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EUROCODE-BASED SEISMIC ASSESSMENT OF FRAME STRUCTURE WITH NON-LINEAR DYNAMIC ANALYSIS

Nonlinear analysis enables engineers to control various aspects of seismic behavior. Utilizing nonlinear analysis, engineers can directly determine inelastic deformations of elements (e.g. rotation), as well as deformations of the structure (inter-storey drift). In addition, non-linear analysis can be used to check the bearing capacity of the elements that should remain in elastic region of deformations in order to prevent brittle failure. Subject of this paper is comparison of the results obtained from linear and non-linear seismic analysis of concrete frame structure designed and detailed according to principles of Eurocode 8 and capacity design method. Linear-elastic seismic analysis of multi-story frame structure was performed, with design and detailing of critical regions according to results and relevant requirements of Eurocode 8. Seismic analysis was performed for seismic zone which corresponds to PGA=0.36g (peak ground acceleration) according to EC8, and for ground type C. Assessment of structure performance during strong ground motions was performed with non-linear time history (dynamic) analysis using software PERFORM 3D (Nonlinear Analysis and Performance Assessment of 3D Structures). Non-linear dynamic analysis was performed for four groups of seven ground motion records that were chosen in order to comply with spectrum defined by Eurocode 8 for analyzed frame structure. Comparison of characteristic results is presented at the end of paper, with conclusions, recommendations and critical assessment of regulation.

Keywords: Eurocode 8, non-linear time history analysis, capacity design method, frame structure.

1. INTRODUCTION

Design process for earthquake-resistant structures is pursuit of balance between the seismic capacity of structures and the expected seismic demand they may be subjected. Seismic excitation is an accidental natural phenomenon, which is manifested through alternating movements of the ground, and thus the movement of the supports-foundations of the structure. Determining the impact in the structure due to this load is much more complex than in the case of other types of loads. An additional complication is cost-effectivness to design structures to the full value of seismic impacts for strong earthquakes, taking into account the probability of occurrence. For these reasons, the concept of seismic analysis with reduced seismic forces was adopted. Reference methods for the seismic analysis of new buildings in Eurocode 8 [1] (hereinafter EC8) are linear-elastic methods. Linear-elastic methods are based on elastic response spectrum with 5% viscous damping reduced with behavior factor. The behavior factor takes into account the ductility capacity of the structure and the energy dissipation capacity. EC8 prescribes also application of capacity design method for the detailing of critical regions. The main feature of the capacity design method is to determine critical regions that will enter into plastic area of deformation during strong earthquakes and the regions that will remain in the elastic area. In that sense, when designing, special attention is to be paid on detailing of the connections between the structural elements and the areas where nonlinear behavior is predicted. Also it is necessary to provide sufficient load bearing capacity of elements that should remain in elastic region in order to prevent brittle failure [2-3]. Calculation algorithm in EC8 also prescribes an analytical criterion for taking into account (P-Δ) effects.

In order to perform a detailed evaluation of the expected seismic response of a structure designed in accordance with linear analysis and estimate deformations and displacements, EC8 recommends use of nonlinear analyses. These analysis can directly determine inelastic deformations of the element (e.g. rotation), as well as deformations of the structure (inter-storey drift). In addition, this analysis can be used to check the bearing capacity of the elements that should remain in the elastic region in accordance with the capacity design. EC8 prescribes use of non-linear static (pushover) analysis and non-linear time history (dynamic) analysis (NDA). [4-5]

2. MODELING AND BUILDING DATA FOR LINEAR ANALYSIS

The analyzed example is a 6-storey spatial reinforced concrete frame structure shown on figure 1. Columns and beams are modeled as prismatic 3D beam elements. Stiffness properties were taken as one-half of the corresponding stiffness of the uncracked elements. For the adopted return period of 475 years, the reference peak ground acceleration is equal to a_{gR}= 0.36g. The ground type at the structure location is C. The structure is designed for high ductility class DCH.

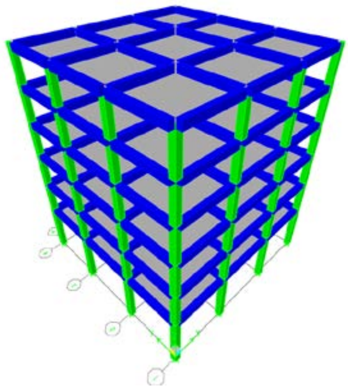

Figure 1. Spatial model of frame structure

The base of the building is rectangular in shape, measuring 16.8 × 16.8 m. In the X and Y directions layout has 3 bays, with dimensions 5.4, 6, 5.4 m respectively. The floor height of the ground floor is 5m, and the other floors 3.2m. The load from the reinforced concrete slab, thickness d = 15 cm, is transferred to the beams with dimensions b / d = 40x60, and from the beams to the columns. The columns have square cross section: the outer columns are 50x50cm and the inner columns are 60x60cm. The concrete strength class according to EC2 is C30/37. Transverse and longitudinal reinforcement is B500 class C. The building is modelled in the ETABS software package. Self-weight of the structure is automatically determined in the software package. The adopted behavior factor is 5.85. The total mass includes the dead load, 15% of the live load and 30% of the snow load. The total mass of the inner frame with the corresponding load on the surface of the slab is M = 5285kN.

The linear analysis was done with lateral force method. In addition, second-order effects had to be taken because the value of the interstorey drift sensitivity coefficient θ was calculated with a value of 0.17 according to the article 4.4.2.2.3 EC8 [1]. Second order effects were taken approximately by scaling the effects according to the calculation below.

$$\frac{1}{1-\theta} = \frac{1}{1-0{,}17} = 1.205 \qquad (1)$$

Based on the results of the calculation, taking into account (P-Δ) effects, detailing was done in accordance with capacity design, and the adopted reinforcement is presented on figure 2.

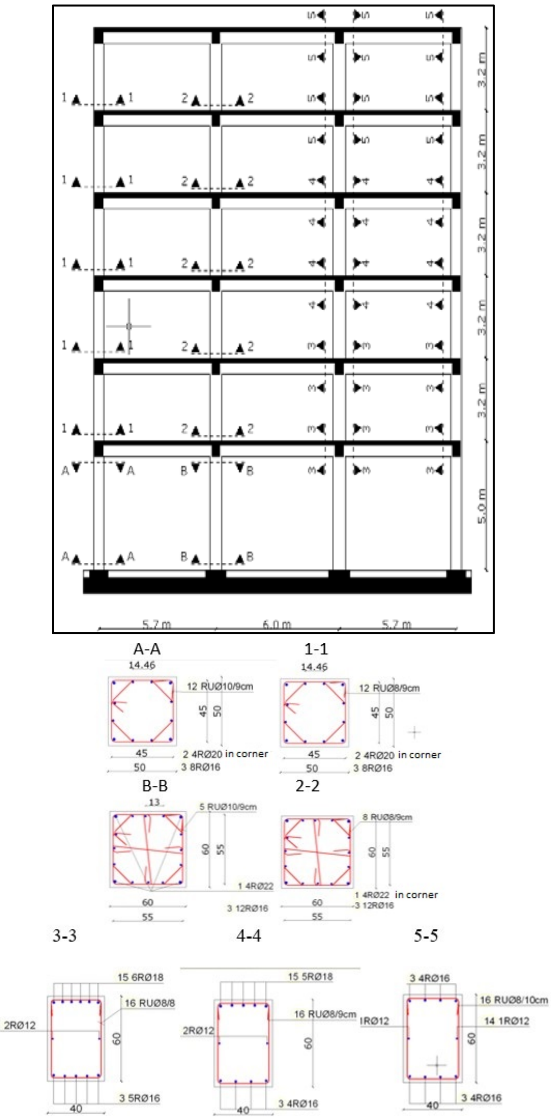

Figure 2. Reinforcement of beams and columns for frame 2-3-B-C

3. MODELING FOR NON-LINEAR ANALYSIS

PERFORM 3d software package [7-9] was used for non-linear modelling. On the location of every slab rigid diaphragm slaving was modelled with masses concentrated on each floor.

In practical modelling for nonlinear seismic analysis, static or dynamic, the reinforced concrete structure is modelled at the element level, and then the element connections are defined. The beam element consists of rigid zone, plastic hinge zone and a central elastic part. Rigid zones are modelled as “default end zones” that are pre-defined in the software and have ten times greater stiffness of the beam and a length equal to ½ the width of the column.

The central part of the beam is modelled as an elastic section with defined load-bearing characteristics in order to be able to control the influences during the seismic action. When modelling, a bilinear moment-curvature model was adopted for beam plastic hinge in accordance with the results obtained in the Xtraxt software (cross sectional analysis of components) for the adopted reinforcement. A model that takes into account the interaction of axial force and moment during an earthquake was chosen for the columns. The elastic-ideal plastic (EPP) moment-curvature model for hinges has been adopted and defined through the component "P-M2-M3 hinge". In addition, for beam and column elements strength sections were assigned (bending and shear) as well as the deformation capacities of plastic hinge to be controlled during the analysis. Beam column joints are modelled as elastic zones ("Elastic panel zone" elements), to behave elastic during earthquake. In accordance with this, the stiffness and load-bearing capacity of every joint was defined. Perform model of joint, beam and column are given on figures 3, 4 and 5.

Figure 3. Beam compound component in PERFORM 3D

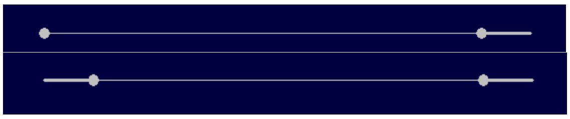

Figure 4. Column compound component for ground storey and other storeys in PERFORM 3D

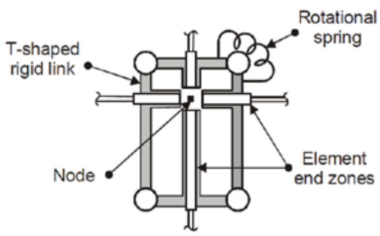

Figure 5. Model of joint with beam and column

Total damping is the sum of elastic and inelastic damping. In nonlinear dynamic analyses, inelastic hysteresis damping is modelled directly, while viscous damping defines elastic damping. PERFORM ("Perform components and elements-Defining the standard for practical performance based design") [7-9] defines two types of viscous damping, namely: modal damping and Rayleigh damping. In this analysis 5% modal damping was combined with

Rayleigh damping and a small amount of βk damping in order to include "higher" modes damping. Second order effects were taken as $(P-\Delta)$ effects.

Following limit states were defined for structure:

- Curvature on the corresponding length of the beam and column plastic hinge in accordance with the no collapse limit state [1]. The curvature capacity of hinge was calculated with Xtraxt software (cross sectional analysis of components) utilising confinement models defined in EC2 and EC8;
- Load capacity of the joint in accordance with the no-collapse limit state. The capacity of the joint is checked for the transmission of moments from the beams on both sides of the joint;
- Shear force capacity for beams and columns in accordance with the no-collapse requirement. The shear capacity of beams and columns is checked whether the elements remain in the elastic area regarding shear during the seismic action;
- Inter-storey drift relative to the damage limitation request is checked;

4. NONLINEAR TIME-HISTORY (DYNAMIC) ANALYSIS

Earthquake records were selected in accordance with EC8 recommendations [1]. One-way records were used considering that the analysed nonlinear model is planar. For analysis with a single earthquake component, EC8 requires that the response due to records does not fall below 90% of the damped elastic spectrum in the 0.2T1 to 2T1 periods.

REXEL v.3.3 (beta) software was used to select the earthquake records. The software was developed at the University of Naples *(Università degli Studi di Napoli Federico II)* by *Iervolino I., Galasso C., Cosenza E* [11]. The subject software enables the selection of records from the database that are compatible with response spectrum. In addition, other seismographic conditions that need to be met by an earthquake (magnitude, epicentral distance, earthquake intensity, type of soil on which the earthquake was recorded) can be defined during the selection. The software includes the *European Strong-motion Database* (ESD), the *Italian Accelerometric Archive* (ITACA) and selected records for the assessment and design of buildings in accordance with the *Selected Input Motions for Displacement-Based Assessment and Design* (SIMBAD) [11].

The response of the structure was calculated for four groups of 7 earthquakes. As a representation, one of 7 record groups is shown on figure 6 and in table 1. This group is selected from Selected Input Motions for Displacement-Based Assessment and Design (SIMBAD), magnitude 6.5 <M <7.5, soil type C, maximum epicentre distance from earthquake 30km and within 90% to 130% of the spectrum value.

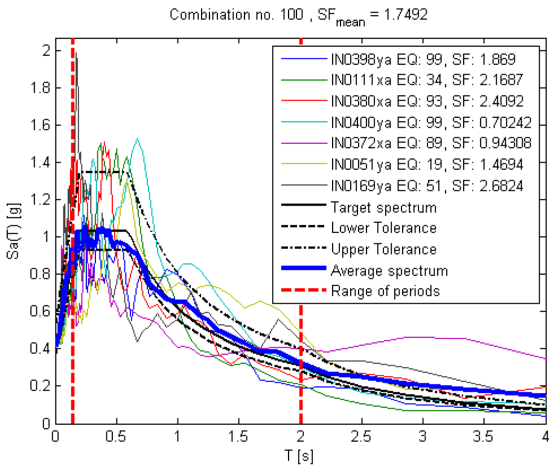

Figure 6. Selection of records in accordance with response spectrum for ground type C

Table 1. Group of selected records from SIMBAD base, ground type C, magnitude 6.5-7.5

ID	Name	ED [km]	PGA [m/s²]
398	Northridge	20	2.17
111	Hyogo - Ken Nanbu	17.45	1.87
380	Superstition Hills	19.51	1.68
400	Northridge	20	5.78
372	Imperial Valley	27.47	4.31
51	NW Off Kyushu	26	2.76
169	Iwate Prefecture	27	1.51
mean		22.49	

5. RESULTS AND DISCUSSION

Presentation of results for four groups of 7 earthquake records are compared in order to verify the basic design objective and defined limit states. As a representation of software output, response of the structure due to the earthquake record "Imperial Valley" which caused largest response from structure was presented. Deformations of plastic hinge did not exceed the fracture limit X defined in F-D diagrams in PERFORM 3D [7-9]. The figures 7 and 8 presents the frame after the earthquake with the D/C ratio of plastic joints, as well as the frame with all the components that entered the plastic area during the earthquake. It can be concluded that plastic hinges appeared in defined locations. Plastic hinges appeared on top of the columns on the second, third and fourth story. Hinges didn't appear on the other end of the columns so "soft story mechanism" was avoided. The maximum D/C factor ratio of 0.83 was recorded in the beams at the first floor.

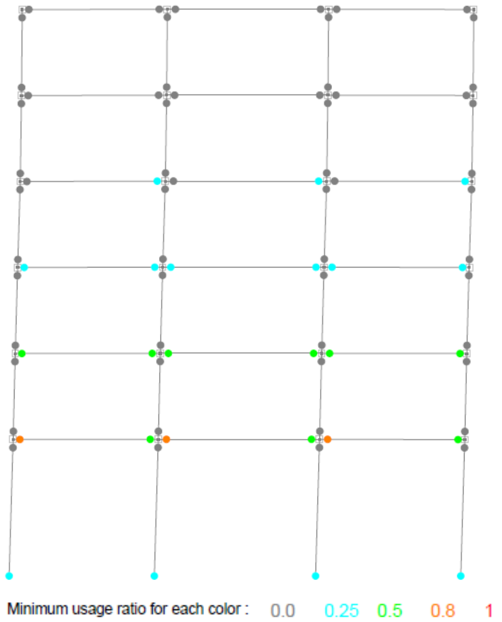

Figure 7. D/C ratio for plastic hinges for „Imperial Valley"

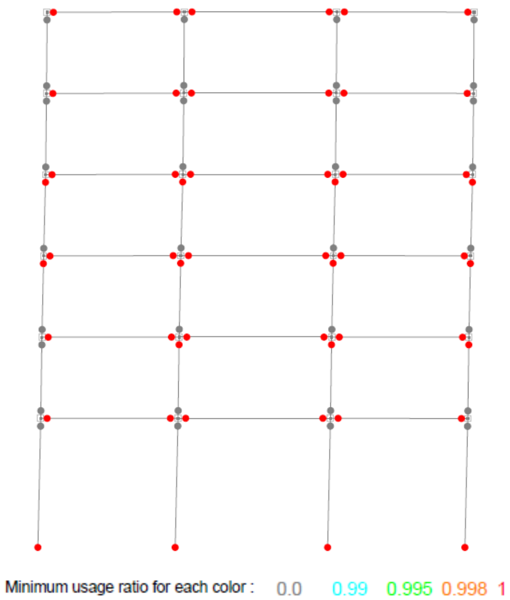

Figure 8. Hinges that have yielded „Imperial Valley"

The table 2 gives the maximum value of D/C ratio for rotation capacity of plastic hinges four groups of 7 records in accordance with the "no collapse" requirement.

Table 2. Overview of maximum values of D/C ratio for the limit state of deformation of plastic joints of beams and columns

Hinge rotation capacity – D/C ratio	**Beam**	**Column**
1. group of 7 records	0.60	0.34
2. group of 7 records	0.65	0.34
3. group of 7 records	0.84	0.40
4. group of 7 records	0.82	0.35
Medium value of 28 records	**0.73**	**0.37**

Load capacity of the beam-column joint was defined via moment which joint can transmit and load capacity of beam and column was defined for shear strength of cross section. Results of analysis for example earthquake shown on figure 9 and 10 present that load bearing capacity of elements was sufficient.

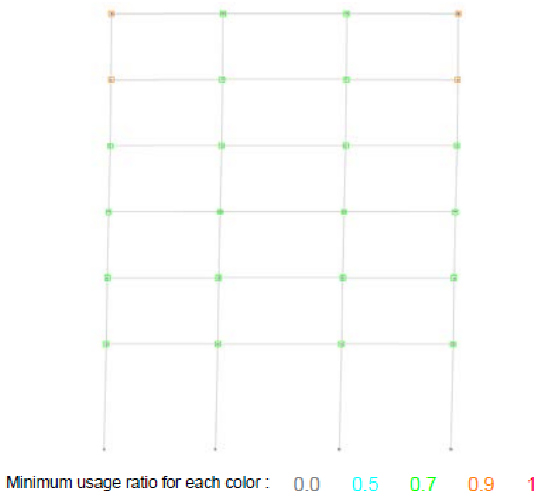

Figure 9. D/C ratio joint capacity „Imperial Valley"

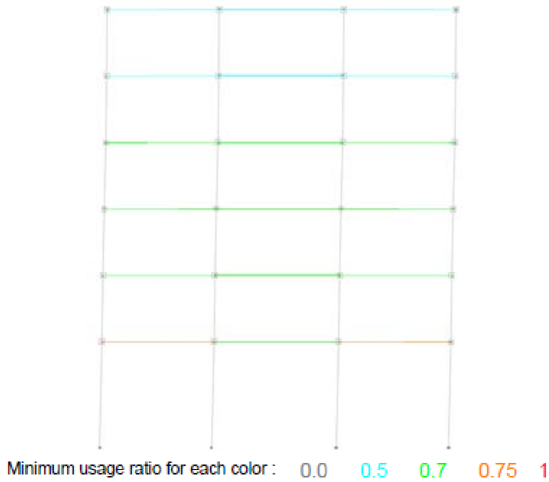

Figure 10. D/C ratio shear strength capacity of beams „Imperial Valley"

The table 3 provides information on the maximum value of D/C ratio (demand/capacity) for shear capacity of joints, beams and columns for four groups of 7 records in accordance with the “no collapse” requirements.

Table 3. Overview of maximum values of D/C ratio for the limit state of load bearing capacity of beams, columns and joints

Load bearing shear capacity – D/C ratio	Joint	Beam	Column
1. group of 7 records	0.87	0.71	0.34
2. group of 7 records	0.88	0.72	0.34
3. group of 7 records	0.89	0.72	0.34
4. group of 7 records	0.89	0.72	0.34
Average value of 23 records	**0.88**	**0.72**	**0.34**

The “damage limitation requirement” stipulates that the structure must be designed and constructed to withstand a seismic action that are more likely to occur than the design seismic action corresponding to the "no-collapse requirement”, without the damage and restrictions in use. This requirement is fulfilled if inter-storey drifts are checked in accordance with article 4.4.3.2. (a, b, c) [1]. The control of inter-storey drifts were checked and results are presented on figure 11.

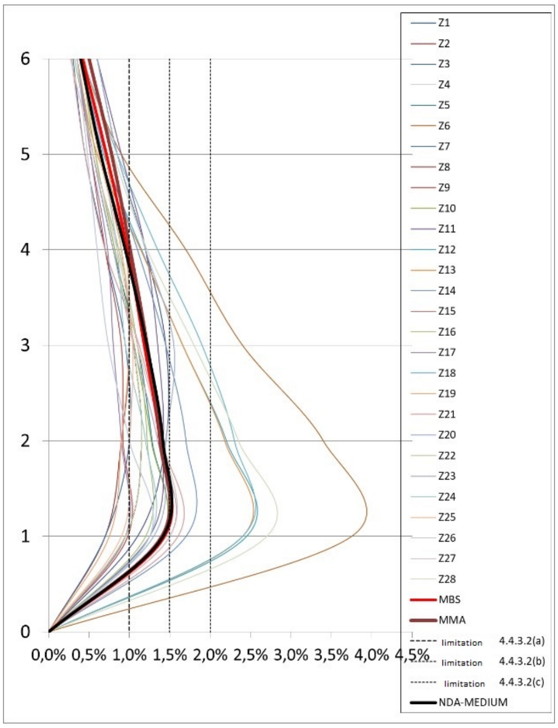

Figure 11. Inter-storey drift from linear and non-linear NDA analysis

In the figure 11 representation of inter-storey drifts are given for linear analysis and NDA for all 28 records including mean diagram from all 28 records. In all NDA analysis the maximum inter-storey drift occurred on the first floor. NDA analysis have large dispersion of results. The maximum NDA drift exceeds 150% value obtained from to linear static analysis. On the other hand the mean diagram of all 28 NDA deviate ± 10% from the values obtained by linear analysis. It must be clarified that diagrams shown in Figure 11 for NDA are determinate for the design seismic action and are not suitable for controlling the damage limitation requirement explicitly because NDA results cannot be linearly scaled.

5. CONCLUSIONS

Based on the results presented in this paper, the following conclusions can be drawn:

• EC8 requires that beams and columns are safe for unfavourable brittle failure due to shear in plastic hinges, which in the case of a analysed structure, was satisfied in each section. In addition, with a larger amount of transverse reinforcement, a higher capacity of rotation of plastic joints is enabled, i.e. ductility of curvature.

• Provided load-bearing capacity of the joints as a result of the capacity design was sufficient in each joint of the frame in accordance with the results of the NDA analysis. The joint reinforcement calculation model from EC8 recommends a higher amount of reinforcement compared to the Paulay and Priestley model which provides additional joint protection [6].

• The rotational capacities for plastic hinges of the beams and columns were satisfied. D/C ratio ranged up to 83%. This way, the EC8 recommendation that the D/C ratio for deformation was met with the appropriate safety coefficients for constituent materials.

• Damage limitation state defined in EC8 for inter-storey drift was met in linear-elastic analysis for limit b and c, and for NDA with 3% margin. It can be concluded that the structure has met the “damage limitation requirement“ for buildings with ductile non-structural elements. NDA analysis have large dispersion of results. The maximum NDA drift exceeds 150% value obtained from to linear static analysis. On the other hand, average value of drift, considering that more than min 7 EQ ground motion are used for NDA, correspond to the values obtained from linear analysis.

In accordance with all the results and conclusions, the linear-elastic seismic analysis of reinforced concrete frames regular in plan and elevation very well describes the behaviour of the structure under the action of the design seismic load. Further research on this subject should be performed on the RC structures with other structural types and structures classified irregular in plan and elevation in order to provide critical assessment of regulation.

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