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SEISMIC PERFORMANCE ASSESMENT BASED ON DAMAGE OF STRUCTURES – PART 1: THEORY*

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Abstract. *The paper presents methodology for safety assessment and design of earthquake resistant structures based on application of damage spectra. The damage spectrum can be used for seismic evaluation of vulnerability of structures with given properties and can provide information of damage potential of the recorded ground motions. Damage spectrum represents a variation of a damage index versus structural period for a single-degree-of-freedom system subjected to an earthquake ground motion. The improved damage index, based on plastic deformation and hysteretic energy dissipation, is applied. It depends on maximal plastic deformation, ductility capacity and function including cumulative damage effects. This function, besides the parameter including influence of deterioration, depends on the history of cyclic deformations and on both cyclic and accumulative ductility.*

Key words: *non-linear analysis, damage index, ductility, hysteretic energy dissipation, cumulative damage effects.*

1. INTRODUCTION

The actual concept of seismic protection is still based on the design of the structure for the so-called designed earthquake action, to which refers the referent return period of the seismic event of $T_r \approx 500$ years [6]. For this seismic event a corresponding calculation for ultimate limit strength of the structure is carried out. The necessary strength capacity of the structure has been determined for effects due to seismic forces corresponding to the given design level. The magnitude of design seismic forces is determined through the 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 local or total collapse, if real earthquake characteristics (peak ground acceleration, frequency content and duration of ground oscillations) correspond to

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the adopted seismic hazard [18]. For the design earthquake action, whose probability of exceedance is relatively small (about 10%), codes prescribe a partial break in the serviceability of the building and occurrence of damage, but local or general collapse of the building must be avoided.

The deficiency of the actual concept of seismic protection is that the force-based design cannot provide an adequate insight into the damage level of the structure [12], while the examining of damage in partition walls and equipment of the building is carried out in a way that is not completely satisfying. Experiences from recently occurred earthquakes indicate that this design approach does not provide a uniform risk [24]. The proclaimed aim of aseismic design and construction is the requirement to prevent human injuries and limit the damage. According to the actual approach of 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 major seismic event [10]. Besides, it is necessary to point out that the usual design level of seismic action ($T_r \approx 500$ years) is not the strongest possible seismic action and that there is a certain probability of occurrence of a stronger earthquake during the lifetime of the structure. However, the actual design concept does not give an insight into the ultimate deformation capacity and safety against the collapse due to seismic action having a smaller probability of occurrence than the design seismic action, i.e. an earthquake with a longer return period than the designed one [11].

2. Ground Shaking and Structural Response

The actual concept of seismic design is based on the reduction of design seismic forces that act onto the structure [20]. Required strength capacity is determined according to the effects of design seismic forces, which are determined by reducing real seismic action through reduction factor [13]. Thus, for the expected seismic action, the non-linear response of the structure is allowed, and according to this a certain damage of the structure. However, the deficiency of this concept in design is that in this way it is not possible to fully control the level of damage [12].

Ground motion during an earthquake is measured by strong motion instruments, which record the ground acceleration. Three orthogonal components of ground accelerations, two in the horizontal direction and one in the vertical, are recorded by the instrument. The characteristics of ground motion that are important in earthquake engineering applications are: 1) peak ground motion (peak ground acceleration, peak ground velocity, and peak ground displacement); 2) duration of strong motion; and 3) frequency content. Each of these parameters influences the response of a structure. Peak ground motion primarily influences the vibration amplitudes. Duration of strong motion has a pronounced effect on the severity of shaking. A ground motion with moderate peak acceleration and a long duration may cause more damage than a ground motion with a larger acceleration and a shorter duration. Frequency content strongly affects the response characteristics of a structure. In a structure, ground motion is amplified the most when the frequency content of the motion and the natural frequencies of the structure are close to each other.

Simplified procedures of the analysis and inadequate input data are often used to assess the level of damage after earthquake ground motion. In practice the intensity of seismic action is usually estimated through analysis of accelerogram, 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 recognized only through analysis of the recorded ground motion data [14]. Hence, in the engineering practice various measures have been introduced to assess damage potential of the ground motions – elastic response spectra, spectrum intensity SI [8], Arias intensity AI [2], drift spectrum [9], RMS acceleration a_{rms}, characteristic intensity I_c [1] etc. Some other parameters have also been defined important for the estimate of earthquake ground motion effects on structures, such as frequency content and predominant period of ground oscillations, mean period T_m [25], duration of strong ground motion t_D [3], [26], etc.

All of the aforementioned variables are based on the analysis of the accelerogram or on using linear-elastic analysis of structure's response. Through the analysis of accelerogram it is not possible to describe well enough the earthquake action on the structure, because it does not only depend on ground motions, but also on the mechanic characteristics of the structure [7]. Some of the mentioned parameters (e.g. spectrum intensity SI and drift spectrum) take into account the structural response to the earthquake action, 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, especially for asymmetric structures, cannot be obtained on the basis of linear analysis [17]. 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.

For a single-degree-of-freedom (SDOF) system, subjected to horizontal earthquake ground acceleration $\ddot{u}_g(t)$, the equation of motion can be written as:

$$m\,\ddot{u}(t) + c\,\dot{u}(t) + f_s[u(t),\dot{u}(t)] = -m\,\ddot{u}_g(t) \tag{1}$$

where m is the mass, c is the viscous damping coefficient, f_s is the restoring force, u is relative displacement of the mass with respect to the ground, and u_g is the earthquake ground displacement. For linear elastic system, the solution of the equation (1) determines the structural response $u(t)$ to the specified seismic ground motion [20]:

$$u(t) = -\frac{1}{\omega_d}\int_0^t \ddot{u}_g(\tau)\,e^{-\xi\omega(t-\tau)}\cdot\sin\omega_d(t-\tau)\,d\tau \tag{2}$$

where ω_d is circular frequency of damped vibration: $\omega_d = \omega\sqrt{(1-\xi^2)}$.

Restoring force for a linear elastic system is proportional to the stiffness of the structure k:

$$f_{se} = k\,u(t) \tag{3}$$

The conventional approach of reducing the seismic forces using a single reduction factor to arrive at the design force level is widely utilised in seismic codes:

$$F_y = f_{se}/R \tag{4}$$

where f_{se} is the elastic strength demand and R is the force reduction factor. In the analysis the adoption of this factor depends on the structural system, materials used and the assumed global behaviour of the system [10].

According to the concept of the current world regulations, the determination of the value of reduction factor is based on the principle of equal displacements of elastic and inelastic (non-linear) systems. Force-based design procedures, which include a final check of deformations, are likely to remain as the primary seismic design method for some time since other design alternatives are still in the development phase. The traditional concept of designing is based on the controlled decrease of structure's strength capacity F_y. Thus for a really possible (expected) seismic action the non-linear response of the structure is allowed, which implies the occurrence of a certain damage level.

Standard occupation structures subjected to severe earthquake ground motion experience deformations beyond the elastic range. To a large extent, the inelastic deformations depend on the intensity of excitation and load-deformation characteristics of the structure and often result in stiffness deterioration. Because of the cyclic characteristics of ground motion, structures experience successive loading and unloading and the force-displacement or resistance-deformation relationship follows a sequence of loops known as hysteresis loops. The loops reflect a measure of a structure's capacity to dissipate energy. The shape and orientation of the hysteresis loops depend primarily on the structural stiffness and yield displacement. Factors such as structural material, structural system and detailing of the structure in general and of critical regions or structural elements in particular, influence the hysteretic behaviour. The measure of inelastic response is displacement ductility demand μ. It depends on maximal inelastic deformation:

$$\mu = u / u_y \tag{5}$$

where u is the maximum inelastic displacement during a ground motion and u_y is the yield displacement. 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. [15]. 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 [14].

The non-linear behaviour of the structure during real earthquakes can also be perceived through energy balance [27]. Energy is imparted to a structure subjected to earthquake ground motion. A part of this energy is stored in the structure in the form of kinetic and strain energy, and the rest is dissipated by damping and hysteretic behaviour in the structure during the ground motion. The various energy terms are defined by integrating the equation of motion, Eq. (1), with respect to u, as:

$$\int_0^u m\ddot{u}\,du + \int_0^u c\dot{u}\,du + \int_0^u f_s(u,\dot{u})\,du = -\int_0^u m\ddot{u}_g\,du \tag{6}$$

The right-hand side of Eq. (6) is the total energy input to the structure E_i:

$$E_i(t) = -\int_0^u m\ddot{u}_g\,du \tag{7}$$

The first term on the left-hand side of Eq. (6) is the kinetic energy E_k of the mass associated with its motion relative to the ground:

$$E_k(t) - \int_0^u m\ddot{u}\,du = m\dot{u}^2/2 \tag{8}$$

The second term on the left-hand side of Eq. (6) is the energy dissipated by viscous damping E_d:

$$E_d(t) = \int_0^u c\dot{u}\,du = \int_0^t c\dot{u}^2(\tau)\,d\tau \tag{9}$$

The third term on the left-hand side of Eq. (6) is the sum of the energy dissipated by yielding (hysteretic energy) and the recoverable strain energy of the system E_s:

$$E_s(t) = \frac{|f_s(t)|^2}{2k} \tag{10}$$

where k is the initial stiffness of the system. Thus the hysteretic energy E_h can be expressed as:

$$E_h(t) = \int_0^u f_s(u,\dot{u})\,du - E_s(t) \tag{11}$$

Based on these energy quantities, Eq. (6) can be rewritten as:

$$E_k(t) + E_d(t) + E_s(t) + E_h(t) = E_i(t) \tag{12}$$

Equation (12) describes a dynamic system with the energy balance as opposed to the force balance in Eq. (1) from which Eq. (12) is derived. Equation (1) defines the instant deformation response through the time history with the interest in the maximum values occurring some time during the vibration. This is the traditional approach, assuming the maximum response is responsible for structural failure. Equation (12), on the other hand, specifies the cumulative effect of the ground shaking on the structure [16]. The kinetic energy E_k and elastic strain energy E_s are related to instant response of the system. They are very small compared with E_d and E_h, which are cumulative over time during a strong excitation, and vanish at the end of the vibration in an inelastic system [10].

Equation (12) emphasizes the cumulative nature of the seismic response, assuming the inelastic performance of the structure. It also implies that the energy approach does not have practical application if the structure remains elastic when E_h vanishes and E_d becomes the dominant contributor to dissipate the input energy. Excessive elastic force has to appear to help balance the input energy. This is because the strain energy in an elastic system is determined by elastic strain energy $E_s(t)$. Without $E_h(t)$ in Eq. (11), $E_s(t)$ has to take a large portion of input energy, causing significant increase in elastic force $f_s(t)$.

Hysteretic energy E_h is a measure of inelastic energy dissipation caused by the earthquake action on the structure. Hysteretic energy dissipation depends on maximum inelastic deformations, accumulation of inelastic deformations and on earthquake duration. During the elastic response of the structure, hysteretic energy equals zero by definition.

3. Estimation of Structural Damage

3.1. Definition of damage index

Although ductility demand and amount of dissipated hysteretic energy are important 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. The degree 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:

$$DI = D / C \tag{13}$$

where *D* is the maximum inelastic response quantity (e.g. displacement, curvature, etc.) during a ground motion and *C* is capacity of the structure. 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, 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 [22]. 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

For the values of damage index $DI < 0{,}2$ the structure is exposed to minor damage, which is accompanied by visible cracks in structural elements and partition walls of the building. For this degree of damage there is no delay in the functioning of the facility, and rehabilitation of structures is relatively easy to implement. Damage index between 0,2 and 0,5 corresponds to a moderate degree of damage, which is accompanied with the appearance of reinforcement yielding in critical regions of some structural elements, including spalling of the concrete cover, as well as the formation of larger cracks in plastic hinge zones. The value of damage index $DI = 0{,}5$ is usually considered as the boundary between moderate and severe degrees of damage, i.e. the boundary between the damage that can be repaired and the damage that is irreparable or the cost of their repairs is economically unacceptable. For severe damage ($DI > 0{,}5$) at the critical sections of structural elements comes to the appearance of concrete crushing and local buckling of longitudinal bars in plastic hinge zones. The failure of the structure corresponds to the value of damage index $DI \geq 1$. It is usually accompanied by loss of shear and/or axial load bearing capacity of some structural elements, resulting in partial or complete collapse of the building.

3.2. Review of most commonly used damage indices

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 u_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 u_{mon}, has been used as the available deformation capacity during the earthquake motion (Fig. 1).

Structural damage indices usually consider a measure of the deformation demands in the structure. While some consider the maximum deformation demand, others take into account the cumulative plastic deformation demands.

Damage index based on plastic deformation under monotonically increasing lateral deformation was proposed by Powell and Allahabadi [23]:

$$DI_{\mu} = \frac{u - u_y}{u_u - u_y} = \frac{\mu - 1}{\mu_u - 1} \tag{14}$$

where u is the maximum inelastic displacement during a ground motion, u_y is the yield displacement and u_u is an ultimate displacement capacity of the system under a monotonically increasing lateral deformation. In equation (14), μ is the maximum ductility demand during an earthquake ($\mu = u / u_y$), while μ_u is the monotonic ductility capacity, defined as:

$$\mu_u = u_u / u_y \tag{15}$$

Damage index DI_{μ} is based on displacement ductility, i.e. on the maximum inelastic deformation. It gives an accurate value of damage due to static unidirectional load (Fig. 1), but displacement ductility itself does not reveal information on the repeated cycles of inelastic deformation and energy dissipation demand during an earthquake.

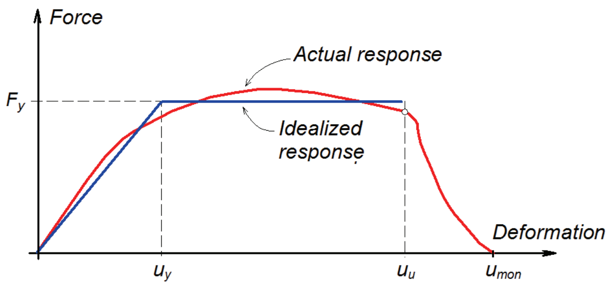

Fig. 1. Force-displacement relationship under monotonically increasing deformation

During an earthquake ground motion a number of the repeated cycles of inelastic deformations occur, so the other structural response parameters are 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 [21] defined normalized hysteretic energy ductility μ_h:

$$\mu_h = 1 + \frac{E_h}{F_y\, u_y} \tag{16}$$

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

$$\varepsilon = \frac{E_h}{F_y\, u_y} \tag{17}$$

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 that 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:

$$DI_h = \frac{E_h}{F_y\,(u_u - u_y)} = \frac{\mu_h - 1}{\mu_u - 1} \tag{18}$$

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

$$DI_h = E_h / E_{hu} \tag{19}$$

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

Park and Ang [22] were proposed the damage index as a linear combination of damage caused by the maximum inelastic deformation and by the cumulative damage resulting from repeated cyclic response:

$$DI_{PA} = \frac{u}{u_u} + \beta \frac{E_h}{F_y\, u_u} = \frac{\mu}{\mu_u} + \beta \frac{E_h}{F_y\, u_y\, \mu_u} \tag{20}$$

where β ($\beta > 0$) is a dimensionless constant that depends on the structural properties, while all other terms 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 (20) 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 equation (20) 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 indices have been defined. Bozorgina and Bertero [4] have proposed two damage indices in the following form:

$$DI_1 = (1-\alpha_1)\frac{\mu-\mu_e}{\mu-\mu_u} + \alpha_1 \frac{E_h}{E_{hu}} \tag{21}$$

$$DI_2 = (1-\alpha_2)\frac{\mu-\mu_e}{\mu-\mu_u} + \alpha_2 \sqrt{\frac{E_h}{E_{hu}}} \tag{22}$$

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):

$$\mu_e = u_e / u_y \tag{23}$$

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. Damage indices 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 Lađinović [19], is obtained by modifying the well-known Park-Ang model through eliminating its deficiencies associated 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:

$$DI_m = \frac{u-u_y}{u_u-u_y} + \alpha\beta \frac{E_h}{F_y\,(u_u-u_y)} \tag{24}$$

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:

$$\alpha = 1 - \mu_c / \mu_{ac} \tag{25}$$

where μ_c denotes cyclic ductility:

$$\mu_c = u_{c,max} / u_y \tag{26}$$

and μ_{ac} accumulative ductility:

$$\mu_{ac} = 1 + \frac{\sum_i |u^+_{p,i}| + \sum_i |u^-_{p,i}|}{u_y} \tag{27}$$

Cyclic ductility depends on maximum plastic excursion $u_{c,max}$ independent on sign of displacement, while accumulative ductility depends on the sum of inelastic displacements $u_{p,i}$ (both positive and negative) during all of the plastic excursions (Fig. 2). Accumulative ductility is associated with the history of cyclic deformations during earthquake and it depends on the number of plastic excursions.

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 [16], 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.

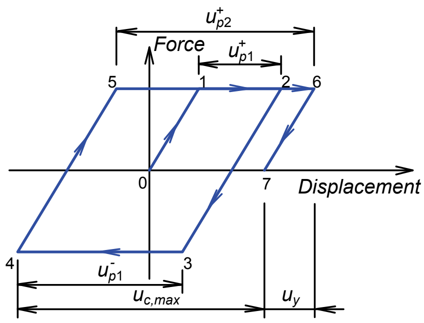

Fig. 2. The history of cyclic deformations during earthquake

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 (24) and (14), 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 energyε, as:

$$DI_m = \frac{\mu_p}{\mu_u - 1}\left(1 + \alpha\,\beta\frac{\varepsilon}{\mu_p}\right) = \frac{\mu_p}{\mu_u - 1}F(\varepsilon,\mu) \tag{28}$$

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.

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 earthquake ground motions (e.g. ordinary, long duration or impulsive earthquakes).

4. CONCLUSION

Seismic performance of structure, i.e. behaviour of structure during the earthquake and its degree of damage, can be quantified by using damage index. It provides information on the state of structure after the earthquake, and on the basis of its value it is possible to estimate the building damage caused by considered seismic action. The value of this index is determined on the basis of various parameters of structural response caused by the earthquake action on the structure, such as the magnitude of induced seismic forces, the maximum inelastic deformation, earthquake duration, the inelastic energy dissipation, the history of cyclic deformations and number of plastic excursions during earthquake ground motion.

For any definition of damage index it is possible to develop the damage spectrum on the basis of the structure's non-linear dynamic response. The procedure for determining the damage spectrum will be presented in the second part of this paper. 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 spectrum can be used for seismic performance evaluations of structures with given properties and can provide information of damage potential of the recorded ground motions.

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PROCENA SEIZMIČKIH PERFORMANSI NA OSNOVU OŠTEĆENJA KONSTRUKCIJA – DEO 1: TEORIJA

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U radu je prikazana metodologija za procenu sigurnosti i projektovanje seizmički otpornih konstrukcija na osnovu primene spekatara oštećenja. Na osnovu spektra oštećenja se može proceniti povredljivost konstrukcija datih svojstava i razorni potencijal razmatrane seizmičke pobude. Spektar oštećenja predstavlja promenu indeksa oštećenja sistema sa jednim stepenom slobode, koji su izloženi dejstvu nekog specifičnog zemljotresa, u funkciji prigušenja i perioda vibracija. Prikazan je poboljšani indeks oštećenja koji je zasnovan na plastičnoj deformaciji i histereznoj disipaciji energije. Njegova vrednost zavisi od maksimalne plastične deformacije, kapaciteta deformisanja konstrukcije i funkcije koja obuhvata kumulativne efekte oštećenja. Ova funkcija, pored parametara kojima je obuhvaćen uticaj deterioracije, zavisi i od istorije cikličnih deformacija, kao i od ciklične i akumulurane duktilnosti.

Key words: *nelinearna analiza, indeks oštećenja, duktilnost, disipacija histerezisne energije, kumulativni efekti oštećenja.*