Comparison of Structural Models for Seismic Analysis of Multi-Storey Frame Buildings

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SUMMARY:
Nonlinear static pushover analysis provides more realistic determination of the interaction between the components of critical support structure and substantially better estimation of strength capacity and deformation of buildings exposed to earthquakes. Its application makes it possible to take into account the redistribution of forces occurring during the nonlinear response and therefore this method provides a better insight into the actual behaviour of structures rather than linear methods of analysis. This paper presents an overview of modelling methods and results of the analysis obtained for the designed model of multi-storey frame using the programme SAP2000 and OpenSees. The strength and deformation capacity of ductile concrete elements of the multi-storey frame structure is determined by the analysis of moment-curvature based on the expected (adopted) material properties. The nonlinear behaviour of structural elements is idealized by plastic hinges "set" in pre-selected locations.

Keywords: seismic demands, multi-storey buildings, pushover analysis, nonlinear modelling, plastic hinges

1. INTRODUCTION

Required strength of buildings under the current concept of seismic protection is determined for the structural effects due to seismic forces corresponding to the so-called design level (return period $T_r \approx 500$ years). These forces are determined by applying the reduction factor which is adopted depending on the presumed deformation capacity of structures. The structure that is designed in this manner can usually withstand earthquake effects without collapse, under the condition that the actual characteristics of the occurred earthquake ground motions correspond to adopted seismic hazard. The disadvantage of this concept is reflected in the fact that, on the basis of the design, there is no insight into the size of nonlinear deformation, and hence into the level of the structural damage.

The contemporary concept of reinforced concrete structural design includes the nonlinear behaviour of the structural elements in the moderate and strong earthquakes in the pre-specified sections for the dissipation of seismic energy without structure failure. The sections, in which the appearance of inelastic deformations is allowed, next to the capacity, need to have the necessary ductility, i.e. adequate deformation capacity. The demanded bending and shear strength of the section has to be fulfilled both inside and outside the zone of the critical regions (plastic hinges), and it is achieved by appropriate longitudinal and transverse reinforcement. The ductility in the regions of potential plastic hinges which reinforced concrete sections need to have, i.e. the demand to have the necessary capacity deformations without significant loss of strength when plastic deformation occurs, is achieved by structural measures related to placing sufficient confinement reinforcement (stirrups) in cross sections. Monitoring of the nonlinear deformations of the structure, in qualitative and quantitative terms, is possible only with the application of nonlinear analysis. The paper presents some results of nonlinear static pushover analysis conducted using SAP2000 and OpenSees, with the usage of fibre plastic hinge models.

The main parameters of the seismic analysis of structures are load carrying capacity, ductility, stiffness, damping and mass. The design can be divided into two main steps. First, a linear analysis is conducted with appropriate dimensioning of all structural elements, ensuring the functionality of the structure after minor earthquakes, and then the behaviour of structures during strong earthquakes has to be controlled using nonlinear methods.

Since the uncertainty (deviation from the real parameters) of input data for seismic analysis is expressed as inaccuracies related to geometrical and mechanical characteristics of the structure, especially inaccuracies describing the nonlinear behaviour of structures and seismic effects, the consequence is the corresponding errors in the analysis results. For all the foregoing, to achieve adequate seismic stability of structures, it is necessary to provide special attention to the building disposition, the choice of an appropriate mathematical model for the numerical analysis, design and detailing of structural elements.

2. NONLINEAR STATIC PUSHOVER ANALYSIS

The real behaviour of a structure during an earthquake can be the best simulated using the nonlinear dynamic time-history analysis (THA). However, THA is still too complicated for practical application, and for the last two decades, calculation methods based on nonlinear static pushover analysis of structures have been intensively developed.

The conventional nonlinear static analysis uses a constant load pattern during the design with a gradual increase of structural effect intensity. Rectangular distribution of lateral forces can be utilized, and it is recommended when the masses of all floors are the same. The equivalent lateral force distribution in height is formed following the pattern $S_i = m_i h_i^k$, where h_i is the height of the i-th floor, m_i is the mass the of the i-th floor, and the coefficient k depends on the fundamental period T_1 of the structure. It has the following values: $T_1 \leq 0.5$ s $\Rightarrow$ k = 1, for $T_1 \geq 2.5$ s $\Rightarrow$ k = 2, while for the value of T_1 the first mode between 0.5 s and 2.5 s interpolation is performing between the values of 1 and 2. The distribution of lateral forces in accordance with the mode shape is determined by the pattern $S_i = m_i\, \phi_{i1}$, where h_i is the height of the i-th floor, and the vector ϕ_i is based on the components of the fundamental mode shape, where the distribution is recommended only if at least 75% of the total mass of the system participates in the first mode of vibrations. The distribution of lateral loads can be introduced into the calculation as a force that is proportional to the inertial force, i.e. proportional to the mass system distribution where, in the case of equal masses of all the system nodes, the distributed load is obtained.

Adaptive nonlinear static analysis uses the correction of the lateral load scheme during the design. The distribution of lateral forces is used in accordance with the mode shape of vibrations, and the model is adjusted after each change in stiffness as a consequence of plastic phenomena in certain sections. Stiffness changes, resulting in a change of the first vibration mode shape, which subsequently causes a new scheme of the transverse load acting on the structural system. For the adopted value of the maximum horizontal displacement of the characteristic structural node, the intensity of the lateral load distributed in accordance with the form of the first vibration mode incrementally increases, as in the case of conventional analysis. After that, in accordance with the plastification of certain sections in the structure, the new design of the stiffness matrix of the system, a new form of the first vibration mode, and a new distribution of the lateral forces in accordance with the shape of the new vibration mode, is calculated. Hence, the nonlinear static pushover analysis is performed again. Thus described steps are repeated until the failure mechanism occurs. In this way, the change in the shape and vibration of the system is included as a result of the appearance of plastic hinges instead of the conventional analysis, resulting in the increased accuracy of the design.

The nonlinear behaviour of the structure can be introduced into the analysis through the geometric and material nonlinearities. The simplest geometric nonlinearity can be introduced into the calculation via the P-Δ effect, which is reflected in the increase of bending moments in the structural elements due to the existence of the vertical load over the horizontal displacements, which are a consequence of lateral seismic loads. Material nonlinearity is included in the analysis through the possibility of cross-section plastification of structural elements. In the calculation, the elements with the possibility of the occurrence of concentrated plastic hinges, as well as inelastic deformation occurring along the element, can be utilized. From the aspect of numerical modelling, we can use models with plastic hinges defined on the moment-rotation relation and the fibre models. There is also a model that consists of multiple parallel line elements (MVL), which represents a compromise between the concentrated plasticity which is introduced through the plastic hinge and the complex fibre model. In the case of plastic hinges it is necessary to introduce the relationship between the force and the displacement or the moment and the rotation (curvature), and it is necessary to define the axial and moment plasticization hinges. Recommendations for defining the characteristics of plastic hinges can be found in the regulations (ATC-40, FEMA 273 and FEMA 356). In the case of forming the plastic fibre hinges, in the case of reinforced concrete structures it is important to define the cross-sectional parts consisting of confined and unconfined concrete and reinforcement, with the definitions on the relationship between stresses and strains. The advantages of a concentrated plasticity model introduced through the plastic hinges are reflected in the modelling simplicity, easy control and clear physical meaning of used hysterisis rules. The deficiencies are reflected in most cases in the inability to be used in the case of biaxial bending, as well as in the case of assessment of local response. Advantages of fibre models are reflected in the possibility of local response estimation and in the application of biaxial bending, while the disadvantages are reflected in the complexity of the analysis and modelling, as well as in the more complex result control. The advantages of MVL elements are observed in the relative and easy implementation, clear physical meaning of used hysterisis rules, the possibility of obtaining local responses, and possible applications of biaxial strength. The disadvantages of this model are observed in the need for iteratively determination of the number of elements as in the request for the usage of several elements to obtain reasonable estimate of structural response.

3. MODELLING A STRUCTURE FOR NONLINEAR STATIC ANALYSIS

The paper considers some aspects of modelling plastic hinges using the programmes SAP2000 and OpenSees. SAP2000 and OpenSees programmes offer more opportunities for the selection of material models, elements and solution algorithms for nonlinear analysis, depending on the type of material, designing of structural elements, cross sections and the type of analysis.

3.1. Fibre model

Fibre model allows biaxial bending interaction with axial force acting at the same time, and can be used for nonlinear static pushover analysis and for nonlinear dynamic time analysis. Also, the usage of such model is possible in the case of planar and three-dimensional analysis. SAP2000 programme has the possibility of introducing the fibre plastic hinge model (“Fibre PMM Hinge”) into the analysis.

There are several possibilities to describe the nonlinear behaviour of line elements in OpenSees. One of them is "NonLinear Beam – Column Elements" which includes the possibility of plasticization propagation along the element using the fibre cross-sectional model. Fibre cross-sections associated with the corresponding Gauss's integration points along the element and the cross-sectional model of reinforced concrete beam or column are formed by the division into layers of the cross-sectional height, which particularly represent the concrete part of the section and the reinforcement.

3.2. Stress-strain relation

Model “Concrete02” (OpenSees) presents the typical concrete behaviour which takes into account the concrete tensile strength (Fig. 1). The behaviour in compression is defined by a maximum compressive strength f_{pc} for the strain ε_{c0} and the residual strength f_{pcu} achieved at the ultimate strain ε_{cu}. Part of the relation that describes the tensile behaviour is determined by the maximum tensile strength f_t and the slope coefficient that determines the decrease of the tensile strength E_{ts}. Model “Concrete02” has a typical hysteresis behaviour, shown in Fig. 1.

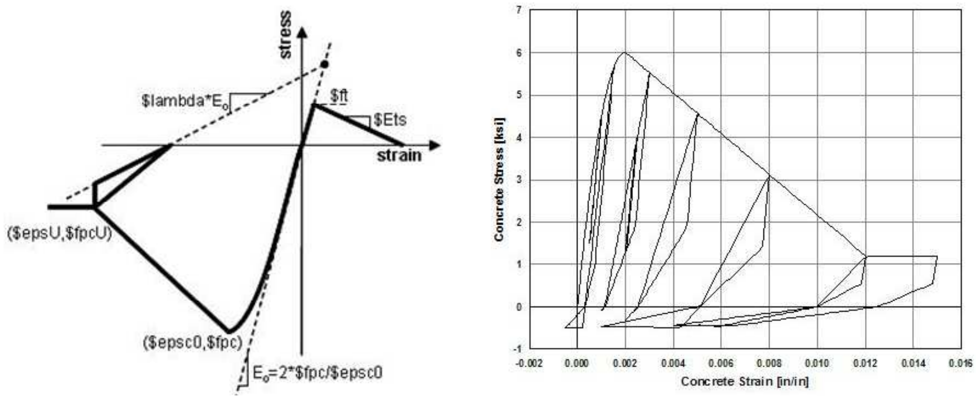


Figure 1. Stress-strain relation (left) and hysteresis behavior (right) – “Concrete02” (OpenSees)

Model “Concrete07” (OpenSees) applies Chang's and Mander's model of behaviour from 1994 (Fig. 2). The model requires eight parameters to define the behaviour of concrete. The main input data is the maximum compressive strength of concrete f_{c0}, and other values are determined in dependence on whether the concrete is considered as confined or unconfined.

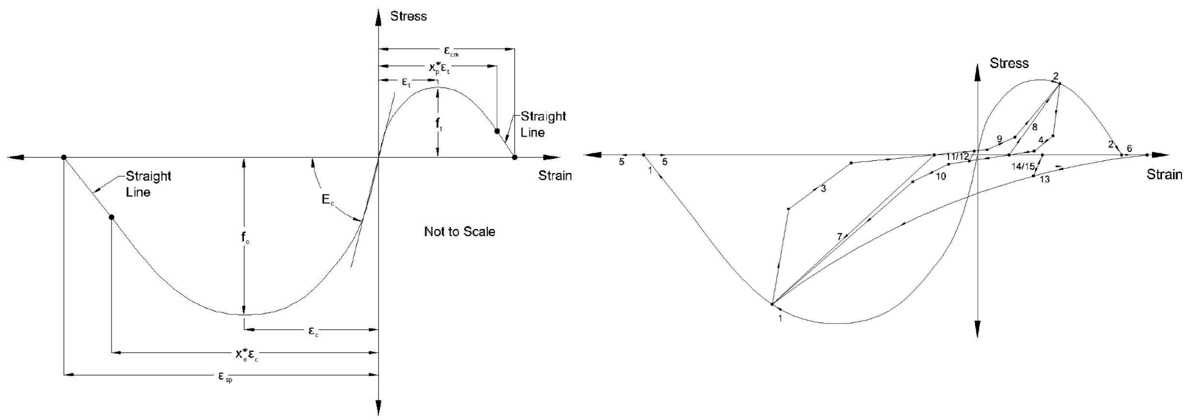


Figure 2. Stress-strain relation (left) and hysteresis behavior (right) – “Concrete07” (OpenSees)

Stress-strain relation and hysteresis behaviour of the model “Steel02” (OpenSees) are shown in Fig. 3 and Fig. 4. Input data to define this relationship are: elastic modulus E, yield strength f_y and the coefficient which defines the relationship between the elastic modulus and the slope of tangent after reaching the yield point.

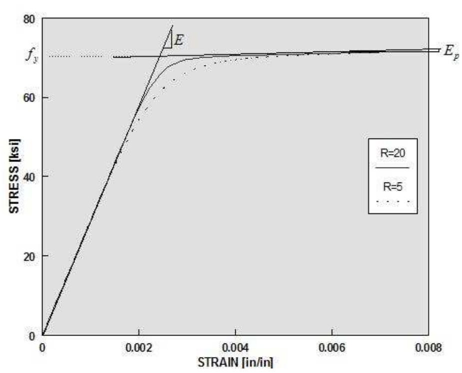
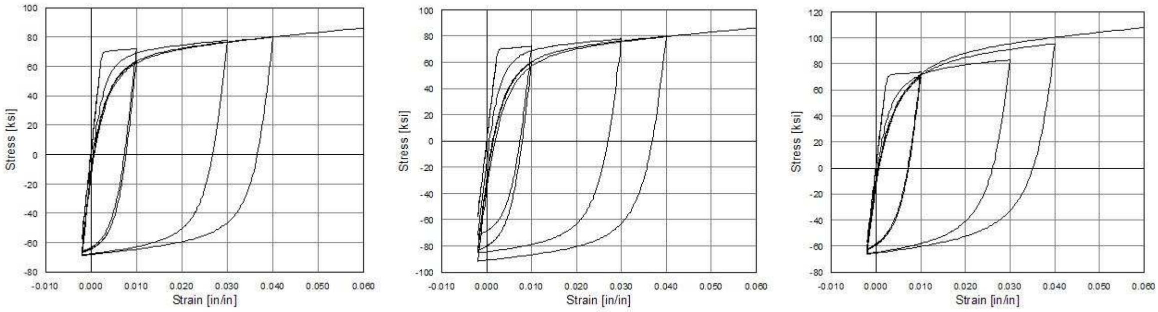


Figure 3. Stress-strain relation – “Steel02” (OpenSees)



without Hardening | with Hardening in Compression | with Hardening in Tension

Figure 4. Hysteresis behavior – “Steel02” (OpenSees)

The SAP2000 programme has the possibility of introducing the non-linear analysis of stress-strain relationship for concrete over the "Mander"'s models, and for the reinforcement through the "Simple" model (Fig. 5). The basic input data for defining the stress-strain relationship for concrete are: the maximum compressive strength f_c for the strain ε_c, the maximum strain at failure ε_u and the elastic modulus E. The basic input data for defining the stress-strain relationship for steel are: yield strength f_y and ultimate strength f_u at the proper strain ε_u.

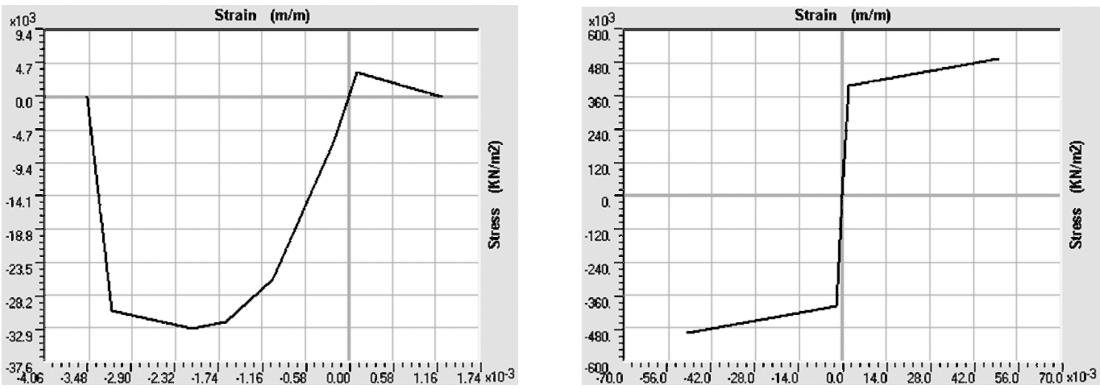


Figure 5. Stress-strain relation for concrete (left) and for reinforcement (right) – SAP2000

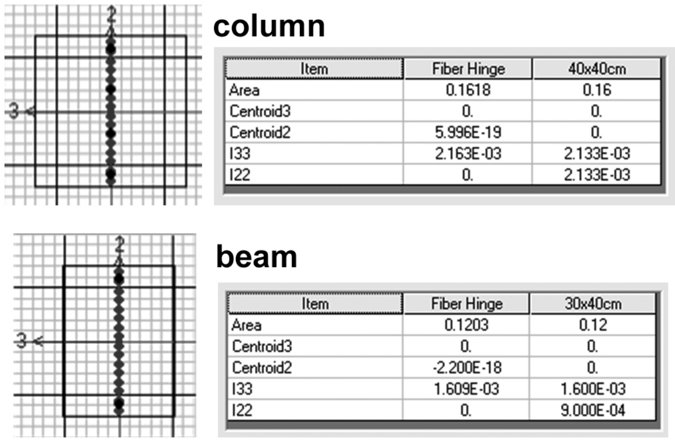
4. NUMERICAL EXAMPLE

4.1. Description of the model

The regular reinforced concrete frame of equal length spans (3×5.0 m) with five stories (5×3.0 m) is analyzed. The cross sectional dimensions of the columns are 40×40 cm, and they are reinforced symmetrically with 12∅14. The frame beams have cross-sectional dimensions of 30×40 cm, and in the critical regions are reinforced symmetrically with ±4∅10. The quality of concrete is C25/30 and the quality of reinforcement is S400 (Class C).

In the calculations, three models are used for the relationship of the stress-strain for unconfined concrete (“Conrete02” and “Concrete07” – OpenSees and Mander’s model – SAP2000). For the model Concrete 02 (OpenSees) the following input data are used: the maximum compressive strength f_{pc} = 33 MPa for the strain ε_{c0} = 0.0021, the ultimate strength f_{pcu} = 19.8 MPa for the strain ε_u = 0.0035 and the maximum tensile strength f_t = 3.3 MPa. Model Concrete 07 has the following input data: the maximum compressive strength f_{pc} = 33 MPa, the strain at the maximum compressive strength ε_{c0} = 0.0021, the elastic modulus E = 31.5 GPa, and the maximum tensile strength f_t = 3.6 MPa for the strain ε_{c0} = 0.00023. When Mander's model of concrete (SAP2000) is applied, the following input values are taken into account: the maximum compressive strength $f_{pc} = 33$ MPa for the strain $\varepsilon_{c0} = 0.0021$ and the ultimate strain $\varepsilon_u = 0.0035$. In the calculations for the stress-strain relation of reinforcement the model Steel02 (OpenSees) is used, with the following input values: the yield stress $f_y = 400$ MPa and ultimate stress $f_u = 500$ MPa. These basic values are also used for the steel model in SAP2000.

The possibility of cross-sectional plastification is introduced through the cross-sectional fibre model. Cross-sectional fibre model for the columns consists of 16 layers for concrete and four layers for reinforcement (Fig. 6 – above). For the beam cross sections the fibre model consisting of 16 layers for the concrete part of the section and two layers for reinforcement is formed (Fig. 6 – below).



column

Item	Fiber Hinge	40x40cm
Area	0.1618	0.16
Centroid3	0.	0.
Centroid2	5.996E-19	0.
I33	2.163E-03	2.133E-03
I22	0.	2.133E-03

beam

Item	Fiber Hinge	30x40cm
Area	0.1203	0.12
Centroid3	0.	0.
Centroid2	-2.200E-18	0.
I33	1.609E-03	1.600E-03
I22	0.	9.000E-04

Figure 6. Fibre model for the column and beam cross sections (SAP2000 and OpenSees)

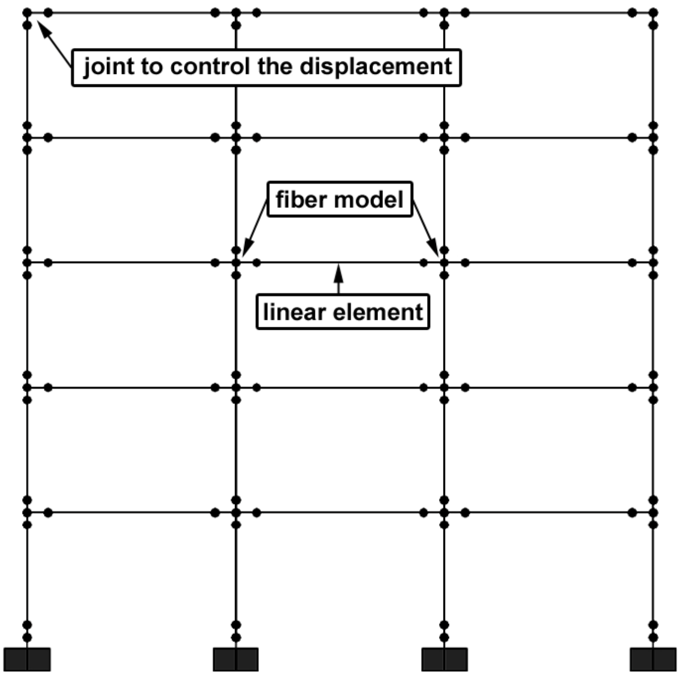


Figure 7. Frame model – SAP2000

Line framework model in OpenSees for the numerical integration was formed with five Gauss-points for determining the distribution of curvature along the nonlinear element. The line frame model in SAP2000 (Fig. 7) is formed by the member divided into three finite elements. The elements at the end parts of the beams are modelled as nonlinear (with plastic hinges), while the middle part of the member is modelled by the finite element with linear behaviour. The elements at the end parts of the beams, with a length equal to the length of a plastic hinge, apply the fibre plastic hinge placed next to the frame joint. Length of the plastic hinge is equal to the cross-sectional height. For the nonlinear analysis, using SAP2000, two numerical models in terms of effective values of the geometric characteristics of the cross sections are used. The first model (model B) has the characteristics of homogeneous cross-section, while the second model (model A) has the corresponding axial and shear surfaces reduced by the coefficient 0.8 and the reduced sectional moment of inertia with the coefficient 0.5.

Nonlinear static pushover analysis is performed taking into account the geometric nonlinearity through P-Δ effects. The distribution of the transverse load is adopted as linear per height, while the influence of the gravity load (w_g = 20 kN/m) is taken into account in the analysis (Fig. 8).

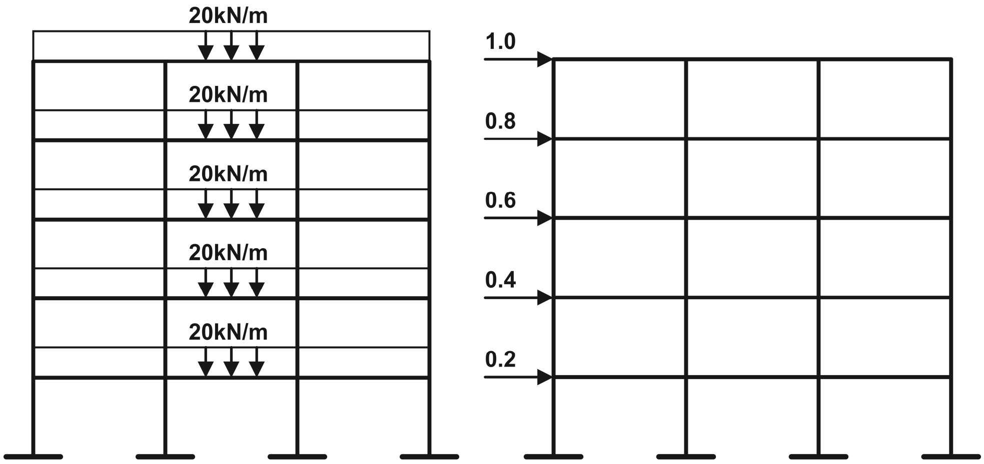


Figure 8. Gravity load (left) and distribution of the lateral forces at the structural height (right)

4.2. The results of the analysis

The obtained pushover curves using different calculation models are shown in Fig. 9. For the parts of the curves in which the nonlinear character of the load – displacement relation (displacement in the range from 0 to ~0.05 m) is not very pronounced, there is a good matching of characteristic curves B, C and D, with the deviation of the model with the reduced geometric cross-sectional characteristics (model A – SAP2000).

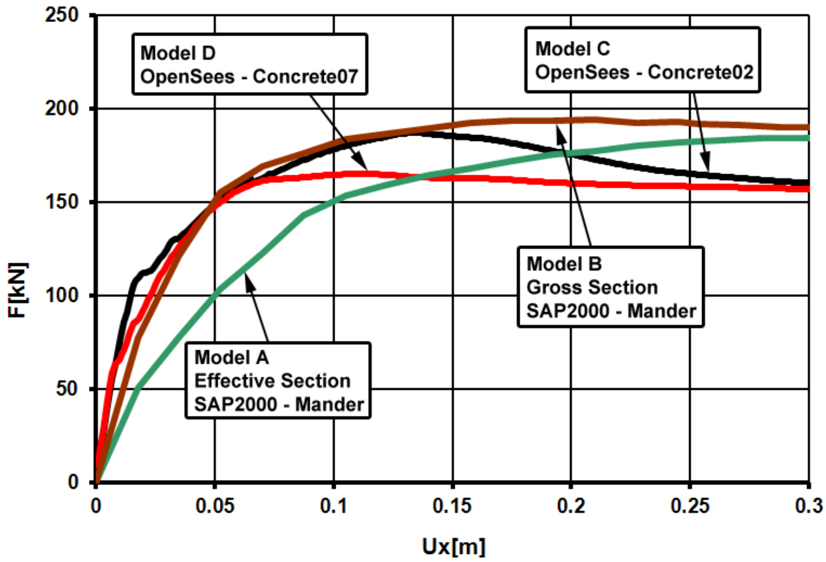


Figure 9. Pushover curves

In more pronounced nonlinear behaviour, significant differences in the character of the force – displacement relationship can be observed. Model B (SAP2000) and model D (OpenSees) are characterized by a very slight force decrease in the increment of the displacement, and to achieve the same value of displacement, more lateral loading in model B (SAP2000) is necessary._Model A (SAP2000) has a permanent increase of the force, while in model C (OpenSees) is observed force decrease for increase of the displacement. In models A and B significant differences in the lateral load values are observed at the same horizontal displacement of the control node due to different values of the cross-sectional effective stiffness.

5. CONCLUSION

With the development of computers in the engineering practice, the methods of analysis based on the nonlinear behaviour of structures are gradually introduced. Nonlinear relationship between force and displacement in multi-storey building structures may be determined easy enough with the application of nonlinear static pushover analysis. The paper presents different possibilities for modelling plastic hinges for the nonlinear static analysis of reinforced concrete plane frame, using the programmes SAP2000 and OpenSees, and using a fibre model for the introduction of the forces of nonlinear distortions into the calculations. A variety of stress-strain relationships for concrete and reinforcement, and a comparison of pushover curves obtained by different designed models, are analyzed.

On the basis of the numerical analyses, it can be concluded that the results of the nonlinear static pushover analysis, obtained using the programmes SAP2000 and OpenSees, satisfactory match from the point of view of character changes in the force – displacement relationship, except in the cases of calculations with effective cross-sectional characteristics (model B). In applying this calculation model, the significant differences of character changes of the force – displacement relationship in relation to the curve obtained using the models A, C and D are obtained.

AKCNOWLEDGEMENT
The work has been done within the scientific research project TR 36043 “Development and application of a comprehensive approach to the design of new and safety assessment of existing structures for seismic risk reduction in Serbia”, which is funded by the Ministry of Science of Serbia.

REFERENCES

Ady Avram, Kevin R. Mackie, Božidar Stojadinović (2008): Guidelines for Nonlinear Analysis of Bridge Structures in California, Pacifik Earthquake Engineering Research Center, PEER 2008/03.

FEMA 356 (2000): Prestandard and Commentary for the Seismic Rehabilitation of Buildings, ASCE, Federal Emergency Management Agency, Washington D. C.

CSI (2005): SAP2000 – Linear and Nonlinear Static and Dynamic Analysis and Design of Three-Dimensional Structures: Basic Analysis Reference Manual. Computers and Structures, Inc. Berkeley, California.

Lađinovic Đ., Ćosić M. (2008): Pushover analiza armiranobetonskih okvira. Simpozijum SGIT Srbije, Sokobanja, str. 113-120.

Radujković A., Rašeta A., Lađinovic Đ. (2006): Mogući mehanizmi loma petospratne ramovske konstrukcije, 12. kongres JDGK, Vrnjačka Banja, str. 47-52.

Silvia Mazzoni, Frank McKenna, Michael H. Scott, Gregory L. Fenves, et al. (2009): Open System for Earthquake Engineering Simulation: User Command-Language Manual, Pacific Earthquake Engineering Research Center, University of California, Berkeley, OpenSees version 2.0.