Seismic vulnerability of pre-code reinforced concrete shear wall buildings in Romania

M Barnaure

'Faculty of Civil, Industrial and Agricultural Buildings, Technical University of Civil Engineering, Bucharest, 122 – 124, Lacul Tei Blv, 020396, Bucharest, Romania

E-mail: mircea.barnaure@utcb.ro

Abstract. Prior to 1963, when the first earthquake design code was implemented in Romania, mid height (over 8 storeys) shear wall buildings were already being built in some of the major cities. These buildings have important structural weaknesses when assessed using the current regulations. An 11 storeys residential building in Bucharest built in 1962 is analysed. Results show that the building is extremely vulnerable on the longitudinal direction, where the lateral resisting system is composed by only two shear walls. If bending capacity is lacking mainly at the base of the building, shear strength deficiencies are present over the entire height of the building, due to a lack of shear reinforcement in the walls. An analysis of several possible strengthening solutions is made, and conclusions are drawn regarding the feasibility of these solutions.

1. Introduction

A large part of Romania, including some of the major cities, are in earthquake prone areas [1]. While lateral loads were sometimes taken into account for the design of buildings even prior to the introduction of the first official provision in 1963 [2], the strong earthquake from March 4, 1977 [3] showed the structural vulnerability of previously constructed buildings. The Romanian earthquake design codes were amended several times based on both local and international expertise, with the latest version being adopted in 2013: P100-1:2013 [4]. The vulnerability of existing buildings is assessed by comparing their strength with that of a code-compliant building. The current Romanian code, P100-3:2019 [5] is in many aspects similar to the European norm EN 1998-3:2005 [6].

Of particular interest when analysing the seismic vulnerability of existing buildings are the mid-rise (over 8 storeys) buildings constructed prior to the introduction of the earthquake codes. Such buildings, with a reinforced concrete structural system, were built in the 1960’s in some of the major Romanian cities, including Bucharest. These buildings are unsafe and require structural strengthening. But a reasonable intervention solution might be difficult to choose, as the implications of the possible structural engineering solutions are multiple: economic, legal, functional, aesthetic, and so on. Based on the analysis of an actual building, some intervention possibilities are discussed, and some general conclusions are made.

2. Description of the analysed building

A basement, ground floor and 10 storeys apartment building built in 1962 in Bucharest was analysed. It consists of two very similar (from a structural point of view) sections, each with a rectangular shape in plan, 25.85 m by 11.55 m, separated over the full height by a seismic joint. These sections are called b1 and a5. They were named as such in the original project, as the analysed building is one of many similar buildings constructed in that period, and these buildings were made by adjoining several “a” or “b” section types. The height of the ground floor and of the stories is 2.75 m, while the basement is a technical space of 1.90 m height. The ground around the building is at level -0.75 m (measured from the level of the ground floor). The level at the top of the slab above the 10th storey is +30.17. Throughout the paper the references will only be made to results obtained for section b1, as results obtained for section a5 are very similar qualitatively and quantitatively.
2.1. The structural system

The structural system is with reinforced concrete shear walls, this constructive solution being typical for the design of similar structures from that period (before or immediately after the appearance of the first normative document of seismic design, P13-63 [2]).

The lateral resisting system (needed to withstand the earthquake forces) is dominated in the longitudinal direction by two “T”-shaped walls (in the longitudinal direction these walls take over practically the entirety of the seismic forces). On the longitudinal facades, there are reinforced concrete frames, but their stiffness is very low when compared to the shear walls from the axis of the building. In the transverse direction, there are a series of lamellar walls. The thickness of the walls is generally 15 cm, but they sometimes have flanges or small columns at their ends. Reinforced concrete floors are 11 cm thick, except for the balconies (8 cm) and the staircase (13 cm) – figure 1.

![Figure 1. Structural configuration of the stories – section b1 (extract from the original project).](image)

![Figure 2. Structural configuration of the basement – section b1 (extract from the original project).](image)

The vertical structural elements in the superstructure are continuous in the basement. The thickness of the structural walls in the basement is higher than in the superstructure (25 cm) and a few additional columns 25x25 cm or 25x50 cm act as supports in the central area of the slab above the basement – figure 2.
Figure 3.
Structural configuration of the foundations – section b1 (extract from the original project).

The shallow foundations (figure 3) are continuous (with an average width of approximately 250 cm) under the main shear walls, while isolated foundations exist under the columns. The connection between the foundations is made on the longitudinal facades by high (182 or 142 cm) beams beneath the slab above the basement. The base of the foundation is at a level varying from -3.35 m to -5.25 m.

2.2. Materials
The strength classes for the concrete mentioned in the original project are:
- B50 (C 2.8/3.5) in lean concrete foundation blocks
- B110 (C 6/7.5) in reinforced foundation beams
- B200 (C 12/15) in the structural elements for ground floor up to the 5th storey
- B170 (intermediate class between C8/10 and C12/15) from the 6th story upwards

For the typical concrete in the superstructure, the \( f_{cm} = 20 \text{ N/mm}^2 \) was considered within the study. The rebars (not deformed) are made of mild steel OL38 type. The \( f_{ym} = 284 \text{ N/mm}^2 \) value was considered within the study.

2.3. The reinforcing detailing for the shear walls
The reinforcement of the elements is specific for the period of the 1960s. Thus, the columns and the intersections of the walls are reinforced (at the base of the building) longitudinally with \( \Phi 14 - \Phi 20 \) (less often \( \Phi 25 \)) rebars and stirrups \( \Phi 8 / 30 \text{ cm} \) (less often \( \Phi 8 / 25 \text{ cm} \)). The diameters of the longitudinal bars and of the stirrups are lower in the upper floors. The stirrups are generally more densely spaced on the overlapping zones of the longitudinal reinforcements.

In figure 4 the reinforcing details of one of the main shear walls are shown. It is to be noticed that even in the lower storeys there is very little shear reinforcement in the web (\( \Phi 8 / 30 \text{ cm} \) in basement and ground floor, \( \Phi 8 / 25 \text{ cm} \) in storeys 1 and 2) while in the upper storeys there is no continuous reinforcement in the web, but only in the intersections between web and flanges.
Figure 4. Reinforcing details for one of the main shear walls in the longitudinal direction – section b1 (extract from the original project).
2.4. Previous damages
In the approximately 60 years of existence, there were several earthquakes of Vrancea origin (the 1977 one having the largest magnitude). No reliable information was found about damages caused by these earthquakes. Still, an investigation report done on the building in 1993 mentions that damage was observed (cracks in the walls, cracks in the beams and lintels) and also that the building was damaged both during the 1977 earthquake (requiring repairs), as well as after the 1986 and 1990 earthquakes. The site investigation performed in 2020 identified no such damages. The only visible damage with possible seismic causes were cracks in the slabs in the staircase area (developed in the transverse direction of the building).

During the discussions with the tenants of the building, some of them living in the building in 1977, with one exception, all replied that they did not know that the building had suffered any damage to the structural elements or that structural interventions had been required and made. The mentioned damages were vertical cracks (cracks in plaster) in the areas where the reinforced concrete structural walls were continued with masonry (non-structural) partition walls. Also, the tenants who had recently carried out refurbishment works in the apartments said that they did not notice any cracks (even repaired) or other previous interventions when they removed the old finishes. Only one tenant living in the building in 1977 described damage to one of the walls (deep cracks, which required resin injection and wall cladding), but the investigations carried out on that wall (radar survey) did not validate the existence of these cracks.

However, as the 1993 report mentioned that: "the building was strengthened after the 1977 earthquake, when local repairs were made consisting in injections and local plastering of the shear walls with epoxy resin and fiberglass fabric", the existence of previous damage can not be excluded. In fact, as discussed further below, the area where the tenant mentions structural damage during the 1997 earthquake (see figure 5) corresponds to one of the most vulnerable areas of the structure.

![Figure 5. Reinforcing details for one of the main shear walls in the longitudinal direction – section b1 in story 3 (extract from the original project). The highlighted area is the one where the owner of one of the apartments mentioned damages (inclined cracks) appearing during the 1977 earthquake.](image-url)

3. Numerical modelling
Several numerical models were developed using the ETABS software [7]. The shear walls were modelled using shell elements, the beams and columns were modelled using frame elements, while slabs were modelled as membrane elements. Some numerical models only include the superstructure of the building, while others also include the substructure. In the models that include the substructure, the foundations were modelled as point or surface elements, while the restraints were defined as linear springs, with characteristics in accordance with the results of the geotechnical survey. The analysis for the seismic combination was made considering static loads. No significant differences in results were observed between the equivalent lateral load and the response spectrum methods. Linear analyses were performed, considering the ductility factor of the building \(q=2.5\).
Figure 6. Numerical 3D model including the substructure.

The vertical structural elements over the height of one storey for the b1 section, as defined in the numerical models, are shown in figure 7.

Figure 7. Vertical structural elements over the height of a storey for the b1 section of the building the numerical 3D (plan view and 3D view respectively).

For the b1 building considered fixed at its base, the natural periods of vibration are 1.2 s (transversal translation), 0.96 s (torsion) and 0.93 s (longitudinal translation). The modal participation mass ratios are over 70% for each of the main 3 modes. For the a5 building, values are slightly lower (periods 1.1 s, 0.85 s and 0.77 s for transversal translation, longitudinal translation, and torsion respectively.)
4. Results

4.1. Preliminary qualitative assessment

The qualitative assessment, based on the analysis of the distribution and values of stresses in the shear walls – figure 8 -, indicated a significant susceptibility to damage under earthquake loads for the walls in the longitudinal direction and in particular to the wall b1-x2 (longitudinal wall from body b1 adjacent to the joint to section a5). This is the wall where potential previous damage during the 1977 earthquake had been mentioned by one of the occupants of the building.

In terms of story drifts, the computed values for ULS are within the acceptable limit (2.5%), but for SLS the maximum computed value is twice the acceptable (0.5%) limit, which shows a risk related to damage of the non-structural elements.
4.2. Bending capacity check
The capacity calculation for each wall was determined using CSiCOL software [8]. The software determines the stress distribution in a given element, based on the geometry of the element, the properties of the materials (concrete and rebars) and the loads. An example of results provided by the software is shown in figure 9. Based on the capacity of the element and the actual level of stresses, the software computes the capacity ratio in bending.

![Figure 9. Ultimate stresses in one of the two principal shear walls on the longitudinal direction in the b1 section of the building under earthquake loads in the direction of the wall.](image)

4.3. Shear capacity check
Shear capacity check was made following the rules in EN 1998-3:2005 [6], using $\mu = 4$, as stipulated in the P100-3 code [5].

By analysing the capacity ratios for bending and shear, shown in table 1, several observations can be made. The first one is that the building is much more vulnerable in the longitudinal direction, with capacity ratios as low as 19%, while capacity ratios in the transverse direction are higher than 0.55. The second observation is that, while bending capacity is low mostly in the levels at the base of the building, an important lack of shear strength is observed over the entire height of the building, due to the lack of shear reinforcement in the walls.

It is to be noted that in table 1, the values for the capacity in bending are average values (considered more relevant than individual values for each wall), as the ductile failure mechanism would lead to a redistribution of loads between the walls in a given direction. On the contrary, for shear stresses, the minimal value for the walls is shown. There is no real possibility of shear stress redistribution between the walls, mainly in the longitudinal direction. The brittle failure of one of the main walls in the longitudinal direction will cause the transfer of shear forces to the second wall and a rapid domino effect, which can lead to local or even global collapse of the building.

| Story | Capacity ratio in bending | Capacity ratio in shear |
|-------|---------------------------|------------------------|
|       | Longitudinal direction | Transverse direction | Longitudinal direction | Transverse direction |
| GF    | 0.25                      | 0.55                   | 0.22                    | 0.68                   |
| 1F    | 0.26                      | 0.62                   | 0.19                    | 0.67                   |
| 3F    | 0.31                      | 0.73                   | 0.20                    | 0.73                   |
| 7F    | 0.88                      | 1.0                    | 0.43                    | 0.90                   |
4.4. Substructure check
The maximum pressure on the foundation ground in the fundamental load combination is lower than the 390 kPa allowable pressure determined in the geotechnical survey. Under earthquake loads along the longitudinal direction, the pressure levels remain moderate, with a maximum value of 310 kPa, but important shear forces develop in the foundation beam in the longitudinal axis (between the long shear walls) – figure 10.

Figure 10. Qualitative representation of shear forces in the foundation beams under earthquake loads.

The maximum values develop in the area between the two main shear walls in section b1.

In the case of earthquake load in the transverse direction, the global tilting effect is important, with the maximum allowable pressure under the external foundations corresponding to a value of the earthquake load equal to 55% of the code load.

4.5. General earthquake safety of the building
The analysis of the building under earthquake loads showed that the overall capacity of the building is only 20% of the required one, the most unfavourable failure pattern being shear failure of the two main walls along the longitudinal direction, either at the ground level or 1st story (where shear forces are the highest, but continuous shear reinforcement in the webs of the walls is scarce) or at 3rd and 4th level, where shear stresses are somewhat lower but there is no continuous web reinforcement.

This value corresponds to classifying this building in the Rs I (worst) earthquake risk class as defined by the P100-3 [5] code. As explained in the code, this category of buildings is susceptible to local or total collapse when submitted to the design earthquake corresponding to the Ultimate Limit State. Retrofit measures are mandatory for such buildings, with the minimal strength level to be attained being 65% (this value is the lower value for a building to be included in the Rs III class).

4.6. Discussion on the results
It is interesting to analyse a few of the reasons that make such a building which did not suffer extensive damage during the 1977 earthquake be considered today in the highest risk category. Even more as the analysed building is definitely not a singular case. In fact, even in the P100-3 [5] code, an example assessment analyses a different building from the 1960’s, and that building also resulted to correspond to the highest risk class.

- The risk assessment is made for a reference earthquake (estimated return period of 225 years), which corresponds to peak ground acceleration values much higher than the values actually recorded in Bucharest in 1977;
- As the building is higher than 28 meters, it falls into the 2nd category of importance [4], which corresponds to a coefficient \( \gamma_e = 1.20 \) (20% higher design earthquake loads);
- The evaluation methodology takes into account the uncertainties regarding the structure of the building through the confidence factor, \( CF=1.20 \), which translates in a reduction of the strength of elements taken into account for the assessment;
- In the case of brittle failures, the current codes (both national [5] and European [6]) impose
considering the safety factors for different materials when establishing the strength of the elements;

- The building is particularly vulnerable in the longitudinal direction, while the 1977 earthquake might have had a different principal direction;
- There are various possible mechanisms (rocking at the base, over-strength of rebars, energy dissipation due to degradation of floors, beams, non-structural elements, etc.) that may have contributed to reducing the level of damage to structural walls in 1977, but these mechanisms are not controllable and cannot be considered in the structural assessment;
- Although the level of damage suffered by the building in 1977 is uncertain (in 2020 only cracks in the floors were identified, although a previous report also mentions cracks in walls and beams that required local retrofit), a building that has already been damaged in a previous seismic event has lost some of its post-elastic deformation capacity and is likely to be much more severely damaged in the event of a new similar earthquake;
- Results indicate a significant risk of brittle wall failure; thus, until the strength of the structural walls is reached, the building can resist, without damage or with minor damage, but when one of the two longitudinal walls fails, it causes an immediate transfer of the entire lateral load to the second wall, with a sudden domino effect that might lead to local or even global collapse.

5. Possible retrofit solutions

As previously mentioned, the minimal acceptable capacity ratio for a strengthened building in Romania is 65%. Based on the results in the structural assessment, for the analysed building this goal would require interventions in the substructure (foundations and basement elements) as well as interventions in all the superstructure storeys. Under these conditions, the economic implications may be much greater than those relating exclusively to the strengthening of the structure, as they involve for example costs related to refurbishment of finishes damaged by the structural interventions or costs related to providing accommodation for the current building tenants during the intervention. In fact, studies analysing the decision-making processes after the L’Aquila [9] or Christchurch [10] earthquakes concluded that the solution for intervention (either repair or demolish and rebuild) is most often based on financial considerations. Still, some other variables, including among others the perception of risks and government decisions influence the final decision, which is rarely solely based on the structural requirements.

Several possible solutions are further discussed. It is to be noted that these solutions exclusively cover the structural safety aspects. Yet, an intervention project should be based on a complex and comprehensive study that analyses all the implications, not only technical, but also financial, legal, functional, aesthetic, and so on.

5.1. Solution type 1

The first retrofit solution consists in RC jacketing for all the vertical structural elements (walls and columns) in all the stories where a superior strength is required. The jacket could be 7 cm thick when applied on both sides of an element, or 15 cm thick for one-side intervention. This solution allows to:

- Favour ductile failure in bending and avoid brittle shear failure;
- Limit the compression stress levels in the vertical elements;
- Develop the compressed area at the free ends of the wall section;
- Direct the potentially plastic areas of the walls at their base.

In order to increase the capacity of foundations, two types of interventions are needed:

- Extending the surface of the foundations, especially on the longitudinal facades, by creating new foundation blocks between the existing foundation blocks;
- Jacketing the areas where high stresses are transmitted by the walls (mainly along the central axis of the building), and thus creating a continuous reinforced concrete foundation beam.

The advantages of this solution are:
Interventions are performed differently from one element to another and from one floor to another, depending on the required increase in strength.

The ground surface occupied by the building is not modified (or just to a very small extent).

As a sidenote, such a solution is also proposed in the case of the mentioned example building in the P100-3 [5] code.

At the same time, the solution has a significant disadvantage:

- As it is necessary to retrofit most of the elements (at least on the first floors), and as the longitudinal shear walls need to be jacketed over the whole height of the building, the implementation of the solution will inevitably involve extensive damage to finishes in all the apartments (with associated repair / replacement costs), as well as the need to relocate the tenants during the works.

Another disadvantage is related to the fact that, by jacketing the vertical elements, the usable area of the apartments is reduced, while some free-passage areas (corridors, staircases, etc.) become smaller (with possible functionality problems). Given the disadvantages of this solution, the authors consider that, as far as possible, intervention solutions 2 or 3, discussed below, should be favoured.

5.2. Solution type 2

The second intervention solution (figure 11) consists in adding reinforced concrete cores and reinforced concrete frames on the outside of the building. These newly introduced structural elements would withstand most of the lateral forces, especially in the longitudinal (deficient) direction, so that existing elements in the superstructure would no longer need interventions inside the building (or only small interventions).

Figure 11. Type 2 strengthening solution (3D and current level view).

The newly introduced cores could be positioned in the balconies area and possibly in the staircase area. The main advantage of this solution is that most of the interventions could be carried out without
the need to relocate the occupants of the apartments for long periods of time, and the interventions would not involve extensive degradation of interior finishes.

![Figure 12. Possible intervention solution for reducing the stiffness of the existing longitudinal walls.](image)

However, the solution also raises certain problems:
- The ground footprint of the building and its exterior appearance change;
- It is difficult to connect the new exterior frames to the existing elements on the facade as to ensure the transfer of seismic forces to the cores;
- As the new elements should be very stiff compared to the existing structure (in order to limit the forces transmitted to the existing elements), the new RC cores should have important in-plane dimensions as well as limited door / windows openings;
  - In order to reduce the stiffness of the existing longitudinal walls in the axis of the building (and thus the lateral loads on them), voids could be created (by cutting with a diamond disc) – as shown in figure 12. This type of intervention, even if it is done inside the apartments, is much less invasive than wall jacketing.
- The foundations of the newly introduced cores would most likely need to be deep foundations (piles), as the new elements will be very stressed in bending, but with a low level of vertical loads; this implies high costs for the foundations.

5.3. Solution type 3
The third intervention solution is a mixed intervention, which has some of the advantages, but also of the disadvantages of the previously discussed solutions. The solution consists in adding reinforced concrete frames on the outside of the building, on the longitudinal facades, combined with additionally jacketing the elements with insufficient strength.

![Figure 13. Type 2 strengthening solution (3D view of the superstructure).](image)

The main advantage of this intervention solution is that it allows making intervention in two steps. Such a possibility is allowed by the P100-3 code [5].

Thus, in the first stage:
- Interventions are made at the foundations on the main facades and in the central axis;
A system of reinforced concrete frames is created, on the outside of the building, on its entire height; the frames are connected to the existing structure using chemical anchors fixed in the existing beams and pillars on the façade;

The intervention on the facades can be completed with thermal rehabilitation works for the building; the façade repair works are useful to avoid further damage to the building until the funds are available to carry out the remaining major repair works;

Following this first stage, the building can pass from the current Rs I seismic risk class into Rs II class (capacity ratio higher than 35%), limiting the risk of collapse in case of a severe earthquake.

In the second stage:

- Jacketing is conducted inside the building, but, as frames were introduced on the façade in the previous stage, the intervention extent for the elements inside the apartments is reduced;
- Following this second stage of intervention, strength of the building will correspond to the Rs III seismic risk class (capacity ratio higher than 65%).

6. Conclusions

The analysis of a reinforced concrete shear wall structure built prior to the introduction of the seismic design code provisions in Romania led to the conclusion that the existing building is highly vulnerable and is susceptible to local or total collapse when submitted to the design earthquake corresponding to the Ultimate Limit State. The main deficiency is the very low shear strength along the longitudinal direction, which is observed not only at the base of the building, but also in the upper storeys. Extensive interventions are needed for the structural strengthening of the building. This situation poses difficulties in choosing the most appropriate solution. Several intervention possibilities were analysed, and their advantages and disadvantages discussed. As the vulnerabilities of studied building are actually representative for many similar buildings from the 1960’s, further analysis should be carried out in the direction of identifying the optimal retrofit solution for this building type. Such analysis should not only involve structural engineers, as the possible technical solutions have different architectural, economic and legal implications that should be taken into account.

7. References

[1] Oncescu M C, Marza V I, Rizescu M and Popa M 1999 The Romanian earthquake catalogue between 984–1997 In Vrancea earthquakes: tectonics, hazard and risk mitigation Springer, Dordrecht

[2] P13-63 Conditioned Standards for Civil and Industrial Constructions Design in Seismic Regions (in Romanian)

[3] BERG G V 1980 Earthquake in Romania, March 4, 1977: an engineering report. National Academy Press.

[4] P100-1:2013 Seismic Design Code. Part 1: Design Provisions for Buildings (in Romanian)

[5] P100-3:2019 Seismic Design Code. Part 3: Seismic assessment of existing buildings (in Romanian)

[6] EN 1998-3:2005 Eurocode 8: Design of structures for earthquake resistance – Part 3: Assessment and retrofitting of buildings

[7] CSI ETABS 18.1.1 2018 Computers & Structures, Inc.

[8] CSI Column 9.0.1 2014 Computers & Structures, Inc.

[9] Polese M, Di Ludovico M and Prota A 2018 Post-earthquake reconstruction: A study on the factors influencing demolition decisions after 2009 L’Aquila earthquake Soil dynamics and earthquake engineering 105 139

[10] Marquis F Kim J J Elwood K J and Chang S E 2017 Understanding post-earthquake decisions on multi-storey concrete buildings in Christchurch, New Zealand Bulletin of Earthquake Engineering 15(2) 731