BULETINUL INSTITUTULUI POLITEHNIC DIN IAŞI
Publicat de
Universitatea Tehnică „Gheorghe Asachi" din Iaşi
Tomul LIV (LVIII), Fasc. 2, 2008
Secţia
CONSTRUCŢII. ARHITECTURĂ

# SIMPLIFIED ASSESSMENT OF *R3* NOMINAL ASSURANCE DEGREE TO SEISMIC ACTION OF THE EXISTING MASONRY DWELLINGS

BY

**MAGDA BROŞTEANU[1,*] and TEODOR BROŞTEANU[2]**

**Abstract.** This paper refers to the assessment of the performance level of a building for a given seismic hazard level. Building performance level describes the expected seismic performance given by the computation of *R3* Nominal Assurance Degree to Seismic Action of the Existing Masonry Dwellings and Monumental Buildings according to the Romanian Norm P100:1992 [1], modified on 1996 with the chapters **11** and **12**, until the Part 3 of P100-1:2006 [2], will be performed for the Assessment and Strengthening Structural Design of the Seismic Vulnerable, Existing Buildings, in the frame of SR EN 1998-1:2004 EC8 [3].

The framing of damages into the potential risk degrees has a social and economic impact. Assessment and retrofitting of the existing buildings have represented a huge engineering challenge as a distinct problem *versus* a new building design. The performance level of a vulnerable existing building shows us the expected seismic performance level due to the classified damages, the pattern of cracks, the interruption of function, the economic losses and the needed interventions, all in function of the importance class of building on next life span of use.

On recommends the computation of *R* (R3) Nominal Assurance Degree to Seismic Action of the Vulnerable Dwellings for the assessing and strengthening design, in comparison to both norms because of *the bearing conventional seismic load* computed by [1], will result less than the value which will be computed by the Part 3 of P100-1:2006, *i.e.* the norm P100:1992 is more severe. In the case of the breakable fracture probability of the existing structural masonry members, one recommends a bigger value of $\Psi$ – reduction factor unless the given values by [1] for a new structure with a high ductility, especially for the deflections calibration on the same limit state.

**Key words:** nominal assurance, seismic action, masonry dwellings.

## 1. Background

There are dwellings and monumental buildings made by structural masonry and placed on seismic hazard zones. The existing buildings with classified damages need an upgrading of cross-walls structures in order to reduce the seismic risk for the users' safe on next life span of use.

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* Corresponding author. *E-mail address*: brostean@ce.tuiasi.ro (M. Brosteanu)

## 2. Diagnostic Surveys

Final aim of diagnostic surveys of the overall pattern of cracks is the framing of the existing buildings with classified damages into 4 potential classes of seismic risk [1].

a) RS I; there is a high risk of collapse;

b) RS II; there is a low probability of collapse but there are major damages of bearing elements;

c) RS III; the damages of bearing elements do not disturb the structural safety but there are major damages of non-bearing elements;

d) RS IV; the expected seismic answer of the existing building and the designed seismic answer for a new building are the same.

Note that should be kept in mind the influence factors of quantifying: the seismic zone, the structural pattern on height, the seismic conformity of structure, the nominal assurance degree to a-seismic action, the probable nature of collapse of bearing elements (either a ductile nature, a semi-ductile one, or a breakable one), the reinforcement details, the building life span, and the building inventory.

*Geometrical surveys* can be performed either by using topographic methods and photogram-metric techniques, old documents, or buildings technical books. Cracks surveys are important in assessing the static conditions of masonry structures and identifying the possible causes of the instability. In particular, the boreholes can be inspected by climbers using rotating micro TV camera. *Sonic and/or radar surveys* will provide information of the elastic and mechanic characteristics of masonry about. *Non-destructive surveys* using the flat jack technique make possible to assess the static pattern of masonry structures over large sample areas. The state of stress in the masonry, the deformability characteristics, masonry compressive strength, the shear strength of mortar layers are the potential results of flat jack technique, as input data in the assessment. *Destructive/non-destructive tests* by laboratory activities and statistic-probabilistic analysis are important to evaluate the effective values of materials strengths and module of elasticity.

*Simplified Assessment of Nominal Assurance Degree to Seismic Action of Simple Masonry Structures* will be performed by using of software (*CSZ-Exe*, *CASIN*, *CAZANS* [4], *Turbo-CASE*). *Simplified Assessment of Nominal Assurance Degree to Seismic Action of Reinforced Masonry Structures* with concrete columns and girdles and/or reinforced jacketing will be performed by using the Norms for Reinforced Masonry Design [5], [6]. Note that the existing vulnerable structures may be only formally modelled because of the computation defects will eliminate the theoretic assumptions.

### 3. Engineering Approach

The a-seismic protection level of the existing building will be established by the quantitative assessment concern of checking if or if not a damaged building has satisfied the performances of limit state and the design seismic associated load, at the same time with. The aims of either *the assessment method to static equivalent lateral load, at the new 1st or 2nd levels or the old E2 procedure* prescribed by this norm are:

a) To compute *the nominal assurance degree to seismic action R (R3)* of the existing building,

b) To compute *the bearing conventional seismic load*, $S_{\mathrm{cap}}$, (*the total load carrying effect for the shear force*) of the existing building,

c) To identify the weak zones,

d) To check-up the criteria of ductility to avoid a breakable fracture,

e) To compute the rigidity of structure to lateral displacements,

f) To compute the *conventional seismic load*, $S_{\mathrm{nec}}$, (*the static equivalent seismic force as the base shear force*, $F_b$) of the existing building as a new one,

g) To frame the building into the classes of seismic risk,

h) To make the final decision of retrofitting.

Nominal assurance degree to seismic action, *R (R3)*, of the existing building depends on a great number of factors such as: rules, placement, layout, structure, details, materials, residual strengths, technology, quality costs etc., all very difficult to be controlled. One recommends the computation in comparison of *R (R3)* Nominal Assurance Degree to Seismic Action of the Existing Vulnerable Dwellings for the assessing and strengthening design by both norms because of the bearing conventional seismic load computed by [1] will result less than the value which will be computed by the Part 3 of P100-1:2006, *i.e.* the norm P100:1992 is more severe.

Pre-design of load-carrying capacity of the entire building by the 1st (2nd) level method consists of the simplified computation of the load-carrying capacity for the shear force and comparing with the static equivalent seismic force at the basement level. Nominal Assurance Degree to Seismic Action of the Existing Buildings made by un-reinforced structural masonry will be computed using the alternative following eqs.

By [1] By [2]

$$R = \frac{S_{cap}}{qS_{nec}} \geq R_{\min}; \qquad R3 = \frac{S_{cap}}{q\,F_b} \geq R3_{\min}, \tag{1}$$

where: $R_{min}(R3_{min}) = 0.70(1.00),...,0.50(0.35)$ is the minimum values of the Nominal Assurance Degree to Seismic Action function of the important class of building (or four classified potential classes of seismic risk); $S_{cap}$ is the load carrying total effect for shear force (it will be computed on the direction in which the masonry area, $A_{z,min}$, is a minimum one); $q$ is the increasing factor for a structure with a small ductility (or the correction factor of the behaviour);

$$(2) \qquad S_{cap} = m\ D\sum_{1}^{n} T_{cap,\min,J}\ ; \qquad S_{cap} = A_{z,\min}\tau_k\sqrt{1+\frac{2}{3}\frac{\sigma_0}{\tau_k}}\ ,$$

where: $m$ is the serviceability factor; $D$ - the damaging coefficient; $\sum_{1}^{n} T_{cap,min,J}$ - the minimum bearing shear force for all $J$ cross-walls among the following three values: $T_{C,M,J}$ - the capable shear force for the eccentric compression in the "J" cross-wall plane; $T_{C,F,J}$ - the capable shear force for the slippage of the horizontal mortar joint; $T_{C,P,J}$ - the capable shear force for the principal stretching stresses; $\tau_k$ - the reference value of masonry shear strength; $\sigma_0$ - the compressive unit stress computed with the damaged masonry strengths;

$$(3) \qquad S_{nec} = \eta\, S_r = \eta\,\alpha\, k_S \beta_r\, \varepsilon_r \psi G \qquad F_b = \gamma_I S_d(T_1) m\lambda$$

where: $\eta$ - the loading factor of torsion effect; $S_r$ - the horizontal earthquake resultant load for $r$ self shape of vibration; $\alpha$ - the occupancy importance factor; $k_S$ - the seismic zone coefficient; $\beta_r$ - the dynamic amplification factor; $\varepsilon_r$ - the equivalent factor; $\psi$ - the reduction factor for the energy dissipation capacity of structure; $G$ - the total gravity load; $\gamma_I$ - the importance factor of building; $S_d(T_I)$ - the ordinate of design response spectrum for the fundamental mode of vibration; $T_I$ - the proper fundamental period of vibration of building in the plane which contains the considering horizontal direction; $m$ - the total mass of building; $\lambda$ - the correction factor for of the associated modal mass around.

The capable shear force associated to breakdown for the eccentric compression in the each $J$ cross-wall plane, at each $K$ level will be computed for un-reinforced masonry:

$$(4) \qquad T_{c,M} = \frac{M_c}{Z_k} = \frac{N\,e_0}{Z_k} = \frac{1.25\,R_z\,S_c}{Z_k}\ ; \qquad V_{f1} = \frac{N_d}{c_p\lambda_p}v_d\left(1\text{-}1.15v_d\right),$$

where: $M_c$ - the maximum design capable bending moment of *J* cross-wall at the *K* level; $Z_k$ - the height from the *K* level to the horizontal resultant load application level; $\boldsymbol{e_0}$ - the eccentricity, *i.e.* the distance from the centroid of compressed area to the compressive load point of application; *N* - the design compressive load; $S_c$ - the static moment of masonry compressed area face to the centroid of the cross-section; $R_z$ - the design compressive strength of masonry; $N_d$ - the design axial compressive force; $c_p$ - the factor for the edge props; $b=\dfrac{H}{l_w}$ - the shape factor; *H* - the wall height; $l_w$ - the wall length; $v_d=\dfrac{\sigma_0}{f_d}$ - the proportion factor; $\sigma_0=\dfrac{N_d}{t\,l_w}$ - the average value of the compressive unit stress; *t* - the wall thickness; $f_d$ - the design masonry compressive strength.

Capable shear force for slippage of the horizontal joint of mortar for each *J* cross-wall plane, at each *K* level will be computed for un-reinforced masonry.

$$T_{c,F}=\frac{A_i}{\mu_i}\left(R_f+0.7f\sigma_0\right);\qquad V_{f2.1}=f_{vd}\,l_{cw}t\,, \tag{5}$$

where: $A_i$ is the cross section area; $R_f$ - the design shear strength of masonry; $f$ - the friction coefficient; $\sigma_0$ - the normal unit stress; $\mu_i$ - the correction factor for the tangent unit stress distribution; $f_{vd}$ - the design shear strength for slippage of horizontal joint; $l_{cw}$ - the wall compressed length.

Capable shear force for the principal stretching stresses for each *J* cross-wall plane at each *K* level will be computed for un-reinforced masonry.

$$T_{C,P}=\frac{R_pA_i}{\mu_i}\sqrt{1+0.8\phi\frac{\sigma_0}{R_p}}\,;\qquad V_{f2.2}=\frac{tl_wf_{td}}{b}\sqrt{1+\frac{\sigma_0}{f_{td}}}\,, \tag{6}$$

where: $R_p$ is the design principal stretching strength of masonry obtained by the multiplication of the tangent tension strength $R_t$ with the serviceability factor *m*; $\phi$ - the correction factor; $f_{td}$ - the design shear strength for principal stretching.

The stages of structural assessing are on site inspection and at office calculation and making decision of *R (R3)* Nominal Assurance Degree to Seismic Action of the Existing Vulnerable Dwelling about:

a) Identification of masonry type: is there or is not un-reinforced masonry, confined masonry, reinforced masonry, or masonry in-filled?

b) Identification of conceptual design of shear walls structure: is there or is not a structural simplicity, uniformity, symmetry and redundancy, a bi-directional resistance and stiffness, a torsion resistance and stiffness?

c) Identification of the quality of system plane and elevation: is there or is not the regularity in plane and in elevation, a rigid or a flexible floor system, a connection in two orthogonal horizontal directions and in the vertical one between walls and floors provided by reinforced concrete ring beams or by steel ties?

d) Identification of noticed cracks by surveys.

e) Identification of masonry strengths by non-destructive tests after de-covering.

f) Dividing the structure in $J$ members on each direction and computation of their features.

g) Computation of vertical and lateral loading and capable loading effects due to on each member.

h) Computation of bearing conventional seismic load of the vulnerable building.

i) Computation of conventional seismic load as fundamental seismic shear force for the existing building as a new one, using a bigger value of $\psi$ reduction factor unless given value by [1] for a new structure with a high ductility, in the case of breakable fracture probability of members.

Note that the Romanian Norm P100:1992 modified in 1996 indicates the low values of $\psi$ reduction factor for a new structure with a high ductility, especially for the deflection calibration on the same limit state: $\psi$ =0.30, for the structures made by un-reinforced masonry; $\psi$ =0.25, for the structures made by confined masonry with concrete columns and girdles.

## 4. Case Study

Case-study refers to the assessing of three-storey dwelling made by structural un-reinforced masonry and placed on seismic zone $a_g = 0.08\,g$. Lateral-load resistant system of two spans times three bays is symmetrically arranged along two main horizontal axes. Dwelling has 120 m$^2$ built area in layout and 3 × 2.80 m height in elevation. Dynamic response of structure shall be analyzed in terms of the inertia forces. Design seismic force will be computed on both directions by both codes.

By [1]: On transverse direction $R = \dfrac{(0.55)(0.50)622}{(1.50)289} = 0.39 < 0.50$ for the 3$^{\text{rd}}$ class of building importance. On longitudinal direction $R = \dfrac{(0.55)(0.50)417}{(1.50)289} = 0.26 < 0.50$ for the 3$^{\text{rd}}$ class of building importance; where: $m$ = 0.55 - serviceability factor; $D$ = 0.50 - damaging coefficient; $q$ =1.50 - the increasing factor for a structure with a small ductility; $\sum_{1}^{n} T_{cap,\min,J}$ = 622.60 kN - capable shear force associated to capable bending moment; $S_r$ = 289.08 kN - the horizontal earthquake resultant load for $r$ self shape of vibration; $\eta$ = 1 - the loading factor of torsion effect; $\alpha$ = 1.0 - the occupancy importance factor; $k_S$ = 0.16 - the seismic zone coefficient; $\beta_r$ =2.5 - the dynamic amplification factor for $T_r < T_c$; $\psi$ =0.30 the reduction factor for the energy dissipation capacity of simple masonry structure. $G$ =7300 kN - total gravity load; $\varepsilon = \dfrac{\left(\sum G_k h_k\right)^2}{G\left(\sum G_k h_k^2\right)} = 0.33$ - the equivalent factor; $T_r = 0.045n = (0.045)3 = 0.13 < T_c =$ 0.7 - the proper period of vibration versus corner period of vibration.

By [2]: On transverse direction $R3 = \dfrac{622}{(1.5)806} = 0.46 < 0.65$ for the 2nd class of seismic risk. On longitudinal direction $R3 = \dfrac{417}{(1.5)806} = 0.31 < 0.35$ for the 1st class of seismic risk; where: $\gamma_I$ =1.0 - the importance factor of building; $T_1$ =0.13 s - the proper fundamental period of vibration of the building in the plane which contains the considering horizontal direction; $T_B = 0.07$ s and $T_C = 0.7$ s - the control periods for MIR = 100 years medium interval of recovery of the earthquake magnitude; $\beta_0$ =2.75 - the dynamic amplification factor; $m = \dfrac{G}{g} = \dfrac{7300}{g} \times 10^3$ kg - the total mass of the building; $S_d(T_1) = \dfrac{S_e(T_1)}{q} = a_g \dfrac{\beta(T_1)}{q} = a_g \dfrac{\beta_0}{q} = 0.08g \dfrac{2.75}{1.65} = 0.13g$ - the ordinate of the design response spectrum for the fundamental mode of vibration; $S_e(T_1)$ - the elastic response spectrum; $\beta(T_1) = \beta_0$ for $T_B < T_1 < T_C$ - the normalized elastic response spectrum; $q = q_n \dfrac{\alpha_u}{\alpha_I} = (1.5)1.1 = 1.65$ - the correction factor of the behaviour; $\dfrac{\alpha_u}{\alpha_I} = 1.1$ - the super-resistance factor; $\lambda$

=0.85 for $n > 2$ stories - the correction factor for $T_1$ of the associated modal mass around.

## 4. Conclusions

The existing vulnerable structures may be only formally modelled because of the computation defects will eliminate the theoretic assumptions.

The method is useful for the assessing and redesign of damaged dwellings. One only recommends the computation of *R3* Nominal Assurance Degree to Seismic Action of the Existing Masonry Buildings for the assessing and strengthening pre-design by the seismic structural safety criteria until the Part 3 of P100-1:2006 [2] will be performed.

The Romanian Norm P100:1992 modified in 1996 does not present the influence of the reduction factor, $\psi$, for the masonry existing buildings in order to avoid the over-estimating of strength reserves of damaged structures. One recommends $\psi$ = 0.3…0.5 for retrofitted masonry buildings and $\psi$ = 0.5…0.75, for masonry existing buildings without strengthening.

For the considered case-study, the Nominal Assurance Degree to Seismic Action of the Vulnerable Structural Masonry Dwelling computed by [1] has resulted with 16% less than the value computed by the Part 3 of P100-1:2006, i.e. the norm P100:1992 is more severe. Structural safety criterion by LSDM for the ultimate limit state of strength is not satisfied on both directions of the structure. On recommends the strengthening design for the building retrofitting.

*Received, October 28th, 2008*

*Gheorghe Asachi" Technical University, Jassy,*
*[1]Department of Civil and Industrial Engineering,*
*and*
*[2]Department of Transportation Infrastructure and Foundations*

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## EVALUAREA SIMPLIFICATĂ A GRADULUI NORMAT DE ASIGURARE SEISMICĂ R3 A CLĂDIRILOR DIN ZIDĂRIE EXISTENTE

(Rezumat)

Se evaluează nivelul de performanţă al unei clădiri pentru un nivel dat de hazard seismic. Nivelul de performanţă a clădirii descrie performanţa seismică aşteptată prin gradul normat, *R3* de asigurare seismică a clădirilor existente din zidărie, aplicabil şi la calculul clădirilor monumentale, în conformitate cu P100:1992: Codul de proiectare seismică [1] modificat în 1996 cu capitolele 11 şi 12, până când partea a 3-a a P100-1:2006 [2] va fi aplicabilă, în cadrul general oferit de SR EN 1998-1:2004 EC8 [3].

Încadrarea degradărilor în grade potenţiale de risc are un impact social şi economic. Evaluarea şi proiectarea consolidării clădirilor existente reprezintă o provocare de inginerie civilă ca o problemă distinctă *versus* proiectarea unei clădiri noi. Nivelul de performanţă a unei clădiri vulnerabile este cel aşteptat de performanţă dat de degradările clasificate, schema de distribuţie a fisurilor, întreruperea funcţiunilor, pierderile economice şi intervenţiile necesare, toate în funcţie de clasa de importanţă a clădirii pentru perioada următoare de folosinţă.

Se recomandă calculul gradului normat de asigurare seismică a clădirilor vulnerabile după ambele norme de evaluare şi predimensionare după criteriul siguranţei structurale la acţiuni seismice deoarece capacitatea de rezistenţă a pereţilor structurali pentru forţa laterală în plan [1] va rezulta mai mică decât cea calculată după partea a 3-a a P100-1:2006, normativul P100:1992 fiind mai sever.

În cazul probabilităţii de distrugere prin rupere a elementelor componente ale unei structuri existente din zidărie, se recomandă o valoare mai mare a factorului de reducţie, $\Psi$, decât valorile date de [1] pentru o structură nouă cu ductilitate mare, în special pentru calibrarea deplasărilor la aceeaşi stare limită.