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«Performance-based» seismic methodology and its application in seismic design of reinforced concrete structures

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ABSTRACT

This paper presents an analysis of the „Performance-Based" seismic design method, in order to overcome the perceived disadvantages and limitations of the existing seismic design approach based on force, in engineering practice. Bearing in mind, the specificity of the earthquake as a load and the fact that the seismic resistance of the structures solely depends on its behavior in the nonlinear field, traditional seismic design approach based on force and linear analysis is not adequate.

„Performance-Based" seismic design method is based on nonlinear analysis and can be used in everyday engineering practice. This paper presents the application of this method to eight-story high reinforced concrete building with combined structural system (reinforced concrete frame structural system in one direction and reinforced concrete ductile wall system in other direction). The nonlinear time-history analysis is performed on the spatial model of the structure using program Perform 3D, where the structure is exposed to forty real earthquake records. For considered building, large number of results was obtained.

It was concluded that using this method we could, with a high degree of reliability, evaluate structural behavior under earthquake. It is obtained significant differences in the response of structures to various earthquake records. Also analisis showed that frame structural system had not performed well at the effect of earthquake records on soil like sand and gravel, while a ductile wall system had a satisfactory behavior on different types of soils.

Contents

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Introduction

In every day engineering practice, approximate methods are used for seismic design, according to them structures are designed on the reduced seismic forces, so damages appear in them during stronger earthquakes (nonlinear deformations), but in a way that the collapse does not happen. In the structure which is designed in this way, during the earthquake, by the rule, the ultimate capacity of the structure elements are reached and the behaviour of the structure exceed from the elastic domain, and it reaches domain of plastic behaviour, where, it is necessary to obtain enough ductility for its survive, or in other words, the capacity of deformations. However, seismic resistance of structures depends mostly on capacity of deformations, but not on strenght capacity. Regarding the fact that seismic resistance of structures exclusively depends on possibility of its survive in the plastic area, it can be concluded that traditional approaches of design, which are based on forces, are not the best solutions for design of seismic resistant structures. The concept of seismic design called *„Performance-Based Seismic Design“* –which is based on displacement, presents the future in design of seismic resistant structures. In this paper, advantages of the new concept of seismic design, based on the behaviour, will be presented on the example of reinforced-concrete structure [1-18].

«Performance-based» seismic methodology

In the last years in America, new seismic design concept has been developed, called, „Performance-Based Seismic Design“ – seismic design based on the structure behaviour, which is based on displacement. This concept presents the example to overcome the noticed disadvantages of the existing seismic designed approach in the engineering practice. At this concept, the non-linear dynamic analysis is used as the basis analysis for the calcuated-design of earthquakes effects, and it presents the only analysis which can surely predict local and global structure behaviour. Results of non-linear dynamic analysis, maximal deformations in plastic zones of structural elements (plastic hinges) are compared with the deformations capacities and in that way it is checked if the structure has obtained enough capacities of deformations which are necessary to survive the possible earthquake. Seismic design based on the structure behaviour - Performance-Based Seismic Design starts with the definition of one or more performance objectives. By the performance objective, the desired level of the structure behaviour is defined for the certain earthquake intensity. Performance levels are described by the possible, accepted losses. Losses present the possible levels of the structure damages which are structure and non-structure elements, the number of victims, economic losses, the time during which the structure will be out of function, repair costs and others. Nowadays seismic design based on the performance are used in the practice through American regulations ASCE41 (ASCE41 2007) where four basic building performance levels are defined, they are Collapse prevention, Life Safety, Immediate Occupancy and Operational. These performance levels define the damages scope which the building has during earthquake and they present the descrete points on the continual scale of building performance state. For comparison, EN 1998-1 defines performance objective which is reached by filling two requests, they are: the request which corresponds to the performance level of Collapse prevention for the earthquake for the return period of 475 years and the request which approximately responds to the performance level of immediate occupancy for the earthquake for the return period of 95 years. In the figure 1, the general algorith of *„Performance-Based“* method is presented.

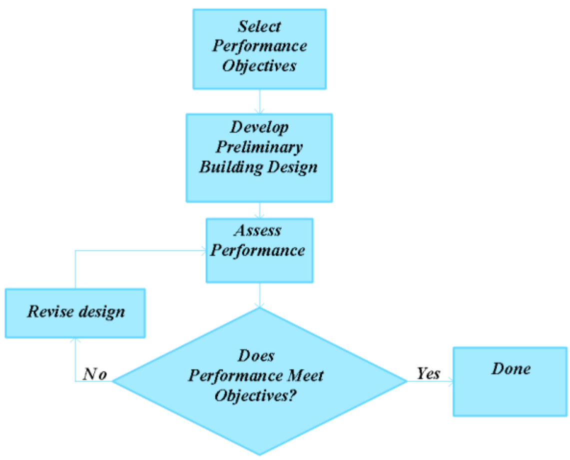

Figure 1: General algorithm of „Performance-Based“ seismic design

Description of the considered reinforced concrete strusture

1 Basic data about RC structure

RC structure that is analysed in this paper, presents eight-story building with total height of 24.0m and the storey heigh of 3m. Floor plan of the building is shown on Figure 2. Building has 5m spans in both directions, four spans in the X direction and three spans in the Y direction. Structural system is RC frame system in X direction and RC wall system in the Y direction. The floor construction is RC slab with thickness of 15cm. The dimensions of the beams are 40/45cm and 20/45cm. The dimensions of the columns are 55/55cm. Thickness of the walls is 20cm. The class of concrete is adopted C35/45 according to Eurocode 2 for all elements. Yield strength of longitudinal reinforced steel is f_y=400 MPa, f_y=240 MPa for transverse reinforcement and f_y=500 MPa for mesh reinforcement. Preliminary design of the considered structure was done according to European regulations EN1998-1 (CEN 2004). The fundamental periods of structure for X and Y direction, obtained from linear analysis, are T_{1x}=1.28s and T_{1y}=0.57s.

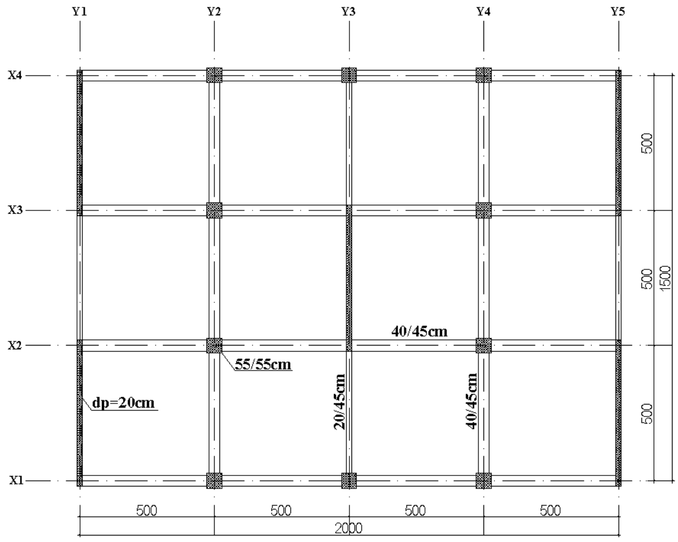

Figure 2: Floor plan of the building

2 Modeling of RC structure for nonlinear analysis

For the purpose of performing nonlinear time-history analysis, model of the structure was created using *Perform-a 3D* (Perform 3D Product of Computers & Structures, Inc. 2006). Modeling of inelastic beams and columns was performed using *Fema chord rotation model* (ASCE41 2007), by which they are modeled with two plastic hinge at both ends and elastic segments between them. Plastic hinges at the ends of the element are rotation hinges. This method of inelastic behavior modeling of beams and columns is quite satisfactory in the case of usual frame structure in which appearing of plastic hinges is expected at the ends of the elements. Trilinear behavior with strenght loss for hinge moment-rotation relationship and the hysteresis loop with stiffness degradation is adopted (see Figure 3).

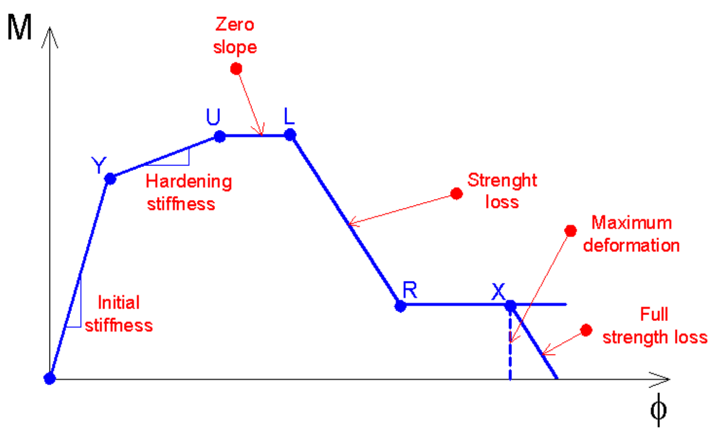

Figure 3: Trilinear behavior with strenght loss for hinge moment-rotation relationship

Modeling of beam-column hinge was performed using the *"Panel Zone"* (ASCE41 2007) element. It consists of four rigid links, hinged at the corners and rotational spring that provides strength and stiffness. *Shear wall* element (ASCE41 2007) was used for the modeling of wall. Wall are modeled by defining the fiber cross-section. The area and location of reinforcement within the cross-section and the properties of the concrete are defined using individual fibers. Concrete and reinforcing steel are modeled with nonlinear characteristics. Deformation Capacities for the performance level, Collapse Prevention are defined with the help of ASCE 41 (ASCE41 2007) which define the deformations capacities for the wide spectar of structural elements and they are showed in the table 1.

Table 1: Capacities of the plastic hinge rotations for the particular elements of the considered structures

	Capacity of the plastic rotation (radians) for the particular performance levels		
Elements	**Collapse prevention CP**	**Life Safety LS**	**Level of Immediate Occupancy IO**
Beams 40/45cm	0.025	0.02	0.01
Beams 20/45cm	0.025	0.025	0.01
Columns 55/55cm	0.0165	0.01266	0.0045
Walls 20cm	0.013	0.0093	0.0047

For the considered structure, performance objective is defined which corresponds to the level of Collapse Prevention for the earthquake with the return period of 475 years and it will be checked.

Selection of ground motions

The considered structure is exposed to the action of forty ground motions from the European strong-motion database (Ambraseys et al. 2000). Twenty ground motions are recorded on the rock (the first set), and the other twenty ground motions are recorded on soft soil (second set). These motions were characterized by surface-wave magnitudes, M, in the range between 6 and 7 and closest distances to the rupture surface, R, between 9 and 50 km. The motions recorded on rock, are scaled so that median of their PGA is 0.4g, while the motions recorded on soft soil are scaled so that median of their PGA is 0.3g. It should be noted that in the EC8 regulations, it is prescribed that recorded motions should be individually scaled to the value of the design ground acceleration, until here scaled motions have different maximum ground accelerations. This approach has the advantage of including different earthquake intensities and because of that it is possible to establish relationship between the intensity measure and response of the structure (see Eqn.5.1.), wich is necessary in the application of "performance based" methodology in probabilistic format (Moehle 2005).

In Tables 2 and 3 are shown the values of PGA of all ground motions after scaling, separately for both sets of records.

Table 2: PGA for motions recorded on rock and their median value

	Number of ground motion									
	1	2	3	4	5	6	7	8	9	10
PGA	0.16g	0.17g	0.21g	0.22g	0.28g	0.29g	0.30g	0.31g	0.34g	0.38g
	11	12	13	14	15	16	17	18	19	20
PGA	0.42g	0.46g	0.48g	0.50g	0.52g	0.54g	0.56g	0.60g	0.65g	0.70g
Median value=0.40g										

Table 3: PGA for motions recorded on soft soil and their median value

	Number of ground motion									
	1	2	3	4	5	6	7	8	9	10
PGA	0.11g	0.12g	0.13g	0.15g	0.16g	0.17g	0.24g	0.27g	0.28g	0.29g
	11	12	13	14	15	16	17	18	19	20
PGA	0.31g	0.36g	0.38g	0.39g	0.45g	0.47g	0.50g	0.51g	0.55g	0.60g
										Median value=0.30g

Results of nonlinear time-history analysis

The response of the structure is calculated for the forty earthquake records, independently in both orthogonal directions of the structure (X direction and Y direction). Large number of results of the nonlinear analysis are gained for both directions of earthquake records. Analyzing the results the performance of the structure is evaluated, independently in both earthquake directions. In the table 4, there are D/C (demand/capacity) relations (mediana of all earthquakes records) for characteristic limit states for the performance level CP. Limit states are satisfied if all relations D/C of that limit state are less than the unit.

Table 4: D/C relations (median of all earthquake records) for particular limit states for NDA

Limit states	Results NDA
Rotation of the plastic hinges of beams for performance level CP	D/C=0.57
Rotation of the plastic hinges of the column for performance level CP	D/C=0.45
Bending rotations in the plastic hinges and zones out, for the walls for performance level CP	D/C=0.35
Tension dilatation in the zone of plastic hinge of the walls	D/C=0.53
Tension dilatation out of zone of plastic hinges of the walls	D/C=0.41
Beams – checking shear capacity	D/C=0.81
Columns - checking shear capacity	D/C=0.63
Beam-column joints – checking shear capacity	D/C=0.72

According to the presented results from the table 4 it can conclude that the considered structure has the satisfied performance for the defined performance objective. The examption is the behaviour of the structure during the effects of several earthquake records, registered on the bad soil in X direction (exp. Ulcinj Hotel Olimpic EW). In the figure 4, the deformed shape of the structure is presented with the position of the formed plastic hinges during the earthquake load Ulcinj Hotel Olimpic EW in X direction. It can conclude that RC frame structural system is not a good solution of the seismic resistant structure, in the conditions where the structure is founded in the bad soil (soil type C according to EC8, soil on the territory Ulcinj where the Hotel Olimpic is placed) and where the earthquake record can be appeared similar to the record Ulcinj Hotel Olimpic EW. Montenegrin earthquake from 1979. confirms this conclusion, because a large number of structures of the RC frame structural system had collapse. The considered structure should change the structural system, in X direction, where one of the ways is placing of ductile wall in that direction. The structural system with ductile walls showed the satisfied behaviour for both cases of the considered soil types.

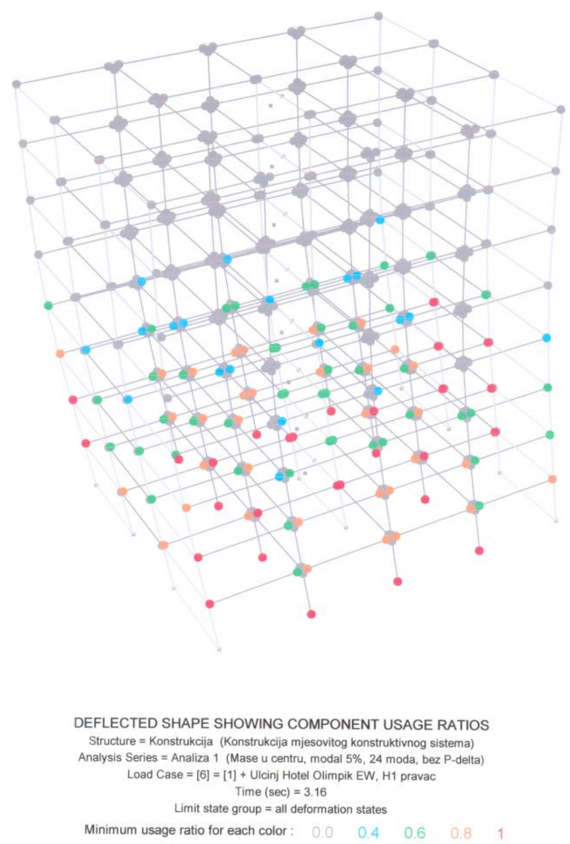

Figure 4: Deformed shape of the structure with the placement of the formed plastic hinges during the earthquake loads Ulcinj Hotel Olimpic EW in X direction

Larger rotations of walls appear in sections on the places of fixity in foundations where it is predicted that plastic hinge appear, which present the desirable behaviour during the earthquake. From the shear D/C relations, it can be seen that shear capacity are obtained in all structure elements, that is the structure is saved of any forms of brittle fracture by the application of the capacity design method. It can be concluded that the structure behaviour satisfied the defined performance objective.

Conclusions

According to the analysis and the detailed studying of the new seismic design approach, based on performance of the structure and its usage on the concrete reinforced-concrete structure, the following conclusions appear in this paper:

„Performance-Based“ seismic design presents the modern method of seismic design, which, in the best way, overcome disadvantages and faults of the existing methods of the seismic design in the engineering practice. Seismic resistance of the structure does not depend on its ultimate strenght capacity, but on the capacity of the deformations which it has, so it can be concluded that the basic parameters of the analysis of this method of deformations, are correctly chosen. „Performance-Based“ seismic design, with the help of nonlinear dynamic analysis during the estimation of the structure behavior leads to the designing of seismic resistant structures in relation to the existing seismic design methods. With the help of it, disadvantages of the designed structure are noticed in the best way.

Structures, designed by the principles of capacity design method, accordng to the Eurocode 8, has large capacities of ductility and enough shear capacity, and with the adequate confinement of all elements, the structure is provided against any kind of brittle failure. Frame structural system is not a good solution of the seismic resistant structure, in the conditions where the structure is founded in the bad soil. The structural system with the ductile walls has the satisfied behavior for both founding in bad soil and in the rock. Analyses results show significant differences in the structure response depending on the different earthquake records. It can conclude from this that the structure performance depends on large number of earthquake characteristics. Beam-column joint must be considered as the separate structural element, and such as that it must be separately designed and calculated. In this paper, it comes to the conclusions that designing of beam-column joints are proposed in the New Zeland literature by Paulay and Priestley, and presented in this paper the best solution. The suggestion given in Eurocodes, about the quantity of horizontal reinforcement in the beam-column joint is excessive. During the adoption of the longitudinal reinforcement in the plastic hinges, the beam must not be overdimensioned. With overdimensioned sections of the plastic hinges of the beams, the moment of appearance of plastic hinge in them is timely dislocated, in that way the plastic deformations are moved to the other structural elements, for example plastic hinges in columns on the places of their fixity in foundations where there is a big danger from fast development of big deformations in them, which can cause the structure collapse.

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«Характеристическая методология» сейсмического проектирования и ее применение на армированно-монолитной конструкции

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УДК 69	Подана в редакцию 10 мая 2015 Принята 30 мая 2015	характеристическая методология, монолитное здание, нелинейный анализ, рамная система, пластичная стена8

АННОТАЦИЯ

Эта работа представляет собой анализ «характеристического метода» сейсмического проектирования, который имеет в качестве цели превзойти замеченные недостатки существующего проектного подхода, который основывается на силах в инженерной практике. Имея в виду специфичность землетрясения как нагрузки, и тот факт, что сейсмическая устойчивость конструкции исключительно зависит от ее поведения в нелинейной области, традиционный подход к проектированию основывается на силах и линейном анализе, которые не являются неадекватными. "На основе характеристик" - метод сейсмического проектирования основывается на нелинейном анализе и может быть использован в ежедневной инженерной практике. В данной работе через конкретное применение данной методики на 8-ми этажном армированно-монолитном здании смешанной конструктивной системы (армированно-монолитные рамы в одном направлении и армированно-монолитные дуктильные стены в другом направлении) рассмотрены все преимущества данной концепции проектирования. Нелинейный динамический анализ для рассматриваемого здания проведен для действия 40 реальных землетрясений, по имеющимся их записям с помощью программы Perform 3D. Из полученных результатов проведенного анализа на примере рассматриваемого здания заключили, что с помощью данной методологии с большой степенью надежности можно оценить реальный ответ конструкции на действие землетрясения. Получены существенно различные значения сейсмических ответов для различных записей землетрясений. Также проведенный анализ показал, что армированно-монолитные рамные системы имеют неудовлетворительное поведение для записей землетрясений, снятых на слабом грунте. Тогда как конструктивные системы с дуктильными стенами имеют удовлетворительные поведение для записей, снятых на различных типах грунтов.

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