Scientific Papers. Series E. Land Reclamation, Earth Observation & Surveying, Environmental Engineering. Vol. IV, 2015
Print ISSN 2285-6064, CD-ROM ISSN 2285-6072, Online ISSN 2393-5138, ISSN-L 2285-6064

EVALUATION OF SEISMIC RESPONSE - FACULTY OF LAND RECLAMATION AND ENVIRONMENTAL ENGINEERING -BUCHAREST

Camelia SLAVE

University of Agronomic Sciences and Veterinary Medicine of Bucharest,
59 Marasti Blvd, District 1, 011464, Bucharest, Romania

Corresponding author email: camelia_slave@yahoo.com

Abstract

Predicting seismic response of structures to future earthquakes contains a large dose of uncertainty. This is primarily due to inability to know exactly the characteristics of future earthquakes, and in the second simplifying assumptions used to calculate the structural response. One of these simplifications is that current design methods used elastic calculation, while the response of several structures under the action of an earthquake is inelastic.

Key words: *calculation method, evaluation, earthquake, seismic response.*

INTRODUCTION

Evaluation of seismic response using static calculation methods (method of lateral forces) in a local dynamic analysis is another major simplification. Uncertainty of determining the seismic response of a structure is amplified by other issues, including the inability to accurately predict the value and distribution especially of gravitational loads, structural elements contribution to stiffness, strength and damping resistance of the main structure.

In these analysis procedures, the maximum earthquake-induced base shear and deformation for an uplifting structure are computed directly from the earthquake response spectrum. It is demonstrated that the simplified analysis procedures provide results for the maximum base shear and deformation to a useful degree of accuracy for practical structural design (Chopra, 2000).

Therefore the conceptual design of structures located in seismic areas is very important, in order to have a proper seismic behaviour.

Basic conceptual issues are related to:
- Simplicity of structure
- Uniformity, symmetry and redundancy
- Strength and lateral stiffness in any direction
- Strength and torsion stiffness
- The realization that the diaphragm floors
- Proper foundations

P100-3/2008 - the new code provides 3 seismic evaluation methodologies for evaluation of construction, defined by the conceptual level of refinement of calculation methods and the level of checking operations detail:

A. Methodology Level 1 is a simplified methodology;

Level 2 methodology is the methodology commonly used type current ordinary construction;

Level 3 methodology using calculation methods applied to nonlinear and complex construction or of particular importance when data require. Level 3 is recommended methodology for current type construction due to higher confidence provided by the method of investigation or the classification in a risk group based on R3 coefficient is not obvious.

The calculus has been making for the body A of the Faculty of Land Reclamation and Environmental Engineering at the University of Agronomic Sciences and Veterinary Medicine of Bucharest.

The building is executed between 1968-1970, and has a resistance structure made of the reinforced concrete frame, designed according to design standards at the time, so that does not meet many of the requirements of the current seismic design codes. Bucharest area according P100-1 / 2006 is characterized by a peak acceleration ag = 0.24 g and a control period (corner) response of the spectrum Tc = 1.6 sec.

The following is a part of an assessment methodology based on Level 2. In this methodology the earthquake effects are

Print ISSN 2285-6064, CD-ROM ISSN 2285-6072, Online ISSN 2393-5138, ISSN-L 2285-6064

approximated by a set of forces applied to conventional construction. Size of lateral forces must be choose as the movements obtained from a linear structure calculation under those forces approximate the structure deformations imposed by seismic loads.

MATERIALS AND METHODS

If the fundamental building period is greater than the corner period Tc of the spectrum is applied the so-called rule of "equal displacement" which states that displacement elastic response represents an upper limit of the nonlinear seismic displacement. Consequently, the lateral forces applied to these situations corresponds to the elastic seismic response of the structure, assessed using the response spectrum by the non-reduce factor q.
The building subject to review has a capacity of over 200 people in total area exposed; it requires its classification in to importance class II, characterized by an importance factor of 1.20.
But when the fundamental building period is less that the corner period the effective inelastic displacement exceeds the elastic response and their assessment needs corrections. Thus, for Vrancea earthquakes recorded in the Romanian Plain whit Tc = 1.6 sec, most existing buildings fall in the range 0 - Tc. Therefore, to assess the ultimate limit state displacements it must to correct the offset values by elastic seismic loads (unabated) amplification coefficient "c" (P100-1/2006, Appendix E).
The level 2 methodology, checking structural elements is made for the ultimate limit state and the service limit state under similar conditions as in P100-1/2006 for new structures design. For the service limit state is imposed to check only lateral displacements, while for ultimate limit state is imposed to check also the structural resistance. In order to determine the displacements and sectional efforts into structural elements of reinforced concrete is has been developed a three-dimensional model of the building resistance structure. For a short presentation, in the case study is presented only the analyzes results that consider seismic effects in transverse direction of the building. The program used is SAP 2000.

The modal analysis revealed the following modes of vibration, according Table 1.

Table 1. Periods of vibration modes on the transverse direction

Mod of vibration	Period (s)	Modal participation factor (λ)	Amount of participation factors $\sum \lambda$
1	0.888	0.775	0.775
2	0.300	0.131	0.907
3	0.164	0.050	0.957
4	0.109	0.026	0.983
5	0.081	0.017	1.000

Unlike the level 1 methodology, when the total mass of the building was evaluated at approximately 3800 t, the level 2 methodology provided a value of 3620 t for the total mass of the building. Consequently, the resulting base shear elastic seismic response found is:

$$F_b = \gamma_1 S_d(T_1) m\lambda = \gamma_1 \frac{a_g \beta(T_1)}{gq} \lambda(mg) = 1{,}2 \frac{0{,}24g}{g} \frac{2{,}75}{2{,}5} 0{,}85G \quad (1)$$

$$F_b = 0{,}67G = 0{,}67 \times 3620 \times 9{,}81 \Rightarrow F_b = 23790 \quad (2)$$

The lateral force was distributed vertically according to the fundamental shape of vibration mode on transverse direction.
Checking relative level displacements According to P100-1/2006 code, relative displacements associated to the service limit state are obtained by multiplying the corresponding elastic response with a reduction factor taking into account the seismic recurrence interval associated to the verification in the service limit state. For buildings classified in Class II, this factor has the value of v = 0.4. Similarly, the ultimate limit state elastic displacements are amplified by a gain factor which takes into account the building fundamental period of vibration with site corner period lower and the inelastic displacements are higher than those corresponding to the elastic seismic response (P 100-3/2008).

This coefficient is equal to:

$$l \leq c = 3 - 2,5\frac{T}{T_c} \leq 2 \Rightarrow c = 3 - 2,5\frac{0,88}{1,60} = 1,625 \quad (3)$$

The level relative movements are, present in Table 2

The acceptable values of relative level displacements are of 0.5% to 2.5% for limit state service (LSS),, equation (4) and ultimate state service (ULS), equation (5) and follows:

$$R_3^{d,SLS} = \frac{d_{r,adm}^{SLS}}{d_{r,max}^{SLS}} = \frac{0,5}{1,15} \Rightarrow R_3^{d,SLS} = 0,43 \quad (4)$$

$$R_3^{d,SLU} = \frac{d_{r,adm}^{SLU}}{d_{r,max}^{SLU}} = \frac{2,5}{4,69} \Rightarrow R_3^{d,SLS} = 0,53 \quad (5)$$

Table 2. Level relative movements associated limit state service (LSS) and ultimate state service (ULS)

Level	Elastic dispalcemnts c (m)	Level hight (m)	Drift as elastic displacements (%)	Drift as LSS (%)	Drift as ULS (%)
4 Floor	0.435	3.80	1.68	0.67	2.73
3Floor	0.371	3.80	2.60	1.04	4.23
2Floor	0.272	3.80	2.89	1.15	4.69
1floor	0.162	3.80	2.71	1.09	4.41
Ground floor	0.059	3.73	1.59	0.63	2.58

RESULTS AND DISCUSSIONS

Verification of reinforced concrete structural elements. Carrying out resistance for Ultimate Limit State depends on the ductile or brittle failure of the structural element s under the considered effort. Failure modes of reinforced concrete elements are defined in P100-3/2008 - Appendix B (P 100-3/2008).

Sectional efforts computed for elements of inelastic behaviour are assessed under the new code seismic evaluation based on the principle relationship: Ed = (E * $_E$ / q) + Eg, where E * $_E$ is the effort of seismic load considering the elastic response spectrum (non - reduce), Eg is the effort resulted from associated non-seismic loads combination including seismic load, and q is the behaviour factor function the element analyzed, yielding the of the type of effort.

For ductile failure elements capacity is determined by dividing it by the partial average resistance safety coefficients and confidence factor CF = 1.20 level of knowledge associated with "normal" KL2.

For fragile failure the verification represents a comparison of efforts resulted under lateral and gravitational forces associated with the plastic state of ductile elements of the structure, with the value calculated with the minimum load capacity of materials resistance (typical values divided by CF and partial safety factors).

According to P100-3/2008 - Appendix B - factor values for reinforced concrete beams of such behaviour are depending on the behaviour (ductile or inductile), the reinforcement ratio at the top and bottom of beam and shear strength of calculation (P 100-3/2008).

Because the critical areas at the beams edges:

(1) the upper edge has not at least two bars shaped surface 14 mm diameter;

(2) there is at least one quarter of the maximum reinforcement from the top provided continuously along the beam length;

(3) the compressed area is provided with at least half of the large reinforcement section

(4) distance between stirrups in critical areas violates the condition of s≤ min {hw / 4, 150 mm, 7 d_{bl}} (where hw is the height of the beam cross section and d_{bL} is the minimum diameter of longitudinal bars), it was considered that the composition and the reinforcement of existing beams partially fulfils the conditions of new design standards. Consequently, the behaviour factor values were obtained by interpolation of the corresponding q values respectively non ductile behaviour.

The following is an example of how to conduct inspections of resistance to a floor beam over transverse current frame. In the spread sheet calculation presented below we used the following values and formulas, this are presents in Table 3 (Slave, 2010).

Values of the structural seismic assurance degree of the transverse beam show that the beam is less reinforced that the earthquake design claims. The minimum value of the indicator is recorded in the central opening in the traffic corridor between the axes B and C, where reinforcement of the bottom edge of the beam is about 10 times lower than that associated computing moment When the seismic load is oriented on transverse direction in the positive direction of axis OY, we have, Table 6 (Slave, 2010).

Table 3 – Values and formulas

f_{cd}	=13.9 MPa (10.5 MPa) – concrete C12/15 class - compression resistance under ductil failure/ fragile failure
Fctd	=1.1 MPa (0.76 MPa) – concerte C12/15 class -tensile resistance under ductil g=failure/ fragile failure;
fyd	=236 MPa (175 MPa) yield strength steel OL38 brand for ductile type failure (or weak);
p_{max}	=ζB(f_{yd} / f_{cd}) maximum reinforcement ratio (corresponding balance point);
p, p', p_e	- reinforcement ratios (tensile efforts in bars, compressive efforts in bars, stirrups;
V_{Ed}	design shear force;
M_E* ,V_E*	bending moment, shear force that generated the seismic action considering the elastic response spectrum;
M_g ,V_g	bending moment, sheer force of the actions that non seismic associated load combinations including seismic action;
M_{Ed}	- (M_E* /q) + M_g - bending moment calculation of the inelastic behavior associated with that section of the beam
M_{Rd}	- $A_{s1}f_{yd}$(d-a) - Bending moment capacity in this section
$\boldsymbol{R_3^M}$	- M_{Rd}/ M_{Ed} - degree of assurance in structural seismic bending moment;
$\boldsymbol{q_M^{plastif.}}$	- behavior factor associated to the plastic state of beam cross section;
$\boldsymbol{V_{Ed}^{plastif.}}$	- calculation shear force associated to the plastic state of beam cross section under bending moment
$S_{i,cr}$	- normalized horizontal projection of inclined crack critical, according STAS10107-0/90
V_{eb}	shear force capable no dimension according STAS10107-0/90
$\boldsymbol{R_3^V}$	- V_{Rd}/ V_{Ed} – degree of assurance in structural seismic shear

Table 4 Geometry and reinforcement transverse beam over the 1st floor

Level	Ax	b (mm)	H (mm)	A_a^{jos} (mm^2)	A_a^{sus} (mm^2)	n_e	A_{ae} (mm^2)	a_e (mm)	P (%)	p' (%)	p_e (%)
1 st floor	A	250	650	1119 (2ϕ20 + 1ϕ25)	245 (5ϕ25)	2	50.3	200	0.73	1.60	0.20
1 st floor	B_{dr}	250	500	402 (2ϕ16)	1473 (3ϕ25)	2	50.3	200	0.35	1.27	0.20
1 st floor	C_{dr}	250	650	982 (2ϕ25)	1473 (3ϕ25)	2	50.3	200	0.64	0.96	0.20
1 st floor	B_{stg}	250	650	628 (2ϕ20)	1473 (3ϕ25)	2	50.3	200	0.41	0.96	0.20
1 st floor	C_{stg}	250	500	402 (2ϕ16)	1473 (2ϕ25)	2	50.3	200	0.35	1.27	0.20
1 st floor	D	250	650	1473 (3ϕ25)	2454 (5ϕ25)	2	50.3	200	0.96	1.92	0.20

Print ISSN 2285-6064, CD-ROM ISSN 2285-6072, Online ISSN 2393-5138, ISSN-L 2285-6064

Table 5. Sectional efforts – transverse beam over the 1st floor

Level	Ax	M^*_E (kNm)	V^*_E (kN)	M_g (kNm)	V_g (kN)
1 st floor	A	2891.9	-100.5	882.3	-88.7
1 st floor	B_{dr}	1786.3	-27.2	1388.9	-17.5
1 st floor	C_{dr}	2116.5	-100.0	678.4	-93.7
1 st floor	B_{stg}	-2335.6	-65.5	882.3	74.4
1 st floor	C_{stg}	-1824.8	-31.9	1388.9	21.0
1 st floor	D	-2581	678	-132	104

Table 6. Structural seismic assurance degree of transverse beam over the 1st floor under the moment bending - Earthquake in the transversal direction (+OY)

Level	Ax	$\frac{p-p'}{p_{max}}$	$\frac{V_{Ed}}{bdf_{ctd}}$	Degree of compliance to provide the seismic structure	q	M_{Ed}	M_{Rd}	R_3^M
1 st floor	A	0.295	0.246	60 %	5.80	398.1	184.0	0.46
1 st floor	B_{dr}	0.313	1.458	70 %	3.55	475.9	49.0	0.10
1 st floor	C_{dr}	0.109	0.040	60%	5.80	264.9	161.5	0.61
1 st floor	B_{stg}	0.187	1.201	60%	3.40	-752.4	-242.2	0.32
1 st floor	C_{stg}	0.313	1.756	70%	3.55	-546.0	-179.6	0.33
1 st floor	D	0.217	1.219	60%	3.40	-898.9	-403.5	0.45
Average 0.38								

Table 7 Structural seismic assurance degree of beam above a floor under moment bending - Earthquake in the transversal direction (+OY)

Level	Ax	$\frac{p-p'}{p_{max}}$	$\frac{V_{Ed}}{bdf_{ctd}}$	Degree of compliance to provide the seismic structure	q	M_{Ed}	M_{Rd}	R_3^M
1 st floor	A	0.295	1.284	50 %	2.95	-1079.2	-503.5	0.37
1 st floor	B_{right}	0.313	1.728	50 %	2.94	-635.5	179.6	0.28
1 st floor	C_{right}	0.109	1.137	50%	3.14	-773.8	-242.2	0.31
1 st floor	B_{left}	0.187	0.329	50%	4.50	453.3	103.3	0.23
1 st floor	C_{left}	0.313	1.430	50%	2.94	589.5	49	0.08
1 st floor	D	0.217	0.043	50%	4.38	449.4	242.2	0.54
Average 0.30								

Table 8. Structural seismic assurance degree of the 1st floor beam under shear force - Earthquake in the transversal direction (-OY)

Level	Ax	$q_M^{plastif}$	$V_{Ed}^{plastif}$	$\overline{S_{i,cr}}$	$\overline{V_{eb}}$	V_{Rd} (kN)	R_3^V	$R_3^M \le R_3^V$
1 st floor	A	9.5	-181.1	1.69	1.49	175.9	0.97	Yes
1 st floor	B_{right}	11.7	-135.9	1.66	1.36	121.2	0.89	Yes
1 st floor	C_{righ}	14.9	-139.3	1.54	1.27	149.5	1.07	Yes
1 st floor	B_{left}	13.8	10.7	1.54	1.27	149.5	4.50	Yes
1 st floor	C_{left}	22.5	-40.6	1.66	1.36	121.2	2.94	Yes
1 st floor	D	6.8	7.4	1.75	1.44	169.8	4.38	Yes
							Average	2.46

Summarizing the average grade of all structural beams transverse the current frame, we have, table:

Table 9. Structural seismic assurance degree of the beams transversal the current frame

Level	Earthquake on the (+ OY)		Earthquake on the (-OY)	
	R_3^M	R_3^V	R_3^M	R_3^V
4 Floor	1.41	2.32	1.54	2.45
3Floor	0.70	2.48	0.49	2.52
2Floor	0.38	2.70	0.30	2.46
1floor	0.38	2.75	0.30	2.46
Ground floor	0.44	2.76	0.33	2.42

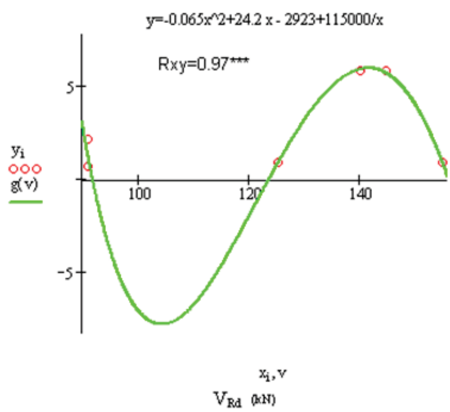
CONCLUSIONS

A Excepting beams from the higher level, the transverse frame beams are substantially under reinforced to bending moment, highlighting the a more pronounced sensitivity to stress when the seismic action is oriented in the negative sense of the axis OY.

B. Seismic evaluation shows also a positive aspect: consistently the values of the seismic structural assurance degree under shear force are superior to those associated with bending moment, suggesting that the fragile failure is inhibited by the flowing of longitudinal reinforcement.

C. The relationship between the degree of structural seismic insurance over 1 floor beam shear - Earthquake in the transverse direction (+ OY)) according to the table 8 is a polynomial regression function, degree III ratio R_{XY} = 0.97 the correlation is very significant.

The graph obtained is done using successive trials with PROFESSIONAL MATHCAD Software.



REFERENCES

Chopra A. – Dynamics of structures – Theory and applications to earthquake engineering, Pretince Hall, 2000

Cod de proiectare seismica– partea A III-A –Prevederi pentru evaluarea seismica a cladirilor existente indicativ P 100-3/2008

Slave C. Seismic risk assessment of existing structures, PhD Thesis, UTCB, September 2010