NONLINEAR SEISMIC TIME-HISTORY ANALYSIS OF RC FRAME ACCORDING TO EN 1998-1

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Summary: *The design of earthquake-resistant structures requires the use of methods that allow a good insight into the value of seismic requirements (stiffness, strength, ductility and energy dissipation). Only nonlinear analysis methods can quantitatively and qualitatively estimate the level of achieved displacements and deformations, which are the key values for analyzing the seismic safety of a building. The most relevant insight into the nonlinear response of the structure can be obtained by non-linear dynamic analysis. This paper presents the results of the nonlinear dynamic analysis of a multi-storey reinforced concrete frame according to EN 1998-1, exposed to seismic actions using the accelerograms.*

Keywords: *Time history analysis, rc frame, stiffness, strength, ductility*

1. INTRODUCTION

The primary purpose of building design in seismically active areas is the provision of adequate safety and performance of buildings during and after earthquakes. The basic behaviour requirements for structural design in seismic areas (EN 1998-1, [1]) demand that the structure does not collapse and limit the damage level of design seismic actions.
The methods used for seismic design of buildings in seismically active areas are based on the method of the linear - elastic analysis. The ability of the structure to 'dissipate' a part of the energy input using the inelastic deformation during an earthquake is taken into account by reducing the seismic forces. The design with the linear methods provides a good insight into the size of seismic forces, but not a good estimate of displacements and deformations, which are crucial for the evaluation of seismic safety of structures. Only the application of nonlinear analysis methods can qualitatively and quantitatively

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achieve the level of damages during strong earthquakes. In EN 1998-1 two methods of nonlinear analysis are prescribed: nonlinear static (pushover) analysis and nonlinear time-history analysis. The highest quality of the dynamic response insight of the structure is obtained using the nonlinear dynamic analysis. However, one must not forgot that the uncertain input data related to seismic loads and structural behaviour in the nonlinear range cause uncertain analysis results.
Application of response spectrum is the basic way to determine the seismic actions in the case of building construction. However, the synthetic and/or real accelerograms could be used for the alternative views on an earthquake action. Regulation EN 1998-1, [1], prescribes the requirements for the analysis that use artificial and/or real accelerograms. The minimum number of accelerograms is three; if less than seven time records are used, the worst results will be adopted; and for seven or more records the mean values can be used. The averaged values of acceleration for zero period should not be smaller than the a_gS for the observed location in the interval period from $0.2T_1$ to $2T_1$; no value of the mean 5 % damping elastic spectrum, calculated from all time histories, should be less than 90 % of the corresponding value of the 5 % damping elastic response spectrum.

2. NUMERICAL EXAMPLE

According to EN1992-1-1, [2], and EN1998-1, [1], five-storey structure is designed for ductility class DCH. The structure is symmetrical in both directions, with the range of 3x5m and the storey height of 3m. As the structure satisfies the criteria of the regularity basis, it can be analyzed as 2D structure, [1], Figure 1.

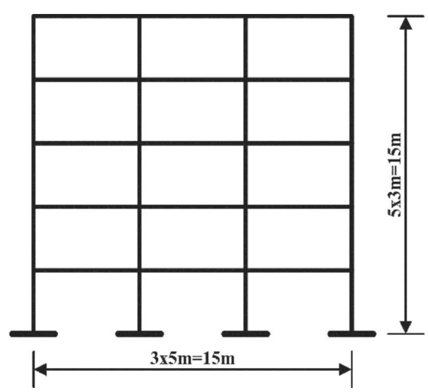

Figure 1. The observed frame of the structure

Numerical analysis uses the planar model of the inner frame with the T-section beam and the effective width of 170 cm. Slab thickness is 15cm, cross-sectional dimensions of beams are 30/45 cm, while the dimensions of the columns are 45/45 cm. In order to consider the influence of cracks, flexion and shear properties of the elements are reduced to the half of the uncracked section values. For the ductility class DCH concrete class C 35/45 is used, with the Poisson ratio $\nu = 0$ (cracked concrete), and steel S 500 class C. The permanent load "*G*" includes the self weight of the elements and the dead added permanent load in the amount of 2.5 kN/m^2. The imposed load "*Q*" for the building category B, according to [3], is taken as equally distributed with 2.5 kN/m^2 intensity. The seismic action is represented by the horizontal elastic response spectrum of type 1 and the soil category C with a maximum acceleration of soil 0.2 g. Values of the period and soil factor that describe the shape of the elastic response spectrum are $T_B = 0.2$ s, $T_C = 0.6$ s, $T_D = 2$ s, $S = 1.15$, and for the damping correction factor is $\eta = 1$. The building is classified as a building of significant class II and the importance factor is $\gamma = 1$. Elastic analysis is carried out based on the design response spectrum, which is reduced in comparison to the elastic spectrum using the behaviour factor q. The value of the design seismic load determined for the ductility class DCH is $q = 5.85$. Inertial effects of the seismic design action are calculated based on the weight associated with all gravity loads (permanent and imposed) that occur in the corresponding combined actions of roof $G + 0.3\ Q$, and for other stories $G + 0.15\ Q$, [4]. The mass of the top storey of the frame structure is 53.57 t, and for other stories is 54.2 t. The total mass of the designed frame structure is 271.65 t. Two dominant tones are $T_1 = 0.791$ s and $T_2 = 0.254$ s. Adopted reinforcement is shown in Figure 2.

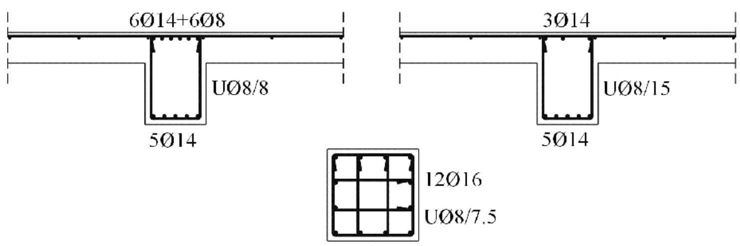

Figure 2. Adopted reinforcement: beam-support, beam- field and column

Seismic action for nonlinear dynamic analysis is introduced over seven recorded (real) accelerograms. Characteristics of applied accelerograms are: $a_{max} \approx 0.23g$, $v_{max}/a_{max} = 0.08$ s to 0.15 s and $M_s = 6.6$ to 7.8. The average response spectrum for all applied accelerograms is shown in Figure 3.

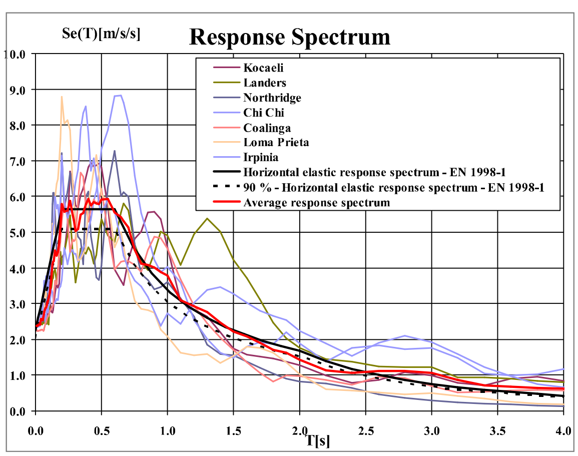

Figure 3. Response spectrum

For the nonlinear analysis, software package SAP2000 V14-2 has been used. The cross-sections are presented as: confined part of the section (the core), unconfined part of the section (a protective layer of concrete to reinforcement) and reinforcement. Relation stress-strain for unconfined and confined part of the cross-section for beam-support and column, for the concrete class C 35/45 and for the steel S 500 are shown in Figure 3.

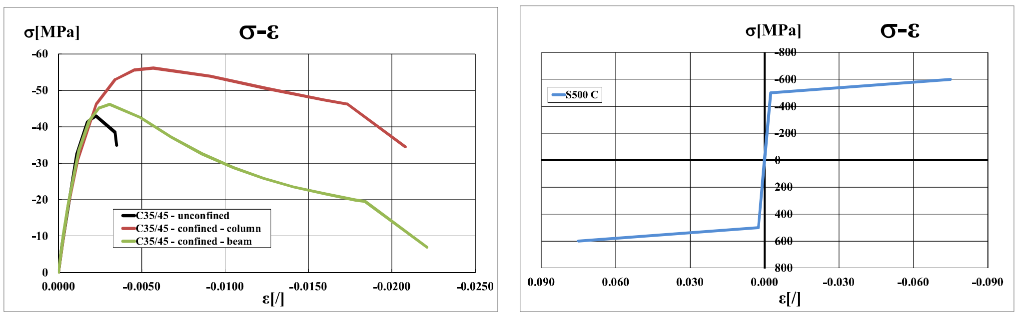

Figure 3. Relation stress-strain for concrete C 35/45 and for steel S 500

The extreme story displacement and interstorey drift of the frame due to the effect of the accelerogram are shown in Figure 4.

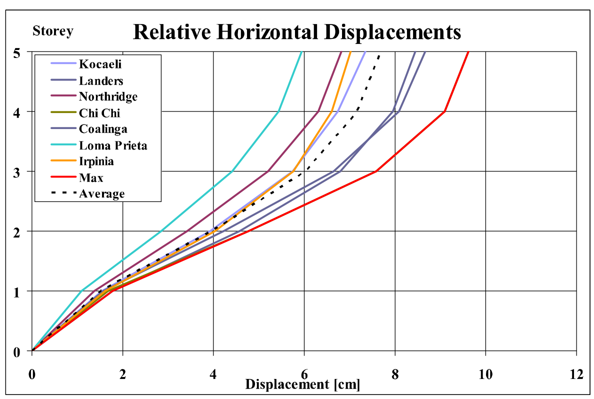

Figure 4. Story dispalcement of the frame

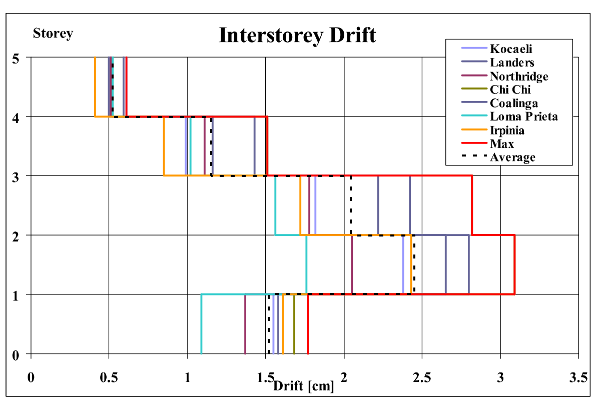

Figure 4. Interstorey drift of the frame

Based on the results of nonlinear time-history analysis it can be concluded that the damage, i.e. inelastic deformation, occurs only in beams and that the behaviour of the columns remains in the linear elastic range. The characteristic arrangement of damages is shown in Figure 5.

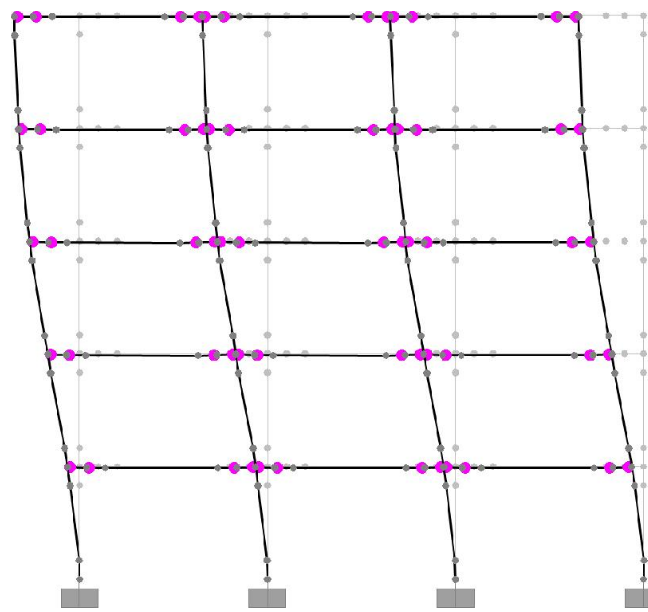

Figure 5. Formation of the plastic hinges (accelerogram Coalinga)

3. CONCLUSIONS

The basic behaviour requirements for structural design in seismic areas (EN 1998-1, [1]) demand that the structure does not collapse and limit the damage level of design seismic actions. Capacity significantly impacts the seismic response of the structure, whether it will be linear or non-linear behaviour during the earthquake. However, to provide the sufficient seismic stability of the structure, the capacity of deformation or displacement is of crucial importance. Elastic acceleration spectrum, which indicates maximum expected seismic forces, may be reduced depending on the behaviour mode and the damage degree that a structural engineer wants to allow. The linear method design gives a good estimate of seismic forces, but for the estimate of seismic performance displacements and deformations are primarily relevant. To achieve the desired behaviour of structures during earthquakes, the balance between stiffness, strength and ductility is the most important. This results in the demand of non-linear analysis methods in the design. The most accurate insight into the achieved level of damages can be obtained by applying the nonlinear time-history analysis.

Real earthquake records, practically, cannot achieve an exact match between the project and the average elastic spectrum according to applied accelerograms. The differences between the maximum relative storey displacements and between interstory drifts are a

consequence of different characteristics of the applied accelerograms, despite the fact that their average response spectrum has a relatively good agreement with the project elastic spectrum.
Based on the results of the nonlinear dynamic analysis it can be concluded that the appearance of the damages occurs only at the ends of the reinforced concrete frame beams. The behaviour of the columns remains practically in the linear-elastic range. This response of the structural system can be considered reasonable.

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НЕЛИНЕАРНА СЕИЗМИЧКА АНАЛИЗА ВРЕМЕНСКОГ ОДГОВОРА АБ РАМА ПРЕМА EN 1998-1

Резиме: *Пројектовање сеизмички отпорних конструкција захтева примену поступака који омогућавају добар увид у величину свих сеизмичких захтева (крутост, носивост, дуктилност и дисипација енергије). Нелинеарним методама анализе се може једино квантитативно и квалитативно проценити ниво достигнутих померања и деформација, а то су кључне величине на основу којих се може анализирати сеизмичка сигурност објекта. Најквалитетнији увид у нелинеарни одговор конструкције се може добити нелинеарном динамичком анализом. У раду су приказани резултати нелинеарне динамичке анализе АБ рама према EN 1998-1 који је изложен сеизмичкој побуди применом акцелерограма.*

Кључне речи: *Нелинеарна динамичка анализа, АБ оквир, крутост, носивост, дуктилност*