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NON-LINEAR ANALYSIS OF BUILDING STRUCTURES IN SEISMIC AREAS ACCORDING TO THE EUROPEAN STANDARDS, CASE STUDY

Abstract:

For the design and construction of buildings in seismic areas the European Standard EN 1998-1: 2004 offers two non-linear methods, namely: a non-linear pushover based static method and a non-linear dynamic method. This paper discusses those methods which differ from one another in respect to accuracy, simplicity and transparency. Non-linear static procedures were developed in the world with the aim of overcoming the insufficiency and limitations of linear methods, whilst at the same time maintaining a relatively simple application. All procedures incorporate performance-based concepts paying more attention to damage control. Application of the presented procedures is illustrated by means of an example of an eight-story reinforced concrete frame building.

Keywords: non-linear static procedure, pushover, non-linear dynamic time-history analysis, seismic demand, seismic capacity

НЕЛИНЕАРНА АНАЛИЗА КОНСТРУКЦИЈА ЗГРАДА ПРЕМА ЕУРОПСКИМ НОРМАМА ЗА СЕИЗМИЧКА ПОДРУЧЈА, НУМЕРИЧКИ ПРИМЈЕР

Сажетак:

За пројектирање и изведбу зграда у сеизмичким подручјима Европска норма ЕН 1998-1:2004 прописује двије нелинеарне методе и то: нелинеарну статичку методу поступног гурања и нелинеарну динамичку методу у времену. Овај рад презентира ове методе, које се међусобно разликују с обзиром на точност, једноставност, транспарентност и теоријске подлоге. Нелинеарни статички поступци развијени су у свијету с циљем превладавања недостатака и ограничења линеарних метода, истодобно пружајући релативно једноставну примјену. Нелинеарни статички поступак презентиран у овом раду укључује "*performance-based*" концепт. Примјена представљених поступака илустрирана је на примјеру осмокатне оквирне армирано-бетонске конструкције зграде.

Кључне ријечи: нелинеарна статичка анализа, поступно гурање, нелинеарна динамичка анализа, сеизмички захтјев, сеизмички капацитет

1. INTRODUCTION

In order to pay greater attention to damage control in the past ten years new methods of seismic analysis containing performance-based engineering concepts have been developed in the world. For obtaining seismic actions more realistically the displacement-based approach has proven itself as a much better choice than the traditional force-based approach.

The European standard EN 1998-1: 2004 [1] offers four methods of design for the effects of earthquake forces, Table 1: two linear methods (the force-based approach) and two non-linear methods (the displacement-based approach), namely non-linear static pushover method and nonlinear dynamic time-history method.

Between the linear methods and non-linear dynamic analysis, a non-linear static approach based on the pushover analysis is being imposed as a link and the most economical solution at the moment.

Table 1. Metods of seismic analysis of structure defined in EN 1998-1:2004

Analysis of structure	Static	Dynamic
Linear	Linear analysis using equivalent static forces	Modal analysis using response spectra
Nonlinear	Non-linear static analysis (pushover-based method)	Non-linear dynamic time-history analysis

The most precise description of the problem is by far the non-linear dynamic seismic analysis, made by applying time-history records which, in the long term, represents the correct development path. Yet, due to its complexity and high standards it goes beyond the frames of practical application and is appropriate only for the research and analysis of structures of special significance.

Nonlinear static structural analysis is based on the N2 method. This pushover-based method was developed at the University of Ljubljana [2], [3] and has found its place in the European standard EN 1998-1. Originally, this method was intended to be use in design of regular structures where only the first mode is predominant. The development of N2 method has resulted in a wide range of its use, namely:

- Analysis of regular constructions in which higher modes are also taken into account [4],
- Analysis of structures with special impact of torsion [5].

In the second generation of the European standard EN 1998, which is in the process of being adopted and which contains radical changes to the current standard, torsion effects and higher-mode effects have been taken into account through correction factors [6]. Specifically, the second generation of the current European standard EN 1998-1: 2004 is divided into two parts, EN 1998-1-1 and EN 1998-1-2. Part EN1998-1-1 defines new elastic and reduced response spectra and data relating to the application of all parts of the second generation of Eurocode 8. Part EN 1998-1-2 applies to buildings only. A complete second generation of Eurocode 8 is expected to be technically complete by the end of 2022, followed by its translation into the official EU languages [6].

There are nonlinear static methods that have already been developed in the world, e.g. in the USA [7], which will not be discussed here.

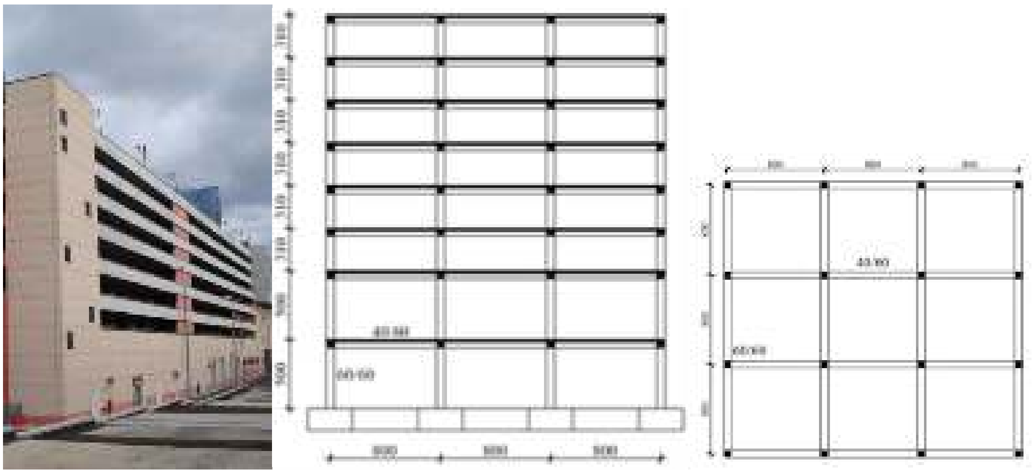
Both nonlinear methods are presented in this paper using the example of the reinforced concrete frame structure, Fig. 1, i. e. the obtained results for nonlinear static method are compared to the "accurate" results reached by a non-linear time-history analysis.

2. DESCRIPTION OF THE BUILDING AND THE LOADING (SEISMIC DEMAND)

Application of non-linear static and dynamic procedures is illustrated here by means of an example of an eight-storey reinforced concrete frame building. The first two storeys are 5.00m high and the other 3.10m (Fig. 1).

In order to perform nonlinear analysis, cross sections and the amount of reinforcement must be assumed first (Fig. 2), and as a result deformations (displacements and relative displacements), i.e. structural damage (plasticization of certain cross sections) will be obtained, while in linear methods the reinforcement is obtained as the ultimate result.

All the columns have dimensions 60x60cm with steel reinforcement equal for all cross sections. The beams have dimensions 40x60cm, also with steel reinforcement equal for all cross sections, Fig. 2. The plate is 20cm thick. The concrete is C25/30 class and the steel reinforcement is B500. Story frame mass for 3.10m high stories is 66.96t and story mass for 5.00m high stories is 73.80t which results in total mass of 549.36t.



(a) (b) (c)

Figure 1 *(a) Building of garage of the Tower Centre, Rijeka, Croatia; (b) Cross-section; (c) Plan of one segment of the garage structure*

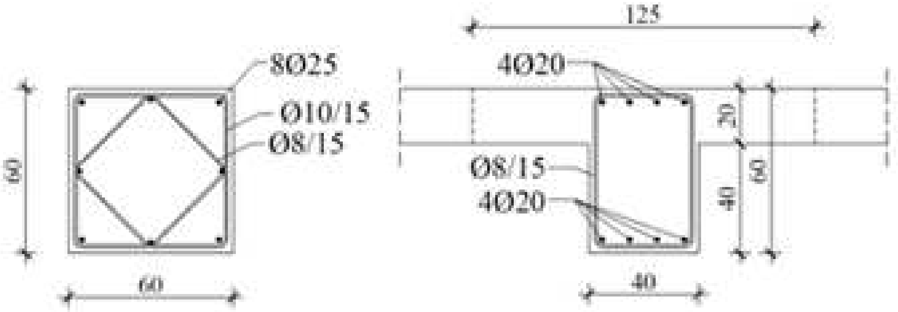


Figure 2 *Cross-sections of columns and beams with steel reinforcement*

The structure in Fig. 1 was designed according to the European standard EN 1998-1: 2004 with the following parameters: ground type B, importance class II (γ_I=1), Type 1 elastic response spectra (the expected surface-wave magnitude M_s larger than 5.5) and viscous damping ratio (in percent) $\xi = 5\%$. The analysis will be performed for the reference peak ground acceleration $a_{gr} = 0.3g$. A behaviour factor $q = 5.85$ was taken into account for the DCH (Ductility Class High) structures.

Since the structure meets the regularity requirements by its plan view and by its height, the current analysis was made on one plane frame, Fig. 3. Due to symmetry only one direction of seismic action was analysed and the fundamental period $T_1 = 1s$ for plane frame is obtained. According to the previously introduced parameters, the elastic acceleration response spectrum and the corresponding design spectrum are presented in Fig. 4 which represents the seismic demand. The fundamental period is in the spectrum range with constant velocities ($T_C < T_1 < T_D$).

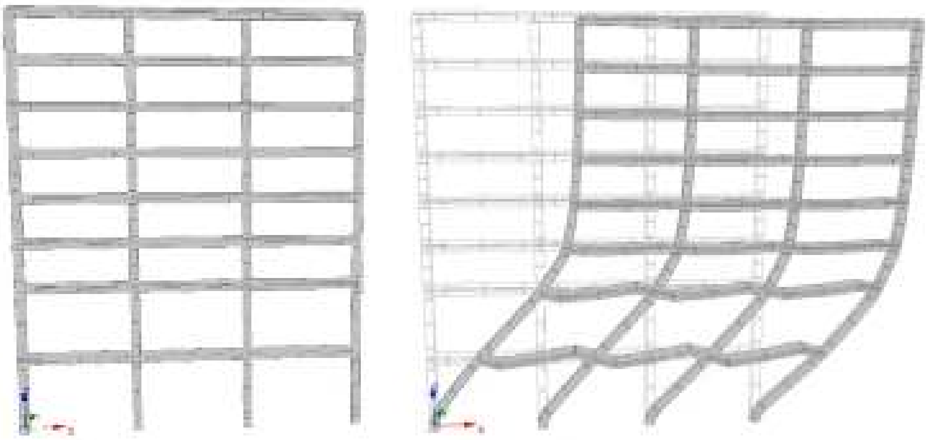


Figure 3 *Plane frame of the current structure and its fundamental mode (*$T_1 = 1s$*)*

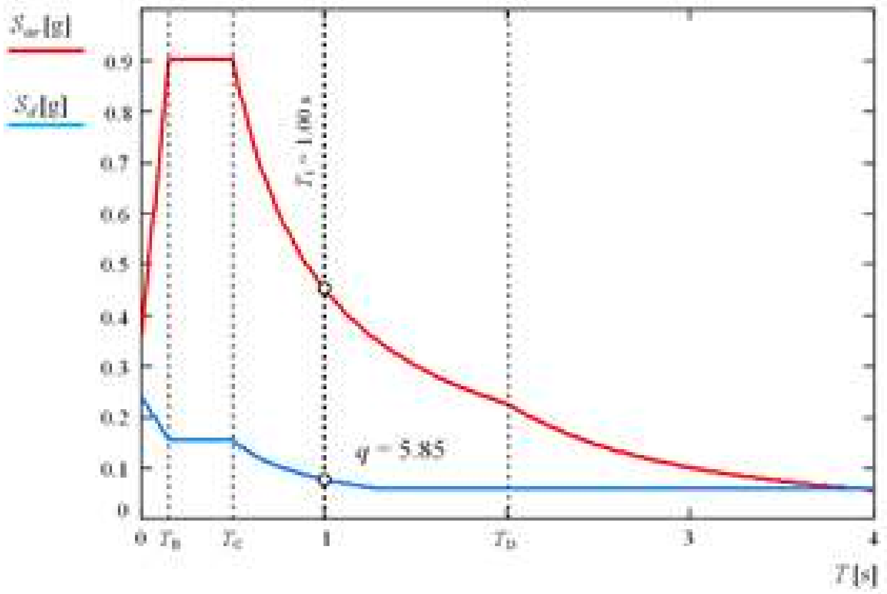


Figure 4 *Elastic acceleration response spectrum (read) with 5% viscous damping ratio for peak ground acceleration 0.3g, ground type B and corresponding design spectrum for behavior factor 5.85 (blue)*

The pushover and time-history analyses were performed by using the SeismoSoft programs [8], [9] whose main purpose is nonlinear static and dynamic analysis of frame structures. Other software packages may be used for the same purpose [10], [11]. Material inelasticity and the cross-section behavior are represented through the so-called fiber modelling approach where each fiber is associated with a uniaxial stress-strain relationship. Each cross section has a number of fibers (200 to 400) and for each fiber a non-linear ratio ε-σ is defined.

A typical reinforced concrete section consists of unconfined concrete fibers, confined concrete fibers and steel fibers, Fig. 5. A non-linear constant confinement concrete model and bilinear steel model with kinematic strain hardening are used. An incremental iterative algorithm with the employment of Newton-Raphson procedures is used to obtain the solution. The dynamic time-history analysis is computed by direct integration of the equations of motion with the Newmark scheme.

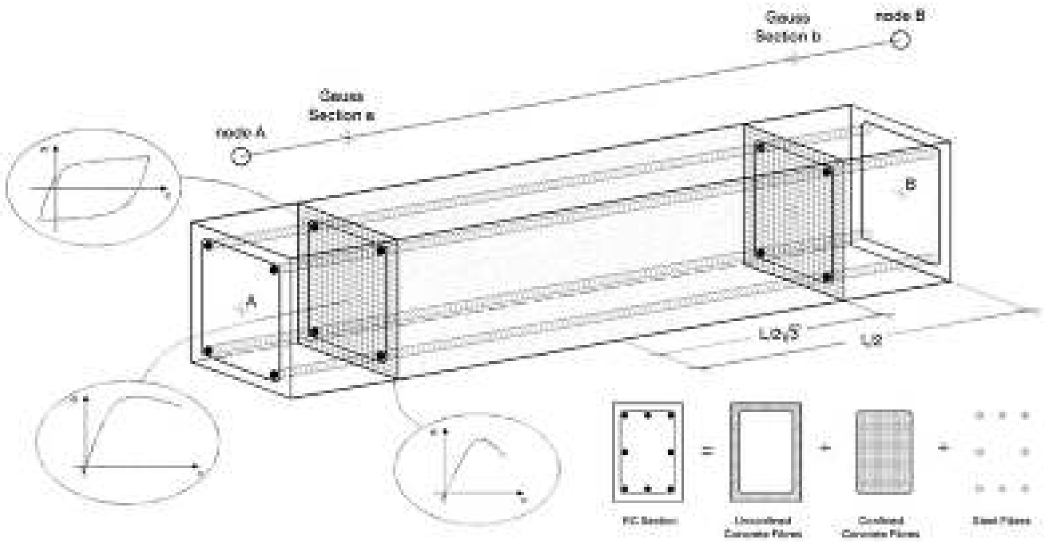


Figure 5 *Fiber modelling approach: unconfined concrete fibers, confined concrete fibers and steel fibers*

3. THE NON-LINEAR STATIC PUSHOVER METHOD

The non-linear static pushover method used in Eurocode 8 [1] is based on the N2 method [2], [3]. The N2 method combines the pushover method of model with several degrees of freedom with a spectral analysis of the equivalent system with one degree of freedom, hence the name. The letter N states that it is a non-linear analysis and the number 2 states that two mathematical models are applied.

The main assumptions in the N2 method are: 1) its application to the structures which have no significant contribution of higher vibration modes; 2) the predominant mode does not change when the seismic intensity is changed (due to the formation of plastic hinges).

In case of the analyzed structure in Fig. 1, the acceleration spectrum which represents **the seismic demand** is plotted in Fig. 4.

The N2 method gives in the given procedure the elasto-plastic **capacity of structure**, Fig. 6.

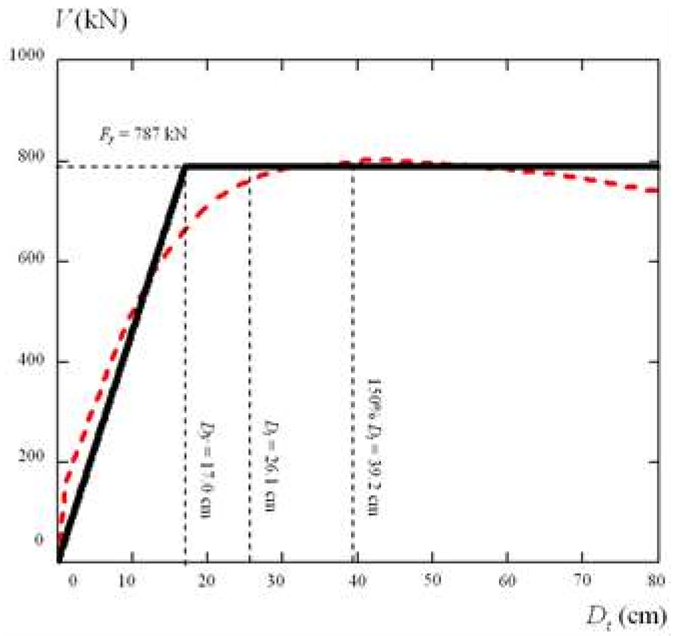


Figure 6 *Capacity curve for the assumed triangular displacement form: the dotted line for the real capacity curve and the solid line for the elasto-plastic idealization*

The method uses a non-linear spectrum in the acceleration – displacement (AD) format, which is obtained in the procedure given in [2], [3], [12]. The format AD enables **the simultaneous review of seismic demand and structural capacity**, Fig. 7. Intersection of seismic demand and structural capacity curves represents **the required target displacement** which corresponds to the required ductility μ.

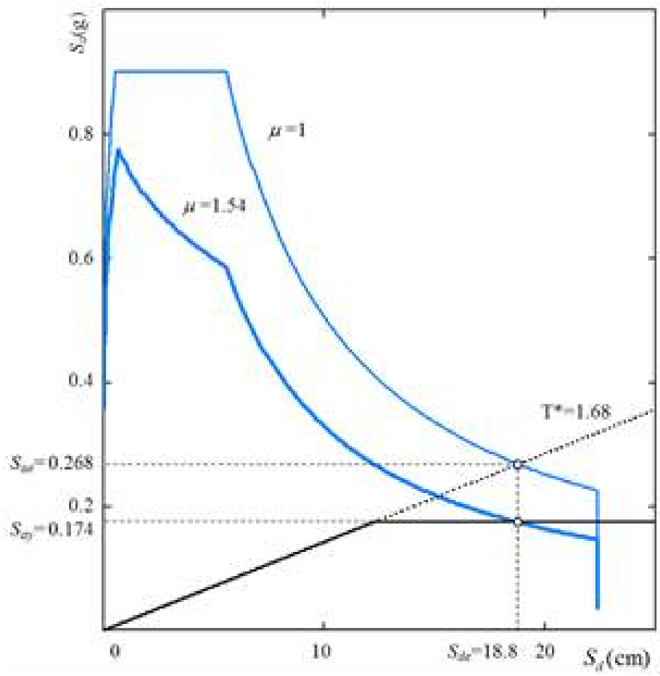
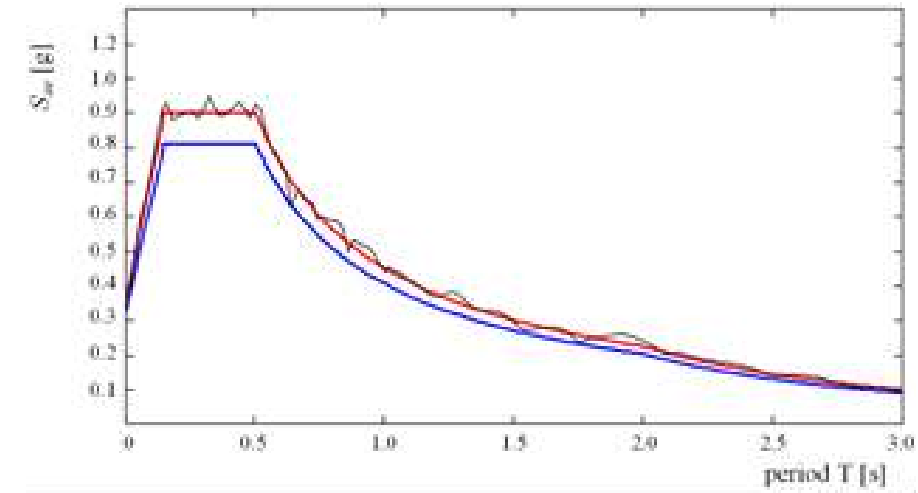


Figure 7 *The demand spectrum for ground acceleration 0.3g (soil class B) and capacity spectrum for structure in Fig. 1 ($\mu = 1.54$)*

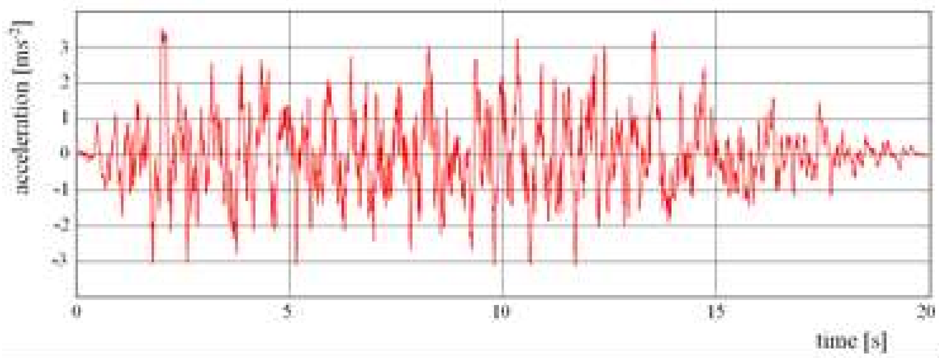
4. THE NON-LINEAR DYNAMIC TIME-HISTORY METHOD

In order to evaluate the results obtained by non-linear static methods, a time-history dynamic analysis was conducted first by using a total of 14 time-history records, seven of which were artificial, and the remaining seven as authentic (real). The artificial time-history records were used for obtaining the mean value of structural responses which corresponds to the specified seismic demand. Processing of artificial and real time-history records was performed by using the *SeismoSignal* program.

Seven artificial time-history records for this example were generated by the program SIMQKE_GR (SIMulation of earthQuaKE GRound motions – Massachusetts Institute of Technology) [13] for peak ground acceleration of 0.3g and soil class B with a 5% viscous damping ratio. The earthquake duration was set on 20s. In the zone near the fundamental period there is no value of elastic spectrum which is calculated from all artificial time-history records, which is less than 90% of the corresponding value of the elastic spectrum response [1] (Fig. 8).



(a)

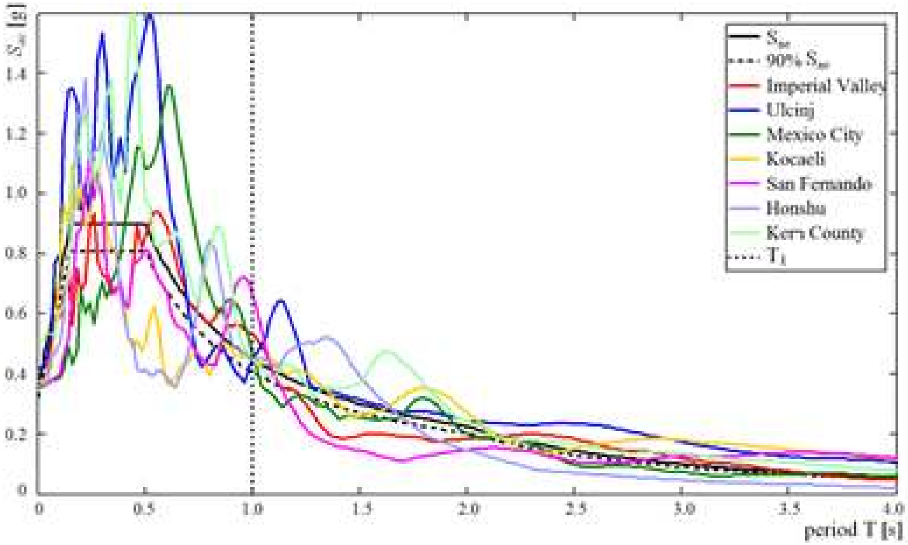


(b)

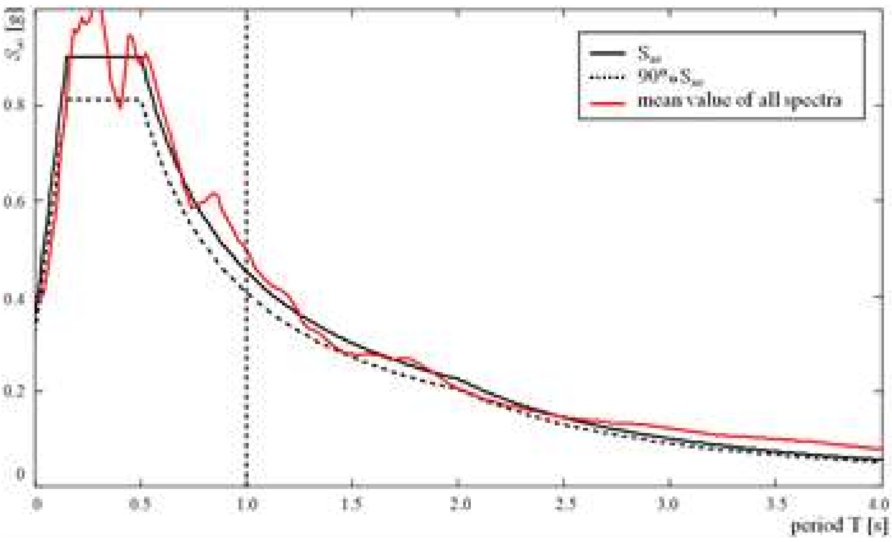
Figure 8 *(a) Response spectrum with 5% viscous damping ratio for ag = 0.3g and soil class B (in red), its 90% value (in blue) and the response spectrum for the artificial time-history record no. 1 (in black); (b) the corresponding artificial digitalized time-history record no. 1*

The real time-history records were taken from the libraries of The National Information Service for Earthquake Engineering, Berkeley, California and The Canadian Association for Earthquake Engineering (CAEE) [14]. All the selected real time-history records were registered on the soil class A or B. The ratio of the maximum velocity to maximum acceleration (v_{max}/a_{max}) for all the selected records lies within the interval from 83 to 125, which corresponds to earthquakes of medium intensity [15]. The following records were selected for obtaining the acceleration response spectrum, Fig. 9:

- Imperial Valley, California, USA (May 18th, 1940, El Centro);
- Ulcinj, Montenegro (April 15th, 1979, Hotel Albatros, Ulcinj);
- Mexico City, Mexico (September 19th, 1985, La Villita, Guerrero Array);
- Kocaeli, Turkey (August 17th, 1999, Sakaria);
- San Fernando, California, USA (February 9th 1971, 3838 Lankershim Blvd., L.A.);
- Honshu, close to the east coast, Japan (August 2nd, 1971, Kushiro Central Wharf);
- Kern County, California, USA (July 21st, 1951, Taft Lincoln School Tunnel).



(a)



(b)

Figure 9 *(a) Acceleration response spectrum of the selected real earthquakes together with the required response spectrum and its 90% value; (b) mean value of all spectrum*

5. COMPARATIVE ANALYSIS OF OBTAINED RESULTS

Fig. 10 presents displacement shapes and storey drifts obtained by using artificial records. Fig. 11 shows the results of using real records. It is observed good agreement between the mean values of responses obtained by artificial records to the mean values of responses obtained by real records.

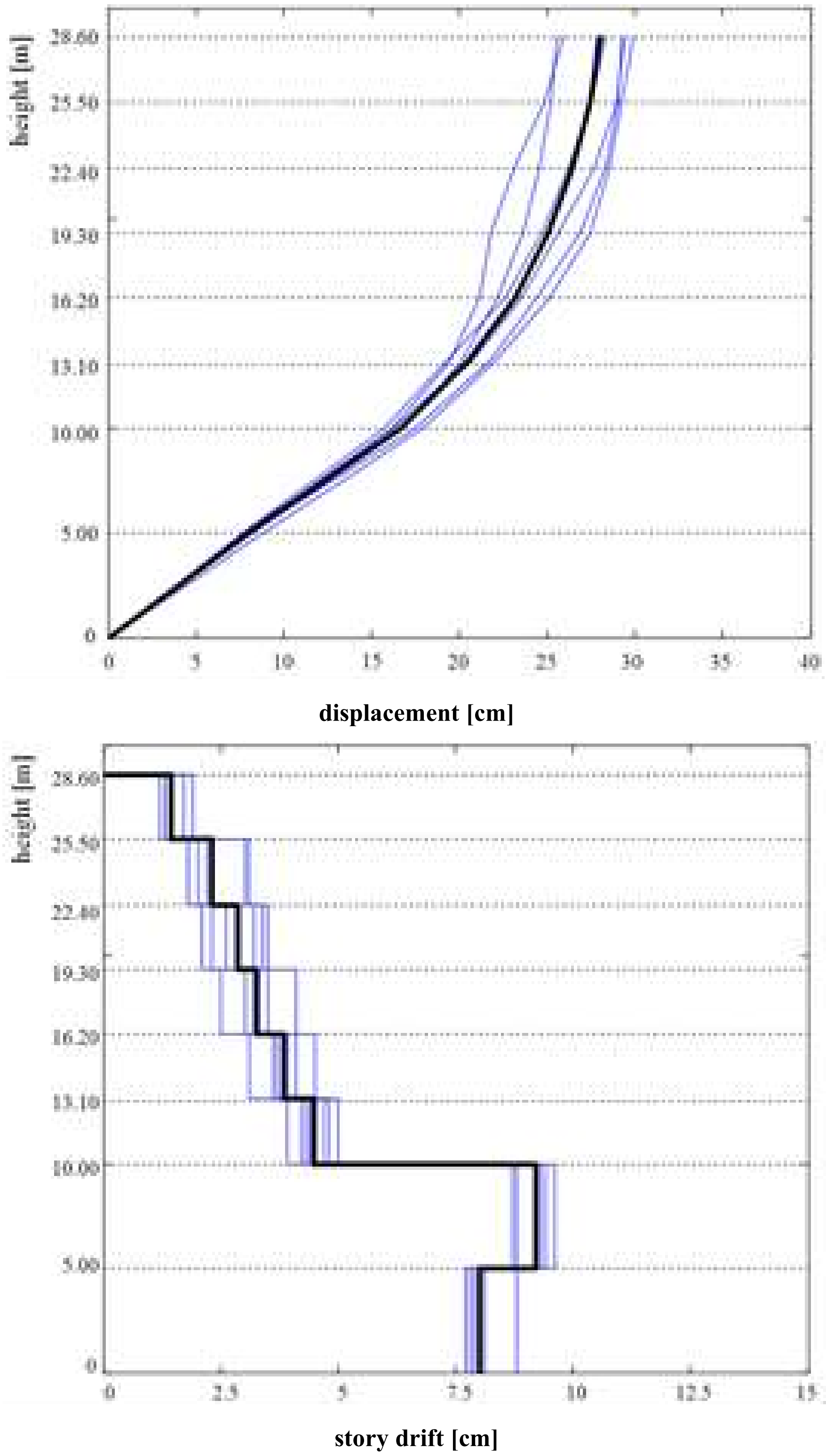


Figure 10. *Displacements and story drifts for structure in Fig. 1 for all seven artificial time-history records (in blue) and their mean value (in black)*

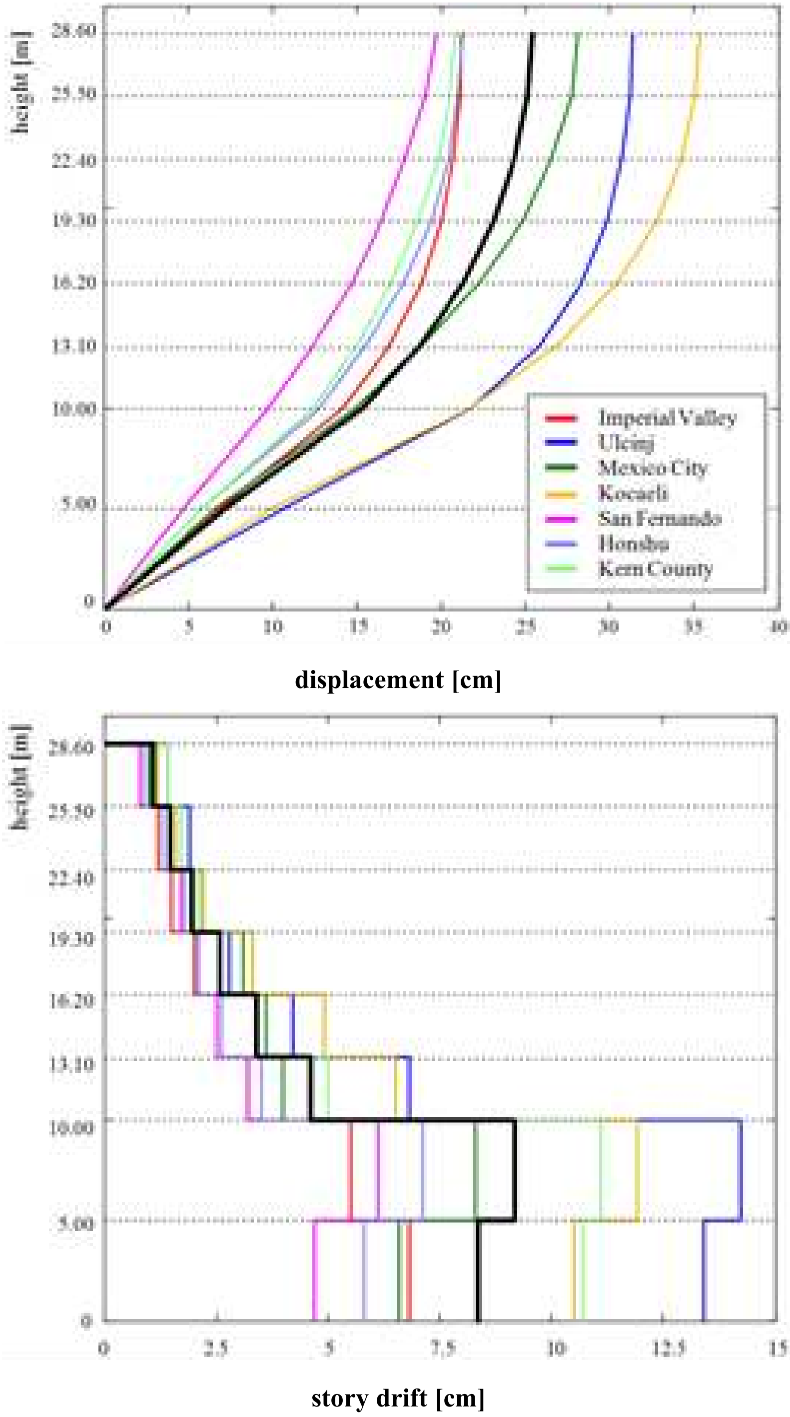


Figure 11. *Displacements and story drifts for structure in Fig. 1 for all 7 real time-history records and their mean value (in black)*

Fig. 12 and Fig. 13 present the comparison of maximum absolute displacements and maximum storey drifts calculated by all methods presented in Table 1, i. e. by equivalent static forces, modal analysis of response spectra for both un-cracked sections and cracked sections, nonlinear static method and nonlinear dynamic method using 7 artificial time-history records and 7 real time-history records.

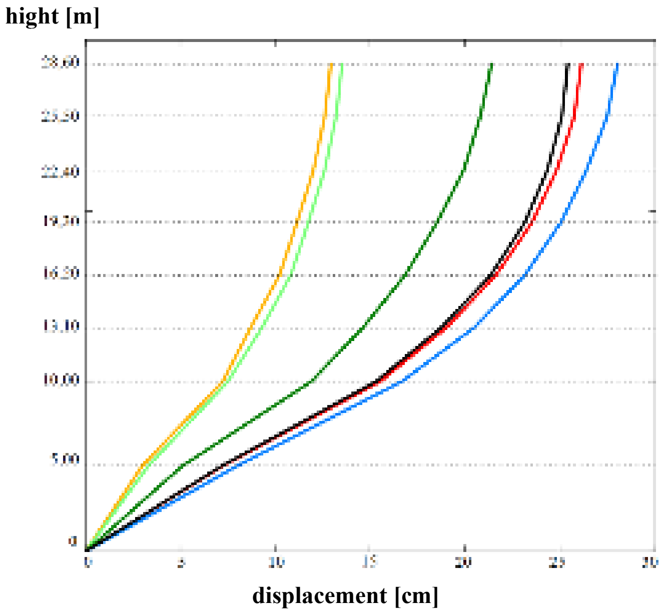


Figure 12. *Comparison of maximum absolute displacements calculated by equivalent static forces (yellow), modal analysis of response spectra for un-cracked sections (light green) and cracked sections (green), N2 method (red) and non-linear dynamic analysis using 7 artificial time-history records (black) and 7 real time-history records (blue)*

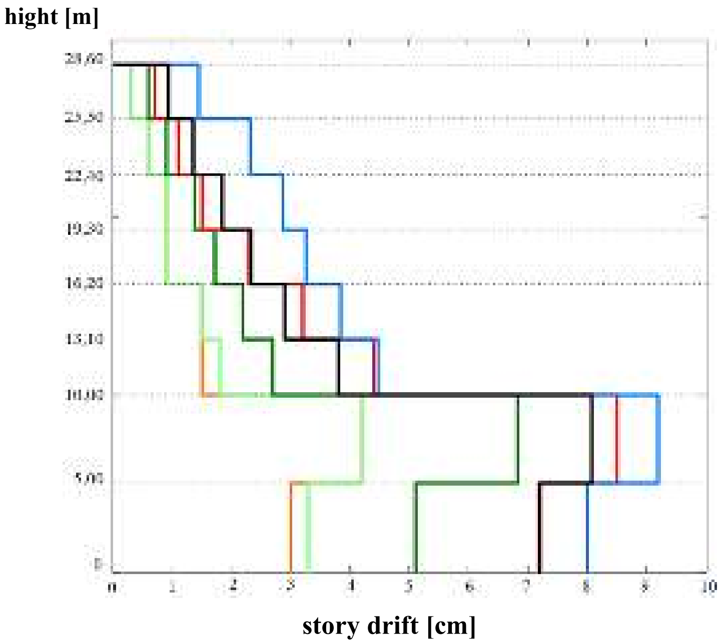


Figure 13. *Comparative representation of maximum story drifts calculated by equivalent static forces (yellow), modal analysis of response spectra for non-cracked sections (light green) and cracked sections (green), N2 method (red) and non-linear dynamic analysis using 7 artificial time-history records (black) and 7 real time-history records (blue)*

6. CONCLUSIONS

- Designers will soon only be doing more accurate nonlinear calculations of structures loaded by earthquake forces, especially for important structures. Applying more accurate methods of calculation takes more time to calculate, but in a nonlinear dynamic analysis, considering the actual behavior of the structure in an earthquake (performance-based analysis) after dimensioning, a more economical construction is obtained.

- Simplifications of calculations for concrete structures result in an increase in the amount of reinforcement, Fig. 12 and Fig. 13.
- If nonlinear analysis of structures with their cracked elements is not applied, in modelling and calculation by linear methods the characteristics of non-cracked concrete and masonry elements (their shear and flexural rigidity) should be taken with 50% of values for cracked sections, corresponding to [1], Fig. 12 and Fig. 13.
- In order to perform nonlinear analysis, cross sections and the amount of reinforcement must be assumed first as presented in Fig. 2 and as a result deformations (displacements and storey drifts), i.e. structural damage (plasticization of certain cross sections) will be obtained, while in linear calculation methods the reinforcement is obtained as the ultimate result.
- In the second generation of the European standard Eurocode 8 (will enter into power in 2022), which is in the process of being adopted and which contains radical changes to the currently valid standard, both nonlinear methods described in this paper are remained and extended for their use. For example, torsion effects and effects of the influence of higher modes in the nonlinear static method will be taken into account through correction factors [4], [5], [6].

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