13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1796

SEISMIC VULNERABILITY OF RC BUILDINGS IN BUCHAREST, ROMANIA

Radu VACAREANU[1], Raluca RADOI [2], Caterina NEGULESCU [3],Alexandru ALDEA[4]

SUMMARY

The paper presents the seismic vulnerability assessment of RC buildings using HAZUS [1] and ATC-40 [2] methodology. Some inconsistencies of the mentioned methodologies are highlighted and an alternative approach is applied. The Monte-Carlo simulation is used to calibrate the fragility function parameters. Five case studies for representative residential buildings in Bucharest-capital city of Romania, are presented and compared. The paper also underlines the specificity of the demand spectra in Bucharest, that is significantly different from the HAZUS ones.

INTRODUCTION

According to the number of people lost in earthquake disasters during 20th century as well as in a single event (March 4, 1977: 1574 deaths, including 1424 in Bucharest), Romania can be ranked the 3rd country in Europe, after Italy and Turkey. Romania is followed by the former Yugoslavia and by Greece (Bolt [3] Coburn and Spence [4]). The World Bank [5] loss estimation after the 1977 earthquake indicates that from the total loss (2.05 Billion US $) more than 2/3 were in Bucharest, where 32 tall RC buildings collapsed. Half of the total loss was accumulated from building damage.

Nowadays there is a high concern of civil engineers and Romanian Government for the assessment and the reduction of seismic risk in Romania.

The World Map of Natural Hazards prepared by the Münich Re [6] indicates for Bucharest: “Large city with Mexico-city effect”. The map focuses the dangerous phenomenon of long (1.6s) predominant period of soil vibration in Bucharest during strong Vrancea earthquakes. Bucharest and Lisbon are the only two European cities falling into Mexico-city category.

---

[1] Associate Professor, Technical University of Civil Engineering, Bucharest, Romania.
Director, National Center for Seismic Risk Reduction, Bucharest, Romania. E-mail: vradu@mail.utcb.ro
[2] Research engineer, Technical University of Civil Engineering, Bucharest, Romania
Research engineer, National Center for Seismic Risk Reduction, Bucharest, Romania.
[3] Research engineer, Technical University of Civil Engineering, Bucharest, Romania
Research engineer, National Center for Seismic Risk Reduction, Bucharest, Romania.
[4] Lecturer, Technical University of Civil Engineering, Bucharest, Romania.
Chief of Division, National Center for Seismic Risk Reduction, Bucharest, Romania.

In Bucharest, the seismic risk is well identified: the most vulnerable buildings are mid and high rise buildings built before 1978 earthquake resistant design code. The buildings before 1941 were built without considering earthquake action, and between 1941 and 1978 the design spectrum was not appropriate for mid and high rise buildings when considering the characteristics of strong ground motions recorded in the city during 1977 and 1986 earthquakes. Romanian Government and local authorities started after 1989 a national program for evaluation of seismic resistance of vulnerable buildings, program that was later integrated into a national strategy for seismic risk reduction. The action of identification of vulnerable buildings is a continuous one. For the buildings for which expert reports are already available, the Seismic Risk Reduction Commission of Ministry of Transports, Constructions and Tourism established priority lists for retrofitting. For example in Bucharest 115 residential buildings were classified as having the highest seismic risk in case of an earthquake similar or stronger to the 1977 one, and now 8 are in retrofitting works.

EARTHQUAKE RESISTANT DESIGN CODES AND BUILDING STOCK

The codes for earthquake resistance of buildings and structures in Romania during the last 60 years are classified in the HAZUS format (Lungu [7]), Table 1.

Table 1. Classification of codes for earthquake resistant design of buildings in Romania (1940-2000)

Period		Code for earthquake resistance of structures
Pre-code, before 1963	Prior to the 1940 earthquake and Prior to the 1963 code	P.I. - 1941 I - 1945
Low-code, 1963-1977	Inspired by the Russian seismic practice	P 13 - 63 P 13 - 70
Moderate-code, 1977–1990	After the great 1977 earthquake	P 100 - 78 P 100 - 81
Moderate-code to High-code, after 1990	After the 1986 and the 1990 earthquakes	P 100 - 90 P 100 - 92

After the 1977 event, ductility rules for reinforced concrete structures were imported into Romanian codes from American Concrete Institute (ACI) codes of practice. Those ductility rules were improved in 1990 according to the new scientific developments.

The values of yielding acceleration (Figure 1) and ultimate acceleration (Figure 2), as they can be inferred from design codes, are herein presented for the HAZUS [1] building types RC1H (high-rise RC frames) and RC2H (high-rise RC shear walls).
The Bucharest building stock was classified with regards to the building's number of storeys and design code (Lungu [7]), Table 2.

Table 2. Classification of buildings in Bucharest, according to their period of construction

Number of storeys	Number of buildings	Period of construction / Code for earthquake resistance of structures							
		until 1900	1901-1929	1930-1945	1946-1963	1964-1970	1971-1977	1978-1990	1990-1995
		-	-	P.I. - 1941	I. - 1945	P13 - 63	P13 - 70	P100 - 81	P100 - 90
1-2	95484	5562	16205	27275	30524	8413	4391	2893	221
3-7	7514	315	1255	2146	979	804	782	1214	19
>8	5283	41	95	164	378	645	1072	2854	34
Total	108281	5918	17555	29585	31881	9862	6245	6961	274

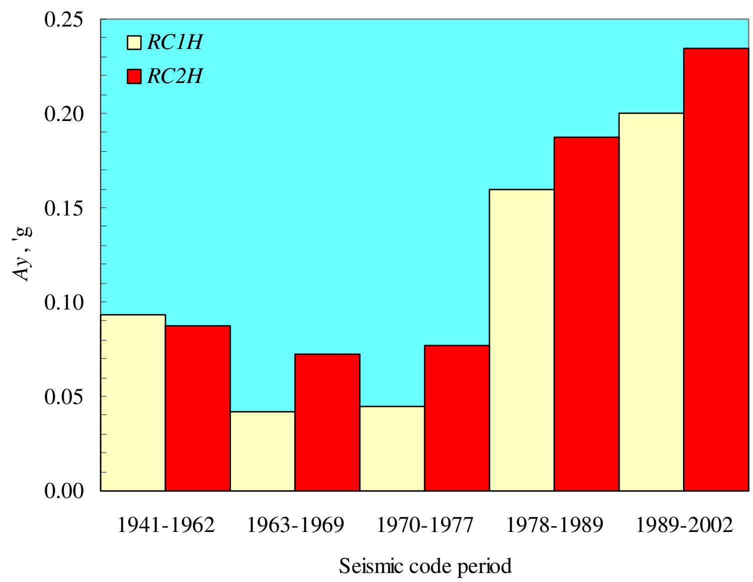


Figure 1. Yielding acceleration according to seismic code period

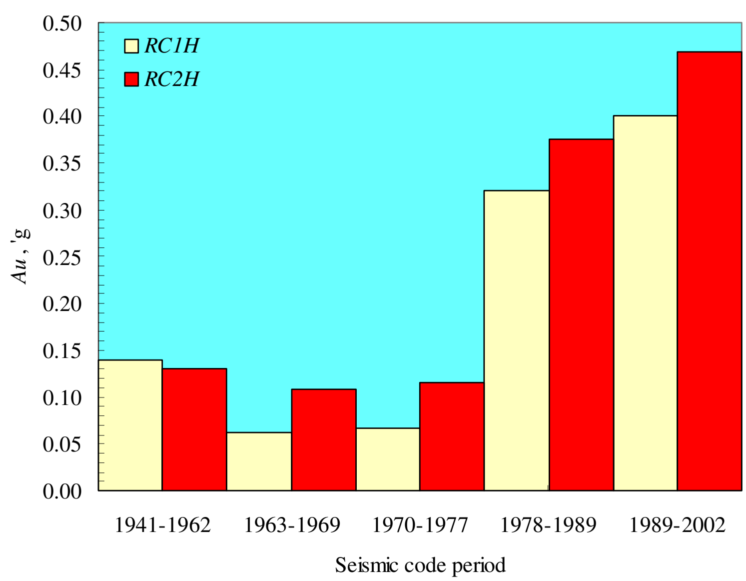


Figure 2. Ultimate acceleration according to seismic code period

The present paper presents the vulnerability analysis of three RC frame structures and of two structures with structural walls. The analysis for one of the RC frame structures is presented in more detail, since it is a typical and representative structure and it is located in the vicinity of INCERC site where the strong ground motion of 1977 Vrancea earthquake was recorded.
For a better understanding of the design of RC frame structures in the pre-code and low-code period, some considerations are herein presented (Excerpts from Romanian Academy [8]).
"Reinforced concrete frames were used for residential buildings before 1977 earthquake mainly on the large boulevards of Bucharest, in two separate periods separated by a time interval of 10 years:

-1956-1963, for isolated buildings located on the main boulevards in city centre : Magheru, 6 Martie, Republicii, etc, having ground-floor and 6÷8 storeys, with columns connected by beams on both directions and RC cast-in-place slabs and having infills of masonry […], the structure being designed for a global seismic coefficient of 3.5%;

- 1974-1976, for large ensembles of structures on main boulevards like Pantelimon, Calea Dorobanti, Titulescu, Obor, Armata Poporului, etc, having ground-floor and 8÷14 storeys, with cast-in-place or prefabricated RC slabs, with infilled walls of autoclaved aerated concrete, the structures being designed according to P13-70 code with a global seismic coefficient of 2.5%.

It should be noticed that the 1974-1976 buildings, in comparison with the older ones (1956-1963) suffered more during 1977 earthquake, being more flexible, with a fundamental period larger than 1 second (consequently being more sensible to the spectral content of the ground motions corresponding to this earthquake), being taller, having commercial spaces at ground-floor, having less stronger infilled walls and being designed for smaller seismic forces according to the regulations in force at that time.

These disadvantages were partially compensated by a better design and by the use of a better concrete but some insufficiencies in the design could not be avoided due to the limitations of the existing regulations, and also some mistakes in execution leaded to structural and non-structural damage.

Generally speaking, the behaviour of RC frame structures in Bucharest during earthquake differed due to the height, design and location, having as result a large variety of damage."

The damage of such RC frame structures is described as follows (Romanian Academy [8]):

"After 1977 earthquake, at some of the RC frame structures built in 1974-1976 were noticed several types of damage, the most serious one being the damage at the columns and beams at lower levels, but in many cases such damage occurred also at higher locations. At the columns, damage occurred especially around the horizontal technological joints […], consisting of cracks with concrete cover expulsion and buckling of the longitudinal reinforcement. Vertical and inclined cracks have been noticed in beams near the supports. Moreover, cracks have been noticed in slabs or in landings, especially in the lower stories. The autoclaved aerated concrete masonry panels had been severely damaged. Local and generalised cracks, bricks or mortar expulsion, have been noticed in the first 4-5 stories; the plaster finishing (some with rather large thickness) had fallen down." Examples of damage to such structures are presented in Fig. 3.

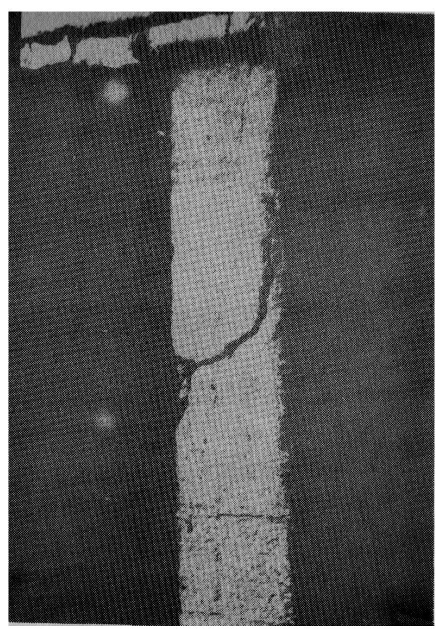


Figure 3. 1977 earthquake in Bucharest: damage to residential RC frame buildings on Pantelimon street (Romanian Academy [8])

METHODS FOR SEISMIC EVALUATION

The advanced methodologies on seismic evaluation of existing buildings such as Earthquake Loss Estimation Methodology – HAZUS [1] and Seismic Evaluation and Retrofit of Concrete Buildings – ATC 40 [2] assess the seismic fragility using the "capacity spectrum method" as a tool for quantifying the expected seismic response of buildings and structures.

The "capacity spectrum method" (Freeman et al. [9]) is based on a graphical procedure that compares the structure's capacity curve with the demand spectrum imposed by the expected seismic motion (Fig. 4).

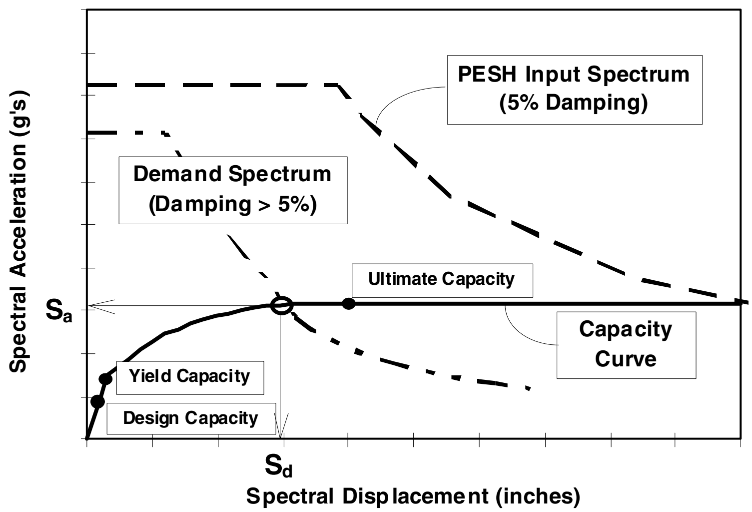


Figure 4. HAZUS [1] - Capacity curve and demand spectrum

The "capacity spectrum method" is aiming to find a performance point on the capacity curve that also lies on the appropriate demand response spectrum (reduced for non-linear effects). The application of "capacity spectrum method" requires that both the demand response spectrum and the structural capacity curve to be plotted in the spectral acceleration vs. spectral displacement domain. This representation format (Mahaney et al. [10]) is commonly termed as ADRS. The inelastic capacity of the structure is represented by the pushover curve. For getting the capacity curve in the ADRS format, it is necessary to do a point-by-point conversion, any point on the pushover capacity curve is converted to the corresponding point on the capacity spectrum. The seismic demand is defined by highly damped (reduced) elastic spectra.

Methodology for assessing seismic fragility according to HAZUS [1]/ATC-40 [2]

The step-by-step procedure to assess the seismic fragility of existing buildings is:

1. Create a model of the structure; perform a pushover analysis; plot the roof displacement – base shear (Δ_{roof} - V_i) curve;
2. Convert the capacity curve to the capacity spectrum; any point V_i, Δ_{roof} on the capacity curve is converted to the corresponding point S_{ai}, S_{di} on the capacity spectrum;
3. Obtain the elastic demand response spectrum in ADRS format; the spectrum might be computed from an actual seismic motion or might be a smoothed design spectrum;
4. Plot on the same graph both the demand and the capacity spectrum;
5. Select a trial performance point on the capacity spectrum; corresponding to this point compute the equivalent viscous damping, β_{eff} (%):

$$\beta_{eff} = \beta_0 + 5\% \tag{1}$$

where β_0 is the hysteretic damping represented as equivalent viscous damping and 5% is the viscous damping inherent in the structure. The term β_0 can be calculated as (Chopra [11]):

$$\beta_0 = \frac{1}{4\pi}\frac{E_D}{E_{S0}} \tag{2}$$

where E_D is the energy dissipated by damping and E_{S0} is the maximum strain energy.

6. Derive the spectral reduction factors SR_A and SR_V. They are given by (Newmark & Hall [12]):

$$SR_A=(3.21-0.68\ \ln(\beta_{eff}))/2.12 \tag{3}$$

$$SR_V=(2.31-0.41\ \ln(\beta_{eff}))/1.65 \tag{4}$$

7. Reduce the elastic acceleration spectrum according to the computed equivalent viscous damping using spectral reduction factors; plot on the same graph the reduced demand spectrum.

8. Obtain the intersection point of the capacity spectrum with the reduced demand spectrum. If the displacement at the intersection of the demand spectrum and the capacity spectrum is within 5% of the displacement of the trial performance point, the selection made in step 5 is correct and the trial performance point becomes the actual performance point. If the displacement is not within the acceptable tolerance, then a new trial point is selected and the process is repeated starting with step 5. The performance point is the maximum spectral displacement (S_d) expected for the specific demand spectrum.

9. Determine the building fragility functions. The conditional probability of being in, or exceeding, a particular damage state, d_s, given the spectral displacement S_d, is defined by:

$$P\left[ds \middle| S_d\right] = \Phi\left[\frac{1}{\beta_{ds}}\ln\left(\frac{S_d}{\bar{S}_{d,ds}}\right)\right] \tag{5}$$

where:

$\bar{S}_{d,ds}$ is the median value of S_d at which the building reaches the threshold of the damage state d_s,

β_{ds} is the standard deviation of the natural logarithm of spectral displacement for damage state d_s, and

Φ is the standard normal cumulative distribution function.

10. For the maximum spectral structural displacement S_d expected for the specific reduced demand response spectrum, determine the damage state probabilities using fragility functions from step 9.

Alternative approach for assessing the expected seismic behaviour of buildings

In some cases, when applying the reduction factors from Eq.(3) and Eq.(4) as recommended by HAZUS [1] and ATC-40 [2], the comparison of the reduced displacement response spectra with the computed non-linear displacement response spectra can present significant differences. Such an example is presented in Figure 5, for the NS component of INCERC 1977 record (Vacareanu et al. [14]). The computed non-linear displacement response spectrum was obtained with NONSPEC software.

In order to avoid this inconsistency, an alternative approach using strength reduction factors is proposed (Chopra & Goel [13]). The proposed procedure uses the constant-ductility spectrum for the demand spectrum, instead of the elastic spectrum in ATC-40 [2] and HAZUS [1] procedures. The expected spectral displacement S_d of an inelastic system with properties T (predominant period of vibration), μ (ductility factor), and f_y (yielding strength) is determined by the following steps (which replace steps 5-8 from original methodology):

1. A constant-ductility design spectrum is established by reducing the elastic design spectrum by appropriate ductility-dependent factors that depend on T.

2. The capacity curve is plotted on the same ADRS graph. The yielding branch of the capacity curve intersects the demand spectra for several μ values. One of these intersection points, which remain to be determined, will provide the performance point. At this point, the ductility factor computed from the capacity curve should match the ductility value associated with the intersecting demand spectrum.
Once the performance point is obtained, the procedure follows steps 9 and 10 from HAZUS/ATC-40 methodology.

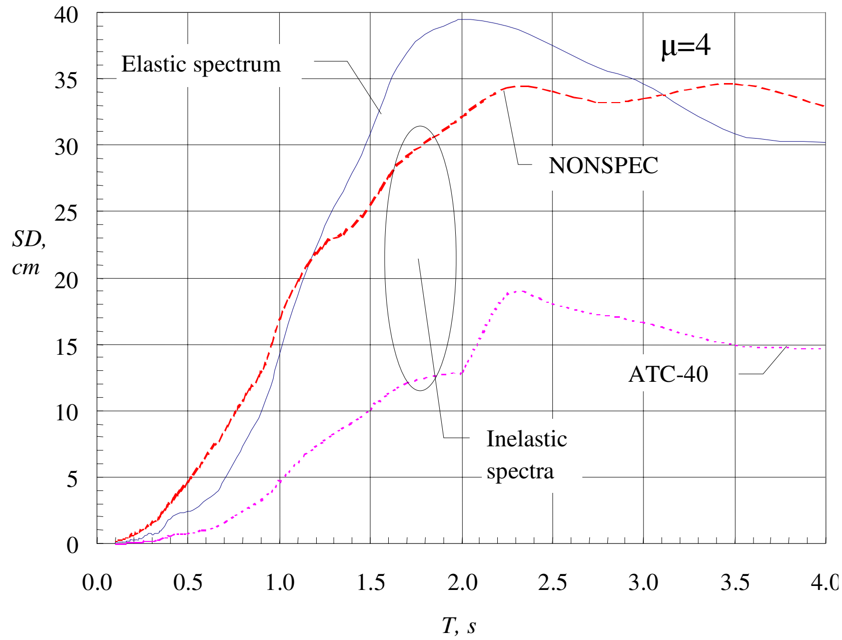


Figure 5. Elastic and inelastic displacement spectra of INCERC (NS) 1977 Bucharest record

Monte-Carlo simulation for obtaining fragility function parameters

HAZUS [1] gives the fragility function parameters $\bar{S}_{d,ds}$ and β_{ds} that are appropriate for each type of building, according to USA practice of design and construction. In order to calibrate the fragility function parameters appropriate for structural systems that are different from USA practice, the Monte-Carlo simulation technique [15] can be used. When applied for the development of fragility function parameters, Monte-Carlo technique involves the selection of values of the input capacity random variables required for pushover analysis, the pushover analysis and the simulation of structural damage.
The Monte-Carlo technique requires a large number of simulation in order to achieve an acceptable level of confidence in the estimated probabilities. The Latin hypercube technique was used in order to reduce the number of simulation cycles. In brief, the simulation technique implies the following steps: (i) simulation of structural parameters; (ii) random permutations of structural random variables; (iii) performing push-over analyses using generated samples; (iv) sample statistics of results of analyses.
The compressive strength of concrete and the yield strength of steel are, as a minimum, the parameters that should be treated as the random variables. Following Galambos et al. [16], a normal probability distribution for concrete strength and a lognormal probability distribution for steel strength might be used.
The outcome of the pushover analyses is a family of capacity curves, which can be described by mean or mean plus/minus one standard deviation capacity curves.
For calibration of fragility function parameters it is necessary to establish a correlation between Park&Ang [17] damage index and interstory drift at threshold of damage state. The slightly modified version of the Park&Ang index, in which the recoverable deformation is removed:

$$D = \frac{Dm - Dy}{Du - Dy} + \beta e \cdot \frac{\int dE}{Fy \cdot Du} \tag{6}$$

where D_m = maximum displacement; D_u = ultimate displacement; D_y = yielding displacement; β_e = strength deterioration parameter; F_y = yielding force and E = dissipated hysteretic energy.

The correlation between Park&Ang damage index and damage state is given in Table 3.

Table 3. Relations between damage index and damage state

Range of damage index	Damage state
$D \leq 0.1$	None (N)
$0.1 < D \leq 0.25$	Minor (Mi)
$0.25 < D \leq 0.40$	Moderate (Mo)
$0.40 < D \leq 1.00$	Severe (S)
$D > 1.00$	Collapse (C)

Using the definition of Park&Ang [17] damage index and the capacity curves, one can determine the correlation between damage index and interstory drift as mean and standard deviation values, Figure 6.
Making vertical sections in Figure 6 for the threshold values of Park&Ang damage index given in Table 2 one can identify the mean and standard deviation values of interstory drift at threshold of each damage state. The median value of spectral displacement at which the building reaches the threshold of the damage state, $S_{d,ds}$ is obtained by multiplying the interstory drift by the height of the building and by the fraction of the building height at the location of pushover mode displacement. The complete damage state corresponds to the collapse prevention limit state and the extensive damage state corresponds roughly to the life safety limit state.
Once the $S_{d,ds}$ and β_{ds}, parameters are obtained, one can compute and plot the fragility functions using equation 5.

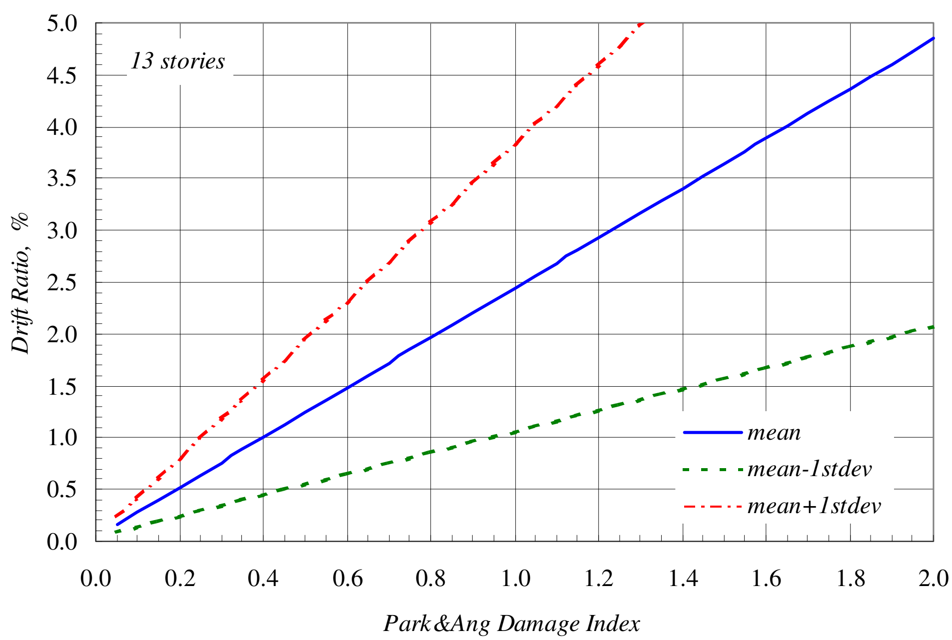


Figure 6. Correlation between Park&Ang damage index and interstory drift (Vacareanu et al. [18])

CASE STUDY: PANTELIMON BUILDING, BUCHAREST

The Pantelimon residential building has a reinforced concrete frame structure and it is a typical frame structure in Bucharest. Erected in 1974, the building was designed using P13-70 seismic design code, which is classified nowadays as a low-code (global seismic coefficient for design 2.5%).
The building is located in Eastern Bucharest, Pantelimon Street nr.258, Building 47, in the vicinity of INCERC seismic station, where the first Romanian strong ground motion was recorded during Vrancea earthquake of March 4, 1977 (moment magnitude Mw=7.5). It is an interesting case study due to at least two reasons: (i) it is a low-code building susceptible to be seriously damaged during strong earthquakes and (ii) it has vibration characteristics close to those of the ground at INCERC site, so resonance phenomenon might be expected. There are many similar building on the main boulevards of Bucharest, so it also has a social significance.
The Pantelimon building is a 12/13 storeys (ground-floor and 11 storeys on the left side and ground floor and 12 storeys on the right side and in the centre) RC frame. The structure consists of 5 spans and 2 bays each having 6m. The staircase is in an external 6x6m section, and was included in the original design of the building as a component part of the building. The building is described in Figures 7 and 8. The columns are rectangular with sections of 60 x 70…80 cm at ground floor and first floor, and with slightly reduced sections at the other floors; the beams have constant sections for all the building, 25x55 cm for marginal beams and 30x60 cm for interior beams; the slabs have 15 cm thickness. The frames are made of cast in place reinforced concrete C 20/25 (concrete strength fc= 24.5 MPa, Young's modulus for the concrete was taken 27500 MPa). The reinforcing yield stress is f_y= 405 MPa. The infills are of Autoclaved aerated concrete (AAC). The storey weight is 11KN/m2.

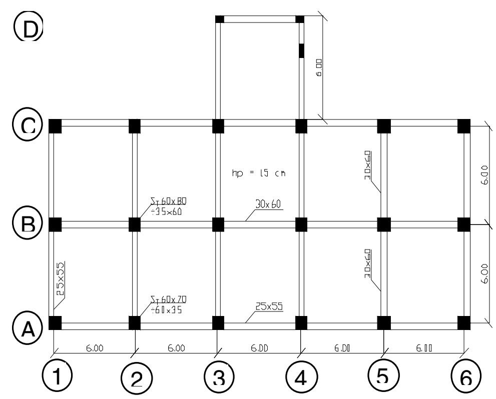


Fig.7 258, Pantelimon street (front view) **Fig.8 258, Pantelimon street (horizontal layout)**

Height floor is 2.8 m, the ground-floor has 4.5 m, the total height of the building is 38.1m (without including the technical room located on top of the building). The basement is a technical one having 2 m height. The foundation is continuous on the building contour and isolated for the interior columns.
The structure belongs to a larger building consisting of 3 similar structures of different heights (Fig.7), but each structure was considered separately during the design.
INCERC site was instrumented for site effects assessment within the JICA Project [19] by National Centre for Seismic Risk Reduction. Two borehole sensors were installed, one at –24m depth and the other at –153m depth. The behaviour during earthquake of typical repetitive buildings is of major interest for seismic risk reduction efforts in Bucharest. Pantelimon building regrouped the attention of both NCSRR, within JICA Project, and Technical University of Civil Engineering, within NEMISREF Project [20]. In July 2003 a microtremor measurement was performed (by NCSRR staff and NEMISREF team) in the nearby structure (ground-floor and 11 storeys). The results of the measurement will be soon available.

Two design spectra were converted in ADRS format for the analysis, one anchored at PGA=0.1g and the other anchored at PGA=0.2g (design acceleration in Bucharest according to design code in force – P100-92). The shape of the design spectra is the shape from the 2003 proposal for a new earthquake resistant design regulation (Lungu et al. [21]). In the first case (PGA=0.1g) the spectral displacement corresponding to the performance point is 16.24 cm, and in the second case (PGA=0.2g) it is of 26cm.

The relations proposed by Fajfar, Cuesta, Aschheim [22] for the determination of the strength reduction factors were used for this analysis:

$$R=c_1(\mu-1)(T/T_c)+1, \quad T/T_c<1 \qquad \text{and} \qquad R=c_1(\mu-1)+1, \quad T/T_c>1 \tag{7}$$

The fundamental period of vibration of the structure is 1.65 seconds. Pushover analysis were performed for the structural system using SAP2000 computer program. The characteristic values of the capacity curve are: (i) yielding spectral displacement 6.2cm; (ii) yielding spectral acceleration 0.07g; (iii) ultimate spectral displacement 30.8cm; (iv) ultimate spectral acceleration 0.078g.
The performance point is identified in the Fig.9 by the intersection between the inelastic demand response spectra and the capacity curve. The obtained fragility functions are presented in the Fig. 10. The result of the analysis is presented in the Fig. 11 in terms of probabilities of being in a certain damage state.

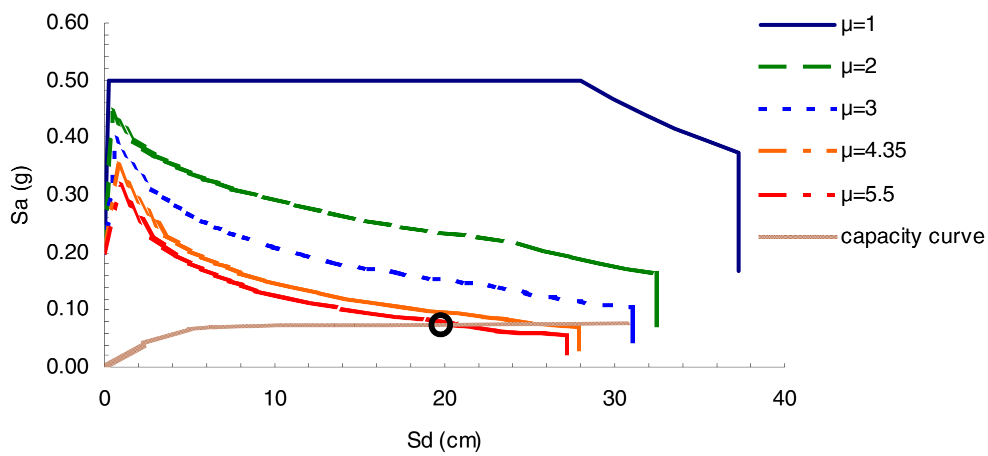


Figure 9. Elastic and inelastic acceleration-displacement response spectra versus capacity diagram

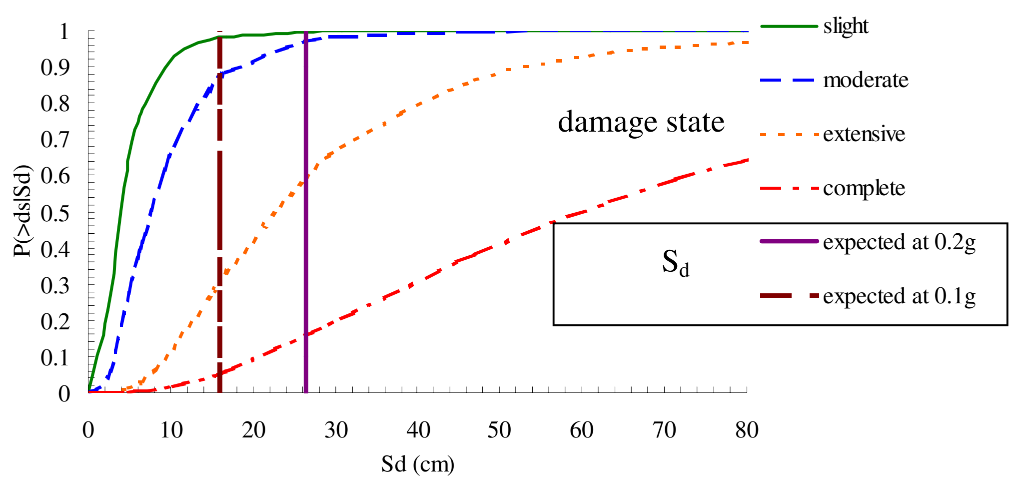
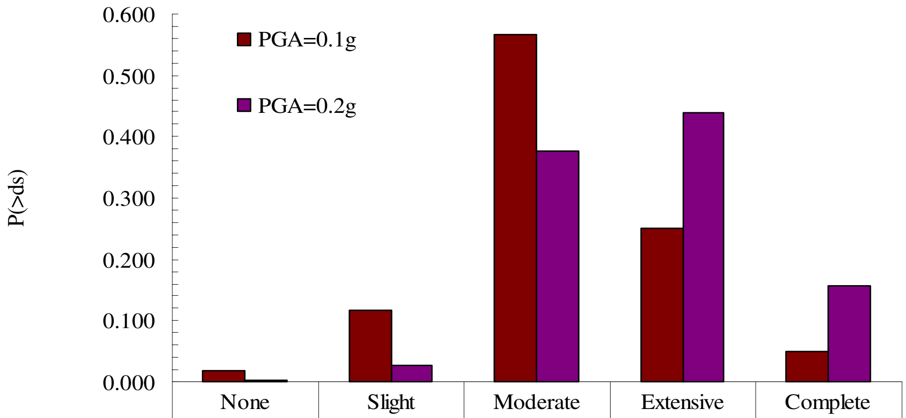


Figure 10.Fragility functions and expected spectral displacement for 0.1g and 0.2g



	None	Slight	Moderate	Extensive	Complete
PGA=0.1g	0.018	0.115	0.566	0.251	0.050
PGA=0.2g	0.002	0.028	0.376	0.438	0.156

Figure11. Pantelimon building - probabilities of being in a certain damage state

Other case studies

The modified methodology presented previously Chopra et al. [13] was applied for other four case studies of representative residential buildings in Bucharest. Details of the buildings are presented in Table 4 (high-rise RC frames and structural walls structures with two levels of seismic code). The horizontal layout of the buildings is presented in Figure 12.

Table 4. Details on the buildings used as case studies

Building name	Structural system - code	No of Stories	Building height, m	Design code	Fundamental period, T_1, s
Armata Poporului	RC frames – RC1	GF+10S	30.8	P13-70 – low code	1.34
Pacii	RC frames – RC1	GF+11S	35.2	P13-70 – low code	1.44
M1F4	RC structural walls – RC2	GF+10S	31.0	P13-70 – low code	0.55
D11F	RC structural walls – RC2	GF+7S	22.0	P100-81– medium code	0.4

Pushover analysis were performed using IDARC 2D computer program [23]. The results of the pushover analysis are presented in Table 5 and Figure 13.

The expected seismic structural response of the buildings was evaluated against the ADRS spectra of the seismic motion recorded at INCERC on March 4, 1977, N-S direction (PGA=0.2g). The results of analyses and the performance points are presented in Figure 14 and in Table 5. Demand spectra in Bucharest are different in comparison with the ones specified in HAZUS [1] (Aldea et al. [24]). HAZUS underlines that it's demand spectra does not apply for the combinations of source and site conditions characterized by significant amplifications at periods larger than 1 second. For such special sites (as Mexico & Bucharest case), HAZUS demand spectra over-estimate the spectral acceleration at low periods and under-estimate it at long periods.

Given the expected seismic response of the buildings and the parameters of the fragility function, the Park & Ang [17] damage index as well as the probability of collapse for each building were determined. The comparative results are presented in Figure 15.

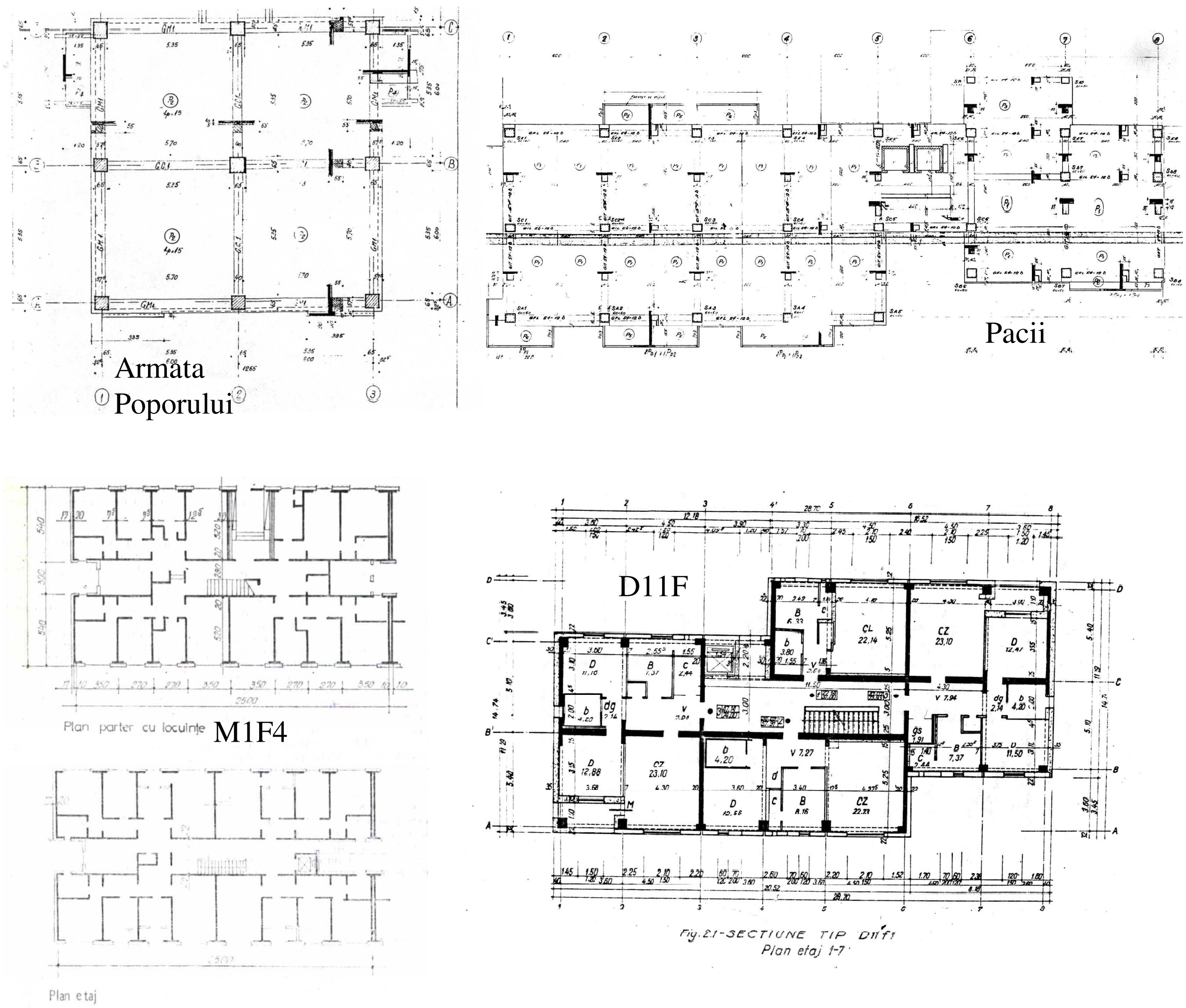


Figure 12. Horizontal layout of the studied buildings

Table 5. Yielding, ultimate and performance points of the buildings analysed

Building name	Yielding acceleration, ‘g	Ultimate acceleration, ‘g	Yielding displacement, cm	Ultimate displacement, cm	Expected spectral displacement, cm
Armata Poporului	0.14	0.20	6	89	24.6
Pacii	0.14	0.24	9	65	22.5
M1F4	0.22	0.30	1.5	7.5	3.6
D11F	0.28	0.34	0.8	6.0	2.0

The highest probability of collapse is get for M1F4 building type. The analysis for this building was performed along the weak direction of the structural system, i.e. in the longitudinal direction, known by the designers to be the weak link for this type of building. In the rest of the cases, the results show lower probabilities of failure. The best performance was noticed for D11F building designed according to a moderate-code (Vacareanu et al. [14]).

The difference between Armata Poporului building and Pacii building comes from the better ductility supplies and better structural regularity in the case of Pacii building.

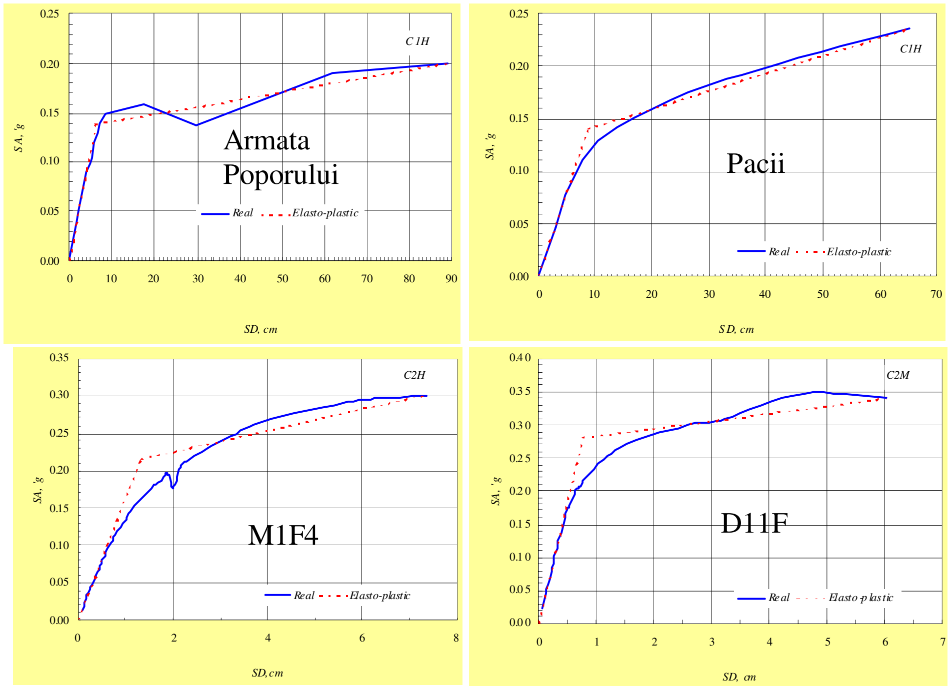


Figure 13. Push-over curves for the studied buildings analysed

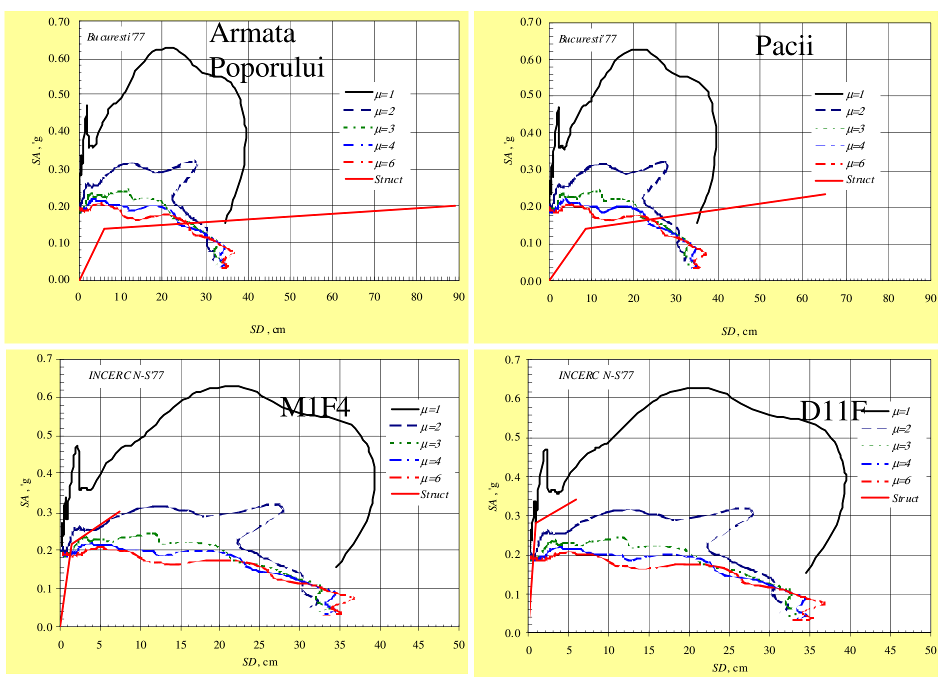


Figure 14. Capacity spectrum method for the studied buildings

CONCLUSIONS

1. HAZUS [1]/ATC-40 [2] methodology is efficient for the evaluation of seismic behaviour and fragility curves.

2. The use of computed inelastic spectra instead of reduced spectra is recommended because it avoids the underrating of spectral displacements.

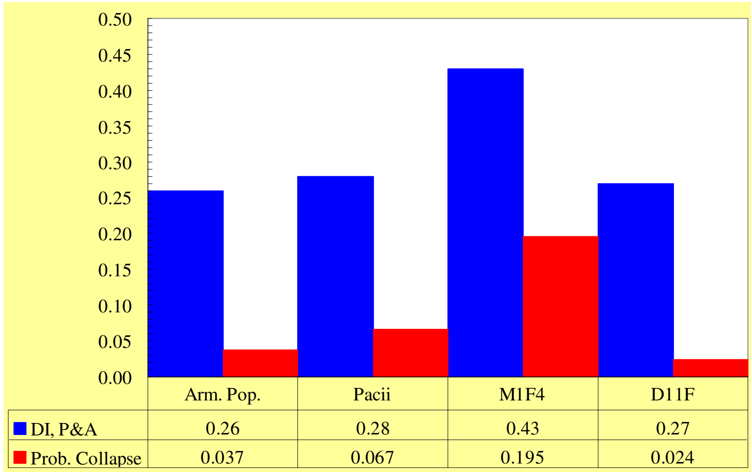


Figure 15. Damage index and probability of collapse for buildings analysed

3. Interstory drift at threshold of damage states can be analytically evaluated for different structural typologies. The values of the interstory drifts can be different with respect to those specified in HAZUS for USA design and construction practice.
4. Monte-Carlo simulation is a powerful tool that can validate and complete the database on seismic behaviour and fragility of buildings.
5. The earthquake damage observed during 1977 and 1986 Vrancea earthquakes showed that the buildings had a better performance in comparison with the analysis results. The methodologies for evaluation of seismic behaviour of existing buildings still requires developments and improvements.
6. The presented methodology enables the ranking of different building types according to the expected probability of collapse and can be regarded as a decision tool for seismic retrofitting of buildings. The probability of collapse must be regarded as nominal probabilities in the sense that they do not represent absolute values, but rather represent relative values enabling comparisons amongst different structural systems.

Acknowledgement
Details of the structures, explanations and advice were kindly offered by Eng. Dragos Badea (S.C. Proiect Bucuresti) to whom we are deeply indebted. The support of JICA Project [19] for the research on seismic risk for Bucharest is also acknowledged.

REFERENCES

1. HAZUS – Technical Manual 1997. Earthquake Loss Estimation Methodology, 3 Vol.
2. ATC 40. Seismic Evaluation and Retrofit of Concrete Buildings, Report ATC-40, Applied Technology Council, Redwood City CA., 1996
3. Bolt B.A., 1995. Earthquakes- revised and updated. W.H. Freeman and Company, New York, 331 p.
4. Coburn, A., Spence, R., (1992), "Earthquake protection", John Wiley & Sons., New York, 355p.
5. World Bank Report No.P-2240-RO, 1978
6. Munich Re, 1998, World Map of Natural Hazards
7. Lungu, D., Aldea, A., Arion, C., 1999. "State, insurance and engineering efforts for reduction of seismic risk in Bucharest", United Nations IDNDR Symposium "The RADIUS initiative - Towards Earthquake Safe Cities", Tihuana, Mexico, 11-14 Oct., 18p
8. "The March 4, 1977 Vrancea earthquake. A Monograph", Romanian Academy, 1981

9. Freeman, S. A., Nicoletti, J. P., and Tyrell, J. V. (1975). Evaluations of existing buildings for seismic risk — A case study of Puget Sound Naval Shipyard, Bremerton, Washington.
Proceedings of 1st U.S. National Conference on Earthquake Engineering, 113-22. Berkeley, Calif.: EERI
10. Mahaney, J.A., Paret, T.F., Kehoe, B.E. and Freeman S.A. (1993). "The capacity spectrum methode for evaluating structural response during the Loma Prieta earthquake", National Earthquake Conference, Memphis.
11. Chopra, A. K. (1995). Dynamics of structures: theory and applications to earthquake engineering. Englewood Cliffs, N.J.: Prentice Hall.
12. Newmark, N. M., and Hall, W. J. (1982). Earthquake Spectra and Design. Berkeley, Calif. Earthquake Engineering Research Inst.
13. Chopra A. K., Goel R. K., 1999 - Capacity-Demand-Diagram Methods For Estimating Seismic Deformation Of Inelastic Structures: SDF Systems, Report No. PEER-1999/02, Pacific Earthquake Engineering Research Center, 72 p.
14. Văcăreanu R., Cornea T., Lungu D. 2001. Evaluation of seismic fragility of buildings, Earthquake Hazard and Countermeasures for Existing Fragile Buildings, Editors: Lungu D. & Saito T., Independent Film, Bucharest Romania, p.163-177.
15. Rubinstein, R. Y. Simulation and the Monte-Carlo method. John Wiley & Sons, New York, 1981
16. Galambos, T. V., Ellingwood, B., MacGregor, J. G., and Cornell, C. A.. Probability based load criteria: Assessment of current design practice. Journal of Structural Division, ASCE, vol. 108, No. ST5, pp. 959-977, 1982
17. Park, Y.-J & Ang, A. H.-S.. Mechanistic seismic damage model for reinforced concrete. Journal of Structural Engineering. ASCE vol. 111 (4): 722-739, 1985
18. Vacareanu, R., Aldea, A., Arion, C., 2003. "Probabilistic risk assessment of current buildings" VIe Colloque National AFPS, Genie Parasismique-Aspects Dynamiques et Vibratoires en Genie Civil, vol III, pag. 153-160
19. JICA Technical Cooperation Project "Reduction of Seismic Risk for Buildings and Structures" with Romania, 2002-2007, Ref.#7241011E0
20. NEMISREF New Methods for Mitigation of Seismic Risk of Existing Foundations (#GRD1-40457) shared-cost RTD project of the GROWTH programme, 5th Framework Programme of the European Commission, 2002-2005. Aldea, A., Lungu, D.,Vacareanu, R., Radoi, R., Negulescu, C., 2003. Work Package 2 Report, "UTCB Real Case", 20p
21. Lungu, D., Arion, C., Aldea, A., Vacareanu, R., 2004. Representation of seismic action in the new Romanian code for design of earthquake resistant buildings P100-2003, Proceedings of 13 WCEE World Conference on Earthquake Engineering, CD-ROM, Vancouver, Canada
22. Cuesta I., Aschheim M., Fajfar P., 2002 "Simplified r-factor relationships for strong ground motion", Earthquake Spectra, Earthquake Engineering Research Institute 19, Issue 1, pp.1-21
23. Valles R. E., Reinhorn A. M., Kunnath S. K., Li C., Madan A. – IDARC2D Version 4.0: A Computer Program for the Inelastic Damage Analysis of Buildings, Technical Report NCEER-96-0010, NCEER, State University of New York at Buffalo, 1996
24. Aldea, A., Lungu, D., Arion, C., 2003. "GIS microzonation of site effects in Bucharest based on existing seismic and geophysical evidence" VI^e Colloque National AFPS, Genie Parasismique-Aspects Dynamiques et Vibratoires en Genie Civil, vol I, p.149-156