

BULETINUL INSTITUTULUI POLITEHNIC DIN IAȘI  
Publicat de  
Universitatea Tehnică „Gheorghe Asachi” din Iași  
Volumul 65 (69), Numărul 3, 2019  
Secția  
CONSTRUCȚII. ARHITECTURĂ

## STRUCTURAL PARTICULARITIES FOR AN IRREGULAR BUILDING, IN PLAN AND ELEVATION, SITUATED IN A SEISMIC AREA

BY

VLĂDUȚ-IONEL IFTODE\*, IOANA OLTEANU-DONȚOV  
and PETRU MIHAI

Technical University “Gh. Asachi” of Iași,  
Faculty of Civil Engineering and Building Services

Received: August 5, 2019

Accepted for publication: September 10, 2019

**Abstract.** One of the requirements when designing a structure in a seismic area is to choose a regular structural system, uniformly distributed in plan and elevation, in order to ensure a good seismic behaviour. The uniform distribution on elevation leads to the reduction of possible effort concentration and ductility requirements in isolated areas of the building. In practice, there are many cases where the additional stiffness from the masonry panels led to the severe damage in the columns causing local mechanisms, respectively, until global collapse. For the case study, a mixed structural system of frames and structural walls, with irregularities in plan and in elevation, was considered. For the infill panels two materials were considered: masonry with vertical voids and autoclaved concrete. The analysis studies the stiffness increase for the considered situations and the cracks development in the linear elements of the frame. Based on the obtained results, it can be stated that the most effective solution for the infill panels of the studied structure is the autoclaved concrete.

**Keywords:** seismic joint; masonry infill; stiffness; seismic behaviour; plastic hinge.

---

\*Corresponding author; *e-mail*: [iftodevlad@yahoo.com](mailto:iftodevlad@yahoo.com)

## 1. Introduction

The use of reinforced concrete frame structures increased due to population growth in both developed and industrialized countries. The World Encyclopaedia of Construction Database contains over 110 reports describing the design and execution of reinforced concrete structures in over 37 countries around the world.

Although recommended in areas with high seismicity, earthquakes revealed major deficiencies for reinforced concrete frame structures. Among the causes that led to significant damage and even collapse of structures in reinforced concrete frames are: superficial design, poor execution and lack of constant checks.

A conceptual design of structures located in seismic areas in order to ensure proper seismic behaviour is very important. Some basic aspects refer to the simplicity and uniformity of the structure, symmetry, redundancy, strength and lateral stiffness in any direction and torsion stiffness, and choosing an adequate foundation system. Structural simplicity implies the existence of a continuous and sufficiently strong structural system that ensures a direct and uninterrupted path for the seismic forces to the foundation soil. No discontinuities should exist in the seismic forces transfer (example: a large gap in the floor or lack of reinforcement to take over the inertial forces). Another example of not recommended structural compliance refers to discontinuities on the column height.

The seismic design objective is to have regular structure, distributed as evenly as possible in plane. If an irregular shape is chosen, it is recommended to divide the structure by seismic joints into simple subassembly, as shown in Fig. 1.

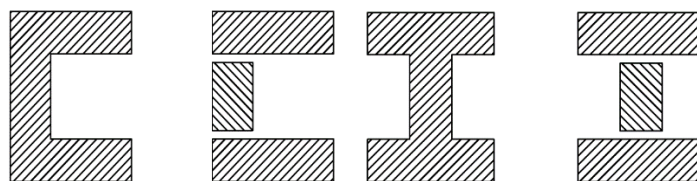


Fig. 1 – In plane structural conformation for structures divided by seismic joints.

Moreover, vertical regularity is required, which reduces effort concentration and ductility requirements in isolated areas of the building. The structural elements which provide lateral stiffness should be placed as uniformly

as possible to limit eccentricities and increased structural redundancy leading to increased seismic energy dissipation capacity throughout the structure. By redundancy, it is ensured that the failure of a single element or a single joint does not lead to global failure.

Seismic joints are designed to divide the structure into separate bodies with different dynamic characteristics in order to allow them to independently oscillate under the seismic action or to limit the effects of eventual collisions at a strength level below the building strength. If the joints separate sections with similar dynamic and constructive characteristics, they may have dimensions established from the dilation-contraction condition. In complex cases, when buildings have very different dynamic characteristics (masses, heights, stiffnesses), or very different lateral strengths (when a new construction is placed in the proximity of an old building with high seismic vulnerability), or have eccentric positions, or have the slabs vertically offset, the joint is dimensioned, imposing the condition that during an earthquake the separated sections do not collide when oscillating.

The necessary seismic joint width,  $\Delta$ , is determined with relation:

$$\Delta \geq \sqrt{d_{1,\max}^2 + d_{2,\max}^2} \quad (1)$$

where:  $d_{1,\max}$  and  $d_{2,\max}$  are the maximum displacements of the two independent structural units under the seismic action, corresponding to the ultimate limit state, determined at the peak height of the building with lower height. The displacement values are computed in accordance with Annex E of P100 / 2013.

It is allowed to adopt smaller values for the joint, than those computed if the impact forces resulting from a dynamic calculation are taken into account in the dimensioning of the two building units and if adequate dampers are placed in the joints.

The number of reinforced concrete frame structures vulnerable to earthquakes is alarming. In industrialized countries, thousands of structures are considered not to meet the necessary conditions, since restrictions on reinforcement detailing in seismic areas were introduced after 1970.

It is generally considered that filling the frames with masonry improves structural behaviour to horizontal actions. This is true, but only for low loads, and as long as the masonry remains intact. Combining two types of material such as concrete and brick leads to inappropriate behaviour in case of an earthquake. The frame structure is relatively flexible and has high ductility, meanwhile unreinforced masonry is stiff and brittle and may explode under the effect of low deformation. At the beginning of an earthquake, masonry takes over much of the seismic action, but as the intensity increases, masonry crashes due to shear forces and slides.

Diagonal cracks are specific for seismic failure. Two structural failure cases can be identified for reinforced concrete frame with masonry infill: the columns are more rigid than the masonry and the masonry is completely destroyed and expelled from the frames panel (Fig. 2) or the columns are less rigid rather than masonry and yields due to large shear forces, often leading to the building collapse.

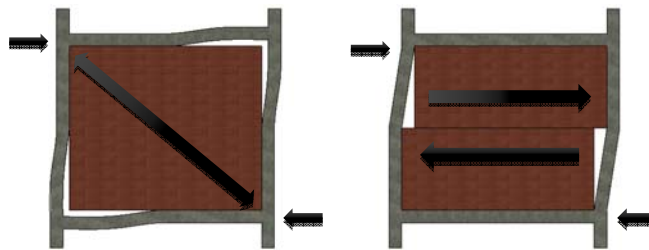


Fig. 2 – Failure modes for reinforced concrete frame structures with infill masonry.

For the seismic design of reinforced concrete or steel frame structures, the computation model will take into account the following unfavourable effects of the masonry panels:

- modifying the structural regularity in plan and elevation and consequently reducing the  $q$  behaviour factor for the entire structure and the conditions of applicability for the calculation methods to determine the basic seismic force;
- modifying the sectional stresses distribution in the vertical structural subassemblies as a result of the increase of the twisting moment by changing the position of the stiffness centre with respect to the centre of mass;
- modifying the geometric computation (modification of the lengths and/or bearing conditions of the frame bars);
- local efforts given by the frame – panel interaction (in particular at the frame nodes and at the corners of the panel).

Due to uncertainties regarding the execution stage and lack of conclusive experimental results regarding the bonding between masonry infill and frame structure, the calculation model for the seismic design of the new buildings will not take into account the favourable effects of the masonry panels. The favourable effect of these masonry will only be taken into account for assessing the safety of existing buildings under the P 100-3 Code, depending on the actual seismic response of the buildings. The unreinforced masonry panels will be limited as follows:

- the area of a panel rested on four sides:

$$A_p = h_p l_p \leq 18.00 \text{ m}^2, \quad (2)$$

• the area of a panel rested on three sides (without column on the free vertical side):

$$A_p = h_p l_p \leq 18.00 \text{ m}^2, \quad (3)$$

• panel height:

$$h_p \leq 3.50 \text{ m}, \quad (4)$$

• length of the panel:

$$l_p \leq 6.00 \text{ m}. \quad (5)$$

For cases where architectural design requirements impose superior dimensions, the panels will be divided by introducing reinforced concrete columns and belts.

The seismic design forces acting in the plane of the masonry panels are computed considering the assembly of the frame and the masonry panels modelled as a triangulated system with diagonals pin connected at the ends in the nodes of the frame. The active width of the diagonal ( $d_p$ ) will be equal to  $0.10D_p$ , where  $D_p$  is the diagonal length for the masonry panel (Fig. 3).

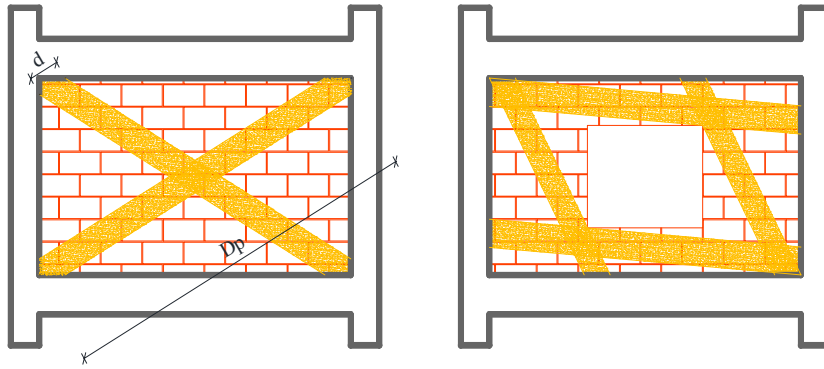


Fig. 3 – Masonry panel. Compressed diagonal systems.

## 2. Case study

The structure considered for the case study develops on ground floor and 6 floors with the following maximum geometric dimensions: length of the

ground footprint 60.10 m and maximum width 19.30 m, Fig. 4. Level heights are at the ground floor of 4.30 m and 3.85 m the rest of the floors. The total height of the building is 32.45 m.

The superstructure has a mixed structural system made of external and internal structural reinforced concrete walls of 20 cm thick and reinforced concrete columns and beams, concrete grade C20/25 and reinforcement of PC52 for longitudinal bars and OB37 for the stirrups. The slabs thicknesses are of 12 cm for section III, 15 cm for section II and 18cm for section I from reinforced concrete, concrete class C20/25, as well as access the stairs. Bricks with vertical voids of 30 cm width will be considered for the infill panels.

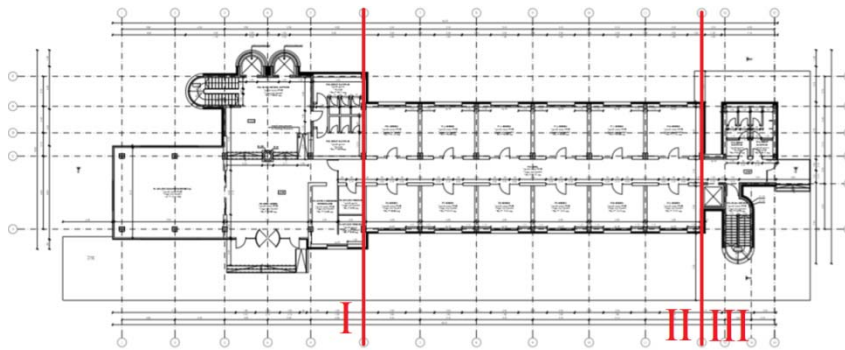


Fig. 4 – Horizontal section and definition of structure's sections.

Static, modal and nonlinear analyses were performed in SAP2000vs16 finite element software. The program performs own vector analyses, static and dynamic analyses, linear and nonlinear analysis in general, and pushover analysis in particular.

Pushover analysis on  $X$  and  $Y$  directions were performed. The results of this analysis revealed the necessity to divide the structure due to the appearance of the C-type (collapse) plastic hinge in the vertical structural elements (columns) at the third floor of section I, can be seen in Fig. 5. The necessity to divide the structure into independent sections is sustained also by specifications in P100/2013 seismic code, where it is stated that the structural system should be monotonically developed on vertical direction, from the foundation to the top of the building.

The joint position is chosen so that the side sections, which have to take the maximum seismic action, to have an increased mass (including an additional number of openings) and/or an increased strength to limit the additional negative effects in these building sections (Fig. 4).

The dimension of the joint established according to relation (1) is also applicable for the finishing works.

$$\Delta \geq \sqrt{d_{\text{Tronson I}}^2 + d_{\text{Tronson II+III}}^2} = \sqrt{198.3^2 + 144.8^2} = 145.97 \text{ mm.} \quad (6)$$

The case study will analyse the behaviour of section II with no infill, with infill of masonry with vertical voids and with infill from autoclaved concrete. For the autoclaved concrete panel, GBN 35 blocks type were used.

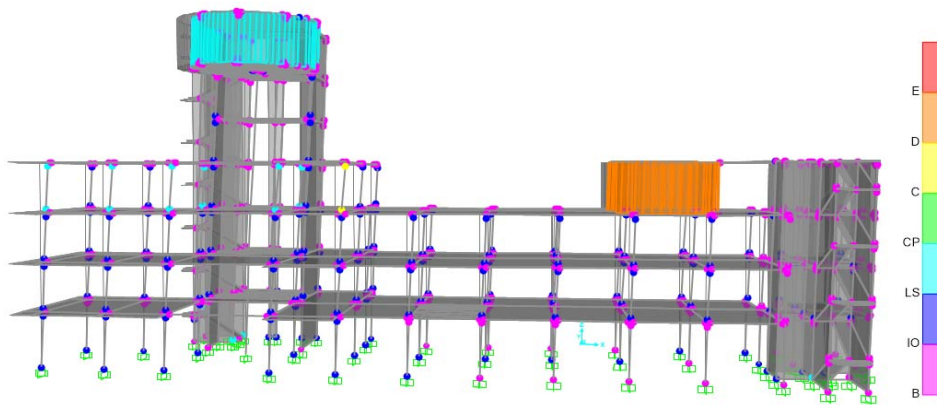


Fig. 5 – Deformation of structure in direction  $X$  with emphasis on plastic joints.

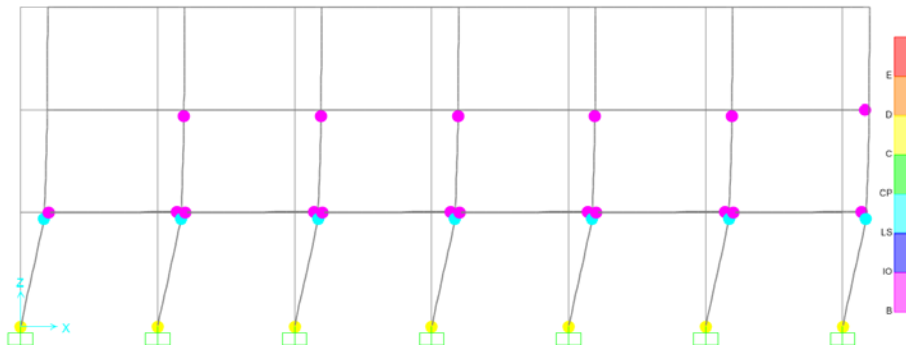


Fig. 6 – Deformation of a longitudinal frame in PUSH  $X$  case.

As can be seen in Fig. 6, the first elements that reach collapse are the columns from the ground floor, which lead to the idea that the structure behaves

as a structure with flexible ground floor. These types of structures are not recommended in areas with increased seismic risk. The capacity curve for the longitudinal frame of section II reaches a lateral force of 270.407 kN and a displacement of 9.3 mm, in the elastic range, then enters the plastic range, where the maximum lateral force is 419.962 kN with a maximum displacement of 100.4 mm.

When the longitudinal frame is filled with masonry brick, the structure behaviour differs substantially from the previous case. As can be seen in Fig. 7 the cracks are distributed in almost all linear elements from the frame, but the failure of the structure is produced in the opposite direction from where the lateral force is applied. The capacity curve for the longitudinal frame with infill masonry bricks reaches a lateral force of 1162.695 kN and a displacement of 8 mm, for the elastic range, meanwhile for the plastic range the maximum lateral force is of 10,905.854 kN with a displacement of 88.7 mm.

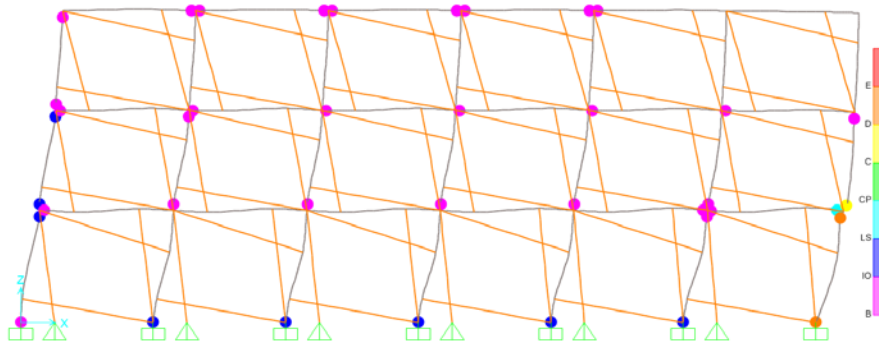


Fig. 7 – Deformation of the longitudinal frame for the frame with brick masonry infill.

The third case considers autoclaved concrete (AAC) in the infill panel. In this case the plastic hinge develops more homogeneously in all linear elements of the frame, Fig. 8. In the capacity curve for the longitudinal frame with AAC infill it can be observed that the elastic range ends at a force of 681.787 kN with a displacement of 10.3 mm, and at a maximum lateral force of 7415.460 kN with a displacement of 167.5 mm for the plastic range.

Figs. 9,...,11 show the plastic hinge development for a transversal frame for the three considered cases. Fig. 9 is for the bare frame case and shows a similar behaviour as in the case of longitudinal frame. The capacity curve obtained through pushover analysis shows a lateral force of 156.550 kN with a



displacement of 6.5 mm for the elastic range which increases until 262.711 kN for the force and 94.7 mm for the displacement in the plastic range.

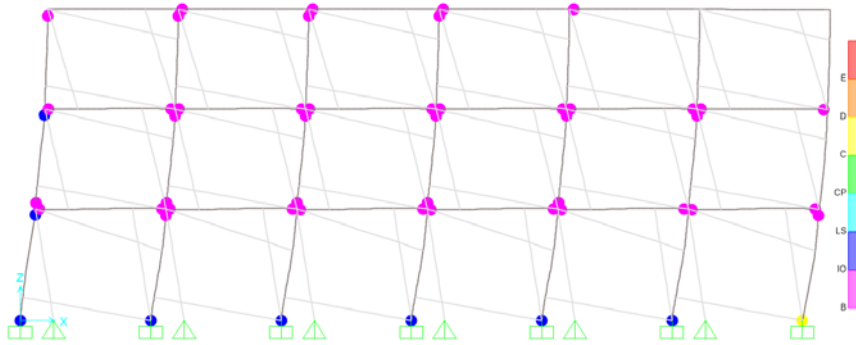


Fig. 8 – Deformation of the longitudinal frame for the frame with AAC masonry infill.

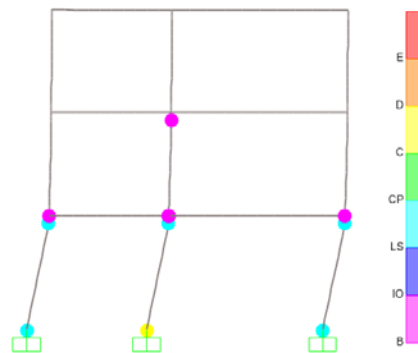


Fig. 9 – Deformation of a bare transverse frame.

The transversal frame, filled with masonry brick with vertical voids, directs plastic hinges into columns. The most vulnerable element is the marginal column from the first floor, Fig. 10. The capacity curve records a lateral force of 433.064 kN with a displacement of 4.2 m, in which the structure operates in the elastic range, then enters the plastic range and reaches a maximum lateral force of 3,038.540 kN with a displacement of 51.3 mm.

The transversal frame, filled with AAC masonry deforms as in Fig. 11. As can be seen, damage appears in all the elements, but the beam is the one that fails first, similarly with the weak beam-strong column design concept from most seismic codes. The capacity curve records a lateral force of 297.131 kN with a displacement of 6.2 mm for the elastic range and maximum lateral force of 3306.672 kN with a displacement of 156.8mm for the plastic one.

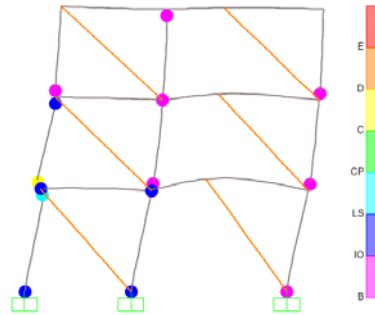


Fig. 10 – Deformation of a transversal frame for the frame with brick masonry infill.

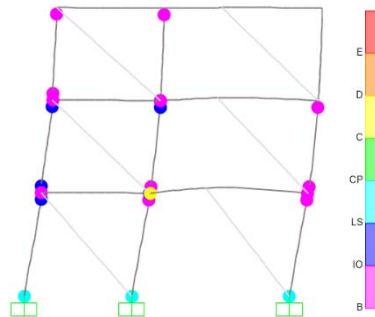


Fig. 11 – Deformation of a transversal frame for the frame with AAC masonry infill.

### 3. Conclusions

From the synthesis of the pushover analysis results, Figs. 12 and 13, a similar tendency is observed for both directions, longitudinal and transversal, for the three considered cases. The capacity curves increase with the stiffness of the structure.

The AAC masonry infill increases with almost 12 times the building stiffness on transversal direction in comparison with the bare frame and 18 times for longitudinal one. The masonry brickwork increases 11 times the stiffness for the transversal frame and 26 times for the longitudinal one. The changes in the capacity curves, as well as in the overall failure mechanism reveal the necessity to consider, at least in very complex structures, the supplementary stiffness introduced in a reinforced concrete structure by the infill panels.

The supplementary stiffness determines the flexible reinforced concrete frame structure filled with AAC/brick masonry to no longer have a good respond to horizontal loads. In practice, there are many cases where the additional stiffness led to column damage, or unfavourable local mechanisms, sometimes reaching to global failure mechanism.

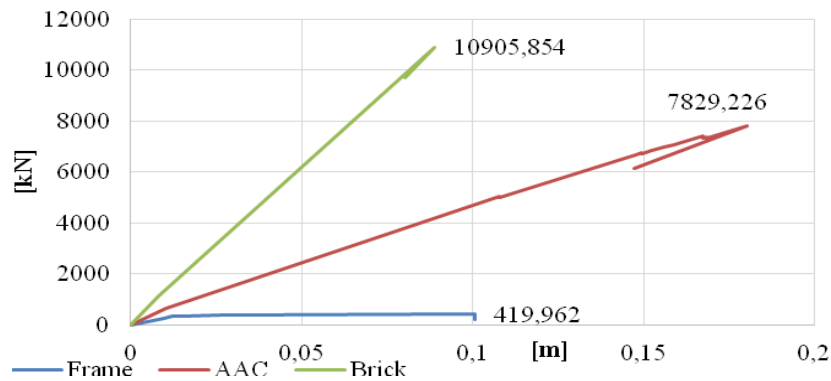


Fig. 12 – Synthesis of longitudinal frame capacity curve results.

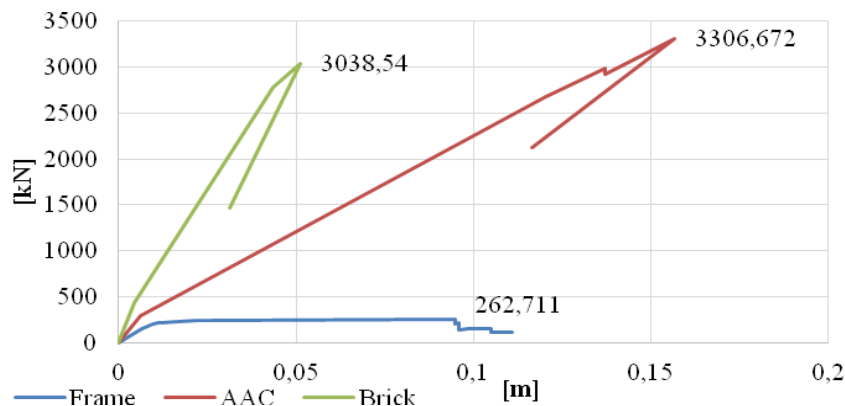


Fig. 13 – Synthesis of transverse frame capacity curve results.

If the highest lateral force is of interest, the results show that the frame with brick masonry infill is the most appropriate one. However, for an efficient seismic design the ratio force/displacement is more interesting for structures in seismic areas. For this case, the results show that the frame filled with AAC brick manages to increase both, the lateral force, as well as the horizontal displacement.

## REFERENCES

- Gelen Gael Chewe Ngapeya, Danièle Waldmann, Franck Scholzen, *Impact of the Height Imperfections of Masonry Blocks on the Load Bearing Capacity of Dry-Stack Masonry Walls*, Construction and Building Materials, Elsevier, **165**, 898-913 (2018).
- Olteanu I., *Evaluarea comportării structurilor în cadre din beton armat supuse la acțiuni seismice*, Ed. Politehniun, Iași, 2011.
- Pujol S., Fick D., *The Test of a Full-Scale Three-Story RC Structure with Masonry Infill Walls*, Engineering Structures, **32**, 3112-3121 (2010).
- Santiago Pujol S., Benavent-Climent A., Rodriguez M.E., Smith-Pardo J.M., *Masonry Infill Walls: an Effective Alternative for Seismic Strengthening of Low-Rise Reinforced Concrete Building Structures*, The 14 th World Conference on Earthquake Engineering, Beijing, China, October 12-17, 2008.
- \* \* *Cod de proiectare a construcțiilor cu pereți structurali de beton armat*, Indicativ CR 2-1-1.1/2013
- \* \* *Design Provisions for Buildings*, Seismic design code indicative P100-1 2013.
- \* \* *Provisions for Seismic Assessment of Existing Buildings*, Seismic design code indicative P100-3 2018.

PARTICULARITĂȚI STRUCTURALE PENTRU O CONSTRUCȚIE  
NEREGULATĂ, ÎN PLAN ȘI ELEVAȚIE, AMPLASATĂ ÎNTR-O ZONĂ  
SEISMICĂ

(Rezumat)

O proiectare conceptuală a structurilor situate în zone seismice care să asigure o comportare seismică corespunzătoare este foarte importantă iar proiectarea seismică trebuie să urmărească realizarea unei structuri cât mai regulate, distribuită cât mai uniform în plan. Elementele structurale care asigură rigiditatea la forțe laterale trebuie dispuse cât mai uniform pentru a permite excentricități cât mai mici și o redundanță sporită a structurii care conduc la o capacitate sporită de disipare a energiei seismice în întreaga structură. Majoritatea structurilor în cadre din beton armat vor fi închise perimetral cu panouri de zidărie acestea în realitate se manifesta ca elemente structurale și până la eventuala lor distrugere la acțiunea unui cutremur puternic, panourile preiau o parte din încărcarea laterală. În practică sunt numeroase cazurile în care aportul de rigiditate adus de zidăria înrămată în cadre a condus la deteriorarea stâlpilor structurilor cauzând mecanisme locale respectiv până la colapsul global. Pentru studiul de caz s-a considerat o construcție cu un sistem constructiv mixt, în cadre și pereți structurali, cu neregularități în plan dar și pe verticală. Aceasta a fost închisă perimetral, pe rând, cu panouri de zidărie din B.C.A. și caramidă cu goluri verticale. În urma analizei se va studia aportul de rigiditate adus structurii în cadre din beton armat, pentru cele două cazuri de închideri perimetrare și dezvoltarea fisurilor în elementele liniare ale cadrului.