DETERMINISTIC AND PROBABILISTIC SEISMIC PERFORMANCE ASSESSMENT OF A LOW-CODE RC FRAME BUILDING

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ABSTRACT

The paper presents the seismic performance assessment of a reinforced concrete frame structure representative for existing buildings in Bucharest. The assessment was performed using deterministic and probabilistic approaches, the last using Response Surface Methodology. Finally, fragility curves were obtained considering the peak ground acceleration as seismic intensity measure.

Keywords: seismic performance, response surface methodology, probabilistic assessment, fragility curves

REZUMAT

Lucrarea prezintă evaluarea performanţei seismice a unei structuri în cadre din beton armat reprezentativă pentru clădirile existente în Bucureşti. Evaluarea a fost realizată în abordare deterministă, dar şi probabilistică, cea din urmă utilizând Metodologia Suprafeţei de Răspuns. În final au fost obţinute curbe de fragilitate considerând acceleraţia maximă a terenului ca măsură a intensităţii seismice.

Cuvinte cheie: performanţă seismică, metodologia suprafeţei de răspuns, evaluare probabilistică, curbe de fragilitate

1. INTRODUCTION

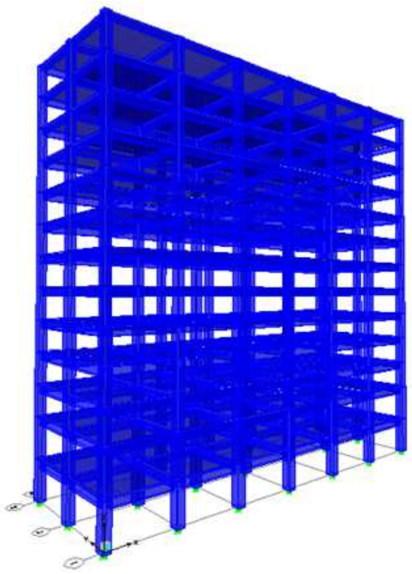

Fig. 1. Reinforced concrete frame structure GF+Mezzanine+11 Floors

The analyzed building has a representative structure for the built environment from Bucharest structure due to its spatial conformation having reinforced concrete frames, which exist in Bucharest especially on the large boulevards, sometimes having retail spaces at first storey and even at the second. A simulated design was performed according to the P13-70 seismic code to determine the reinforcement of structural elements.

The structure used in this study has 13 stories (GF + Mezzanine + 11 Floors), the ground floor having 3.60 m in height and the others, 2.75 m. All bays are 6.00 m long. The concrete used is Bc20 grade and the steel reinforcement is PC52 type.

2. DETERMINISTIC SEISMIC PERFORMANCE ASSESSMENT

In the deterministic analysis, the concrete was considered cracked, with the stiffness reduced to half of that of the uncracked concrete. The mean values of the strengths of materials were used in the analyses. The directions of application of earthquake forces were X (longitudinal) and Y (transversal).

In order to study the effect of masonry infill panels on the structural capacity at lateral forces, the structure was modeled in 3 assumptions: first, the simple structure, without considering the contribution of masonry infill panels, second, the structure with perimeter frames having masonry infill panels made of ceramic compressed bricks (c.c.b.) and third, with infills made of autoclaved aerated concrete (a.a.c.).

As seen in Figure 2, in the case of c.c.b. masonry, the capacity of the structure increases significantly, which obviously has an effect on the stiffness, consequently the fundamental vibration period of the structure being reduced. The a.a.c. masonry - with smaller density, compression, tension and shear strength - has a smaller effect.

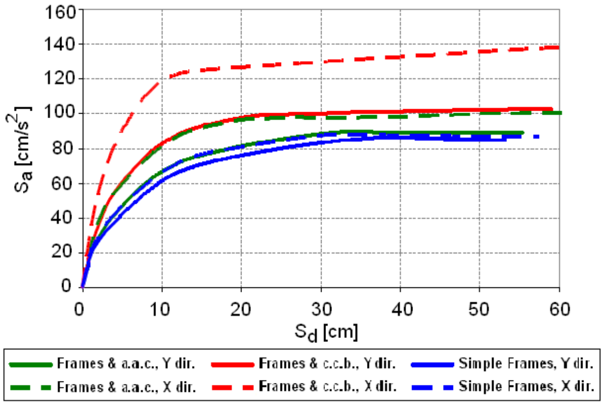

Fig. 2. S_a-S_d capacity curves in X and Y directions for the structure

Figure 3 shows that the capacity of the structure is exceeded in the principal directions for peak ground accelerations (a_g) of 0.24g and 0.36g corresponding to Ultimate Limit State (ULS) and Life Safety Limit State (LSLS) respectively according P100-1/2006 design code.

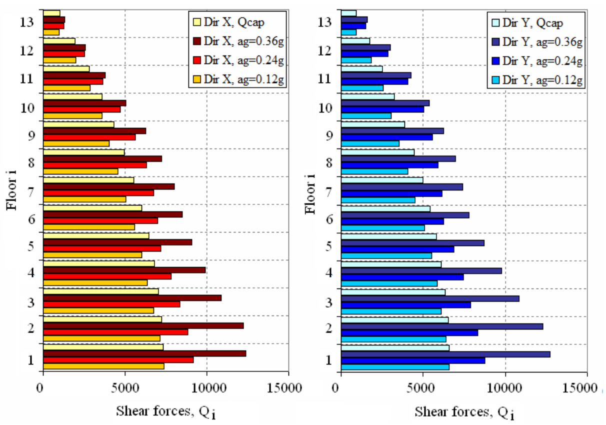

Fig. 3. Design and effective storey shear forces for SLS, ULS and LSLS for simple frames

Seismic displacement demands (spectral displacements, SD) for the two principal directions of the building for the first period of vibration, were determined using inelastic response spectra. For this purpose, 5 synthetic accelerograms compatible with the design spectrum for the Service Limit State (SLS) were used.

In order to illustrate more clearly the behavior of the structure at lateral forces, the spectral displacements at which the first plastic hinges appear in beams and columns and also the first failure of the structural elements at the base of the 1st storey columns are shown in Figure 4.

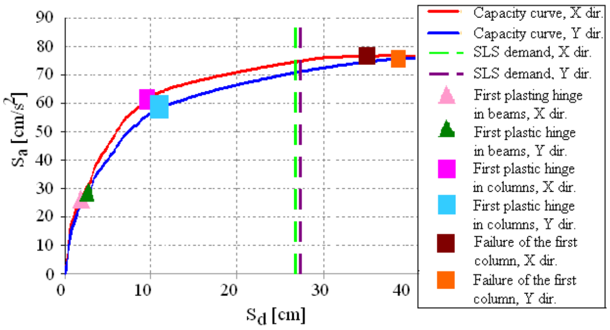

Fig. 4. S_a-S_d curve and the values corresponding the apparition of the first plastic hinges in beams and columns and the failure of the first column in X and Y directions

Regarding the hinge apparition mechanism, it should be noted that the first plastic hinges appear in the beams of the second storey in both directions of the structure, and then the hinges appear at the extremities of other beams as the lateral forces acting on the structure increase.

The first plastic hinges in the first storey columns at their base and their failure takes place before reaching the capacity of other structural elements.

3. PROBABILISTIC SEISMIC PERFORMANCE ASSESSMENT

3.1. Methodology

The Response Surface Methodology (RSM) involves obtaining a response surface through a function with multiple variables and determining the polynomial coefficients

(Myers and Montgomery, 2002), the response surface being, in fact, a polynomial regression to a set of values that must be at least as numerous as the coefficients. If the set of values to be determined is large and it would require a lot of time for analyses, a method by which the number of analyses would be as small as possible will be chosen. The Design of Experiments is a solution in this regard.

Figure 5 shows the process of obtaining fragility curves using RSM. The seismic intensity parameter plays the role of control variable, it is deterministic and it has fixed values, the structural parameters being random variables of the response surface function. The control variable is fixed at a given level of seismic intensity, while the random variables modify their values following Monte Carlo simulations, according to their probability distribution. Metamodels are directly obtained for each level of seismic intensity, by evaluating the response surface at this value.

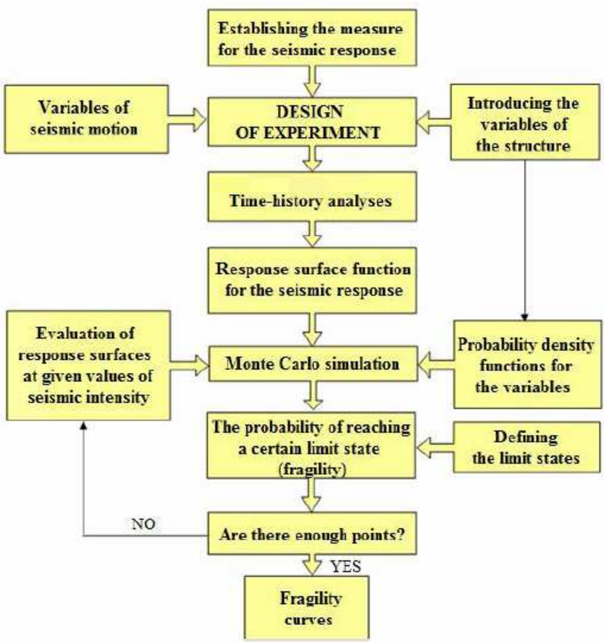

Fig. 5. Determining seismic fragility curves using Response Surface Methodology

In order to evaluate the seismic performance of the frame structure, the following random variables were considered: concrete compressive strength (f_c) and concrete longitudinal elasticity modulus (E_c), yield strength of steel (f_y), the control variable being the peak ground acceleration (PGA).

A normal distribution and a coefficient of variation of 15% for the concrete compressive strength and a lognormal distribution and a coefficient of variation of 7% for yield strength of steel were assumed.

Table 1 presents the values of the random structural parameters considered in the models, along with their coded values (-1, 0, 1), which make easier performing the simulations and obtaining the response surfaces.

Table 1. *Input variables for the RC frame structure*

Random Structural Parameters	Input variables	Lower bound	Center Points	Upper bound
Concrete compressive strength, f_c	ξ_1 (MPa)	11	20	29
	x_1	-1	0	1
Concrete longitudinal elasticity modulus, E_c	ξ_2 (MPa)	8100	14850	21600
	x_2	-1	0	1
Steel yield strength, f_y	ξ_3 (MPa)	377	405	433
	x_3	-1	0	1
Peak ground acceleration, PGA	ξ_4 (g)	0.12	0.36	0.60
	x_4	-1	0	1

Sets of 5 accelerograms compatible with 5% damped design spectrum for Bucharest (as specified by the Romanian seismic design code P100-1 [1]), scaled at PGA of 0.12g, 0.36g and 0.60g were used in the analyses.

Central Composite Design (CCD) was used in the Design of Experiment, resulting 25 combinations of the four variables.

Currently, the deformation demands are among the most effective parameters in predicting structural and non-structural damage. In the current paper, for assessing the RC frame structure performance, the interstory drift ratio (ISD) was chosen as a parameter for determining the damage state at different values of seismic intensity, measured by PGA, for which the structure was analyzed. ISD is defined by

$$\text{ISD}\ (\%) = 100(d_{i+1} - d_i)/H_i \qquad (1)$$

where d_{i+1} and d_i are the displacements of storeys $i+1$ and i, while H_i is the storey height.

The maximum interstory drift (ISD_{max}) was recorded for each time-history analysis and a normal distribution of its values was considered.

3.2. Analyses results

Table 2 presents the mean and the standard deviation values of maximum interstory drifts, determined after performing the 25 nonlinear time-history analyses in the principal directions of the structure. The polynomial coefficients of response functions were determined using these values.

Figures 6 and 7 show the response surfaces for mean and standard deviation of ISD_{max} as a function of the following random variables: concrete compressive strength, concrete longitudinal elasticity modulus and peak ground acceleration.

It can be seen that a drop in the stiffness of concrete influences significantly the value of the interstory drift.

Table 2. *Matrix of the experiment and response values used in generating the response surfaces*

Case no.	Parameters				ISD_{max} (% of H_{storey})			
					X direction		Y direction	
	x_1	x_2	x_3	x_4	Mean	Std. dev.	Mean	Std. dev.
1	-1	-1	-1	1	4.670	0.404	5.273	0.454
2	1	1	-1	-1	0.948	0.070	0.945	0.074
3	1	0	0	0	2.956	0.440	3.035	0.254
4	-1	-1	1	-1	1.146	0.089	1.344	0.135
…	…	…	…	…	…	…	…	…
24	-1	1	1	1	3.309	0.135	3.405	0.140
25	1	-1	-1	1	4.669	0.404	5.272	0.454

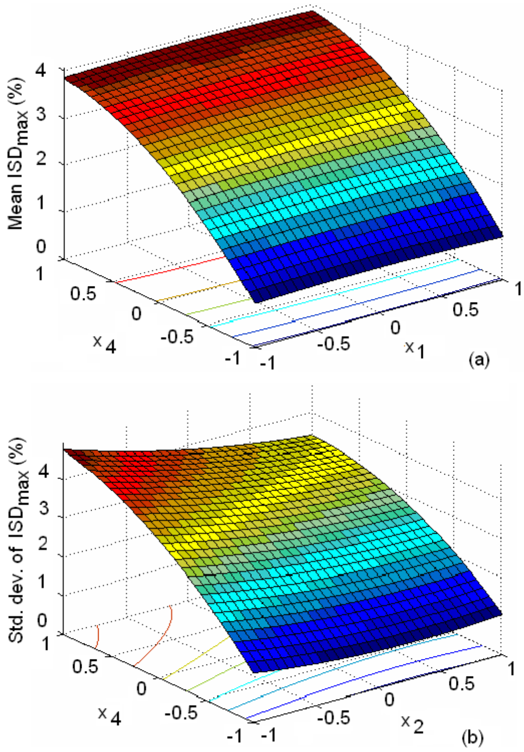

Fig. 6. Response surfaces for mean (a) and standard deviation (b) of ISD_{max}, as a function of f_c (x_1), E_c (x_2) and PGA (x_4) in X direction

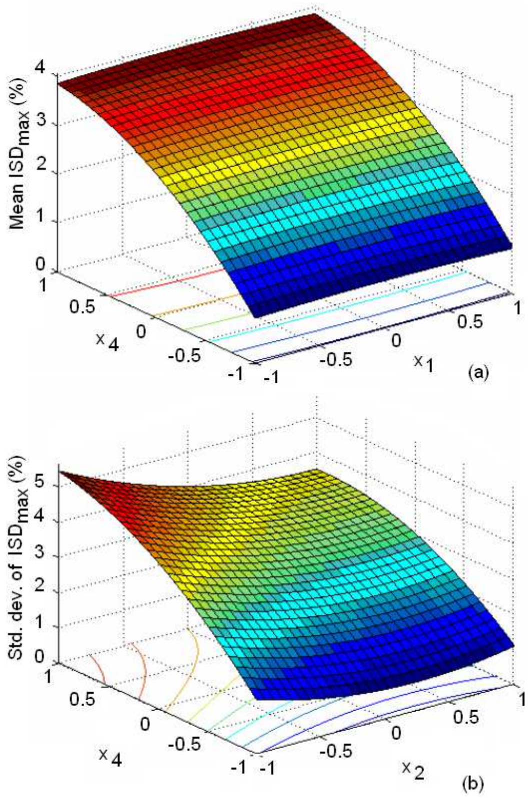

Fig. 7. Response surfaces for mean (a) and standard deviation (b) of ISD_{max}, as a function of f_c (x_1), E_c (x_2) and PGA (x_4) in Y direction

The verification of the accuracy of the model is performed by using the coefficient of determination (R^2) and its modified version (R_A^2).

The value of R^2 ranges between 0 and 1 and it represents the fraction of the total variation of data points. A value of R^2 close to 1 indicates a good fit of the model, but sometimes this is insufficient in the process of statistical validation of the model, its value increasing with the number of variables considered. Therefore, a modified coefficient of determination, R_A^2, is used for the validation, as defined by Equation (3).

Table 3. *Statistical validation of response surfaces*

Statistical Parameters	X directions	Y directions
Error Sum of Squares, SSR	44.3E-04	55.2E-04
Total Sum of Squares, SST	44.3E-04	55.4E-04
Coefficient of determination, R^2	0.9996	0.9961
Modified coefficient of determination, R_A^2	0.9990	0.9912

$$R^2=SSR/SST \tag{2}$$

$$R_A^2=1-(1-R^2)(N-1)/(N-p) \tag{3}$$

In Equation (3), N is the number of combinations (analysis cases) and p represents the number of polynomial coefficients of response surface functions.

As seen in Table 3, the values of R^2 and R_A^2 are very close to unity, indicating the accuracy of the model.

The response surface models were assessed at various levels of PGA. A number of 10,000 Monte Carlo simulations were performed on these models, generating random values for the variables x_1, x_2 and x_3, between their lower and the upper bounds, considering their specific statistical distributions.

Considering the limit values of the maximum interstorey drifts coresponding to the three damage states described in the American document FEMA 356: IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention), the fragility curves in the Figure 8 were obtained.

The probability for the analyzed frame structure to be in one of the three discrete damage states (IO, LS, CP) at a certain value of PGA is computed as the difference between the values for the same PGA of the two consecutive damage functions.

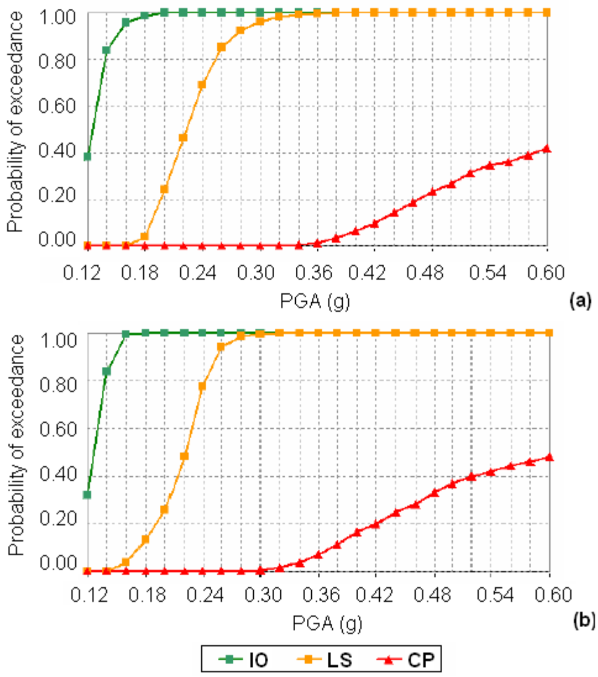

Fig. 8. Fragility curves in X (a) and Y (b) directions for the considered frame structure, corresponding to the damage states in FEMA356: IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention)

As it can be seen in Figure 8, for PGA=0.24 g (mean recurrence interval MRI=100 years, according to the P100-1/2006 code), the conditional probability of being in or exceeding the IO damage state is 100% in both principal directions of the structure. For LS damage state, this probability is 69% in X direction and 77% in Y direction, and for CP damage state, the probability is 0% in both directions.

4. CONCLUSIONS

Following the deterministic and probabilistic analyses that were performed, the RC frame structure showed a low level of seismic protection, as compared to the requirements of the existing Romanian seismic design code, requiring rehabilitation measures

Also, another important aspect revealed by the seismic fragility curves obtained as functions of PGA is that the collapse of the building is unlikely. However, other aspects should be taken into account: the influence of the uncertainties related to the model, the approximations that were used and the fact that the values of the maximum interstory drifts for the three damage states taken from FEMA 356 do not accurately illustrate the behavior of the RC frame structures from Romania.

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