ГОДИШНИК НА УНИВЕРСИТЕТА ПО АРХИТЕКТУРА, СТРОИТЕЛСТВО И ГЕОДЕЗИЯ
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SEISMIC BEHAVIOR OF A PRECAST SINGLE-STOREY SINGLE SPAN REINFORCED CONCRETE FRAME

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Keywords: *Spectral method, Time History Analysis, Behavior Factor*

ABSTRACT

This paper studies the seismic behavior of a precast single-storey single span reinforced concrete frame. An example frame has been analyzed using the linear "Spectral method" and the nonlinear dynamic analysis with accelerograms according to BDS EN 1998-1. As a result the behavior factor of the structure is determined. Conclusions and recommendations for the practical application of the method are made.

1. Introduction

Precast single-storey structures made from reinforced concrete are widespread in the Republic of Bulgaria. Newly designed industrial buildings can be calculated by design codes of "Eurocode" or according to the current national regulations. Horizontal impacts (wind, global geometric imperfections, seismic impact, etc.) in transverse direction are received by frame structures, and in longitudinal direction – by steel vertical bracing or longitudinal reinforced concrete frames. The performance of a single transverse frame subjected to seismic actions is considered in this paper. An overview of the recommendations for design of this type of structures by spectral method according to BDS EN 1998-1 [4] has been carried out. The behavior factor of a precast single-storey frame was investigated.

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2. Design of precast reinforced concrete structures for seismic loading by modal response spectrum analysis, according to BDS EN 1998-1 [4]

Usually single-storey precast structures are designed with fixed columns to foundations and a pin-connected beam to the vertical members. Dissipation of seismic energy occurs in dissipative zones (critical regions) located at the bottom of the columns. These regions are calculated and constructed to have the required local ductility. Significant non-linear rotations develop in these zones when there is an earthquake with an intensity close to the design of the structure. This effect occurs due to the elongation of reinforcement in plastic hinges. In linear elastic analysis, the ability of resistance and energy dissipation in structure by nonlinear behavior of columns is calculated by reducing the elastic response spectrum by a behavior factor q_p.

The value of the behavior factor q_p, according to [4] for this type of structures is determined by the following formula:

$$q_p = q.k_p = q_0.k_w.k_p > 1,50 \,, \tag{1}$$

where $q = q_0.k_w$ is the behavior factor;

q_0 – basic value of the behavior factor, dependent on the class of ductility of the structure, according to Table 5.1 of [4]. For medium ductility class (DCM) the coefficient $q_0 = 3.\alpha_u / \alpha_1 = 3.1,1 = 3,3$;

k_w – factor reflecting the prevailing failure mode in structural systems with walls. For frame structures $k_w = 1$;

k_p – reduction factor depending on the energy dissipation capacity of the precast structure, as well as the type and location of connection joints:

- $k_p = 1,0$ – for structures with connection according to 5.11.2.1.1 (connections located away from critical regions), 5.11.2.1.2 (overdesigned connections), or 5.11.2.1.3 (energy dissipating connections) of [4];
- $k_p = 0,5$ – for structures with other types of connections.

The opinion of the author is that the choice of the value of coefficient k_p in BDS EN 1998-1 [4] and the Bulgarian national annex is not clearly, accurately and unambiguously clarified. The different value of k_p leads to a significant difference in the results (geometric dimensions of the sections, inserted longitudinal reinforcement, etc.) when designing this type of structures for seismic effects.

3. Designed structure

The reinforced concrete frame shown in Fig.1 is designed. The building is located in the Sofia city area. The structure is loaded with following impacts:

- self-weight of reinforced concrete elements – calculated automatically by the software product;

- dead loads from roof triple-layer sandwich panels and installations – 1,0 kPa (distance between main beams (main transverse axes) – 12 m);
- dead loads from wall triple-layer sandwich panels and installations – 0,6 kPa (distance between facade columns – 6 m);
- seismic impact with maximum reference peak ground acceleration on type A ground of $a_{gR} = 0,23g$, at importance class II with coefficient $\gamma_I = 1,0$ and ground type C, according to the stratigraphic profile of the soil conditions.

Used materials are:

- concrete – grade C25/30;
- reinforcing steel – class B500C.

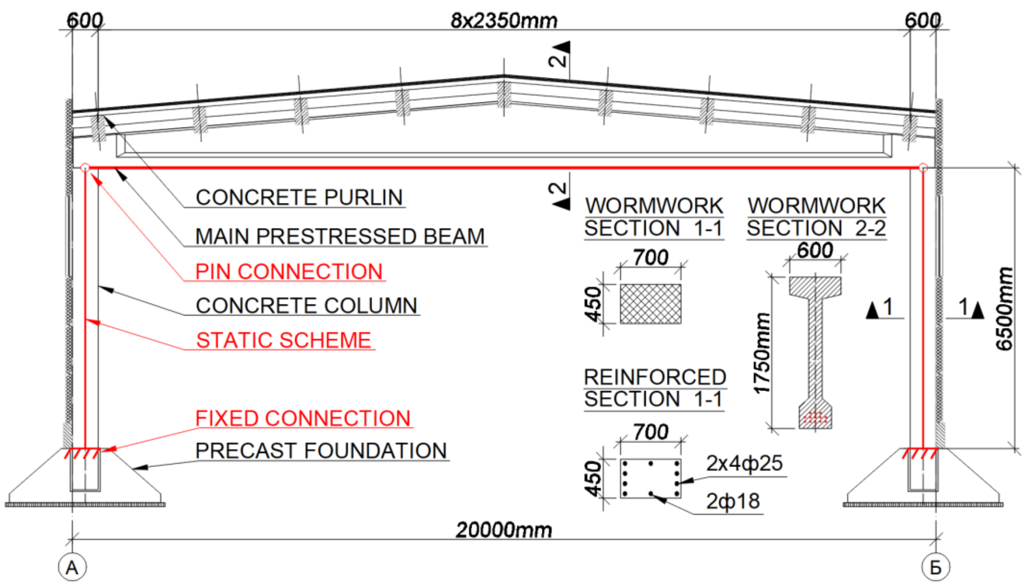


Figure 1. Vertical section of a precast structure

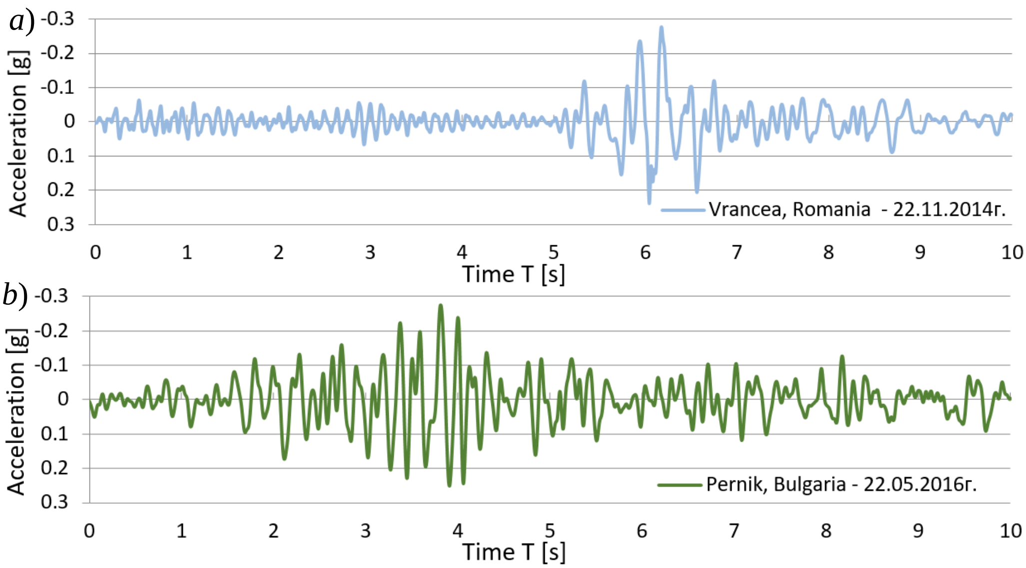
4. Investigation of the behavior of a precast reinforced concrete one-storey single-span structure by Linear Spectral Analysis and Nonlinear Dynamic Analysis with accelerograms (Time History Analysis)

A design model of the frame of Figure 1 was developed using the specialized software product CSI Etabs [6]. The structure is modeled by frame elements for the columns, which are fixed to the foundations, and an inextensible frame element (with EA = ∞) connected to them, representing the main prestressed beam (Fig. 1).

The linear characteristics of the materials were used for the spectral analysis. The behavior of the frame was investigated considering different elastic bending stiffnesses of the columns in the critical regions (50%.EI, 70%.EI and 100%.EI, where EI is the bending stiffness of the uncracked cross-section of the column) similar to [9]. BDS EN 1998-1 [4] recommends that the elastic flexural and shear stiffness properties of concrete elements may be taken to be equal to one-half of the corresponding stiffness of the uncracked elements, while in [1, 2 and 8] reduction is recommended at 30 %. The length of the critical regions is entered according to the requirements of formula (5.14) from [4]. The computational model is loaded with dead impacts and the behavior of the frame is considered only for a seismic (accidental) combination.

For the dynamic analysis with accelerograms, stress-strain diagram of concrete is introduced according to Fig. 3.2 of BDS EN 1992-1-1 [3] and the effect of the dynamic nonlinear behavior of the material is accounted by the "Takeda" hysteresis model. The stress-strain diagram takes into account the tensile strength of concrete. The model considers the increased bearing capacity of the confined concrete in the columns. The dynamic non-linear behavior of the reinforcing steel uses the "Kinematic" hysteresis type. The transverse and longitudinal reinforcement in the frame columns is modeled. Critical regions (plastic hinges) are defined at the bottom of the columns above their connections to the foundations and they are type "P-M2" (Deformation controlled). The seismic activity of Earth through the time was modeled by seven accelerograms (Fig. 2) – six of them are records of real earthquakes in the Republic of Bulgaria and abroad and one is generated in accordance with the elastic spectrum of response at 5 % viscous damping, as recommended in [5]. All accelerograms are scaled taking into account the soil coefficient S. Table 2 presents the predominant frequencies of the Fourier amplitude spectrum and the spectral density of each accelerogram, which were determined by a specialized software product applying Fourier transformation [7, 10]. Non-linear influence of second-order effects (P-Δ effect) on the structure is taken into consideration.

Staged calculation analysis was performed at dynamic nonlinear analysis (Time History Analysis) – initially from permanent loads in seismic combination and subsequently from the seismic activity. In this way, the behavior of the frame (stressed-deformed state) is determined before, during and after the occurrence of the seismic impact.



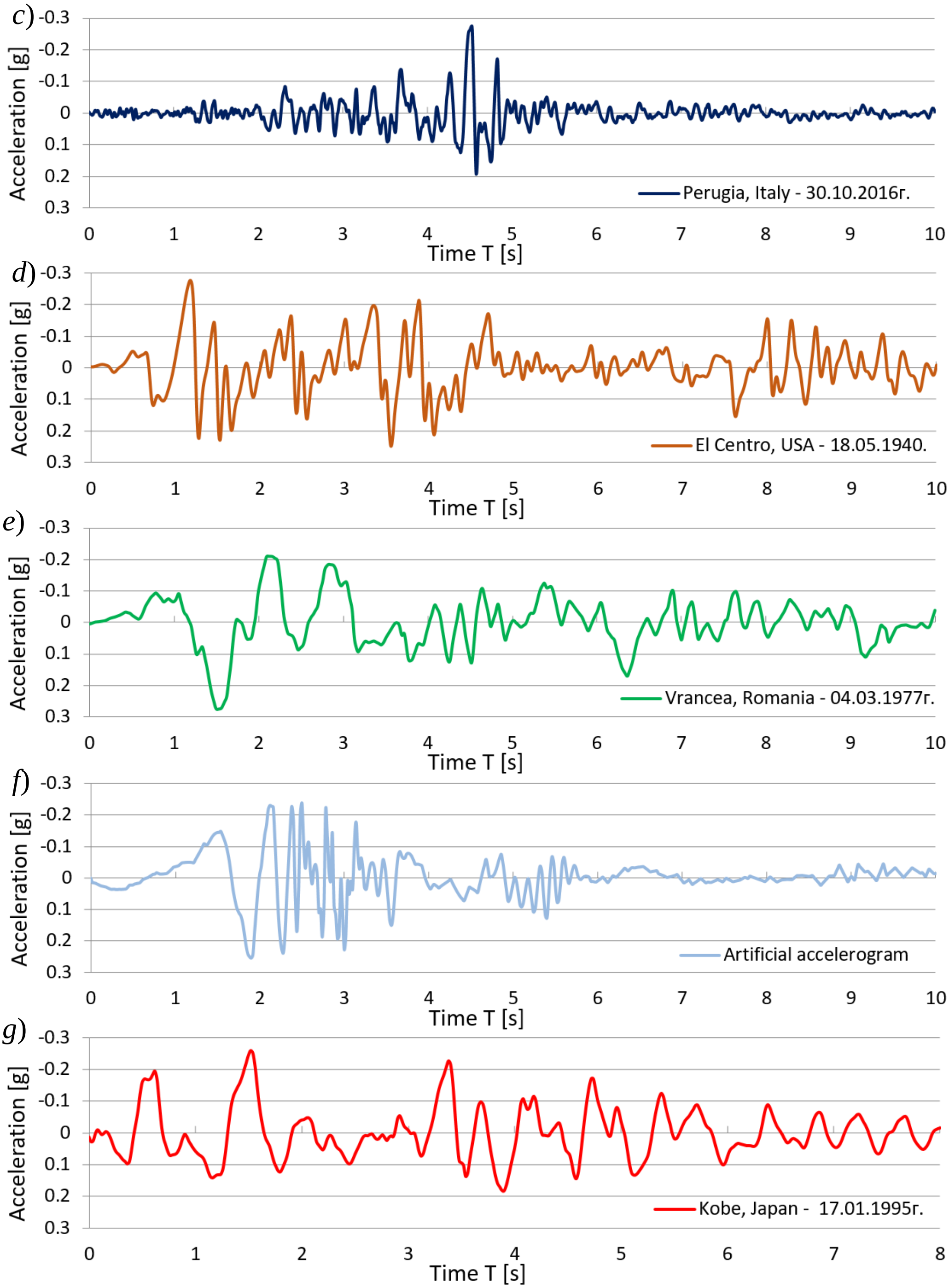


Figure 2. Scaled accelerograms of earthquakes in:

a) Vrancea, Romania – 22.11.2014; b) Pernik, Bulgaria – 22.05.2016; c) Perugia, Italy – 30.10.2016; d) El Centro, USA – 18.05.1940; e) Vrancea, Romania - 04.03.1977; f) Artificial accelerogram; g) Kobe, Japan 17.01.1995

Table 1 presents the results of the linear study of the frame by the spectral method. It is concluded that depending on the reduction of the bending stiffness in the plastic hinge of the columns, the first period of the structure is changed, as well as the arising forces from seismic impact. Calculated shear forces V_{tot} at the base of the structure at different behavior coefficients are presented in Table 1, which are defined at $k_p = 1,0$ and $k_p = 0,5$ for ductility class DCM. When reducing the bending stiffness of columns, it is necessary to approach with caution to the damage limitation requirement.

The necessary amount of longitudinal reinforcement in columns (Fig. 1) in dynamic study of the structure with time history analysis with accelerograms was established by multiple interactions. Designed reinforcement satisfies the forces from all seismic actions. The hysteresis behavior during plastic deformation with strengthening of the longitudinal reinforcement in the critical regions (plastic hinges) is calculated.

Table 1. Results from linear spectral analysis

	Flexural stiffness of columns		
	100 %.EI	70 %.EI	50 %.EI
First natural period of vibration of the structure T [s]	0,705	0,758	0,831
Elastic displacement d_e [mm] at $q = 1$	61,9	65,1	72,55
Elastic displacement d_e [mm] at $q = 1,65$	37,2	39,4	44,00
Elastic displacement d_e [mm] at $q = 3,3$	18,75	19,7	21,98
Shear force at base of frame $V_{tot,el}$ [kN] at $q = 1,0$	534,6	476	440
Shear force at base of frame $V_{tot,q=1,65}$ [kN] at $q = 1,65$	322	288	266
Shear force at base of frame $V_{tot,q=3,3}$ [kN] at $q = 3,3$	162	144	134

Table 2. Results from nonlinear Time History Analysis

	Seismic impact						
	Vrancea 2014	Pernik 2016	Perugia 2016	El Centro 1940	Vrancea 1977	Artif. acce.	Kobe 1995
Predominant Frequencies [Hz]	3,85	8,3	9,5	6,2	0,81	4,3	5,1
Displacement on top of structure d_r [mm]	15,7	16,7	28,5	56,36	66	61,9	100,7
Shear force at base of frame $V_{tot,pl}$ [kN]	136	144	238	262	264	266	272
Limit state, according to paragraph 2.1 of BDS EN 1998-3 [5]	DL	DL	SD	NC	NC	NC	NC
Behavior factor q_p [-]	**3,93**	**3,71**	**2,25**	**2,04**	**2,03**	**2,01**	**1,97**

It is concluded from the data in the tables that despite the same maximum seismic accelerations at the base of the structure from all accelerograms, the nonlinear displacements of the top of the building d_r and the shear forces at the base of the structure $V_{tot,pl}$ differ up to twice the amount. This effect occurs from different amplitude-frequency range of each of the calculated earthquakes. The behavior factor q_p is determined for each earthquake, taking into account the ability of the structure to dissipate energy in critical regions, through the following equation:

$$q_p = \frac{V_{tot,el}}{V_{tot,pl}}, \tag{2}$$

where $V_{tot,el} = m.S_e$ is the maximum shear force at the base of frame from elastic response spectrum S_e with structure mass m;

$V_{tot,pl}$ – maximum shear force at the base of the frame, reported by the nonlinear analysis in time (Time History Analysis), for the corresponding seismic action.

The value of the behavior factor q_p changes in the range from 2 to 4 for the construction under consideration. Longitudinal reinforcement is 2×4ϕ25+2ϕ18 in each column (Fig. 1). The high values of q_p in Vrancea (2014) and Pernik (2016) arise due to the plastic behavior of the structure (hysteresis behavior of the columns in their dissipative zones) during these earthquakes. The low values of q_p in the rest of the seismic impacts – to the elastic behavior.

From the results of the non-linear maximum deformations reached in the longitudinal reinforcement and defined states of damage in the structure, according to paragraph 2.1 of BDS EN 1998-3 [5], the limit state of the building is classified for each of the considered earthquakes (Table 2). For the Vrancea (2014) and Pernik (2016) earthquakes, assumed state of damage for the structure, will be DL (Damage Limitation), for Perugia (2016) – SD (Significant Damage), and for the rest – NC (Near Collapse).

5. Conclusions and recommendations

Based on the aformentioned, the following conclusions and recommendations are drawn:

1. The behavior factor of the sample structure considered in the present study was determined by six scaled seismic accelerograms and one artificially generated accelerogram.

2. The calculation of the value of the behavior factor for one-storey precast structures with linear spectral analysis according to BDS EN 1998-1 [4] is difficult when determining the coefficient k_p. The author recommends clarifying and specifying the conditions for determination in the Bulgarian national annex to [4]. It is necessary to refine q_p by carrying out the national experience in structure design, project implementation and operation of the building, as well as analytical and experimental studies of their behavior under seismic impacts.

3. The study of real frame structures by designers with nonlinear time history analysis is possible in the presence of a specialized software product with relevant capabilities for this type of analysis, as well as a computer configuration with appropriate hardware parameters.

The parametric data of seismic effects are easily available, but it is necessary to be approached with caution by designers when choosing accelerograms that correspond to the seismogenic and soil conditions of the considered construction site.

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ИЗСЛЕДВАНЕ НА ПОВЕДЕНИЕТО НА СГЛОБЯЕМА ЕДНООТВОРНА ЕДНОЕТАЖНА СТОМАНОБЕТОННА РАМКА, ПОДЛОЖЕНА НА СЕИЗМИЧНИ ВЪЗДЕЙСТВИЯ

Хр. Нешев[1]

Ключови думи: *спектрален метод, анализ във времето, коефициент на поведение*

РЕЗЮМЕ

Настоящата статия разглежда поведението при сеизмично въздействие на едноетажни еднoотворни стоманобетонни сглобяеми сгради. Изследвана е примерна равнинна рамка, изчислена чрез линеен „спектрален метод“ и нелинеен динамичен анализ с акселерограми съгласно БДС EN 1998-1, като е определен коефициентът ѝ на поведение. Направени са изводи и препоръки за практическо приложение.

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