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THE DEVELOPMENT OF CONTROLLED DAMAGE MECHANISMS-BASED DESIGN METHOD FOR NONLINEAR STATIC PUSHOVER ANALYSIS

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Abstract. *This paper presents the original method of controlled building damage mechanisms based on Nonlinear Static Pushover Analysis (NSPA-DMBD). The optimal building damage mechanism is determined based on the solution of the Capacity Design Method (CDM), and the response of the building is considered in incremental situations. The development of damage mechanism of a system in such incremental situations is being controlled on the strain level, examining the relationship of current and limit strains in concrete and reinforcement steel. Since the procedure of the system damage mechanism analysis according to the NSPA-DMBD method is being iteratively implemented and designing checked after the strain reaches the limit, for this analysis a term Iterative-Interactive Design (IID) has been introduced. By selecting, monitoring and controlling the optimal damage mechanism of the system and by developed NSPA-DMBD method, damage mechanism of the building is being controlled and the level of resistance to an early collapse is being increased.*

Key words: *damage mechanisms, strains, Capacity Design Method, Nonlinear Static Pushover Analysis, Iterative-Interactive Design*

1. INTRODUCTION

The classic engineering approach to the analysis of buildings, in case of an earthquake, is based on the application of Equivalent Static Method (ESM) or Spectral-Modal Analysis (SMA). The determination of internal forces in columns and beams is carried out by Linear Static Analysis (LSA) according to the Finite Element Method (FEM). Application of LSA-SMA method shows good results in practice, but there are a lot of questions that are hard to answer, such as: the question of the real deformation level on the global and local level

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for the target displacement, the evaluation of the structural capacity from the target displacement level to the initiation of collapse, the development of fracture mechanism and initiation of the collapse phenomenon of the structure. The answers to these questions belong to the domain of Performance-Based Seismic Design (PBSD) of structures within the contemporary Performance-Based Earthquake Engineering (PBEE) methodology. Development of damage, fracture and collapse mechanisms of the building can be carried out by analyzing the structural system with or without introducing additional elements. If we do not introduce additional elements to the structural system, then, in most cases, damage mechanism is only being estimated, not actively controlled. If we introduce additional elements such as dampers, base isolation, special visco-elastic elements (shape memory alloy) and the elements for the concentrated dissipation of hysteretic energy (knee), then it is possible, in certain cases, to actively control damage and fracture mechanism of the system. The research presented in this paper is based on the principle of controlling the damage and fracture mechanism of the system, without the introduction of additional elements to the structural system and it is being implemented in four key steps: identification of optimal mechanism of the building damage and fracture according to the Capacity Design Method (CDM), calculation of the nonlinear building response using the Nonlinear Static Pushover Analysis (NSPA) within the capacity domain, control by comparison of optimal and realized mechanism of the building damage (DMBD - Damage Mechanisms-Based Design) and redesigning a system. By connecting the CDM method, the NSPA analysis and concept of structure designing according to the damage mechanisms a new method has been developed. The title of a new method is the following: "Nonlinear Static Pushover Analysis - Damage Mechanisms-Based Design (NSPA-DMBD)".

2. Summary of Existing Analysis of Mechanisms of Damage, Fracture and Collapse of Buildings

Assessment of the level of damage, analysis of the fracture mechanism development, evaluation and analysis of the collapse of the building according to the PBSD are being carried out by applying: simplified analysis, analytical procedures, energy criteria, damage index, the calculation of system performances using Performance-Based Plastic Design (PBPD), the fragility curves, numerical one step solution analysis, incremental-iterative analysis, the NSPA analysis, the NDA analysis and the Incremental Nonlinear Dynamic Analysis (INDA). The assessment of the damage of the building for a given level of seismic intensity is analyzed through the global damage index [1], [2]. Indicators of the damage can be classified into three groups: noncumulative, cumulative and combined. Global damage indices are classified into two groups: the average weighted indices [3], [4] and the indices that are determined based on the modal parameters [5]. Enhanced damage index of a 2D model building is presented in [6], while [7] shows the damage index of 3D models of buildings. Control of the collapse of the structure has been considered as the storey-safety factor [8] and the drift concentration factor [9], while the assessment of the level of damage based on the residual seismic capacity is being discussed in [10]. Classes of damage of the structure are defined according to [11]. As opposed to consideration of the system behavior during the nonlinear response, the capacity assessment of a structure and analysis of the damage, the performances of the system using the damage response spectra has been analyzed in [12]. Research on controlled mechanism of system damages, using the calculation of structures according to the Force-Based Design (FBD) has been presented in [13]. By applying the energy concept and the yield point spectra method the building damage functions are being discussed through: displacement modification factor, yield strength reduction factor, seismic energy response parameter, damage index and equivalent ductility ratio, [14]. The consideration of the possibility of developing different fracture mechanisms of moment resisting frame systems (MRF), from the storey to combined mechanism, is shown in [15], while in [16] the identification of collapse fracture mechanism system has been executed based on the collapse capacity spectra. The paper [17] shows examples of buildings where the earthquake formed the storey (local) fracture mechanisms with and without a total collapse. The methodology concerning the calculation of building structures using PBPD analysis has been shown in detail in the book [18] based on the adopted places for potential yielding in case of an earthquake. In a study [19] the initiation of the building collapse mechanism and the global dynamic instability of the system is considered as a function of an inter-storey drift and spectral acceleration using INDA analysis. Also, a number of studies on the damage mechanisms and building fractures using the INDA analysis in the capacity domain and the fragility curves are presented in [20], [21], [22], [23], while in [24], using the NDA analysis, a storey quasi-shear fracture mechanism of a 3D model building framework has been identified. Comparative NSPA and INDA analysis of the pre-collapse and collapse state of the framework buildings has been presented in [25], where the infinum of collapse initiation for a static criterion is being defined, as well as the supremum of collapse initiation for dynamic criteria. In comparison to the numerical models of buildings and the analysis using a FEM method, the aspects of numerical modeling by Applied Element Method (AEM) have been presented in [26]. AEM method can very well simulate the mechanisms of damage and fracture, as well as the initiation of pre-collapse domain and a phenomenon of the building collapse.

3. The Concept of the CDM Method in Nonlinear Analysis of Buildings

The concept of the CDM method in the NSPA analysis of framework buildings is developed in order to analyze the incremental development of the damage mechanism, inspection and correction of reinforced concrete sections. The CDM is a method in which the structural elements are selected that are correspondingly designed and shaped in order to dissipate energy at large deformations, while the other structural elements are provided with sufficient load capacity, so the selected type of energy dissipation can be achieved [27]. The main idea of this method is to pre-select the elements and modes of inelastic deformations that can form mechanisms with high-energy storage capacity and that do not compromise the vertical strength of the structure [28]. These elements are deliberately designed so that they are the first to yield, while at the same time, the details of their performance are such that they allow greater dissipation of energy. Other elements are provided with a sufficient strength so that the programmed mechanisms of absorption can be fully accepted. When multi-storey frame buildings are concerned, the areas of beam-column joints, i.e. the ends of the beams and columns are the places where the plastic deformations are developed, and these places should be kept in mind during the analysis of damage and fracture mechanisms. The columns are "stronger" elements and whose collapse could threaten the global stability of frame buildings. Unlike columns, beams are, "weaker" elements in which the strength reduction may threaten global stability of frame buildings to a lesser extent, and for which it is possible to provide sufficient ductility. By applying the concept of the CDM method for the analysis of multi-storey frame buildings, ideal plastic mechanism would be to develop nonlinear plastic deformations at the ends of beams and only at the ends of columns at the supports. It is possible to ensure the development of beam collapse mechanism within the frame buildings, following the principles of the CDM method, according to EC 8 [27], if the ratio of the bending moments at each node is the following:

$$\sum M_{Rc} \geq 1.3 \sum M_{Rb} \ , \tag{1}$$

where ΣM_{Rc} is the sum of calculated values of bending moments of columns and ΣM_{Rb} is the sum of calculated values of bending moments of beams connected to the node.

Preliminary research of 2D framework models has been conducted in order to present and consider the problem of the CDM methods using the NSPA analysis. As a representative, a 10-storey 4-bay reinforced concrete frame model with different options for developing the damage mechanisms has been adopted: I model - plastic hinges placed at the ends of all members to simulate a general damage mechanism, II model - plastic hinges placed at the ends of beams and columns at the supports to simulate the optimal damage mechanism, III model - "strong" columns and "weak" beams and IV model - "strong" beams and "weak" columns. For pre-defined numerical models the NSPA analyses were conducted by controlling the incremental displacement using SAP 2000 [29], and NSPA pushover curves were developed (Figure 1). Generally, the worst case is obtained for the model IV, since only the formation of plastic hinges in columns is permitted and the lowest ductility level is achieved, while the strength is almost the same as for the model I. By analyzing the capacity, stiffness and ductility, much better solution is obtained for models II and III than for models I and IV. In model II, where plastic hinges are set according to the CDM method, a greater strength is achieved in the nonlinear domain, stiffness is positive and ductility is significantly increased, almost doubled comparing with the model I. If we allow only the development of plastic hinges on beams, as is the case with the model III, it is possible to achieve the largest strength in the nonlinear domain. On the other hand, the development of the damage mechanism in such a model would not be sufficiently exhausted as you may additionally allow the formation of plastic hinges at the ends of columns at supports, without jeopardizing the global stability.

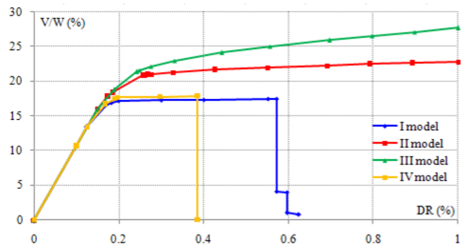

Fig. 1 Developed NSPA pushover curves for all analyzed models of mechanisms

In the next step, applying the Capacity Spectrum Method (CSM) according to ATC 40 [30] the levels of target displacements have been determined in ADRS format (Figure 2). Comparing the levels of target displacement, we can say that they are almost identical, but the difference in the relative value of the total shear forces at the base of a structure (*V/W*) is significant. Models of the mechanisms I and IV have significantly lower values of total shear force at the base of the structure, and models of the mechanisms II and III have significantly higher values, indicating that the models are much stiffer and attract larger seismic forces. Larger seismic forces require larger cross-sections, i.e. correction of the necessary reinforcement. The increase in total shear force at the base of the structure for the model of mechanism II, compared to the model of mechanism I, amounts to 15%.

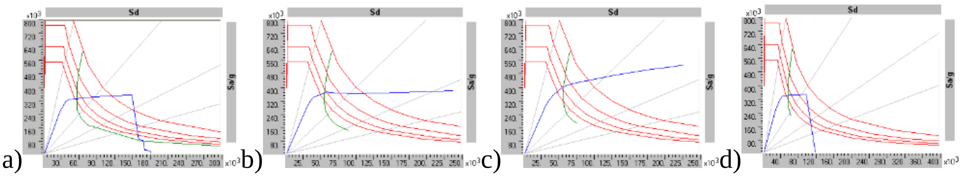

Fig. 2 Pushover curves in ADRS format for model: a) I, b) II, c) III, d) IV

After each incremental situation at different levels of drift, it is possible to analyze the number of formed plastic hinges N_{ph} for inter-performance levels from B to E, according to FEMA 356 [31]. Dominant state of nonlinear deformations in the plastic hinges is B-IO, followed by a IO-LS and LS-CP, provided that in the case of damage mechanism of models I and IV plastic hinges are formed for performance levels D-E and >E up to a maximum value of global drift *DR*=1%, for which the monitoring was carried out (Figure 3). The development of plastic hinges in damage mechanism models I and IV for performance levels D-E and >E, indicates a reduction in strength at significantly lower values of the realized maximum of global structural drift. Using the CDM method with controlled damage mechanism, on model II, we avoid the premature fracture on some parts of the beams and columns, and thus the premature collapse of the structure.

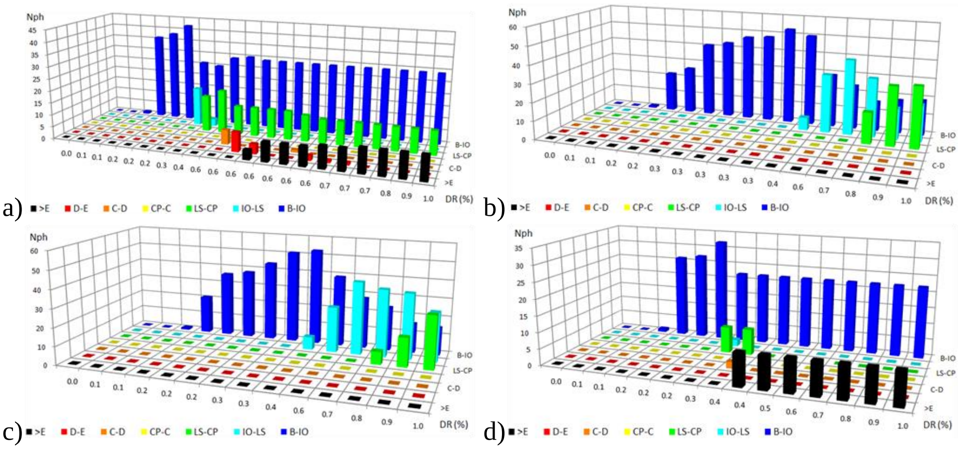

Fig. 3 Number and distribution of formed plastic hinges at different levels of drift for 10-storey MRF building, depending on performance levels: a) I, b) II, c) III, d) IV

4. Performance of Frame Buildings According to the NSPA-DMBD Method

The mathematical formulation of the problem of NSPA-DMBD method, originally developed in this research, is based on the balance A_e=A_i of external work A_e and work of internal elements of the energy dissipation. In general, the work of the external load is the work of volume F and surface forces p during the virtual displacements [32]:

$$A_e = \int_V FdudV + \int_S pdudS \, . \tag{2}$$

The expression (2), in case the system is composed of beams only, can be written as:

$$A_e = \int_S (p_t\xi + p_n\eta)ds + \sum_i P_i\delta_i + \sum_i M_i\beta_i \, , \tag{3}$$

where p_t and p_n represent the components of distributed load in directions of the tangent and the normal, ξ and η are the corresponding virtual displacements in directions of the tangent and the normal, P_i and M_i concentrated forces and bending moments, δ_i and β_i suitable virtual displacement and rotation. Since the analysis of the system is reduced to the effect of lateral concentrated seismic forces and the virtual displacement in the horizontal direction due to the effect of these forces is being considered, the effect of external forces on the displacement (3) becomes:

$$A_e = \sum_i P_i\delta_i \, . \tag{4}$$

In general, the internal virtual work in order to dissipate energy is presented as the work of stresses σ during the virtual strains ε:

$$A_i = \int_V \sigma d\varepsilon dV \, , \tag{5}$$

while the expression (5) in case the system is composed of beams, and per unit of the beam's length can be written as:

$$A_i = N\varepsilon + M\kappa + T\gamma \, , \tag{6}$$

where N is the normal force, M bending moment, T shear force, κ curvature, γ shear strain. In the case that consideration is conducted on the system with the development of concentrated plastic deformations (plastic hinges), then the virtual work of external horizontal concentrated seismic forces P_i on the displacement Δ_i, can be written as:

$$A_e = \sum_{i=1}^{n} P_i\Delta_i = P_1\Delta_1 + \ldots + P_i\Delta_i + \ldots + P_n\Delta_n \, , \tag{7}$$

where $\Delta_i = ih_i\theta$, tg$\theta \approx \theta$, i=1,…n. ih_i is the position (ordinate) of the horizontal concentrated seismic force P_i and θ is rotation of the column at the support. The virtual work of internal forces on the rotation, which comes due to yielding of the columns is:

$$A_{i,c} = \sum_{i=1}^{n} M_c\theta_i = M_{c,1}\theta_1 + \ldots + M_{c,i}\theta_i + \ldots + M_{c,n}\theta_n \, , \tag{8}$$

while the virtual work of internal forces which comes due to yielding of the beams is:

$$A_{i,b} = \sum_{i=1}^{n} M_b \phi_i = M_{b,1}\phi_1 + \ldots + M_{b,i}\phi_i + \ldots + M_{b,n}\phi_n \, . \tag{9}$$

The total virtual work of external and internal forces on virtual displacements is:

$$\theta_{i=1} \sum_{i=1}^{j} P_i i h_i = \sum_{i=1}^{k} M_{c,i}\theta_i + \sum_{i=1}^{l} M_{b,i}\phi_i \, , \tag{10}$$

where j is the number of storeys in the 2D framework model building, k is the number of supports, l double number of beams (two plastic hinges form at each beam) (Figure 4). In case all storeys are of the same height, then the (10) becomes:

$$h\theta_{i=1} \sum_{i=1}^{j} iP_i = \sum_{i=1}^{k} M_{c,i}\theta_i + \sum_{i=1}^{l} M_{b,i}\phi_i \, . \tag{11}$$

Expression (10) is derived for the general case of development of the plastic hinges of columns and beams of different cross-sections and different amounts and distribution of reinforcement. If the analysis is conducted on the system with the propagation of plastic deformations along the beams and through the incremental situation (i), and using the incremental-iterative procedure, then, the work of external horizontal concentrated seismic forces P_i on the displacement Δ_i can be written as:

$$A_e = \sum_{i=1}^{n} P_i^{(i)} \Delta_i^{(i)} = P_1^{(i)} \Delta_1^{(i)} + \ldots + P_i^{(i)} \Delta_i^{(i)} + \ldots + P_n^{(i)} \Delta_n^{(i)} \, , \qquad \Delta_i^{(i)} = i h_i^{(i)} \theta^{(i)} \, . \tag{12}$$

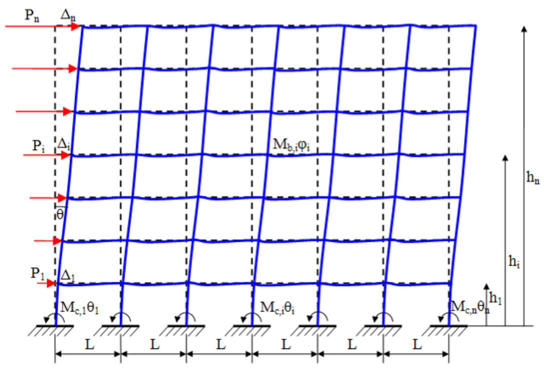

Fig. 4 MRF model of building with yielding of members

The work of internal forces originating from the yielding in columns and beams are:

$$A_{i,c} = \sum_{i=1}^{m} \int_S (N_c^{(i)} \varepsilon_c^{(i)} + M_c^{(i)} \kappa_c^{(i)} + T_c^{(i)} \gamma_c^{(i)}) dS \, , \quad A_{i,b} = \sum_{i=1}^{n} \int_S (N_b^{(i)} \varepsilon_b^{(i)} + M_b^{(i)} \kappa_b^{(i)} + T_b^{(i)} \gamma_b^{(i)}) dS \, , \tag{13}$$

where m is the number of columns, n the number of beams of the building. The total work of external and internal forces on virtual displacements, reads as follows:

$$\sum_{i=1}^{j} P_i^{(i)} i h_i^{(i)} \theta_{i=1}^{(i)} = \sum_{i=1}^{m} \int_S (N_c^{(i)} \varepsilon_c^{(i)} + M_c^{(i)} \kappa_c^{(i)} + T_c^{(i)} \gamma_c^{(i)}) dS + \sum_{i=1}^{n} \int_S (N_b^{(i)} \varepsilon_b^{(i)} + M_b^{(i)} \kappa_b^{(i)} + T_b^{(i)} \gamma_b^{(i)}) dS \,. \quad (14)$$

Since the consideration of the system response is conducted in incremental situations, for the material nonlinear constitutive model of concrete and reinforcing steel the normal forces, bending moments and shear forces can be expressed as:

$$N_{c,i}^{(i)} = \int_{F_c^{(i)}} \sigma_{c,i}^{(i)} dF_c \,, \quad M_{c,i}^{(i)} = \int_{F_c^{(i)}} \sigma_{c,i}^{(i)} z_{c,i}^{(i)} dF_c \,, \quad T_{c,i}^{(i)} = \int_{F_c^{(i)}} \tau_{c,i}^{(i)} dF_c \,, \quad (15)$$

$$N_{b,i}^{(i)} = \int_{F_b^{(i)}} \sigma_{b,i}^{(i)} dF_b \,, \quad M_{b,i}^{(i)} = \int_{F_b^{(i)}} \sigma_{b,i}^{(i)} z_{b,i}^{(i)} dF_b \,, \quad T_{b,i}^{(i)} = \int_{F_b^{(i)}} \tau_{b,i}^{(i)} dF_b \,, \quad (16)$$

where τ is a shear stress. By substituting the expressions (15) and (16) into (14) we obtain the expression for the external and the internal work:

$$\sum_{i=1}^{j} P_i^{(i)} i h_i^{(i)} \theta_{i=1}^{(i)} = \left[\sum_{i=1}^{m} \int_S \left(\varepsilon_c^{(i)} \int_{F_c^{(i)}} \sigma_{c,i}^{(i)} dF_c + \kappa_c^{(i)} \int_{F_c^{(i)}} \sigma_{c,i}^{(i)} z_{c,i}^{(i)} dF_c + \gamma_c^{(i)} \int_{F_c^{(i)}} \tau_{c,i}^{(i)} dF_c \right) dS \right] +$$
$$+ \left[\sum_{i=1}^{n} \int_S \left(\varepsilon_b^{(i)} \int_{F_b^{(i)}} \sigma_{b,i}^{(i)} dF_b + \kappa_b^{(i)} \int_{F_b^{(i)}} \sigma_{b,i}^{(i)} z_{b,i}^{(i)} dF_b + \gamma_b^{(i)} \int_{F_b^{(i)}} \tau_{b,i}^{(i)} dF_b \right) dS \right] . \quad (17)$$

Derived expression (17) is valid in the case of the application of conventional NSPA analysis, since the horizontal seismic forces P_i are not corrected during the analysis, but only have an incremental increase from 0 to 100%. Procedure of derivation of (17) for the adaptive NSPA analysis, reduces to the introduction of the correction of horizontal seismic forces according to the incremental situations [33]:

$$P_i = \sqrt{\sum_{q=1}^{n} (\Gamma_q \Phi_{i,q} m_i S_{a,q})^2} \,, \quad (18)$$

where q is the mode number, Γ_q modal participation factor for the q-th mode, $\Phi_{i,q}$ mass normalized mode shape value for the i-th storey and q-th mode, m_i the mass of the i-th storey, $S_{a,q}$ spectral acceleration for the q-th mode. By substituting (18) into (17) one obtains the expression for the external and internal work of the NSPA analysis:

$$\sum_{i=1}^{j} i h_i^{(i)} \theta_{i=1}^{(i)} \sqrt{\sum_{q=1}^{n} (\Gamma_q^{(i)} \Phi_{i,q}^{(i)} m_i^{(i)} S_{a,q}^{(i)})^2} = \left[\sum_{i=1}^{m} \int_S \left(\varepsilon_c^{(i)} \int_{F_c^{(i)}} \sigma_{c,i}^{(i)} dF_c + \kappa_c^{(i)} \int_{F_c^{(i)}} \sigma_{c,i}^{(i)} z_{c,i}^{(i)} dF_c + \gamma_c^{(i)} \int_{F_c^{(i)}} \tau_{c,i}^{(i)} dF_c \right) dS \right] +$$
$$+ \left[\sum_{i=1}^{n} \int_S \left(\varepsilon_b^{(i)} \int_{F_b^{(i)}} \sigma_{b,i}^{(i)} dF_b + \kappa_b^{(i)} \int_{F_b^{(i)}} \sigma_{b,i}^{(i)} z_{b,i}^{(i)} dF_b + \gamma_b^{(i)} \int_{F_b^{(i)}} \tau_{b,i}^{(i)} dF_b \right) dS \right] . \quad (19)$$

The first special case is obtained if one takes into account that when columns are concerned, the effect of shear forces on the yielding is small, and when the beams are concerned, the effect of normal and shear forces on the yielding is small, then (19) is:

$$\sum_{i=1}^{j} ih_i^{(i)}\theta_{i=1}^{(i)}\sqrt{\sum_{q=1}^{n}(\Gamma_q^{(i)}\Phi_{i,q}^{(i)}m_i^{(i)}S_{a,q}^{(i)})^2} = \left[\sum_{i=1}^{m}\int_S\left(\varepsilon_c^{(i)}\int_{F_c^{(i)}}\sigma_{c,i}^{(i)}dF_c + \kappa_c^{(i)}\int_{F_c^{(i)}}\sigma_{c,i}^{(i)}z_{c,i}^{(i)}dF_c\right)dS\right] + $$

$$+\left[\sum_{i=1}^{n}\int_S\left(\kappa_b^{(i)}\int_{F_b^{(i)}}\sigma_{b,i}^{(i)}z_{b,i}^{(i)}dF_b\right)dS\right]. \tag{20}$$

Another special case is obtained if one takes into account that until a certain level of drift DR_a the yielding in beams is allowed first, and only after they reach a given drift level, the yielding of the columns at supports is possible:

$$0 < DR \le DR_a \qquad (i) = (a):$$

$$\sum_{i=1}^{j} ih_i^{(i)}\theta_{i=1}^{(i)}\sqrt{\sum_{q=1}^{n}(\Gamma_q^{(i)}\Phi_{i,q}^{(i)}m_i^{(i)}S_{a,q}^{(i)})^2} = \sum_{i=1}^{n}\int_S\left(\varepsilon_b^{(i)}\int_{F_b^{(i)}}\sigma_{b,i}^{(i)}dF_b + \kappa_b^{(i)}\int_{F_b^{(i)}}\sigma_{b,i}^{(i)}z_{b,i}^{(i)}dF_b + \gamma_b^{(i)}\int_{F_b^{(i)}}\tau_{b,i}^{(i)}dF_b\right)dS, \tag{21}$$

$$DR > DR_a \qquad (i) > (a):$$

$$\sum_{i>a}^{j} ih_i^{(i)}\theta_{i>a}^{(i)}\sqrt{\sum_{q=1}^{n}(\Gamma_q^{(i)}\Phi_{i,q}^{(i)}m_i^{(i)}S_{a,q}^{(i)})^2} = \left[\sum_{i>a}^{m}\int_S\left(\varepsilon_c^{(i)}\int_{F_c^{(i)}}\sigma_{c,i}^{(i)}dF_c + \kappa_c^{(i)}\int_{F_c^{(i)}}\sigma_{c,i}^{(i)}z_{c,i}^{(i)}dF_c + \gamma_c^{(i)}\int_{F_c^{(i)}}\tau_{c,i}^{(i)}dF_c\right)dS\right] +$$

$$+\left[\sum_{i>a}^{n}\int_S\left(\varepsilon_b^{(i)}\int_{F_b^{(i)}}\sigma_{b,i}^{(i)}dF_b + \kappa_b^{(i)}\int_{F_b^{(i)}}\sigma_{b,i}^{(i)}z_{b,i}^{(i)}dF_b + \gamma_b^{(i)}\int_{F_b^{(i)}}\tau_{b,i}^{(i)}dF_b\right)dS\right]. \tag{22}$$

Using this method, we are able to significantly control the development of damage mechanism system, which enables a higher level of building safety requirements in case of an earthquake. Combining the first and second special case we obtain:

$$0 < DR \le DR_a \qquad (i) = (a):$$

$$\sum_{i=1}^{j} ih_i^{(i)}\theta_{i=1}^{(i)}\sqrt{\sum_{q=1}^{n}(\Gamma_q^{(i)}\Phi_{i,q}^{(i)}m_i^{(i)}S_{a,q}^{(i)})^2} = \sum_{i=1}^{n}\int_S\left(\varepsilon_b^{(i)}\int_{F_b^{(i)}}\sigma_{b,i}^{(i)}dF_b + \kappa_b^{(i)}\int_{F_b^{(i)}}\sigma_{b,i}^{(i)}z_{b,i}^{(i)}dF_b\right)dS, \tag{23}$$

$$DR > DR_a \qquad (i) > (a):$$

$$\sum_{i>a}^{j} ih_i^{(i)}\theta_{i>a}^{(i)}\sqrt{\sum_{q=1}^{n}(\Gamma_q^{(i)}\Phi_{i,q}^{(i)}m_i^{(i)}S_{a,q}^{(i)})^2} = \left[\sum_{i>a}^{m}\int_S\left(\varepsilon_c^{(i)}\int_{F_c^{(i)}}\sigma_{c,i}^{(i)}dF_c + \kappa_c^{(i)}\int_{F_c^{(i)}}\sigma_{c,i}^{(i)}z_{c,i}^{(i)}dF_c\right)dS\right] +$$

$$+\left[\sum_{i>a}^{n}\int_S\left(\kappa_b^{(i)}\int_{F_b^{(i)}}\sigma_{b,i}^{(i)}z_{b,i}^{(i)}dF_b\right)dS\right]. \tag{24}$$

Generally considering, damage mechanism of a system can be monitored through: global ductility of the building, the global drift, the inter-storey drift, local ductility of beams, moment-curvature relationships for the cross-sections and through strains. In relation to the macro aspect of consideration of damage mechanism through global ductility and global drifts, the NSPA-DMBD method was developed on the principle that the development of system damage mechanism is monitored on the level of micro aspects, or through strains. Reinforced concrete columns and beams are modeled using a fiber linear finite elements, and discretization on the cross-section level is carried out in three areas: unconfined concrete fibers, confined concrete fibers and steel fibers. Expression (19) is analyzed after each incremental situation for each cross-section, i.e. fiber, until the limit strain of steel reinforcement $\varepsilon_{s,u}$ is reached in a single column or beam fiber:

$$\varepsilon_{s,i}^{(i)} < \varepsilon_{s,u}\,, \qquad \varepsilon_{s,i}^{(i)} = \frac{\sigma_{s,i}^{(i)}}{E_{t,s,i}^{(i)}}\,, \qquad \varepsilon_{s,u} = \frac{\sigma_{s,adeq}}{E_{t,s,adeq}}\,, \tag{25}$$

or until a limit strain of concrete pressure $\varepsilon_{c,u}$ is reached in a single column or beam fiber:

$$\varepsilon_{c,i}^{(i)} < \varepsilon_{c,u}\,, \qquad \varepsilon_{c,i}^{(i)} = \frac{\sigma_{c,i}^{(i)}}{E_{t,c,i}^{(i)}}\,, \qquad \varepsilon_{c,u} = \frac{\sigma_{c,adeq}}{E_{t,c,adeq}}\,, \tag{26}$$

where E_t is the tangent modulus. When the limit of strain is reached:

$$\varepsilon_{s,i}^{(i)} \geq \varepsilon_{s,u} \quad \text{or} \quad \varepsilon_{c,i}^{(i)} \geq \varepsilon_{c,u}\,, \tag{27}$$

the calculation of the cross-section is performed, in which the limit of strain is reached, and the new amount and the placement of reinforcement is determined, and then the procedure for the NSPA analysis of the building is repeated. Since the procedure of analyzing the damage mechanism of a system according to the NSPA-DMBD method has been iteratively implemented and designing has been checked after the reached strain limit, the term Iterative-Interactive Design (IID) has been introduced.

Flowchart of the developed NSPA-DMBD method is shown in Figure 5. In the pre-processing stage desirable (optimal) system damage mechanism is defined as well as the criteria to indicate the achieved performance states on the material level, for ultimate strain of steel reinforcement and the ultimate strain of confined concrete. Then, using the LSA analysis the calculation is performed for the vertical gravity load, which simulates the behavior of the object in real conditions. After the LSA analysis is conducted, the seismic load is divided into *n* parts and the calculation for each part is done incrementally. The stiffness matrix of the system obtained from the LSA analysis is used as the initial stiffness matrix for the NSPA analysis. Having carried out the first NSPA analysis for $S_{i=1}$ seismic actions, the damage mechanism of the system is being analyzed. If the seismic load distribution is divided into many parts, there is a possibility that the first $S_{i=1}$ NSPA analysis of the damage mechanism could not be developed, since the development of nonlinear deformations in the members is not initiated. At this stage, therefore, we examine the damage mechanism of a system through reinforcement and concrete strains. If it is determined that the unfavorable damage mechanism is developed by using the NSPA analysis where S_i, the level of strain for each fiber, i.e. cross-section, is determined separately. Then we move on to the correction of reinforcement using redesigning in case the limit of strains are crossed, otherwise we move to the testing of criteria concerning the drift ratio for the current calculated situation DR_i and the drift of immediate occupancy (IO) performance level DR_{IO}.

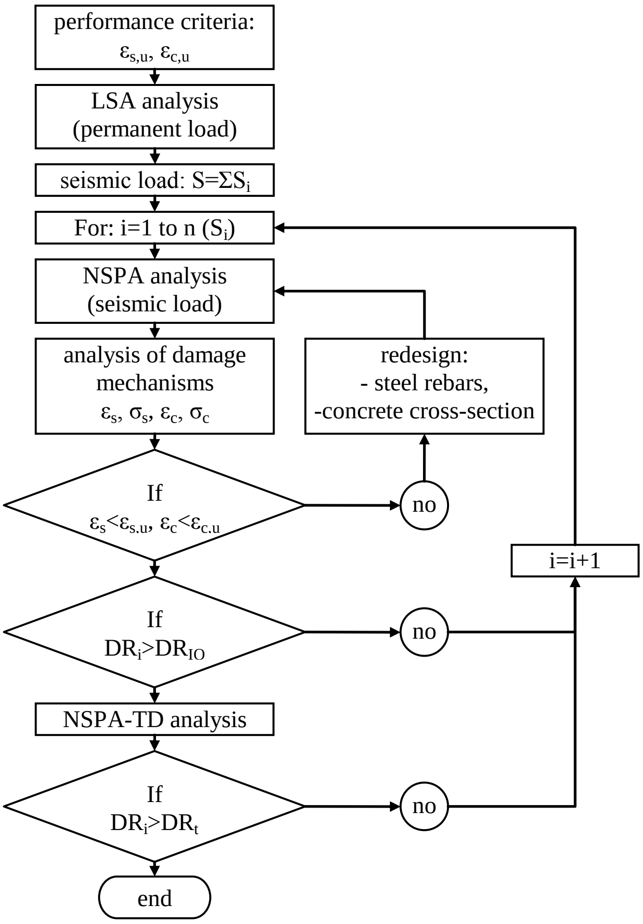

Fig. 5 Flowchart of the NSPA-DMBD method

If it appears that the DR_i>DR_{IO}, then the analysis of target displacement for the NSPA analysis is conducted (NSPA-TD), while in case of the opposite situation, we make a transition to a new seismic load increment *i*+1 and the procedure of criteria and calculation testing described above is repeated. The condition DR_i>DR_{IO} sets in order to ensure the development of damage mechanism according to the CDM method by introducing the IO performance level. Following the NSPA-TD analysis, the criteria of a relationship between a drift for the current calculated situation DR_i and a drift for the level of the target displacement DR_t is conducted. By fulfilling this criterion, the NSPA-DMBD method ends, while otherwise we use a new increment of seismic load *i*+1 and previously described procedure of the criteria and calculation testing is repeated.

Numerical testing of the NSPA-DMBD method was carried out on a reinforced concrete 2D 8-storey 4-bay framework system. Framework system has been previously designed according to EC 8 regulations and then a NSPA analysis has been carried out on the same system. Performance criteria were set prior to the application of the NSPA analysis for limit strains of reinforcement $\varepsilon_{s,u}$=10‰ for bilinear constitutive material model with kinematic strain hardening in the area of nonlinear deformations and reinforced concrete $\varepsilon_{c,u}$=3.5‰ for nonlinear constant confinement concrete model. The total number of iterations carried out within the NSPA analysis is nine, since during the last iteration, the unfavorable damage mechanism has not been realized, even at the maximum global drift value, DR_{max}=3.3%. The indication of the unfavorable damage mechanism development was observed at the level of drift *DR*=1.55%, by exceeding the limit for strains in the concrete according to the NSPA analysis, for the column of the fourth storey (Figure 6a). During the next step, the correction of reinforcement in the third and fourth storey columns, and a new iteration according to the NSPA analysis has been implemented. For such conducted analysis, we identified the unfavorable damage mechanism at the level of drift *DR*=2.22% by exceeding the limit strains of concrete in the second storey (Figure 6b). For the third iteration, within the NSPA analysis, at the level of drift *DR*=2.22%, an unfavorable damage mechanism was achieved by exceeding the limit strains in the concrete for the columns of the fourth storey (Figure 6c), while for the fifth iteration, within the NSPA analysis, at the level of drift *DR*=2.55%, an unfavorable damage mechanism was achieved by exceeding the limit strain in the reinforcement for the column of the first storey (Figure 6d). When we take into consideration the seventh iteration of the NSPA-DMBD method, at the level of drift *DR*=2.77%, an unfavorable damage mechanism is achieved by exceeding the limit strain in the concrete for the column of the second storey (Figure 6e). Generally, it can be said that there is an increase in the level of drift according to the NSPA-DMBD method and a reduction in the level of damage and the risk of collapse, which is a result of the correction of system damage mechanism, from unfavorable towards the optimum.

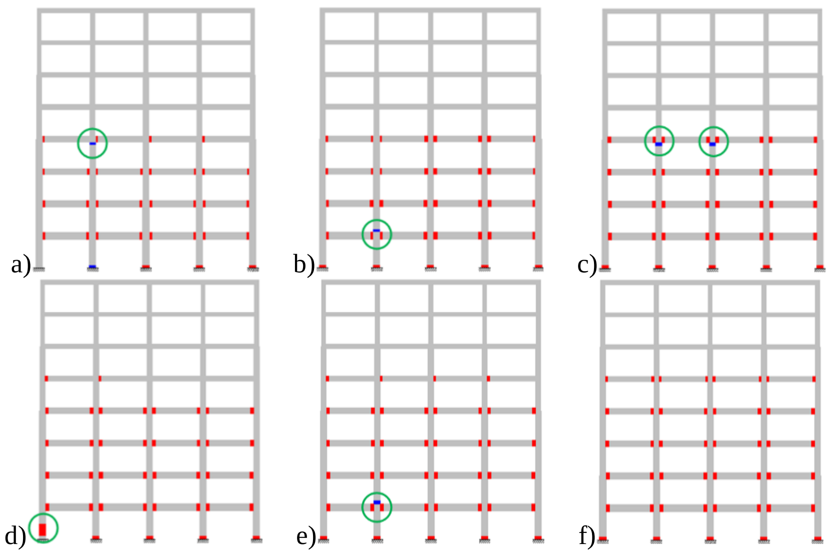

Fig. 6 Iterations of the NSPA-DMBD: a) initial (*DR*=1.55%), b) first (*DR*=2.22%), c) third (*DR*=2.22%), d) fifth (*DR*=2.55%), e) seventh (*DR*=2.77%), f) ninth (final)

Locations of development of damage mechanism at beams are in most cases localized to the beam-column connection, while only a small number of cases have identified growth of limit strain outside the beam-column connection, such as the fifth iteration of the NSPA-DMBD method. Out of nine iterations conducted within the NSPA-DMBD method, we have identified four where the strain limit in the reinforcement was exceed, while for the remaining analyzes, strain limit transgressions in the concrete were authoritative. During the calculation of reinforced concrete frame buildings, using standardized regulations for designing the cross-section of reinforced concrete, the aim is development of a fracture by reaching the limit strains along the reinforcement. In this way, the reinforced concrete elements are provided with a favorable ductile behavior. By applying the developed NSPA-DMBD method and a research done on the model of a framework building, it was pointed out that even during initial NSPA analysis the indicator of unfavorable damage mechanism exceeded the limit of concrete strain.

Generated NSPA pushover curves are shown in Figure 7. The influence of the correction of the system's damage mechanism by applying the NSPA-DMBD method does not have greater importance, globally, up to the value of drift *DR*=1.5%, where almost identical solutions are obtained as for the initial original pushover curve. By further increasing the level of drift and entering the domain of emphasized nonlinear behavior, this difference becomes significant, so that the development and character of the optimal damage mechanism becomes more important. With the initial NSPA pushover curve, the stiffness in a nonlinear domain is constantly reduced, while in the NSPA-DMBD pushover curve, the stiffness in nonlinear domain is being increased as the number of iterations increase.

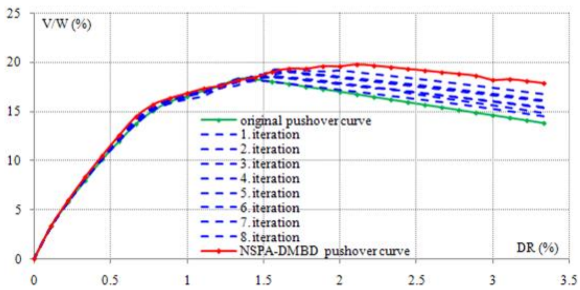

Fig. 7 An original NSPA pushover curve, pushover curves for the iteration phases of the NSPA-DMBD method and the final NSPA-DMBD pushover curve

Figure 8 presents the inter-storey drifts (*IDR*) according to the NSPA-DMBD method for four levels of maximum drifts: DR_{max}=1.5%, DR_{max}=2%, DR_{max}=2.5% and DR_{max}=3%. It is evident that by increasing the number of iterations according to the NSPA-DMBD method, the maximum value of inter-storey drift is reduced at all levels of the considered drifts DR_{max}. Inter-storey drift values are reduced in lower storeys, while in the upper storeys they are increased, thus creating the balance of structural responses according to the NSPA-DMBD method. Figure 9 presents the global drifts (*DR*) according to the NSPA-DMBD method for four levels of maximum drifts: DR_{max}=1.5% and DR_{max}=2%, DR_{max}=2.5% and DR_{max}=3%. There is an apparent reduction of drifts on every storey for the final iteration of the NSPA-DMBD method.

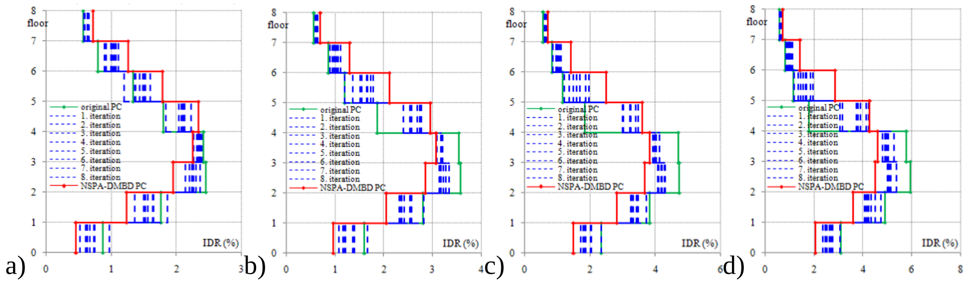

Fig. 8 Charts of the inter-storey drifts *IDR* according to the NSPA-DMBD method: a) DR_{max}=1.5%, b) DR_{max}=2%, c) DR_{max}=2.5%, d) DR_{max}=3%

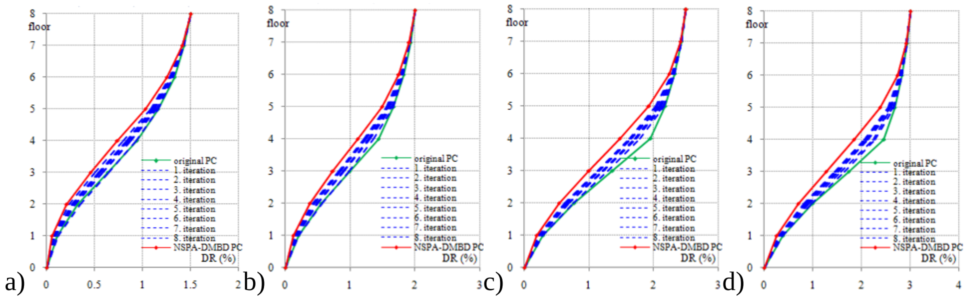

Fig. 9 Charts of the global drifts *DR* according to the NSPA-DMBD method: a) DR_{max}=1.5%, b) DR_{max}=2%, c) DR_{max}=2.5%, d) DR_{max}=3%

5. CONCLUSION

The main concept of mathematical formulation and numerical studies, presented in this research, is that by controlling the system damage mechanism based on the NSPA analysis (NSPA-DMBD) we examine the optimal system response in the case of seismic action. Using the CDM method, we define the optimal system damage mechanism using the yielding on beams and partial yielding on columns at supports. The formation of system damage mechanism in incremental situations is controlled at the level of strains, by examining the relationship of current and limit strain in the concrete and reinforcement. The correction of cross-section, in which the limit strain is reached, is carried out using redesigning, and then the procedure for the NSPA building analysis is repeated from start. Since the procedure of system damage mechanism analysis according to the NSPA-DMBD method is iteratively implemented, and the designing is checked after the limit strain is reached, for this sort of analysis the term Iterative-Interactive Design (IID) is introduced.

By applying developed NSPA-DMBD method and due to the research conducted on the model of the framework building, it was pointed out that even during the initial NSPA analysis the indicator of an unfavorable damage mechanism was related to the limit strains of concrete columns. This is in contrast with a solution that is normally obtained using the standard approach in design of reinforced concrete sections, where the goal is to obtain a fracture by reaching the limit strains in reinforcement. With the initial NSPA pushover curve, the stiffness in the nonlinear domain is constantly reduced, while in the NSPA-DMBD pushover curve, the stiffness in nonlinear domain increases with the increasing number of iterations. Increasing the number of iterations, according to the NSPA-DMBD method, the maximum value of inter-storey drift is reduced at all levels of the considered maximum drifts. Also, the values of inter-storey drifts, according to the NSPA-DMBD method, are reduced at the lower storeys, while at the upper storeys they are increased, thus creating a balance between structural responses. By selection, monitoring and control of optimal building damage mechanism and using the developed NSPA-DMBD method, we control the building damage and fracture mechanism and increase the level of the system resistance to an early collapse.

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REFERENCES

1. M. Williams and R. Sexsmith, "Seismic damage indices for concrete structures: a state-of-the-art review", Earthquake spectra, vol 11, iss. 2, 1995, pp. 319-349.
2. A. Ghobarah, H. Abou-Elfath and A. Biddah, "Response-based damage assessment of structures", Eart. Eng. and Str. Dyn., vol. 28, iss. 1, 1999, pp. 79-104.
3. Y. Park, A. Ang and Y. Wen, "Seismic damage analysis of reinforced concrete buildings", Journal of Str. Eng., vol. 111, iss. 4, 1985, pp. 740-757.
4. Y. Park, A. Ang and Y. Wen, "Damage-limiting aseismic design of buildings", Eart. Spec., vol. 3, iss. 1, 1987, pp. 1-26.
5. M. Roufaiel and C. Meyer, "Reliability of concrete frames damaged by earthquakes", Journal of Str. Eng., vol. 113, iss. 3, 1987, pp. 445-457.
6. Đ. Ladjinović and R. Folić, "Application of improved damage index for designing of earthquake resistant structures", 13th WCEE, Paper No. 2135, Vancouver, Canada, 2004, pp. 1-15.
7. S. Jeong and A. Elnashai, "Analytical and experimental seismic assessment of irregular RC buildings", 13th WCEE, Paper No. 113, Vancouver, Canada, 2004, pp. 1-15.
8. T. Dinh, T. Ichinose and N. Marubashi, "Collapse control of irregular wall buildings using story-safety factor", 13th WCEE, Paper No. 1340, Vancouver, Canada, 2004, pp. 1-14,
9. Z. Qu, L. Ye and A. Wada, "Seismic damage mechanism control of RC ductile frames from a stiffness point of view", 8th International Conf. on Urban Eart. Eng., Tokyo, Japan, 2011, pp. 1-5.
10. M. Maeda, Y. Nakano and K. Lee, "Post-earthquake damage evaluation for R/C buildings based on residual seismic capacity", 13th WCEE, Paper No. 1179, Vancouver, Canada, 2004, pp. 1-15.
11. Y. Nakano, M. Maeda. H. Kuramoto and M. Murakami, "Guideline for post-earthquake damage eval. and rehab. of RC buildings in Japan", 13th WCEE, Paper No. 124, Vancouver, Canada, 2004, pp. 1-15.
12. Y. Bozorgnia and V. Bertero, "Damage spectrum and its applications to performance-based earthquake engineering", 13th WCEE, Paper No. 1497, Vancouver, Canada, 2004, pp. 1-8.
13. A. Manafpour, "A damage-controlled force-based seismic design method for RC frames", 13th WCEE, Paper No. 2670, Vancouver, Canada, 2004, pp. 1-22.
14. C. Loh and A. Chao, "The use of damage function in performance-based seismic design of structures", 13th WCEE, Paper No. 3257, Vancouver, Canada, 2004, pp. 1-15.
15. T. Takada and Nakano, "Seismic load effect directly linked to specified collapse mechanisms in ultimate limit state design", 12th WCEE, Paper No. 2560, Auckland, New Zealand, 2000, pp. 1-9.
16. C. Adam and C. Jager, "Dynamic instabilities of simple inelastic structures subjected to earthquake excitation", Adv. Dyn. and Model-Based Ctrl of Struc. and Mach., Springer, 2012, pp. 11-18.
17. Y. Lieping and Q. Zhe, "Failure mechanism and its control of building structures under earthquakes based on structural system concept", Journal of Eart. and Tsu., vol. 3, iss. 4, 2009, pp. 249-259.
18. S. Goel, S Chao, Performance-Based Plastic Design: Earthquake-Resistant Steel Structures, SEA, 2008.

19. D. Vamvatsikosa and A. Cornell, Seismic Performance, Capacity and Reliability of Structures as Seen Through Incremental Dynamic Analysis, The John A. Blume EE Center, Report No. 151, USA, 2005.
20. L. Ibarra and H. Krawinkler, Global Collapse of Frame Structures under Seismic Excitations, PEER Center, PEER 2005/06, 2005.
21. B. Bradley, R. Dhakal and J. Mander, "Modeling and analysis of multi-storey buildings designed to principles of ductility and damage avoidance", 10th EAPCSEC, Bangkok, Thailand, 2006, pp. 1-6.
22. C. Goulet, C. Haselton, J. Mitrani, J. Beck, G. Deierlein, K. Porter and J. Stewart, "Evaluation of the seismic performance of a code-conforming reinforced-concrete frame building: from seismic hazard to collapse safety and economic losses", Eart. Eng. and Str. Dyn., vol. 36, iss. 13, 2007, pp. 1973-1997.
23. C. Haselton and G. Deierlein, Assessing Seismic Collapse Safety of Modern Reinforced Concrete Moment-Frame Buildings, PEER Center, PEER 2007/08, 2008.
24. S. Krishnan and M. Muto, "Mechanism of collapse of tall steel moment frame buildings under earthquake excitation", Journal of Str. Eng., vol. 138, iss. 11, 2012, pp. 1361-1387.
25. A. Mwafy and A. Elnashai, "Static pushover versus dynamic collapse analysis of RC buildings", Engineering Structures, vol. 23, iss. 5, 2001. pp. 407-424.
26. K. Meguro and H. Tagel, "Applied element method used for large displacement structural analysis", Journal of Nat. Disas. Sci., vol. 24, iss. 1, 2002, pp. 25-34.
27. Eurocode 8, Design of Structures for Earthquake Resistance - Part 1: General Rules, Seismic Actions and Rules for Buildings, European Committee for Standardization, Brussels, Belgium, 2004.
28. T. Paulay and M. Priestley, Seismic Design of Reinforced Concrete and Masonry Buildings, John Wiley & Sons, New York, USA, 1992.
29. SAP 2000, Integrated Software for Structural Analysis and Design, CSI Berkeley, USA, 2010.
30. ATC 40, Seismic Evaluation and Retrofit of Concrete Buildings, Vol. 1, Applied Technology Council, Redwood City, USA, 1996.
31. FEMA 356, Pre-Standard and Commentary for the Seismic Rehabilitation of Buildings, American Society of Civil Engineers, Federal Emergency Management Agency, Washington D. C., USA, 2000.
32. H. Sadd, Elasticity: Theory, Applications and Numerics, Elsevier, New York, USA, 2005.
33. S. Antoniou and R. Pinho, "Advantages and limitations of adaptive and non-adaptive force-based pushover procedures", Journal of Earth. Eng., vol. 8, iss. 4, 2004, pp. 497-522.

RAZVOJ METODE KONTROLISANOG MEHANIZMA LOMA ZGRADA ZASNOVANA NA NELINEARNOJ STATIČKOJ *PUSHOVER* ANALIZI

U radu je prikazana originalno razvijena metoda kontrolisanog mehanizma loma zgrada zasnovana na nelinearnoj statičkoj pushover analizi (NSPA-DMBD - Nonlinear Static Pushover Analysis - Damage Mechanisms-Based Design). Optimalan mehanizam loma zgrade određuje se na osnovu rešenja metode programiranog ponašanja (CDM - Capacity Design Method), a odgovor zgrade se razmatra u inkrementalnim situacijama. Formiranje mehanizma loma sistema u ovakvim inkrementalnim situacijama se kontroliše na nivou dilatacija, ispitivanjem odnosa trenutnih i graničnih dilatacija u betonu i armaturnom čeliku. Pošto se postupak analize mehanizma loma sistema po NSPA-DMBD metodi sprovodi iterativno, a dimenzionisanje proverava nakon dostignute granične dilatacije, to je za ovakvu analizu uveden termin iterativno-interaktivno dimenzionisanje (IID - Iterative-Interactive Design). Selekcijom, monitoringom i kontrolom optimalnog mehanizma loma sistema i razvijenom NSPA-DMBD metodom kontroliše se globalni mehanizam loma zgrada i povećava nivo otpornosti na rani kolaps.

Ključne reči: *mehanizam loma sistema, dilatacije, metoda programiranog ponašanja, nelinearna statička pushover analiza, iterativno-interaktivno dimenzionisanje*