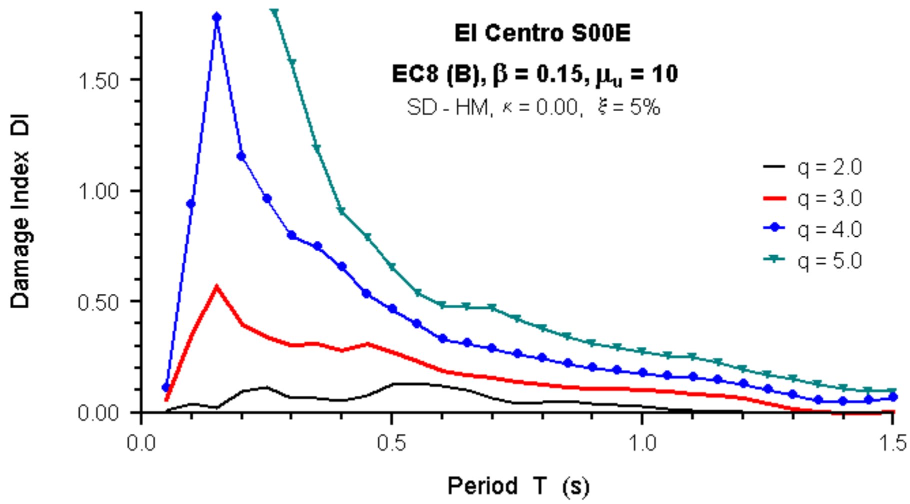

Fig. 1 Damage spectra for different yield strength capacity of the structures

On the basis of the obtained results it is possible to state that the linear decrease of strength leads to a disproportional increase of the structural damage. The structures different in stiffness have different level of damage although their strength capacity is determined by use of the same behaviour (force-reduction) factor. It is also noticeable that damage level of stiff structures is significantly higher. It points out that the current design concept does not provide a uniform risk, thus two different buildings designed according to the same code may experience different levels of damage under a given earthquake.

Unlike E1 Centro record, where the greatest values of the damage index have been obtained for the structures with short periods of vibrations, for Mexico City – SCT N270 record the critical area ($DI > 0.5$) is the range of moderately stiff and flexible structures (Fig. 2). These differences occurred primarily as a consequence of different frequency content of the ground motion. The predominant period of vibrations of accelerogram SCT is considerably longer than E1 Centro S00E record, so in its action the hysteretic energy dissipation is the greatest for moderately stiff and flexible structures [11]. Although for this accelerogram the ductility demand also increases when structural period decreases, for the structure with $T > 1.0$ sec an increase of damage occurs, due to the cumulative effects of inelastic deformations. The number of repeated cycles of inelastic deformations increases if a resonant effect is more expressed. For this accelerogram it happens in the frequency range of moderately stiff and flexible structures.

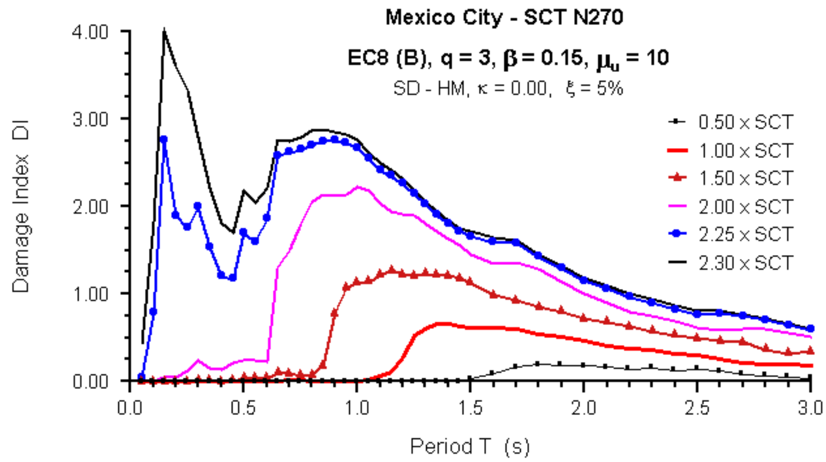

Fig. 2 Damage spectra for different seismic intensity to strength ratio

Although for moderately stiff and flexible structures have been obtained considerably greater values of damage index for Mexico City – SCT record than for usual excitations, the seismic stability of structures were not jeopardized ($DI < 0.5$). However, it should be mentioned that PGA of this accelerogram is only $a_g = 0.171g$, while the yield strength of the structures are determined for considerably higher seismic action ($a_g = 0.40g$) with application of small force-reduction factor ($q = 3$). Also should be taken into account that the relatively great value of available ductility capacity ($\mu_u = 10$) was taken. In order to see the behaviour of structure for different seismic intensity to strength ratio, the damage degree for various values of peak ground acceleration of SCT accelerogram has been analysed (Fig. 2). Based on the obtained results, it can be concluded that the increase of intensity of seismic action leads to damage increase, but that increase is neither proportional to increase of PGA, nor it is equal for the structures different in stiffness. Unfavourable effect of increase of PGA is more expressed for stiffer structures.

DAMAGE BASED DESIGN

Within this paper, researches are directed to formulation and development of the procedures for seismic design that is based on damage evaluation. The proposed procedure, in addition to other design criteria, also includes damage criterion, which allows choosing the structure’s damage degree. It can be used in solving the two basic tasks of the earthquake engineering:

- Determination of the necessary deformation capacity in order that the structure's damage degree after the earthquake is within acceptable limits. Yield strength is known, as well as other structure's parameters (stiffness, damping, structural system, etc.), which are relevant for the inelastic response.
- For the known value of monotonic ultimate deformation capacity, it is necessary to determine yield of the structure that would be sufficient that the damage degree after an earthquake is not greater than the prescribed one.

Although the first problem can be applied in design of the new structures, it is usually used for seismic evaluation of the existing buildings and repair of the damaged structures. Since in these structures the strength and monotonic ductility capacity are known, the problem is reduced to evaluation whether the available structure's ductility is sufficient for satisfaction of the prescribed damage criteria. In such case, it needs to be pointed out that, for the complete insight in seismic resistance of a structure, a number of limit states need to be observed, i.e. it is necessary to check available deformation capacity for seismic actions with different return period. The second problem is usually used to design of the new earthquake-resistant structures. In addition to determination of minimum structure's stiffness, in order to satisfy displacement criteria, it is also necessary to determine a yield strength by which the level of inelastic deformations will be limited.

Ductility demand

In order to obtain a sufficient earthquake resistance of the structure, it is necessary that its available deformation capacity is greater than ductility demand that corresponds to the allowed damage degree. This can be expressed in the following form:

$$\mu_u \geq \mu_r \tag{13}$$

where μ_u is monotonic ductility capacity, μ_r ductility demand including cumulative damage effects, i.e. ductility demand at which the prescribed value of damage index DI is obtained. In further analyses a modified damage index given by the equation (12), is used. It will be presented in the form:

$$DI = \frac{\mu - 1}{\mu_u - 1} F(\varepsilon, \mu) \tag{14}$$

where $F(\varepsilon,\mu)$ is the function:

$$F(\varepsilon, \mu) = \left(1 + \alpha \beta \frac{\varepsilon}{\mu_p}\right) \tag{15}$$

This function depends on parameter β, on maximum plastic deformations during ground motion and on hysteretic dissipation of energy. Function $F(\varepsilon,\mu)$ depends not only on the structure's properties, but also on earthquake characteristics, and it also includes effects of strong-motion duration. This function can be used as an indicator to identify the type of ground motions.

Value of the ductility demand including cumulative damage effects μ_r can be determined on the basis of expression (14), which with a condition ($\mu_u = \mu_r$) leads:

$$\mu_r = 1 + \frac{\mu - 1}{DI} F(\varepsilon, \mu) \tag{16}$$

Ductility demand including cumulative damage effects, besides the ductility demand μ, depends on the function $F(\varepsilon,\mu)$ that is determined on the basis of the other relevant parameters of structural response – equation (15). It is also necessary to know the value of the damage index DI, which depends on the allowed degree of the structural damage (Table 1).

For the illustration of this procedure, a research has been carried out in which has been analysed whether structures with given properties have sufficient deformation capacity to withstand without a collapse the action of E1 Centro S00E, Bar NS, Ulcinj – Hotel Olimpik EW, Mexico City – SCT N270 and Rinaldi RS 228 excitations. The yield strength of the structures is determined according to Eurocode 8 ($a_g = 0.4g$, subsoil class B, and behaviour factor $q = 3$). For all structures the same monotonic ductility capacity ($\mu_u = 8.0$), the same hysteretic model (SD – HM, $\kappa = 0$), the same deterioration coefficient ($\beta = 0.15$) and the same damping ratio ($\xi = 5\%$) have been adopted. Ductility demand including cumulative effects for ultimate limit state ($DI = 1$) is obtained based on the equation (16) and dynamic analysis for each considered structure and each excitation. As illustration, the results obtained for Bar NS record and for structure with T = 0.5 sec, whose normalized strength is $C_y = 0.333$, have been presented. The following parameters of inelastic response were obtained: normalized hysteretic energy $\varepsilon = 15.912$, ductility demand $\mu = 4.269$, cyclic ductility $\mu_c = 5.359$ and accumulative ductility $\mu_{ac} = 55.905$. Based on the equations (11) and (15) the values of parameter $\alpha = 0.904$ and function $F(\varepsilon,\mu) = 1.660$ are obtained. According to (16) the ductility demand including cumulative effects is $\mu_r = 6.427$. Since the available deformation capacity of structure ($\mu_u = 8.0$) is greater than ductility demand at collapse ($\mu_r = 6.427$), it follows that the structure with given properties ($T = 0.5$ sec, $C_y = 0.333$) has sufficient resistance to withstand considered excitation (Bar NS record) without a collapse.

The same method has been applied for other structures and seismic actions as well, and results of performed numerical analyses have been shown comparatively (Fig. 3). Analyses results indicate that the considered structures with given properties (strength capacity according to EC8 and monotonic ductility $\mu_u = 8.0$) have sufficient deformation capacity to withstand action of all excitations without a collapse, except the accelerogram Rinaldi RS 228. Even for this seismic action, which is extremely strong ($a_g = 0.841g$), structures with $T \geq 1.0$ sec will not undergo a failure.

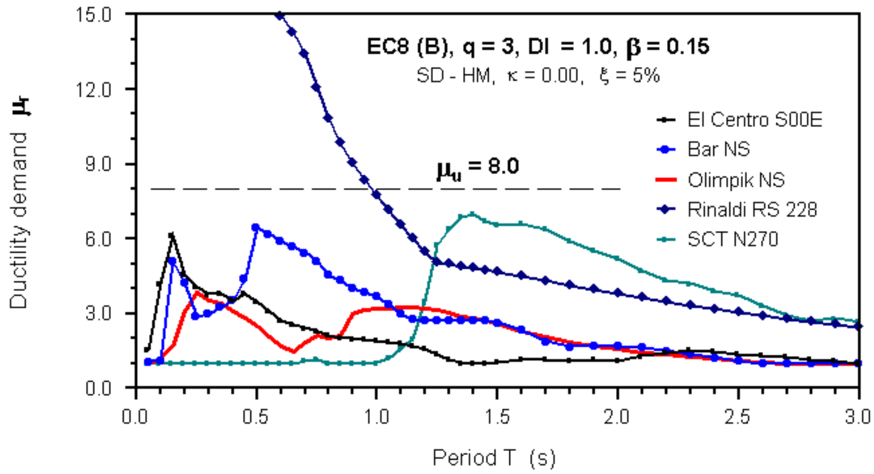

Fig. 3 Ductility demand at collapse during different ground motions

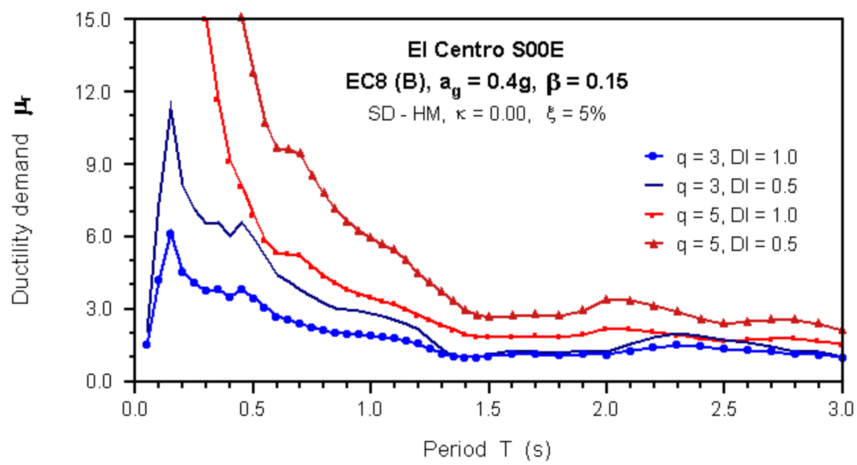

Fig. 4 Ductility demand for different strength capacity and different degree of damage

Calculation results indicate that on ductility demands, including cumulative damage effects, the significant influence have not only the intensity and type of seismic action, but also the structure's stiffness. However, ductility demand, for a specific excitation, essentially depends not only on stiffness, but also on strength capacity and on the allowed damage degree. In order to observe their influence, a dependence of ductility demand including cumulative damage effects from the allowed damage degree and available strength has been analysed. Calculation has been performed for the earlier considered frequency range, for two values of damage index and two levels of strength (Fig. 4). On the basis of the calculation results, it can be stated that all considered parameters have a considerable influence on the deformation demand. Thus, for example, for a stronger structure (q = 3, C_y = 0.333), with period T = 0.5 sec, required ductility capacity at collapse (DI = 1) equals μ_r = 3.074. For a weaker structure (q = 5, C_y = 0.200) and the same limit state, it is obtained μ_r = 6.900. That means that with decrease of strength up to 60% the structure of the given stiffness must have 124.5% greater available deformation capacity to prevent its collapse. Similar effects are also achieved if damage level of structure is intended to be limited. In case of decrease of permitted damage to the expected value DI = 0.5, which corresponds to damage degree for a usual seismic hazard (T_r = 475), the ductility demand μ_r = 5.916 is obtained for the stronger structure. In order to decrease the value of damage index (from DI = 1.0 to DI = 0.5), its available deformation capacity must be increased for 92.5%. Similar relations are also obtained for other structural vibration periods.

Based on the performed analysis, it can be concluded that, for considered range of allowed damage (moderate damage degree – DI = 0.5, and structure's failure – DI = 1.0) and adopted level of strength decrease, the both parameters have essential influence on the deformation demand. For the considered seismic action, with decrease of the strength the greater difference in ductility demand is obtained than in case of decrease of the permitted damage degree. This difference varies depending on frequency range, but influence of the strength capacity of structure, as a rule, is a bit greater.

Strength demand

In design it is necessary to provide such characteristics of the structure that will make possible its favourable response to the expected seismic action. Based on the type of structural system, on the level of axial load and anticipated structural solutions, the global ductility capacity of the structure is adopted. On the basis of relation between the force-reduction factor and the global ductility, the design seismic action could be obtained. With that action the necessary structure's strength is determined and it is checked whether all design criteria given in the codes are met. This calculation method does not provide insight into degree of structural damage under an earthquake whose characteristics correspond to the design seismic action. In that, this kind of concept also does not provide uniform risk, since structures that are designed according to the same codes have a very different level of damage during the same earthquake, what is shown in Fig. 1.

Calculation procedure that makes possible to select the damage degree can be based on determining the strength that will limit the damage of the structure. In that case, the strength demand is determined from the condition that the damage index is equal to the prescribed value:

$$DI \leq DI_p \tag{17}$$

This task is considerably more difficult than the previous one, and cannot be solved directly, but only iteratively. Namely, the value of damage index, which is defined by means of the equation (10), depends on the ductility demand, hysteretic energy dissipation and cumulative effects of inelastic deformations. Taking into consideration that these values directly depend on the strength as well, the equation (17) can be solved only iteratively, through large number of non-linear dynamic analyses. To solve this problem, a special computer program has been developed. Starting from the elastic strength demand the calculation is performed according to the procedure which is applied for determination of damage spectrum, with decrease of strength in small steps until the condition (17) is satisfied.

For illustration of application of the proposed procedure, the results of the analysis in which the strength demands for constant damage are determined. The analysis has been made for six different levels of damage and E1 Centro S00E accelerogram (Fig. 6). Each presented curve gives strength demands at which the structures of different stiffness have the same damage level. If the damage spectrum for the considered seismic action would be determined, with the strength obtained in this way, the damage degree of all structures would be equal, i.e. the constant value of the damage index *DI* would be obtained.

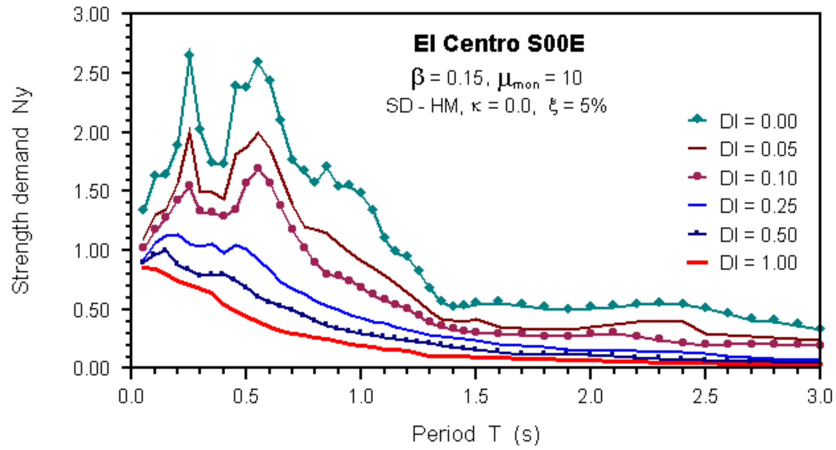

Fig. 5 Constant-damage spectra for El Centro record

With increase of damage index values, smaller strength is simultaneously required, and it depends on the structure's stiffness and the permitted damage degree. Thus, for example, if it is necessary that structure with $T = 0.55$ sec withstands the given action without damage ($DI = 0$), it must have strength capacity $N_y = 2.591$ ($N_y = F_y / ma_g$). Considering that the maximum PGA of E1 Centro S00E accelerogram is $a_g = 0.348g$, the strength of this structure must be at least 90.2% of the structure's weight. For a minor degree of damage ($DI = 0.2$), the required relative strength equals $N_y = 1.076$, while for the moderate damage($DI = 0.5$), which corresponds to the design seismic action, the strength must be $N_y = 0.608$. In order to prevent collapse ($DI = 1.0$) of the structure of the given stiffness, its strength must be at least $N_y = 0.392$ or 13.6% of the total structure's weight.

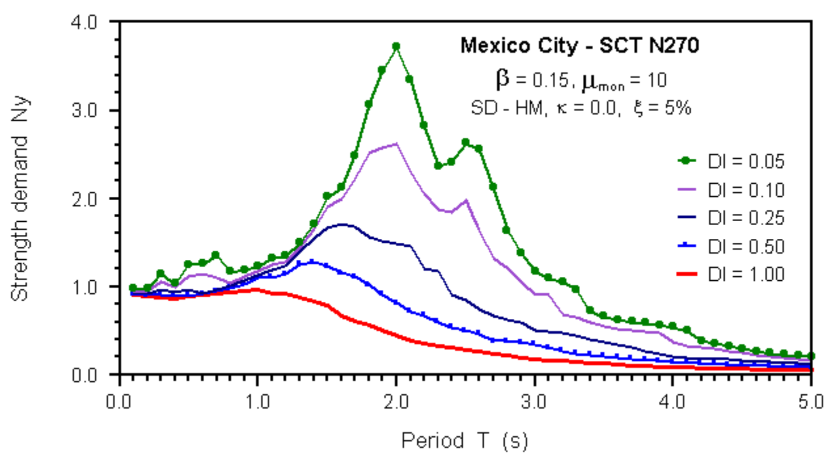

Fig. 7 Constant-damage spectra for Mexico City – SCT record

For E1 Centro S00E record has been obtained a usual shape of strength demand spectrum functions. However, for Mexico City – SCT N270 accelerogram, because of very different frequency content and different strong-motion duration, different shapes of strength demand spectrum functions have been obtained (Fig. 7). For moderately stiff and flexible structures the strength demand significantly depends on the permitted damage degree. That is usual and such a relationship is expected. However, for periods smaller than $T \approx 1.0$ sec, the strength demand to a very small extent depends on the permitted damage level, especially for higher values of the damage index. For example, for $T = 0.6$ sec and damage index values $DI = 0.25$, $DI = 0.5$ and $DI = 1.0$ correspond strength demands $N_y = 0.929$, $N_y = 0.919$ and $N_y = 0.903$ are obtained, respectively. When the strength capacity is decreased for only 2.7%, the structure would be driven from the moderate damage level ($DI = 0.25$) even to the failure ($DI = 1$). The conducted analysis indicates that for the stiff and moderately stiff structures, it is very difficult to provide such characteristics of the structure that will make possible its favourable response to this ground motion. It also indicates the unusually great change of damage degree of rigid structures with given strengths under relatively small change of the seismic action intensity (see Fig. 2).

In order to point out the deficiency of the current concept of seismic design, the analysis has been carried in which were obtained the strength demands for constant damage of structures subjected to the three excitations (E1 Centro S00E, Bar EW and Mexico City – SCT N270). Thus determined strength demands were compared with strength demands according to EC8 for high seismicity zones ($a_g = 0.40g$), soil class B and two values of behaviour factor: $q = 3$ and $q = 5$ (Fig. 8). In the analysis was adopted the damage index value of $DI = 0.5$, which corresponds to the damage degree for design seismic action, i.e. the earthquake with probability of exceedance of 10% in 50 years. For all structures the same value of parameter $\beta = 0.15$ and equal monotonic ductility capacity $\mu_u = 10$ have been adopted. Dynamic analysis has been performed with damping ratio $\xi = 5\%$ and with stiffness degrading hysteretic model without hardening.

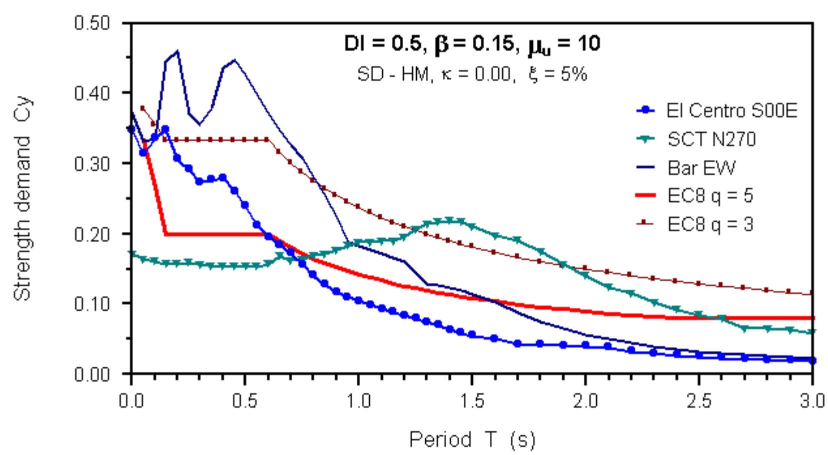

Fig. 8 Strength demand according to EC8 and for equal damage level

Based on the obtained results it can be stated that structures designed according to Eurocode 8 do not have, for all stiffness, sufficient strength that would provide expected damage degree of the structures. That is especially valid in case of application of greater behaviour factor ($q = 5$). The results of the performed analysis indicate that, although Eurocode 8 represents a contemporary design concept of seismic protection [7], it does not always provide sufficient resistance against occurrence of damage within the acceptable limits.

CONCLUSIONS

By application of design (reduced) seismic action and linear elastic model, as it is usual in the engineering practice, it is possible to estimate the real magnitude of seismic forces to which the structure will be exposed during the earthquake ground motions. Although in this way it is possible to determine a real value of seismic forces, it does not mean that strength demand could also be accurately estimated. Strength demand of the structure cannot be determined only by satisfying the criteria that are expressed by forces, but also must satisfy the specific displacements and deformation criteria.

Current design concept is based on reduction of the elastic strength demand. In that way, for the possible real (expected) seismic action, the structure's non-linear response is allowed, which also assumes the occurrence of certain level of damage. Lower design strength generally has favourable effects on the amount of financial investment in construction of the structures. However, that leads to increase of inelastic deformation demands and a greater damage degree of both structural and non-structural elements. Therefore, it is needed to provide minimum yield strength in order to limit the damage degree for moderate earthquakes and adequate deformation capacity to control large inelastic deformations during strong earthquake ground motions. As the ductility demands increase with the decrease of lateral strength, and with regard that measures that can be used to increase ductility are limited, it is necessary to establish corresponding balance between strength, stiffness and ductility.

Based on the obtained results it can be concluded that with increase of seismic action intensity the damage increases, but the damage degree is neither proportional to the increase of peak ground acceleration, nor is equal for structures different in stiffness. With increase of peak ground acceleration, the unfavourable effects are, by a rule, more expressed in a frequency range that is usually considered as a domain of stiff structures. To the very similar effects also leads decrease of the strength capacity – linear decrease of strength can cause a considerable and non-proportional increase of damage degree of structures during strong ground motion.

Structure's behaviour during strong earthquakes can be very different, from very favourable to completely unsuitable which is accompanied with severe damage, and sometimes even with the structure's collapse. It is necessary to point out that the current design concept does not provide an insight into possible damage degree of the structure during earthquake ground motion with characteristics that correspond to the design seismic action. Strength demands depends not only on magnitude of seismic forces, but also on the available ductility, i.e. structure's capability to deform inelastically, and on a number and amplitude of repeated cycles of inelastic deformations. The analysed examples show that, for the structures whose strength was determined by application of the constant force-reduction factor, a different damage degree is obtained. The design based on force-reduction factor results in non-uniform risk, thus different buildings designed according to the same code and with the same force-reduction or ductility factors 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 level of acceptable damage and cumulative effects of inelastic deformations.

REFERENCES

1. Ang AHS. "Reliability bases for seismic safety assessment and design". Proceedings, 4th U.S. National Conference on Earthquake Engineering, EERI, Palm Springs, Vol. 1, 1990: 29-45.

2. Arias. "A measure of earthquake intensity. In Seismic Design for Nuclear Power Plants." R.J. Hansen, Editor, MIT Press.

3. Bolt BA. "Duration of strong motion", Proceedings of the Fourth World Conference on Earthquake Engineering, Santiago, Chile, 1969: 1304-1315.

4. Bozorgnia Y, Bertero VV. "Estimation of damaged potential of recorded earthquake ground motion using structural damage indices". 12th ECEE, London, Paper Ref. 475, Elsevier Science Ltd., 2002: 1-10.

5. Cosenza E, Manfredi G, Ramasco R. "The use of damage functionals in earthquake engineering. Earthquake Engineering and Structural Dynamics", Vol. 22, 1993: 855-868.

6. Eurocode 8. "Design provisions for earthquake resistance of structures". CEN, Roma, ENV 1998-1-1 (May 1994); ENV 1998-1-2 (May 1994); ENV 1998-1-3 (Nov. 1994).

7. Folic R, Ladjinovic Dj, Brujic Z. "Analysis and Design of RC Structures According to EC 8". International Symposium on Earthquake Engineering ISEE 2000, Podgorica, 2000: 355-362.

8. Folic R, Ladjinovic Dj. "Inelastic Demand Spectra for Ground Motions Representing Design Earthquake". 12th ECEE, London, Paper Reference 742, Elsevier Science Ltd., 2002: 1-10.

9. Housner GW. "Spectrum Intensities of Strong-Motion Earthquakes". Symposium on Earthquake and Blast Effects on Structures, 1952.

10. Iwan WD. "Drift spectrum – Measure of Demand for Earthquake Ground Motions". ASCE Journal of Structural Engineering, 1997, Vol. 123, No. 4: 397-404.

11. Ladjinovic Dj. "Damage Based Design of Earthquake Resistant Structure". YuASE, 11th Congress, Vrnjacka Banja, 2002, Vol. 2: 163-168 (in Serbian).

12. Mahin SA, Bertero VV. "An evaluation of inelastic seismic design spectra". ASCE, Journal of Structural Division, 1981, Vol. 107: 1777-1795.

13. Park YJ, Ang AH-S. "Mechanistic seismic damage model for reinforced concrete". ASCE, Journal of Structural Engineering, 1985, Vol. 111, No. 4: 722-739.

14. Powell GH, Allahabadi R. "Seismic damage prediction deterministic methods – concepts and procedures". Earthquake Engineering and Structural Dynamics, 1998, Vol. 16: 719-734.

15. Priestly MJN. "Performance Based Seismic Design". 12th WCEEE, Auckland, 2000, Paper no. 2831.

16. Rathje EM, Abrahamson NA, Bray JD. "Simplified Frequency Content Estimates of Earthquake Ground Motions". Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 1998, Vol. 124, No. 2: 150-159.

17. Trifunac MD, Brady AG. "A Study of the Duration of Strong Ground Motion". Bulletin of the Seismological Society of America, 1975, Vol. 65, No 3: 581-626.

18. Uang C-M, Bertero VV. "Evaluation of seismic energy in structures". Earthquake Engineering and Structural Dynamics, Vol. 19: 77-90.