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SEISMIC RISK ASSESSMENT OF FACULTY OF LAND RECLAMATION AND ENVIROMENTAL ENGINEERING - BUCHAREST

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Abstract

Increased vulnerability of human society to natural hazards is not so much due to a change in the way phenomena manifest, but also to anthropogenic causes, which require more than ever, a pertinent analysis of risk factors and constant involvement of specialists in all fields activity in reducing the negative effects they may cause to people, to the infrastructure or to environmental factors. Safety of structures is one of the main performance requirements for buildings. Expressed in a quality-like manner, this requirement must be completed with quantitative factors.

Key words: *vulnerability, list, scale, collaps, fragility*

INTRODUCTION

Increased vulnerability of human society to natural hazards is not so much due to a change in the way phenomena manifest, but also to anthropogenic causes, which require more than ever, a pertinent analysis of risk factors and a constant involvement of specialists in all fields activity in reducing the negative effects they may cause to people, to the infrastructure or to environmental factors.

Safety of structures is one of the main performance requirements for buildings. Expressed in a quality-like manner, this requirement must be completed with quantitative factors.

Building design based on experience, rules of thumb and intuitive application of the rules of mechanics, has been used for millennia, until the seventeenth and eighteenth centuries (and even later), when basis of mechanics was founded through the works of Galileo Galilei, Hooke, Mariotti, Bernoulli, Coulomb, etc.. The most important element in the development of calculation methods of construction safety concept is philosophy. This concept has evolved over time and has been elaborated on a scientific basis in recent decades.

MATHERIAL AND METHOD

The case study presented below is aimed to establish the seismic risk class of an existing building that has a reinforced concrete structure (1).

The evaluation procedures used are in accordance with the National Annex of seismic design standard EN1998-3: 2005.

The building selected for evaluation is building A of the Faculty of Land Reclamation and Environmental Engineering, University of Agronomic Sciences and Veterinary Medicine of Bucharest. The building was erected between 1968-1970, and has a resistance structure consisting of reinforced concrete frames designed according to the norms of the respective period, therefore it does not meet many requirements of the current seismic design codes.

Building Department for Land and Environmental Engineering is located in the north of the capital, in the same area there are the Village Museum and Romexpo Hall, and the House of the Free Press. The building consists of three independent sections, separated by narrow expansion joints approximately of 50 mm. For example, seismic evaluation was selected for Building A

Assessed building consists of a ground floor and 4 levels, with a height of approximately 19 m non-structural walls with compartmenting role and made of full brick masonry laid in the system of "American" bricks: two plans separated by bricks arranged longitudinally and crossed about from place to place by transversal bricks. The result is a wall system with a lower weight than the classic one, with acceptable properties in terms of insulation, but with the mechanical properties and deformation much lower. In terms of how reinforcement of reinforced concrete frame elements should be noted and how these were designed as "Norm conditioned building design in seismic regions": P13-1963. Given the limited knowledge of seismic engineering at the time, sectional effort design of beams and columns are associated with a base shear of approx. 4.5% of the construction weight. In addition, compliance and reinforcement of concrete elements are strongly influenced by the requirements and design concepts of "gravity" system from STAS 1546-56. Thus, both plates and beams are reinforced in the "gravity" system of straight bars and inclined bars. Based on the information presented above one should establish which is the appropriate level of knowledge. P100-3/2008 defines three levels of knowledge:

- KL1: limited knowledge;
- KL2: normal knowledge;
- KL3: full knowledge. Thus, the selected knowledge level determines the allowed calculation method and the value of the confidence factor (CF).

RESULTS AND DISCUSSION

Assessment methodology based on Level 3

This is the most comprehensive methodology as it involves the use of nonlinear methods of calculation. It applies to major construction meant to be a more accurate assessment of seismic performance. Also, this methodology is useful in case of complex structures for which, methodologies Level 1 and 2 do not provide sufficiently reliable results (2).

For the seismic evaluation based on level 3 methodology, there were used nonlinear static calculation methods (the "push-over") as well as nonlinear dynamic analysis ("a time-history").

This nonlinear calculation method has been applied according to the provisions of Annex D of P100-1/2006 code and set out to determine the resilience (strength inelastic) structure (F_y) and its ability to move (d_u) . This value of the last movement is finally reported to the displacement requirement (d_s) ,thus obtaining the degree of structural earthquake insurance – the R_3 indicator (2).

When defining characteristics of post-elastic deformation (curves M-θ) for the plastic potential areas, the following assumptions have been adopted:

For each opening of the beams, the longitudinal reinforcement varies widely (Fig. 1) and this makes the identification of the plastic potential areas be both difficult and relatively uncertain. Thus, to define the positions of the plastic joints one must compare the beam flexural capacity in each section along the beam with the moments resulting from the conventional static calculation for each direction of seismic action. As both the diagram of capable moments and the related diagrams for the conventional calculation vary along the beam, the procedure of identifying plastic potential areas becomes extremely complicated. To simplify it, we chose to define four possible positions of plastic hinges for each opening of the beams. Their positions were defined taking into account the variations in the capable moments diagram that show sudden capacity jumps for both positive and negative moments.

In the original reinforcement plans one can observe that the bottom longitudinal reinforcement beams have a length of anchoring shorter than the required length according to current seismic design and therefore it is possible that the end joints cannot mobilize the entire bending capacity. But taking into account that: (a) end beaks presence significantly improves the ability of anchoring the smooth bars and (b) experimental test results have shown that under a correct execution, required anchorage length is actually significantly shorter than the length required in current codes, for the nonlinear analysis performed there was considered that the longitudinal bars work at their full capacity.

For nonlinear static calculation there are used two types of plastic joints:
- "bending"plastic joints that are not influenced by the intensity of axial force. These joints were assigned to the beams because within these elements the axial forces are negligible.
- plastic joints with axial force bending that take into account the influence of the axial force on the capable moment and on the plastic rotation. This type of joints were located at the ends of the pillars, on the height of each level (1).

Capable plastic rotations of reinforced concrete elements were evaluated using the relations (b.1a) and (B.1b) of Annex B of the Code P100-3/2008 (2). For the concrete and the reinforcement, there were used resistance values associated with the materials specified in the original drawings that were actually confirmed by limited range of non-destructive tests. Thus the average values of resistance were divided by the confidence factor CF = 1.20 and by the partial safety factor.

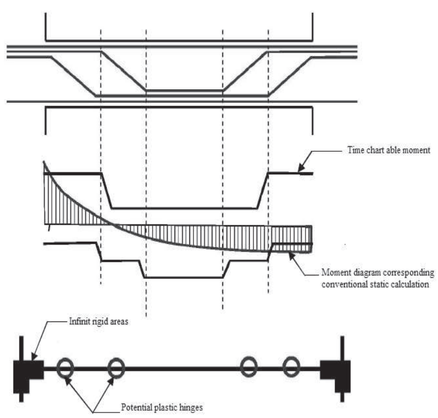

Fig. 1. Plastic potential joint positions for varying the longitudinal reinforcement

The following is an example for the application of this relationship for a beam and a column.

So, at ground level, the end of the axis A transverse beam current framework has a 350x650 mm concrete section and is armed with five longitudinal 5ϕ25 to the top and 2ϕ 20+1ϕ25at the bottom. Transverse reinforcement is made with stirrups ϕ8/200.

Consequently, the following are critical for the axis A value of maximum plastic rotation:

$$\boldsymbol{\theta_{um}^{pl} = \beta\left(\frac{\omega'}{\omega}\right)^{0,3} f_c^{0,2}\left(\frac{L_y}{h}\right)^{0,35} 25^{\propto\rho_x\frac{f_{yw}}{f_c}}} \quad (1)$$

in which

β	It is a coefficient , which has the following β value for beams of=0,008;
H	It is the height of the transversal section; h=650 mm;
L_V=M/V	It represents the shearing branch in the bottom section ; L_V=3.54;
ω, ω’	Reinforcement coefficients of compressed area, respectively wide area. In the end area from the A axis $\boldsymbol{\omega' = \frac{2454}{250\times 615} = 0,016}$ respectively $$\boldsymbol{\omega = \frac{1119}{250\times 615} = 0,0073};$$ $\rho_{S_x}=\frac{A_{S_x}}{b_w s_h}=\frac{2\times 50,3}{250\times 200}=0,002$ represents the transversal reinforcement coefficient ;

α is the confinement effectiveness factor, determined by the following formula, where b_o and h_o is confined core dimensions measured from center of stirrups, and b_i is the distance spacing between longitudinal reinforcement in a stirrup corner or paperclip in the perimeter section, thus:

$$\boldsymbol{\alpha = \left(1-\frac{s_h}{2b_0}\right)\left(1-\frac{s_h}{2h_0}\right)\left(1-\frac{\sum b_i^2}{6h_0b_0}\right) = 0,077} \quad (2)$$

It follows that the maximum plastic rotation of beam end section is:

$$\boldsymbol{\theta_{um}^{pl} = 0,008\left(\frac{0,016}{0,0073}\right)^{0,3} \times 13,9^{0,2} \times \left(\frac{3,54}{0,65}\right)^{0,35} \times 25^{0,077\times 0,002\times\frac{236}{13,9}} \Rightarrow \theta_{um}^{pl} = 0,027} \quad (3)$$

As for items under P100-3/2008 armed with smooth bars, no kinks in critical areas, calculated maximum plastic rotation relationship above is reduced by multiplying by a reduction coefficient value of 0.5. It follows that the final amount of maximum plastic rotation in the end of the beam axis is beam is $\boldsymbol{\theta_{um}^{pl} \approx 1,4\%}$

On the ground floor interior columns of transverse current frame with a cross section of 500x400 mm and are armed with 3ϕ16 on the longitudinal side. Transverse reinforcement is made with a caliper and a diamond perimeter diameter of ϕ6 willing to step 150 mm. According to Equation (B.1a) evaluation of new seismic code, the maximum plastic rotation pole used in checking the ULS is compressed area and the width of the item. It follows that the maximum plastic rotation pole is:

$$\boldsymbol{\theta_{um}^{pl} = \frac{\beta}{4^{v_d}} f_c^{0,2} \left(\frac{L_y}{h}\right)^{0,35} 25^{\propto \rho_x \frac{f_{yw}}{f_c}}} \quad (4)$$

Where β = 0.008 for columns; h=650 mm; L_V=3.54;

$$\rho_{s_x} = \frac{A_{s_x}}{b_w s_h} = \frac{3,41 \times 28,3}{500 \times 150} = 0,0013 \quad (5)$$

and

$$\boldsymbol{v_d = \frac{N_{Ed}}{bhf_c} = \frac{860 \times 10^3}{500 \times 400 \times 13,9} = 0,31} \quad (6)$$

width of the element is compressed.

It follows that the maximum plastic rotation pole is:

$$\boldsymbol{\theta_{um}^{pl} = \frac{0,018}{4^{0,31}} \times 13,9^{0,2} \times \left(\frac{1,75}{0,4}\right)^{0,35} \times 25^{0,443 \times 0,0013 \times \frac{236}{13,9}} \Rightarrow \theta_{um}^{pl} = 0,0152} \quad (7)$$

Since all columns are reinforced with smooth bars, and this amount should be reduced by half, so that the final amount of maximum plastic rotation in the transverse current frame inner pillars at ground level, is $\boldsymbol{\theta_{um}^{pl} = 0,8\%}$

Ways of defining the constitutive laws of plastic potential areas are shown schematically in Fig. 2.

The "flow" moment and the latter moment were determined based on reinforcement details of each item. Since last spins values do not differ significantly from one element to another, same values for global maximum plastic rotations were used to model plastic hinges. These are averagely estimated by applying relations P100-3/2008 Annex B of the code for a small number of elements

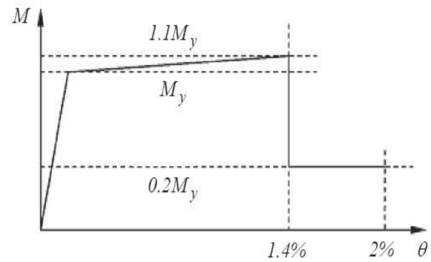

Fig. 2 Low M - θ beams

CONCLUSIONS

Since for the nonlinear dynamic calculation there were not used at least seven movements of the land compatible with the elastic response spectrum of the structure, average values of three analyzes performed could not be used for verification.

Therefore, for the 3 level methodology using nonlinear dynamic calculation methods, the degree of structural seismic insurance at the structure level is the lowest value of those obtained under the action of land movements characterized by the three diagrams for acceleration used:

R_3 = min (0.52, 0, 70, 0, 44) ⇒ R_3 = 0.44 = 44%

Consequently, according to their score indicator R_3 = 44%, Building A building FIFIM falls in seismic hazard class RsII.

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