Universitatea Tehnică de Construcții București
Facultatea de Construcții Civile, Industriale și Agricole

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DOCTORAL THESIS

Seismic rehabilitation of historical heritage built

-SUMMARY-

Doctoral student

Eng. Daniel-Ioan DIMA

Scientific coordinator

Professor Eng. Radu VĂCĂREANU, PhD

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Contents

1. INTRODUCTION

1.1 Topic and objectives of the thesis

General subject of the present doctoral thesis is reflected in the title of the thesis, and refers to the particularities of seismic assessment and structural consolidation of historic buildings, for seismic protection and preservation. Geographic area of the Europe, characterized by a seismicity of global importance is the Mediterranean Basin, the countries in this region being subjected to extremely severe earthquakes.

A important project example in the domain of seismic protection of historical monuments is FP6 PROHITECH International Project "*Earthquake protection of Historical Buildings by Reversible Mixed Technologies*" [1, 2, 3, 4, 5, 6], carried out during 2004-2008, which brought together 12 seismic countries, including Romania. They shared both their experiences of the earthquakes' negative effects on historic buildings and their technical knowledge, in order to invigorating and develop new sustainable national and international policies and strategies, oriented in particular on prioritizing seismic safety of historic monuments.

The unfortunate experiences of the great Vrancea earthquakes of the last century (November 10, 1940 and March 4, 1977), led to the conclusion that Romania can be considered the 3rd European seismic country after Italy and Turkey, and Bucharest city as the most vulnerable European capital in terms of seismic view [1, 7, 8], but the seismic vulnerability of cities in Romania, especially in Bucharest is given by the vulnerability of the existing built fund vulnerable to earthquakes

The thesis topic is still current and of great national importance, in the context in which, in Romania, the rhythm of seismic safety (structural consolidation) of historic buildings is still slow, a cause of this being especially the human factor, because of the indifference and especially because of different interests of people, which do not always converge in this direction

The main objectives of this doctoral thesis are:

- emphasizing the impact of the adverse effects of the Vrancea earthquakes, not only on the safety of human lives, but also on the construction of seismically vulnerable historical monuments, the damage or loss of which cannot be neglected;
- intensifying the interest regarding the seismic safety and preservation of the historical patrimony built in Romania;
- highlighting the advantages and stimulating the interest of use, as a very useful support for structural calculations (seismic action assessment), the method of dynamic recording/ vibration measurement of the building's structure, which is selected to be expertized, known in the literature [10 ÷ 45] as the recording method „Ambiental Vibration Tests” (AVT);
- practical and realistic understanding of how to adopt an optimal solution for consolidation process, taking into account all the parameters involved in: i) *ensuring structural seismic performance (strength and stability), provided in the Romanian seismic assessment code P100-3 / 2019 [46]*; ii) *technological aspects (technical limitations, execution times, etc.)*; iii) *economic aspects (material costs and execution labor); iv) fulfilling the requirement to preserve the character of a historical monument, by limiting, as far as possible technologically, its affectation / diminution (minimally invasive interventions and / or reversible consolidation solutions)*.

1.2 Thesis content

This doctoral thesis consists of an introductive chapter (chapter 1) and four synthesis chapters (chapters 2, 3, 4, 5), the contents of which are directly related or tangential to the topic of the thesis, a chapter of personal contributions (chapter 6), elaborated according to proposed objectives, and the chapter of conclusions and personal contributions, the thesis being concluded with the supporting bibliography.

Chapter 1 presents the general topic and objectives of the doctoral thesis, and some general introductive elements regarding the motivation for choosing the topic, to which was added a synthetic description of the content of each chapter of the thesis.

Chapter 2 presents general aspects regarding the protection of the built historical heritage (the history evolution of the main outlines and concepts for restoration and preservation of historical monuments, Athens Charter 1933, Venice Charter 1964, UNESCO professional organizations, ICOMOS, etc.), international and national state-of-the art in this field (specific projects, legislation and specific documents), human and natural risks to which historical monuments are subjected over time, and also were presented some biographical information about the engineer Alexandru Cișmigiu, who was one of the most dynamic and active specialists in the field of historical monuments rehabilitation and consolidation from our country after the earthquake of March 4, 1977.

Chapter 3 was dedicated to remind the Romanian historical earthquakes of 1802, 1838, 1940, 1977, 1986 and 1990 and their disastrous effects on people (casulities) and on buildings, including historical monuments.

Chapter 4 consist in general elements regarding the most important international and national technical documents as FEMA 356/2000, FEMA 547/2007, AS 3826/1998, Eurocode 8-3, P100-3 / 2019, MP025-04, etc., used in the practice of seismic assessment and design of technical solutions for structural consolidation of seismically vulnerable buildings.

Chapter 5 are presented some technical elements for interventions in existing buildings with load-bearing masonry structures (levels of consolidation process as safety, repair, reinforcement operations), various masonry repair solutions (reinforced plasters, masonry weaving process, injections with mortars, etc.) and both classic/current structural consolidation solutions (reinforced concrete jackets, insertion of concrete columns and belts, floors consolidation through reinforced concrete overflows, metal tie rods, consolidation of foundation through concrete or masonry sub-additions, high-strength mortar micropiles, etc.), as well as modern ones (polymeric materials based on carbon fibers/glass, damping systems and seismic insulators).

Chapter 6 are presented generalities related to the technical expertise of buildings (concepts and specific terminology-qualitative assessment/determination of the indicators R_1 and R_2, and quantitative assessment/determination of indicator R_3, classification in seismic risk class, etc.) and the results of three case studies, these being the main contribution of the author of this doctoral thesis.

2. THE PROTECTION OF HISTORICAL HERITAGE

2.1 The evolution regarding the protection of historical heritage

Material or spiritual *asset* is a concept that today means *national assests*, specific to each people, and which is its own *cultural heritage*. The category of material *assets* also includes constructions with different destinations (civil, military, religious, etc.), which have entered the consciousness of creators (designers) or their owners, as concrete objects with dual value, both cultural and material [47] . Historical heritage consists of a diversity of objects, which share their common belonging to the same past, referring to a type of institution and a type of mentality [47, 48].

2.2 Principles and doctrines of restoration

The doctrine of unity of style. Eugene Emmannuel Viollet-le-Duc (1814 - 1879)[50]

Anti-interventionist doctrine. John Ruskin (1819 –1900) [50].

The scientific doctrine of restoration. Camillo Boito (1836 –1914)

2.3 The Athens Charter, 1933

In 1933, the Athens Charter was adopted, as a result of *the International Congress of Modern Architecture*, which promoted: i) *statistical conception on heritage preservation*; ii) *implicitly, raises the issue of reconciling development and preservation*; iii) and under the influence of the ideology of Corbusian urbanism, thus proclaims "*the cult of past values does not exceed the criteria of social justice*"; iv) anti-stylistic/anti-E.E. Viollet- le-Duc).

2.4 The Venice Charter, 1964

The Venice Charter was drafted and adopted at *the 2nd ICOMOS International Congress of Architects and Technicians of Historic Monuments*, May 25-31, 1964, Venice. The Charter is written of 16 articles, but for the design engineers involved in restoration and rehabilitation/structural consolidation works, the most important are article 2 article 10.

2.5 International state-of-the-art

The most important professional orgazisations in the domain of the historical heritage protection and preservationm are.

UNIDROIT (The International Institute for the Unification of Private Law - Frascati, Rome,1926);
UNESCO (United Nations Educational, Scientific and Cultural Organization, 1945)**;**
ICOMOS (International Council on Monuments and Sites – Warsaw, Poland, 1965);
ICCROM (International Center for the Study of the Preservation and Restoration of Cultural Property - New Delhi, 1956);

2.6 National state-of-the-art

CMI (Commission of Historical Monuments), Romania. In 1892, the first decision-making body in Romania was established by the decision of the high Royal Decree with No. 3658/17 November 1892 [63, 64]. Today, CMI operates under the name of **INP** (National Heritage Institute), and according to official documents made public [65, 66].

UNRMI (National Union of Historic Monument Restorers) is a professional organization of national importance, affiliated to UNESCO, ICOMOS, established in 1991 [67], being composed of specialists with rich experience in the field of restoration and rehabilitation of historic buildings, it is dedicated exclusively in this area.

Below it can be seen some reference projects carried out under the careful guidance of UNMRI, which are part of the current national stage of rehabilitation of the built historical heritage. Thus, we can mention: the Romanian Athenaeum [67] (Figure 2.1); Ploiesti Culture Plate [68] (Figure 2.2); The old church of Sinaia Monastery, Prahova county [69] (Figure 2.3); Unirii Museum in Iași [70] (Figure 2.4).

2.7 International and national projects on seismic protection of built historical heritage

RISK-EU was another large-scale program, whose main objective was seismic protection and seismic risk reduction in European seismic countries.

FP6 PROHITECH (Earthquake Protection of Historical Buildings by Reversible Mixed Technologies) was also one of the large international projects in the field of protection of historical monuments, which took place between 2004-2008) [1, 2, 3, 4, 5, 6], to which Romania.

IRPP / SAAH / RPSEE (Integrated Rehabilitation Project Plan / Survey on the Architectural and Archaeological Heritage) is one of the reference projects of international and national importance, also in the field of protection of historical heritage.

CETERS (Theoretical research and experiments to reduce the seismic risk of buildings in the national cultural heritage), was initiated in 2004-2005, and was entitled, CEEX, MENER ANCS Program, MEC (2006-2009).

2.8 Romanian legislation on the protection of historical monuments

- **Law 422 of July 18, 2001** on the protection of historical monuments;
- **Government Decision no. 493/2004** for the approval of the Methodology regarding the monitoring of the historical monuments registered in the LMI;
- **Order no. 2797/2017** on establishing the types of interventions on historical monuments, buildings in their protection areas or in protected areas […].

2.9 Human and natural risks for the historical built heritage

The history of the two world wars shows us the dramatic picture of a historical heritage built almost completely destroyed (Figure 2.6 ÷ Figure 2.8), a drama that we can see in the civil military conflicts in the Middle East (Syria, Iraq, Palestine), in which many historical monument constructions were severely damaged and even destroyed.

An image regarding the deplorable state of the historically built fund in Bucharest and in the country in general, we can also find it in the *Report of the Presidential Commission for Built Heritage, Historical and Natural Sites, Bucharest*, September, 2009 [83]. Below it can be see some examples in this respect.

Abusive demolition, which often presents a greater danger compared to the effects of major earthquakes (unpredictable phenomena), while abusive demolition are premeditated acts, made against the background of various financial or other interests. In this sense we can exemplify the building located on Str. Alexandru Constantinescu, no. 63 (Figure 2.10a, b) or the building located on Str. Aviator Sănătescu no. 37, in its place being built, in a very short time, a new building (Figure 2.10c, d).

The buildings that were classified with RsI seismic risk, were conventionally marked with "*red dot*", in this way being signaled both the public danger represented by them and the need for urgent consolidation. Of course, many of the owners, in an attempt to evade such a delicate situation, abusively some either removed the markings or applied certain paints on them (Figure 2.11a, b), their visibility was prevented. It is also worth noting the situations in which new ones are built near some historical (Figure 2.12a, b, c, d).

2.10 Alexandru Cișmigiu. His biography and activity in the domain of seismic rehabilitation of historical monuments

Professor of Engineering Alexandru Cișmigiu was one of the leading Romanian specialists in the field of seismic rehabilitation and preservation of historical monuments in Romania. Alexandru Cișmigiu was the pioneer of the idea of "disaster prevention", the idea reached today, after a few decades, the concept of reference worldwide and promoted more and more intensely through education, world technical-scientific organizations, by approaching a specific legislation.

As an official recognition in the field of anti-seismic design, Alexandru Cișmigiu was appointed UNESCO expert in Yugoslavia in 1969, 1970 and 1972, after the devastating earthquakes in Skopje (1963) and Banjaluka (1969-1970), for problems of theory, seismic zoning, prescriptions, consolidation of damaged buildings and design of new ones. Among the many consolidation projects he honored with great professionalism and devotion were the Telephone Palace (Figure 2.13a), Agapia Monastery (Figure 2.13b).

3. HISTORICAL EARTHQUAKES AND THEIR EFFECTS ON PEOPLE AND CONSTRUCTIONS

3.1 General aspects regarding the seismicity of the Romanian territory

The experience of the 1977 and 1986 earthquakes confirmed that the peculiarity regarding the predominantly long periods (1.4 ÷ 1.6 s) of the ground vibration in case of moderate and high intensity earthquakes, identified for Bucharest, is given by the local ground conditions (presence in the surface area of a package of thick layers of mostly clay soil of about 50 ÷ 60 m, in the Eastern, Southern and central areas of Bucharest) [91, 92].

3.2 The historical earthquakes of Vrancea from 1802, 1829, 1838

The earthquakes 1802, 1829 and 1838[91] were considered the largest ones in our country, in the XIX century.

3.3 The earthquake of November 10, 1940

According to [91], the year 1940 was marked not only by the earthquake of November 10 and its aftershocks, but was characterized by a very high seismic activity in Vrancea throughout that year. There were many earthquakes of small magnitudes and intensities such as: June 24, 1940 (M = 5.5 at a depth of 115 km), easily felt in Muntenia and Moldova; October 3, 1940 (M = 4.7 ÷ 5.0 at 150 km depth); October 21, 1940, there were several earthquakes in Vrancea, the most important of which took place at midnight at a depth of 100 km (M = 4.5); etc.

On November 10, 1940, the first large-scale earthquake occurs in modern Romania of the twentieth century, a Romania already consumed in World War I and on the eve of World War II, being preceded on October 22, 1940, at 8:27 of the earthquake with Mw = 6.5, depth 125 km and intensity I = VII, and the one of November 8, with Mw = 5.9, depth 145 km). This seismic event was characterized by a Gutenberg-Richter magnitude of MGR = 7.4, (moment magnitude Mw = 7.7) and occurred at a depth of about 140 km, with Vrancea epicenter.

The consequences of this strong earthquake were serious both in terms of loss of life, with over 350 casualties, and in terms of significant property damage. In Bucharest, the most significant destruction was the complete collapse of the Carlton Block, being the tallest reinforced concrete construction in Romania at that time (47 m high, 12 floors). By November 24, 136 dead had been removed from the rubble of this block [87, 91].

3.4 The earthquake of March 4, 1977

Starting from the work "*Synthesis of the Monograph The Earthquake in Romania of March 4, 1977 and the effects on constructions*", parts I, II, III and IV, Central Institute for Research, Design and Management in Constructions, 1978 [95], which was provided to me by the goodwill of Mr. Professor PhD. eng. Dan Lungu, the second important and large seismic event that shook the territory of our country in the twentieth century, was the one of March 4, 1977, which broke out at 21 and 22 minutes, with a duration of 60 seconds, with the epicenter in the Vrancea area.

It was characterized by a moment magnitude Mw = 7.4, a Guthenberg-Richter magnitude MGR = 7.2 and a depth of 94 km, having a special importance, both nationally, by its seismological characteristics (magnitude, focus mechanism, affected area with high intensity) and socio-economic effects (loss of life, property damage, effects on construction), as well as internationally, it is felt from Sicily to Moscow and Leningrad, and in the South to Greece (according to the macro-seismic intensity distribution map developed by NV Shebalin) [95].

The earthquake of March 4, 1977 was the occasion to record, for the first time in our country, the dynamic characteristics by instrumental methods (the first accelerograms). The first seismic wave trains were registered at different stations in the country, such as: Vrâncioaia, Focșani, Cheia, Bacău, Câmpulung Muscel, Iași, Bucharest, Deva and Timișoara. Below are the INCERC station records, as follows [95]:

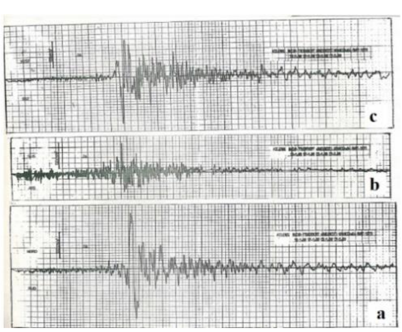


Figure 3.1 - Accelerogram of registration No.1 - INCERC - Bucharest, Sos. Pantelimon 266;
a) the accelerogram of the horizontal movement, Node-South direction;
b) the accelerogram of the vertical movement;
c) horizontal motion accelerogram, East-West direction [95]

- **the record 1** (Figure 3.1): INCERC, Bucharest, Sos. Pantelimon 266, made with a Japanese SMAC-B accelerograph, in the basement of a light ground floor building, this being the most important record, can be considered, practically, as a record of the undisturbed movement of the ground and characterized by weak oscillations, predominantly vertical, with a duration of approx. 18 s, strong oscillations, predominantly horizontal, lasting approx. 15 ÷ 20 s, with destructive effects and oscillations being attenuated, with a duration of approx. 40 ÷ 50 s;
- **the record 2** (Figure 3.2): Block E.5, Balta Albă from Bucharest (a high and relatively rigid construction), provided by the MO-2 accelerograph, mounted on the 9th floor, was characterized by horizontal acceleration values of approx. 3 ÷ 3.5 m / s2 and periods of approx. 0.8 ÷ 1 s, values much higher than conventional for structural calculation;

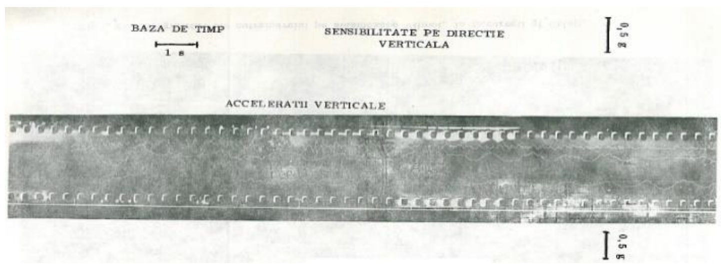


Figure 3.2 - Recording Accelerogram No. 2, Bl. E5 - Balta Albă, Bucharest [95]

- **the records 3 and 4** (Figure 3.3): INCERC, Bucharest, Sos. Pantelimon and Galați, obtained at ground level, with the help of two Wilmot seismoscopes, these highlighting extreme oscillation speeds of approx. 0.42 m / s in Bucharest and approx. 0.27 m / s in Galați.

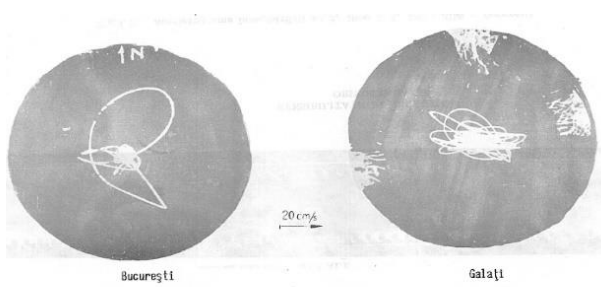
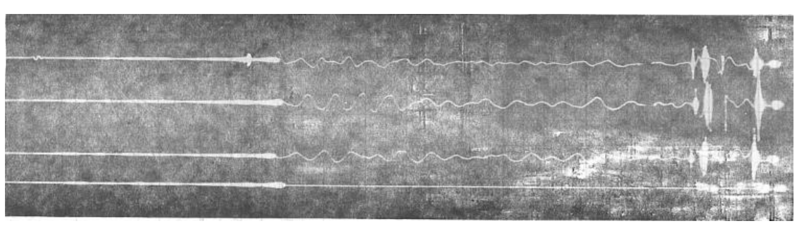


Figure 3.6 - Accelerograms of records Nr. 3 and No. 4, INCERC, Bucharest and Galați [95]

The earthquake was also recorded at the Seismological Observatory "Dr. Cornelius Radu ", Vrâncioaia station, but the movement mechanism of the film did not work, registering only the value of the maximum acceleration, of about 0.31g-0.35g (Figure 3.4) [95].



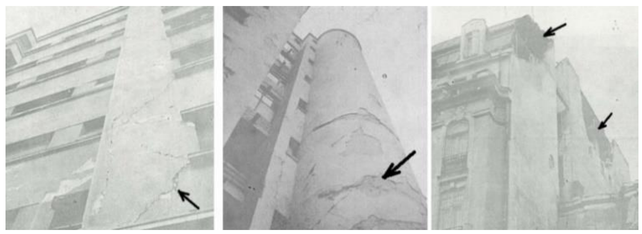
Figură 3.4 - Accelerograma înregistrării de la stația Vrâncioaia – Moldova [95]

At that time, instrumental seismoscope recordings were obtained from the city of Nis - Yugoslavia and recordings provided by seismographs mounted on the structures of buildings in the city of Chisinau - Republic of Moldova, U.R.S.S.

3.5 Causes and effects of the earthquake of 4 March 1977

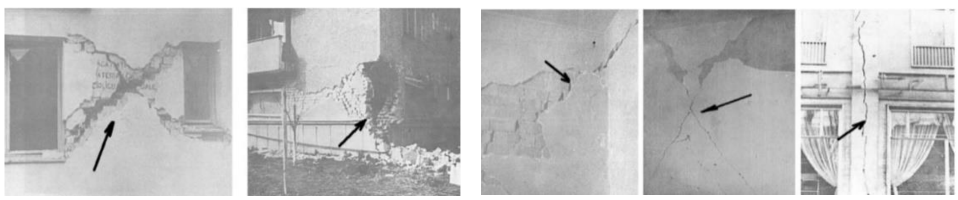
From the point of view of human and economic losses resulted: *i) 1570 casualities and over 11,300 injured, of which approx. 90% in Bucharest; ii) 32,900 homes collapsed or were severely damaged, 35,000 families were left homeless, and tens of thousands of other buildings suffered various damages; iii) after complete evaluations, made later, resulted in material damages amounting to over 2 billion dollars at that time*.

Damages which was found:



a) b) c)

Figure 3.8 - a) cracks inclined at 45º in the facade walls; b) dislocations of the plaster;
a) collapsed masonry heels [95]

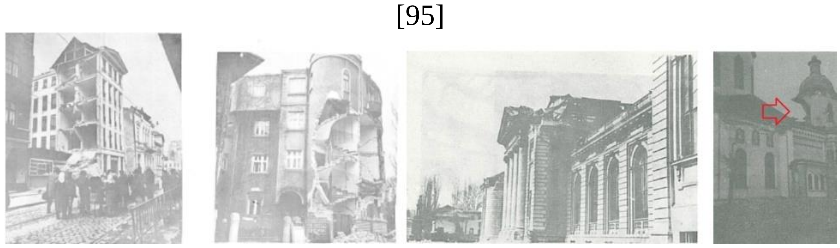


a) b) c) d)

Figure 3.9 - a) “X” cracks in the masonry shoulders; b) damage to exterior walls,
there is a lack of concrete corner pillars; c) cracks at 45º in the masonry walls at intersections;
b) opening an expansion joint in the façade [95]

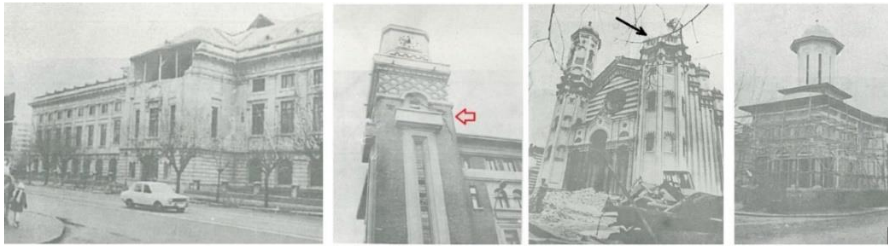
a) b) c) d)

Figure 3.10 - Ministry of Metallurgical Industry ("Carpathians" Block) - Bucharest: a) before the earthquake; b) after the earthquake; c), d) The office block “Republica confectionery” (former Nestor block), Calea Victoriei, no. 63-69, Bucharest, suffered the total collapse of body A from Calea Victoriei [95]



a) b)

Figure 3.11 - a) facade wall completely collapsed; b) wall of the staircase, completely collapsed;
c) damage to the pediment - Faculty of General Medicine / Medical-Pharmaceutical Institute,
Bucharest; d) damaged tower - church of St. Basil the Great, from Ploiești [95]



a) b) c) b)

Figure 3.12 - a) the collapse of a portion of the facade wall - the Palace of Culture in Ploiești; b) cracks in the masonry walls - Central halls in Ploiești; c) St. Spiridon Church - Bucharest, partial collapse of the tower; d) cracks / dislocations of the exterior plasters - St. Eleftherius church (old), Bucharest [95]

3.6 The earthquakes of 30 August 1986 and 30 May 1990

The earthquake of August 30, 1986, had Mw = 7.1, MGR = 7 and a depth of focus of 131.4 km, and those of May 30 and 31, 1990, had Mw = 6.9 and 6.4, MGR = 6.7 and MGR = 6.2 and focal depths at 90.9 km and 86.9 km, respectively [91].

The map obtained for the earthquake of August 30, 1986 (Figure 3.16), highlighted two important characteristics specific to the seismicity of the Romanian territory: i) the general orientation of the ice accelerations in the NE-SW direction; and ii) the existence of large accelerations in the area of Focșani, similarly highlighted in the map obtained after the earthquake of May 30, 1990.

4. TECHNICAL REGULATIONS REGARDING SEISMIC EVALUATION AND DESIGN OF SOLUTIONS FOR CONSOLIDATION OF EXISTING BUILDINGS

4.1 International documents

Thus, starting from the existent literature [6], the most important technical documentations (codes, norms, standards, technical guides, manuals, etc.) are reviewed, regarding the seismic evaluation and rehabilitation of the existing constructions, including historical monuments.

- FEMA 356/2000 (similar ASCE 41-06): *Prestandard și comentarii pentru reabilitarea seismică a clădirilor*.
- FEMA 547/2007, *Tehnici pentru reabilitarea seismică a clădirilor existente*
- FEMA 172 / 1992 NEHRP, *Manual pentru reabilitarea seismică a clădirilor existente*
- ITALIA – *Ministery for Culture Heritage and Activities: Guidelines for evaluation and mitigation of seismic risk to culture heritage 2007*
- Noua Zeelandă – NZSEE 2005 *Assesment and improvement of the structural performance of building în earthquakes – publicat în 2005 în cadrul asociației New Zealand Society of Earthquake Engineering* (NZSEE)
- JBDPA 2001 Guidelines for seismic retrofit of existing reinforced concrete buildings, Japonia;
- etc.

4.2 National legislative and technical documents on seismic risk reduction

P100-3 / 2019 - Code for the evaluation and design of consolidation works for existing seismically vulnerable buildings [97]. This code, like its predecessor P100-3 / 2008, is in fact the elaborated / developed and updated form of code P100-92, chapters 11 and 12 and harmonized with Eurocode 8-Part III.

M.P. 025 / 04– Methodology for risk assessment and intervention proposals required for the construction of historic monuments in their restoration works [98]. This document, adopted on the basis of Order 743 / 19,04,2004, within MTCT (Ministry of Transports, Constructions and Tourism), elaborated within a scientific team within the “Ion Mincu” University of Architecture and Urbanism, Bucharest.

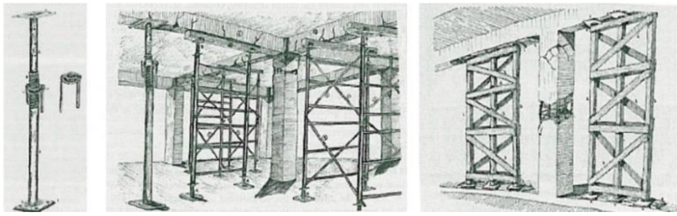
5. SOLUTIONS AND METHODS FOR CONSOLIDATION OF EXISTING BUILDINGS, SEISMIC VULNERABLE

Starting from Annex F, “Seismic Rehabilitation Guide for Existing Buildings”, P100-3 / 2019 [97] and the available literature [1, 2, 3, 4, 5, 6, 99, 100, 102, 103, 104, 105, 106, 107, 108], in this chapter we opted for a synthetic exposition of some classic / current and modern consolidation solutions, applicable to different structural types, including the existing buildings with load-bearing masonry structures, seismically vulnerable.

5.1 Levels of the consolidation process

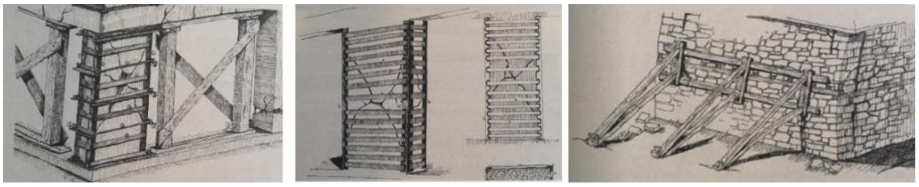
Levels of the consolidation process are: securing, repairing, strengthening, rebuilding. The securing can be done in the form of independent load-bearing elements, applied locally, or complex structures made at the level of a floor or even of the whole structural assembly, some examples being given below:

a) **adjustable wooden / metal props (telescopic)** - (Figure 5.1);
b) **corsets** - (Figure 5.2);
c) **buttresses** - (Figure 5.3)



a) b) c)

Figure 5.1 - a) b) Scaffolding made of adjustable metal props (telescopic) and wooden beams;
b) scaffolding made of wooden elements [1]

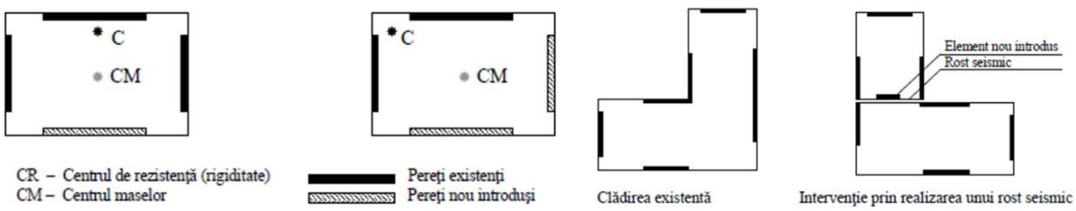


a) b) c)

Figure 5.2 - a) b) corsets made of wooden or metal elements to ensure the resistance elements of the buildings strongly cracked / cracked or broken by shear force; c) Buttresses to support the walls of the facade masonry [1]

5.2 Correction of construction deficiencies of existing constructions

Remedy for irregularities in the plan - interventions in this regard refer to the improvement of the seismic behavior of the structures with important overall torsion effect (Figure 5.4a), and can be made a structural delimitation by "*cutting*" the construction or achieving a seismic purpose (Figure 5.3b).

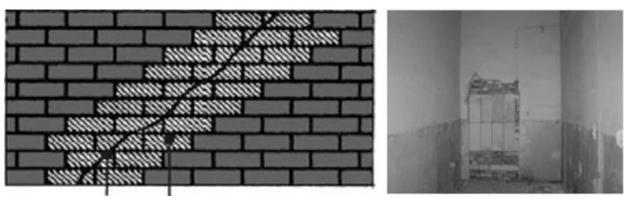


a) b)

Figure 5.3 - a) Remedy of the unfavorable effect of torsion, by introducing rigid and resistant elements; b) introduction of rigid and resistant elements and realization of a seismic joint [6]

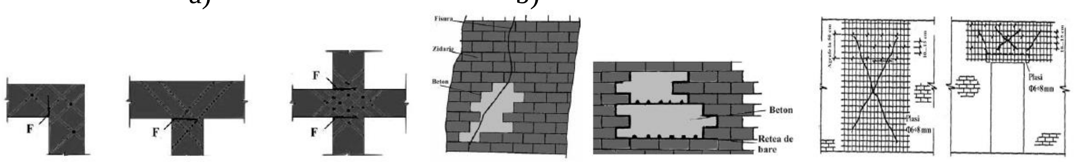
5.3 Repair work on masonry buildings

Repair works are recommended for historical monuments in situations where consolidation interventions cannot be possible without significantly affecting their architectural and historical value.



a) b)

Figure 5.4 - a) Restoration of a degraded masonry by recessing it; b) Restoration of masonry following the closing of a door / window gap [97]



a) b) c)

Figure 5.5 - a) Application of reinforced injections to corners / intersections / branches of walls (F - crack); b) concrete filling of masonry cracks; c) local plating with reinforced plaster [97]

5.4 Consolidation works for masonry structures

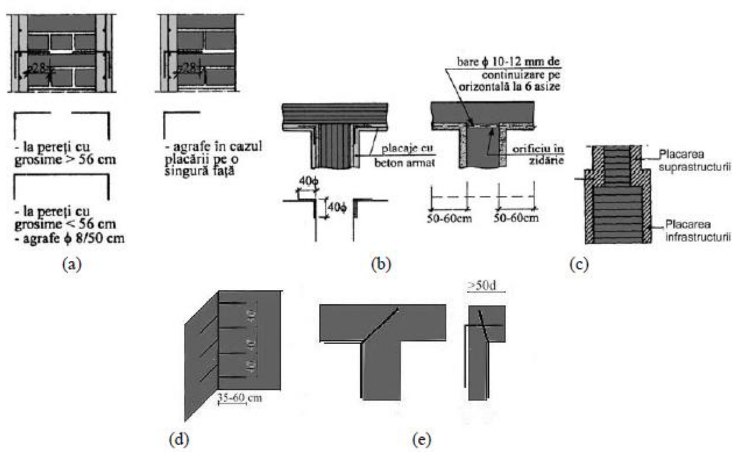


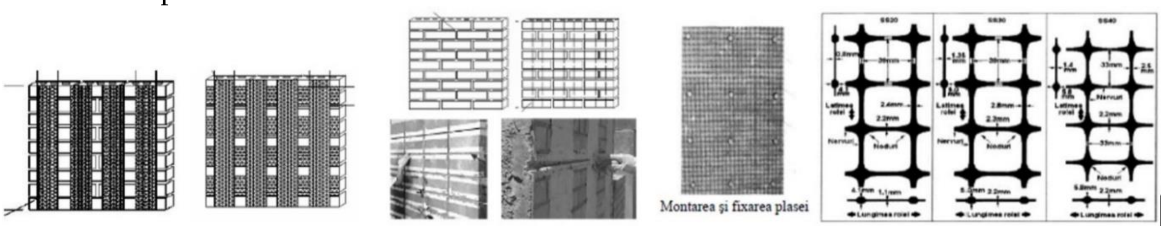
Figure 5.6 - Details of masonry cladding with plaster / reinforced concrete used in Romania:
a) detail in the current field;
b) detail at intersections;
c) detail in the area of modification of the wall thickness;
d) detail of mounting anchors at the corners - view;
e) detail of mounting anchors on the accounts - plan [97]

Reinforcement by concrete / reinforced mortar cladding with welded / welded steel nets (reinforced liners). This is a consolidation process widely used both in Romania and in other countries; this procedure is accompanied, in the case of cracks / cracks, by the injection, in advance, with one of the methods presented above, in the repair chapter (Figure 5.6).

a) b) c) d)

Figure 5.7 - a) reinforced concrete belt / bearing type element, provided for ensuring the embedding of the reinforced concrete jacket; b) the new reinforced concrete slabs will be supported in the "teeth" system; c) at the level of the gaps, new reinforced concrete buinadrugi must be provided; d) reinforced concrete belt housing, provided for the masonry load-bearing wall, from the lower floor (ground floor)

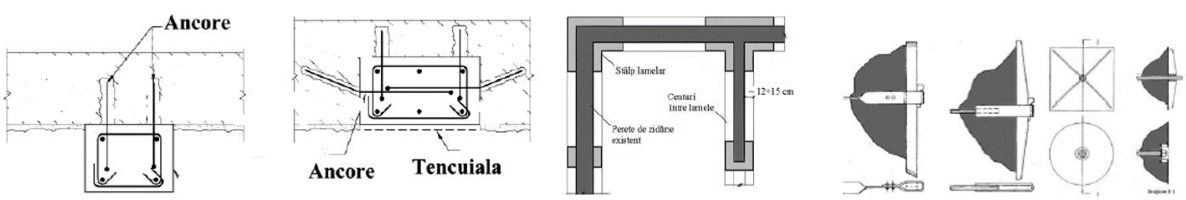
Consolidation by plating with fiber-reinforced polymer (FRP) products [101, 103] is a repair / consolidation process used in Romania as well.



a) b) c)

Figure 5.8 - a) Plating masonry walls with FRP strips / flat strips; b) FRP round bars; c) FRP polymer mesh or grids [97, 101, 103]

Consolidation of masonry by inserting belts and pillars. This method of reinforcement consists in the introduction of belts and pillars of reinforced concrete either apparent or embedded in the existing masonry (Figure 5.9a, b).



a) b) c) d)

Figure 5.9 - Consolidation of masonry structures with reinforced concrete pillars and belts - a) external pillar, apparent; b) pillar embedded in the masonry and with a thickness less than that of the wall; c) Consolidation of masonry with lamellar pillars; d) Tie rod anchor parts (anchor plates) with aesthetic forms [1, 97]

Strengthening the connections between walls and floors.

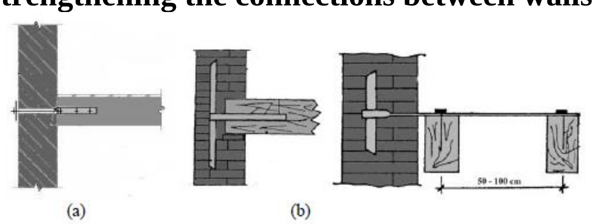
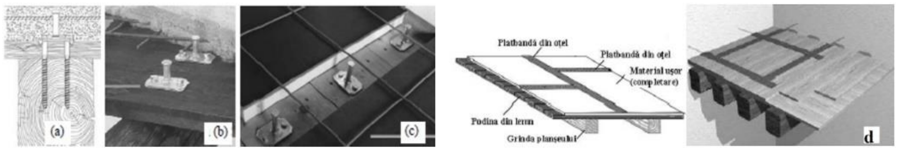


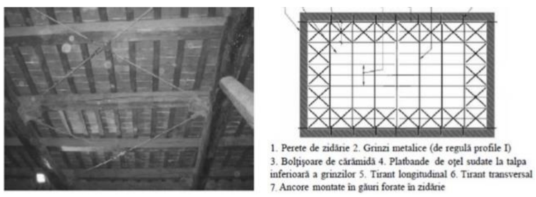
Figure 5.10 - Strengthening the connections between walls and floors - a) by introducing apparent anchors; b) by inserting the anchors in the thickness of the walls [1, 97]

The consolidation of the wooden floors



a) b) c) d)

Figure 5.11 - Consolidation of wooden floors - a) cross section of the floor; b) over-concrete executed directly on the wooden floor; c) over-concrete executed on a thermal insulation layer; d) Reinforcement of wooden floors with metal strips [1, 97]



a) b)

Figure 5.12 - Reinforcement of floors: a) made of wood with metal tie rods (braces in "X"); b) of metal profiles and brick vaults with metal tie rods ("X" braces) [6]

Other solutions practiced: reinforcement by addition with new boards / cabinets placed at 45^{o} in relation to the floor beams; strengthening the masonry construction infrastructure.

The procedures for consolidating the foundations

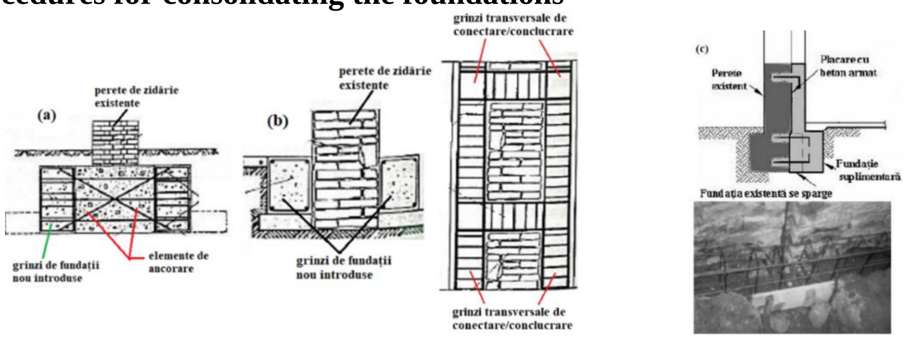


Figure 5.13 - a), b) Consolidation of foundations by widening their base in solutions; c) Reinforcement of foundations by reinforced concrete cladding [1, 97]

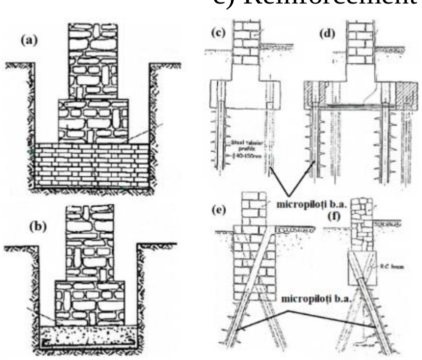
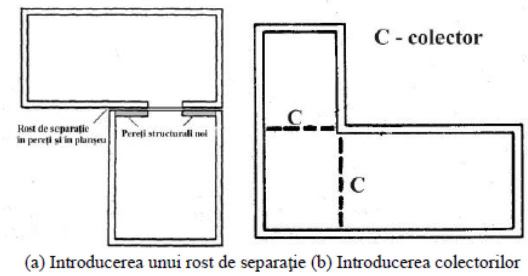


Figure 5.14 - Consolidation of foundations by subsidization - a) made of brick; b) made of monolithic reinforced concrete; c), d), e), f) by using drilled micro-piles [1]

5.5 Consolidation works with modification of the existing structural composition

Interventions in order to eliminate the eccentricity of the center of rigidity: introduction of new walls in positions as far as possible from the center of rigidity of the floor; increasing the rigidity of the contour walls by closing some gaps; elimination of effort concentrator areas; the introduction of the separation joints (Figure 5.15).



Figură 5.15 - Corectarea deficiențelor de alcătuire de ansamblu în soluție a) sau b) [97]

Interventions to ensure the route of vertical and seismic forces to the foundations are necessary especially in the cases of: a) structural walls that are not continued to the foundations; b) when the connections between the floor and some walls are broken on long lengths (eg the stairwell next to the wall); c) when the perimeter belts are not continuous (eg the belt from the floor to the stairwell is missing).

To correct this deficiency, the following intervention solutions can be adopted: introduction of structural walls and / or pillars, and completion with a system of belts.

Interventions to increase the redundancy of the structure. In the case of structures that do not meet the redundancy requirements imposed by the technical regulations in force, P100-1 / 2013, interventions in this regard may be:

i) the addition of new structural elements (walls, masonry / concrete or metal pillars) in areas where the rupture of a single element may cause the loss of the general stability of the structure (eg the case of shovels / pillars with insufficient sectional dimensions or lack of lintels on windows and doors), Figure 5.16;

ii) improving the ductility capacity through adequate consolidation works.

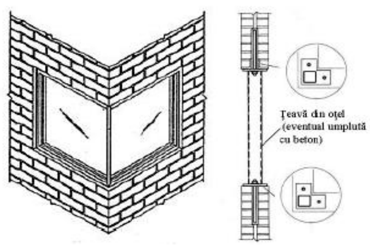
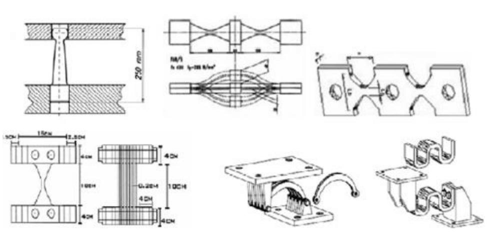


Figure 5.16 - Reinforcement of corners if the masonry rests directly on the window structure [97]

5.6 Seismic damping systems

Displacement-dependent devices that take into account the ductility properties of metals. Their properties are closely related to the deformation capacity of the constituent material (steel, lead and some other special alloys); they can have different shapes: pivot, crescent, butterfly, rail, plate, triangular or in "X" (Figure 5.17). These devices are also called shock absorbers, which can be configured to plasticize at axial stresses, shear forces and bending moment.



a)

b)

Figure 5.17 - a) Types of ductile metal dampers; b) Seismic motion damping device, made in the form of supports for the metal farms of an Industrial Hall, Salermo, Italy [99]

a) b) c)

Figure 5.18 - a) Pall-type seismic motion damping device, used in the composition of "X" braces - Patient Tower, Seattle, USA; b), c) Damptech seismic motion damping device, used in the composition of bracing - Yaguriji Temple, Japan [99]

a) b)

Figură 5.19 - Amortizori vâscoși în diverse sisteme de antenuare a energie seismice [98]

5.7 Base insulation systems

The fundamental principle of isolating the base of a building is to radically change the response of the structure so that the influence of land movement on the site is minimal or even zero (the land moves under construction without transmitting its movement). The ideal of isolating the support base would be to completely detach / separate the land structure, but in reality this cannot be possible, as a minimum number of structure-land contact areas are required [99, 104].

Seismic isolation can be provided with or without additional damping devices. They can be classified, depending on the materials used and how they are made, as follows [97]:

- **low damping insulators**;
- **High damping rubber insulator** (HDRB);
- **rubber core with lead core** (LRB);

6. TECHNICAL EXPERTISE OF EXISTING CONSTRUCTIONS

6.1 Generalities

The technical expertise of the existing constructions aims to identify the levels of fulfillment of the fundamental safety requirements in operation, provided according to the legislation in force “Law 10/1995, on quality in constructions”.

According to article (18), paragraph (2), “Interventions to existing constructions refer to construction works, reconstruction, partial demolition, consolidation, repair, modernization, modification, extension, rehabilitation, thermal rehabilitation, [….].

Table 6.1 - Expertised and framed buildings in Bucharest in seismic risk classes in the period 1993-2013 [115]

Municipiul București	Clasa de risc seismic				
	RsI - pericol public	RsI	RsII	RsIII	RsIV
Imobile	190	184	302	75	6
Apartamente	5363	1276*	11070	1781	86

Table 6.2 - Updated statistics of expert buildings in the period 1993-2021

Nr. total clădiri expertizate	Statistică în funție de perioade în care au fost construite/proiectate clădirilor			
		<1941	[1941; 1977]	>2006
1258	nr. clădiri	1038	194	26
	%	82,51	15,42	2,07
	Statistică în funție de perioada în care au fost expertizate (%)			
		1993	[1994; 2006]	>2006
	nr. clădiri	776	339	143
	%	61,69	26,95	11,37

Table 6.3 - Buildings in Bucharest expertized and included in the seismic risk class Rs I-public danger, in the period 1993-2013 [115]

Perioada / Nr. etaje	< 1900	1901-1910	1911-1920	1921-1930	1931-1940	1941-1950	1951-1960	> 1960	Total
1 etaj	6	-	-	-	-	-	-	-	6
2 etaje	13	3	1	1	-	-	-	-	18
3 etaje	10	1	1	3	1	-	-	-	16
4 etaje	5	2	-	1	11	-	-	-	19
5 etaje	2	-	-	8	16	1	-	-	27
6 etaje	2	2	2	3	17	-	-	-	26
7 etaje	-	-	1	6	24	1	-	-	32
8 etaje	-	-	-	2	16	4	-	-	22
9 etaje	-	-	-	1	10	1	2	3	17
10 etaje	-	-	-	-	3	-	1	1	5
>10 etaje	-	-	-	-	2	-	-	-	2
Total	38	8	5	25	100	7	3	4	190

6.2 Specific concepts and terminology

Levels of knowledge refer to the extent to which the available technical information (partial / incomplete / complete) either from existing technical documents (technical book) or as a result of technical inspections (findings) in the field, ensures an optimal and realistic level of confidence. numerical results specific to quantitative evaluations (structural calculations).

The code provides and defines three levels of knowledge (Table 6.4): i) KL1 = 1.35 - limited knowledge; KL2 = 1.20 - normal knowledge; KL3 = 1.00 - complete knowledge (Table 6.4).

Seismic assessment methodologies consist of all approaches and criteria for qualitative and quantitative assessment (calculation methods), which establish the seismic performance of existing buildings. Code P100-3 / 2019 [97] provides three methodologies differentiated by the level of detail and complexity of the qualitative and quantitative evaluation criteria, provided in the code.

The seismic calculation is made according to the provisions of code P100-1 / 2013. Depending on the structural characteristics and the importance of the construction subject to technical expertise, the seismic design code P100-1 provides for the structural calculation, the following methods: i) the method of lateral forces associated with the fundamental vibration mode; ii) method of modal calculation with response spectra;

Other less common methods for current seismic design and assessment practice are*: i) linear dynamic calculation method; ii) nonlinear static calculation method; iii) nonlinear dynamic calculation method*.

6.3 Instrumental determination of vibration periods for buildings

A direct and practical method by which the value of the fundamental proper period of vibration of the structure of a building can be determined is known in the literature / various studies [10 ÷ 45], as the method "*Ambient Vibration Tests*" (AVT) or the method of environmental vibrations".

The study was coordinated by Prof. Dr. Eng. Petre Trofin, who was at that time vice-rector for science, and by Prof. Stefan Bălan (academician and head of the department of theoretical and applied mechanics at the Bucharest Construction Institute-Faculty of Construction) , together with Assoc. Prof. Constantin Zeveleanu, Chief of Works Dr. Sorin Larionescu, Eng. Alexandru Dobrescu and Eng. Nicolae Dimitriu, the work being done based on the scientific research contract no. 4108/1971 (I.C.B.).

The object of the study was the church of Arnota Monastery (Figure 6.1), historical monument registered in LMI with LMI code VL-II-a-A-09667, located in Bistrița village, Costești commune, Vâlcea county.

The reason for the study was the physical state of progressive degradation in which the church structure was (multiple cracks / dislocations of the mural, Figure 6.2), due to shocks (microseism) caused by explosions during the exploitation of the Pietreni-Bistrita limestone quarry.

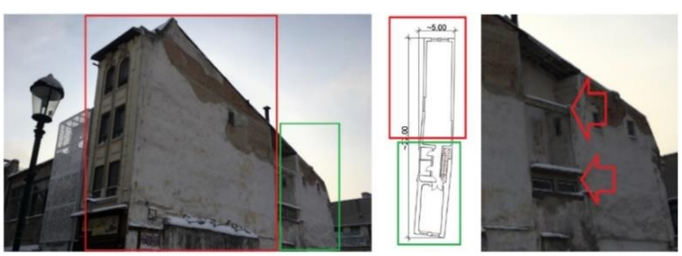
6.4 Case study 1. Evaluation of the dynamic response of an existing construction, using numerical methods (ETABS modeling), direct methods (empirical) and AVT method (Ambient Vibration Tests).

The main objective of this case study is to evaluate the dynamic response of the structure of an existing building, located in the old center of Bucharest, Str. Lipscani no. 66, having height regime Sp + P + 2E + M, with overall dimensions in plan of ~ 5x27 m, and atypical, elongated shape (ratio ~ 1: 6).

The building (Figure 6.4) was built around 1930, has the destination of "house" and is included in the List of Historical Monuments in Romania, being registered with the code B-II-B-19041 [119]. At the time of the study, there was no technical construction book and no history of its design, execution and operation, but, as it was found, from a structural point of view, it has a mixed composition.

a) b)

Figure 6.4 - a) Main facade; b) Side façade [119]



a) b) c)

Figure 6.5 - a), b) Identification of building sections; c) Exterior elements of continuity outside the building - reinforced concrete beams [119]

There were no serious structural degradations, but the floor slabs had multiple cracks (Figure 6.6), and there was also an advanced level of aging of the materials, especially of the masonry mortar and plasters of significant grading.

Figure 6.6 - Cracks in the floor tiles of the building [119]

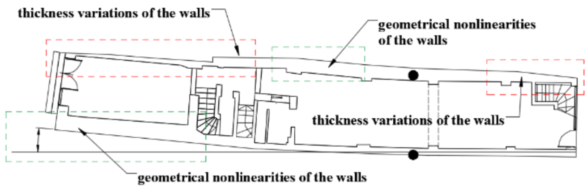


Figure 6.7 - Plan variations of wall thicknesses and geometric nonlinearities

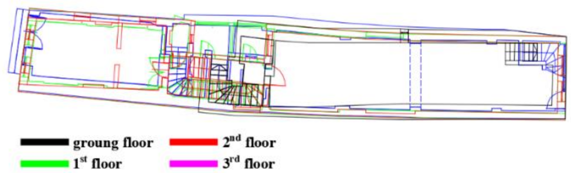


Figure 6.8 - Vertical variations of wall thicknesses and geometric nonlinearities

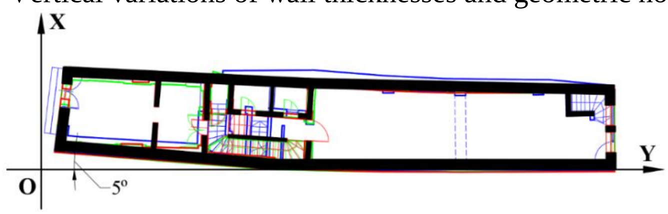


Figure 6.9 - Deviation in plane from rectilinearity by ~ 5°

Determination of the fundamental proper period of vibration by numerical methods.

The determination of the compressive strength of reinforced concrete was used using the sclerometry method, using a specific device (Figure 6.17a), and the compressive strength of the brick masonry was also determined by the sclerometry method (Figure 6.17b), using a specific device for masonry. Evaluations of the quality of the concrete were also performed, using the ultrasonic method (Figure 6.17c).

a) b) c)

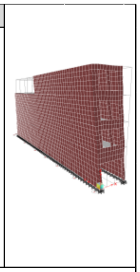
Figure 6.11 - a) Sclerometry method used to determine the compressive strength of reinforced concrete; b) sclerometry method (8x14x28 cm brick masonry elements); c) Ultrasonic method for determining the quality / possible defects of reinforced concrete [119]

Based on the mechanical characteristics determined by the non-destructive tests mentioned above, the following resulted: the modulus of elasticity of reinforced concrete class C8 / 10 in beams and slabs, is Ec, b, s = 25000 N / mm2; the modulus of elasticity of reinforced concrete class C12 / 15 in columns is Ec, c = 27000 N / mm2; a specific gravity γc = 24 kN / m3 was considered. To determine the modulus of elasticity of masonry, the general empirical formula below was used, according to CR6-2013:

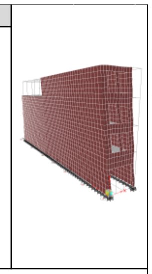
$$E_z = \alpha * f_k \tag{6.1}$$
$$f_k = k * f_b^{0.7} f_m^{0.3} \tag{6.2}$$

For numerical modeling, using the finite element analytical program, ETABS, three reference hypotheses were formulated (Table 6.4, Table 6.5, Table 6.6), which, by combining them, by reducing the stiffnesses of the $E_{c,c}I_c$ columns, the stiffness of the beams, of the plates $E_{c,b,s}I_b$, and of the masonry E_zI_z, 14 final hypotheses were obtained (Table 6.7). Also, in Table 6.8 are presented the results of numerical modeling - ETABS, in terms of fundamental periods of vibration T * [s]

Tabel 6.4 – Reference hypothesis 1

Materials characteristics				T^*[s]	
concrete from columns	C12/15 class		$E_{c,c}$=27000 N/mm^2	0,31	
	concrete stiffness's columns		$E_{c,c}I_c$		
concrete from beams and slabs	clasă C8/10		$E_{c,b,s}$=25000 N/mm^2		
	concrete stiffness's beams and slabs		$E_{c,b,s}I_{b,s}$		
masonry	brick type C150		E_z [N/mm^2]		
	f_b[N/mm^2]=	15	2781		
	lime mortar M4				
	f_m[N/mm^2]=	0,4			
	$k_{ceramic\ elements}$=	0,55			
	α=	**1000**			
	masonry stiffness	E_zI_z			
T^* - fundamental vibration period of the building from numerical model ETABS					

Tabel 6.5 – Reference hypothesis 13

Materials characteristics				T^*[s]	
concrete from columns	C12/15 class		$E_{c,c}$=27000 N/mm^2	0,57	
	concrete stiffness's columns		$E_{c,c}I_c$		
concrete from beams and slabs	clasă C8/10		$E_{c,b,s}$=25000 N/mm^2		
	concrete stiffness's beams and slabs		$E_{c,b,s}I_{b,s}$		
masonry	brick type C150		E_z [N/mm^2]		
	f_b[N/mm^2]=	15	1947		
	lime mortar M4				
	f_m[N/mm^2]=	0,4			
	$k_{ceramic\ elements}$=	0,55			
	α=	**700**			
	masonry stiffness	E_zI_z			
T^* - fundamental vibration period of the building from numerical model ETABS					

Tabel 6.6 – Reference hypothesis 14

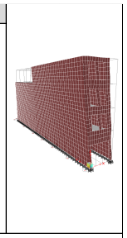
Materials characteristics				T^*[s]	
concrete from columns	C12/15 class		$E_{c,c}$=27000 N/mm^2	0,71	
	concrete stiffness's columns		$E_{c,c}I_c$		
concrete from beams and slabs	clasă C8/10		$E_{c,b,s}$=25000 N/mm^2		
	concrete stiffness's beams and slabs		$E_{c,b,s}I_{b,s}$		
masonry	brick C150		E_z [N/mm^2]		
	f_b[N/mm^2]=	15	644		
	lime mortar				
	f_m[N/mm^2]=	**0,01**			
	$k_{ceramic\ elements}$=	0,55			
	α=	**700**			
	masonry stiffness	E_zI_z			
T^* - fundamental vibration period of the building from numerical model ETABS					

Table 6.7 - Fundamental own periods, obtained in all 14 hypotheses,
including reference hypotheses 1, 13 and 14

Numerical models (ETABS)											
Model i		**E_z; $E_{c,c}$; $E_{c,b,s}$ α (from Table 1)**			**T_i[s]**	**Model i**		**E_z; $E_{c,c}$; $E_{c,b,s}$ α (from Table 2)**			**T_i[s]**
hipothesis	1	E_zI_z	$E_{c,c}I_c$	$E_{c,b,s}I_{b,s}$	**0,31**	hipothesis	8	E_zI_z	$0,5E_{c,c}I_c$	$E_{c,b,s}I_{b,s}$	**0,43**
	2	E_zI_z	$0,5E_{c,c}I_c$	$E_{c,b,s}I_{b,s}$	**0,37**		9	E_zI_z	$E_{c,c}I_c$	$0,5E_{c,b,s}I_{b,s}$	**0,42**
	3	E_zI_z	$E_{c,c}I_c$	$0,5E_{c,b,s}I_{b,s}$	**0,38**		10	E_zI_z	$0,5E_{c,c}I_c$	$0,5E_{c,b,s}I_{b,s}$	**0,44**
	4	E_zI_z	$0,5E_{c,c}I_c$	$0,5E_{c,b,s}I_{b,s}$	**0,40**		11	$0,5E_zI_z$	$0,5E_{c,c}I_c$	$E_{c,b,s}I_{b,s}$	**0,54**
	5	$0,5E_zI_z$	$0,5E_{c,c}I_c$	$E_{c,b,s}I_{b,s}$	**0,48**		12	$0,5E_zI_z$	$E_{c,c}I_c$	$0,5E_{c,b,s}I_{b,s}$	**0,53**
	6	$0,5E_zI_z$	$E_{c,c}I_c$	$0,5E_{c,b,s}I_{b,s}$	**0,46**		13	$0,5E_zI_z$	$0,5E_{c,c}I_c$	$0,5E_{c,b,s}I_{b,s}$	**0,57**
	7	$0,5E_zI_z$	$0,5E_{c,c}I_c$	$0,5E_{c,b,s}I_{b,s}$	**0,50**		14	$0,5E_zI_z$	$0,5E_{c,c}I_c$	$0,5E_{c,b,s}I_{b,s}$	**0,71**

Determination of the fundamental fundamental period of vibration using direct methods starting from the literature [7] there are a number of direct (empirical) methods that provide formulas for calculating the fundamental period of vibration for different structural types or formulas generally valid for all types of structures. For the present case study, the most appropriate direct methods were considered those in Table 6.8, in which the results obtained by each method can be observed.

Table 6.8 - Direct (empirical) methods that can be used for determination of the fundamental proper period of vibration of structures [7]

Nr. crt.	Direct/empirical methods	Formula	Empirical formulas	Building's conformation characteristics			
				n	B [m]	L [m]	H [m]
				4	5	27	13,7
				L_{walls} [m]	A_{walls} [m^2]	d [m^{-1}]	
				218	430	0,57	
				T [s]			
				T_{min}	…		T_{max}
1	**T. Taniguchi method:** The method is based on experimental investigations done on a large number of buildings from Tokyo and Yokohama. The formula 1.1 it was validated for all types of buildings. Also, were validated other two forumulas 1.2 and 1.3 as alternativ ones.	1.1	T=(0.12…0.40)√((2n+1)/3)=	0,21	…		0,69
		1.2	T=(0.07…0.09)n =	0,28	…		0,36
		1.3	T=(0.06…0.10)(n+0,5)=	0,27	…		0,45
2	**F.P. Ulrich and D.S. Carder method:** The method's formulas were obtained based on experimental measurements on 400 buildings, with various structural types.	2.1	T=(0.01…0.035)H=	0,14	…		0,48
		2.2	T=~0.02H=	0,27			
3	**E. Rosenblu - ETH method:** It is recommended only for residencial and office buildings.	3	T=(0.09…0.10)(n+1)	0,45	…		0,50
4	**K. Nakagava method:** The method's formulas are based on the experimental measurements on 53 buildings, tajing account of the ratio H/√B factor.	4.1	T=(0.07…0.13)H/√B	0,43	…		0,80
		4.2	T=(0.10+0.038n)…(0.20+0.064n)=	0,25	…		0,46
5	**M. Takeuchi method:** The method's formula is based on the experimental measurements on 60 buildings from Tokyo and Osaka.	5	T=H/60=	0,23			
6	**A. Arias and R. Husid method:** The method's formula it was validated for buildings from Chile, with RC frames structures and concrete or fill masonry walls.	6	$T=0.024H^{0.71}d^{-0.14}$=	0,17			
7	**M. Baeza method:** The formula's method it was determined for various buildings from Chile.	7	T=0.036n =	0,14			
8	**A. Arias, R. Husid and M. Baeza method:** The method's formulas are based on the experimental results on 34 tall buildings (4...17 stories), from Santiago and Valparaiso citie.	8.1	T=0.012H=	0,16			
		8.2	T=0.035n=	0,14			
9	**H. Sandi and G. Serbanescu method:** The method's formulas were determined based on exeprimental measurements on Romanian buildings, from Bucharest city. The formula 9.1 was validated for transversal period assessment and the formula 9.2, was validate for tall building.	9.1	T=(0.045…0.055)n=	0,18	…		0,22
		9.2	T=0.065 H/√B=	0,40			
10	**J. S. Carmona and J. H. Cano method:** The method's formulas experimentaly obtained for buildings from Argentina. The formula 10.1 was validate for RC frames structures and the formula 10.2 was validate for RC frames and masonry fill walls structures.	10.1	T=0.012H+0.09 =	0,25			
		10.2	T=0.07 H/√B=	0,43			
		10.3	T=H√(0.003/B)+0.0002/(1+30d)=	0,34			
11	**R. Husid, W. Pieber and J. Romo method:** The method's formulas 11.1 and 11.2 were validated for current RC frames structures, and the formula 11.3 was validated for buildings from Chile.	11.1	T=0.04n =	0,16			
		11.2	T=n/69=	0,06			
		11.3	$T=0.04\ H/\sqrt[3]{B}$=	0,32			

12	**M. Ifrim method:** The method's formula was validated for buildings with RC frames and masonry walls structures, with ≤ 6 stories.	12	T=0.09√H=	**0,33**
13	**JOINT COMMIT-TEE ASCE-SEA method:** The method's formula is based on experimental measurements done on 3000 buildings and it was validated for all types of structures.	13	T=0.09 H/√B =	**0,55**
14	**U.S. COAST AND GEODETIC SURVEY method:** The method's formula was validated based on experimental investigations on 212 buildings.	14	T=0.1n =	**0,40**
15	**OFFICE OF CONSTRUCTION VETERAN ADMINISTRATION WASHINGTON method:** The method's formula 15.1 was validated for masonry structures.	15	T=0.15n =	**0,60**
16	**Empirical formulas from Romanian seismic code P100-81:** The formula it was validated for all type of buildings (RC frames or masonry walls).	16	T=0.3+0.05n=	**0,50**
17	**Empirical formulas from Romanian seismic code, P100-1/2013:** In the Annex B (B.2), are recommended some empirical method, validated for buildings with height ≤ 40 m: $T=C_t\ H^{\wedge}(3/4)$, in which $C_t=0.05$ – for masonry structures or other similar.	17	T=0.05H^(3/4)=	**0,36**

In which: T - the fundamental proper period of the structure; n - the number of levels of the building; d - coefficient by which the influence of "wall density" is introduced and represents the ratio between the total length of all existing walls in the building and its total developed surface; B - the width of the structure on the ground floor; L - length of the structure on the ground floor; H - height of the building.

Determination of its own fundamental vibration period using the AVT (Ambient Vibration Tests) method. The dynamic recordings were made with the technical support of Mr. Assistant University. dr. eng. Teodor Pavlu, having at his disposal an equipment made up of the main station MT WHITNEY KINEMETRICS (Figure 6.18a) with 18 channels, called "St"; 4 sensors (accelerometers), which were named "S1", "S2", "S3" and "S4" (Figure 6.18b), and a control sensor (backup epicenter) ESP U3 (Figure 6.18c), called "E "; each sensor was designed to make recordings in all 3 orthogonal directions (triaxial sensors / accelerometers).

a) b) c)

Figure 6.12 - a) MT WHITNEY KINEMETRICS station with 18 channels; b) KINEMETRICS type triaxial sensor / accelerometer; c) ESP-U3 epicenter, with backup role

Also available as a backup was another station (ALTUS K2 KINEMETRICS) and a special software for processing dynamic records (accelerograms), called QUICK TALK KINEMETRICS.

The four sensors "S1", "S2", "S3" and "S4" and the episensor "E", were placed at the level of each floor, in 4 configurations (Figure 6.13, Figure 6.14, Figure 6.15 and Figure 6.16). The main station was located on the ground floor above the ground floor, throughout the recordings.

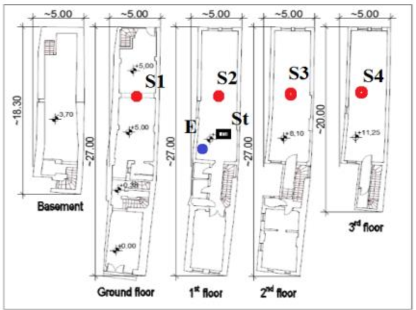


Figure 13 – 1st sensors' configuration

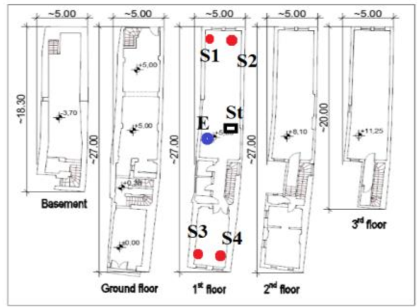


Figure 14 – 2nd sensors' configuration

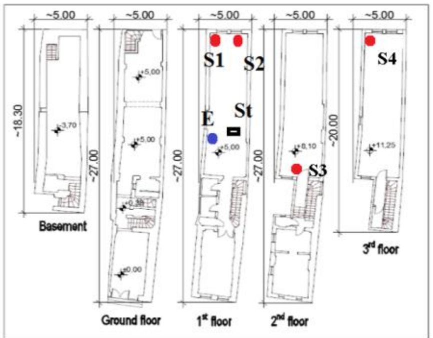


Figure 15 – 3rd sensors' configuration

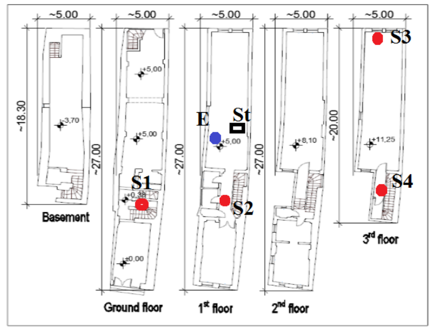


Figure 16- 4th sensors' configuration

The processing of the recorded data was done with the ARTEMIS MODAL PRO 4.0 software (license No. 7030). Thus, the fundamental vibration period of the analyzed building structure resulted in T1AVT = 0.85 s, value associated with the transverse vibration mode.

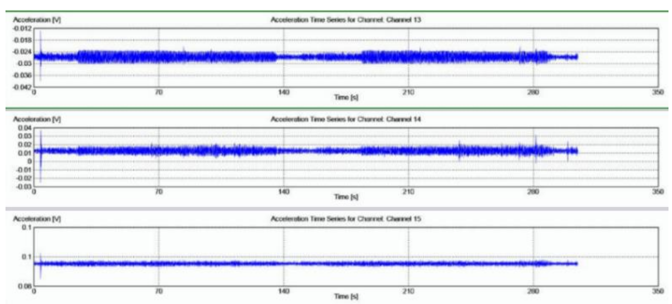


Fig. 6.17 – Recordings (accelerograms) obtained at floor level above floor 3

Comments:

- the maximum value of the fundamental eigenperiod resulting from the numerical modeling of the structure (ETABS), is 0.71 s (hypothesis 14), being an extreme hypothesis, in which a 50% reduction of the rigidities of all structural elements (pillars, beams, plates, with cracks), the coefficient α = 700 (recommended for old masonry structures) and a compressive strength of mortar f_m = ~ 0.001N / mm^2, considered to be associated with a mortar with a high level of aging and degradation /friable);
- the maximum value of the fundamental eigenperiod resulting from the use of direct (empirical) methods is 0.60 s, obtained with the formula of the method offered by the Office of construction veteran administration Washington, formula validated for constructions with masonry load-bearing wall structures;
- the value obtained from the dynamic recordings resulted in 0.85 s (vibration in the transverse direction);

- in this situation, it can be observed that an average of the values obtained by analytical modeling is 0.47 s, and an average of the values obtained by direct (empirical) methods is 0.32;
- it can be observed that in hypothesis 7 of numerical modeling, the value 0.50 s was obtained considering a 50% reduction of the stiffnesses of all structural elements (columns, beams and plates), an α = 1000 and a compressive strength of mortar f_m = ~ 0.4 N / mm^2;
- in hypothesis 13 numerical modeling, the value of the period was 0.57 s, considering a 50% reduction in the stiffness of all structural elements (columns, beams and slabs), an α = 700 (recommended for old masonry structures) and a compressive strength of mortar f_m = ~ 0.4N / mm ^{2}.

The total reduction by 50% of the rigidities of all structural elements, in the current design refers to the final stage (cracked), which would involve a physical condition of the structure materialized by multiple cracks / cracks, but at the time of the study, the structure did not show significant structural degradation, therefore a 50% reduction in stiffness is not realistic, therefore hypotheses 7 and 13 cannot be considered compatible with reality. On the other hand, the value of the dynamic recording 0.85 s far exceeds the values obtained in hypotheses 7 and 13, and an average of the values of the other hypotheses and methods would be ~ 0.40 s.

For the situation of the present case study, it can be admitted that the maximum plausible value of the fundamental fundamental vibration period can be found in the range [0.40; 0.57], considering the value 0.85 s resulting from the dynamic records as one affected by the errors inherent to this method.

6.5 Case study 2. Assessment of the own fundamental vibration periods of existing constructions by numerical methods, using an automatic calculation program (ETABS) and by direct (empirical) methods

In the present case study, the results of the calculations of the own fundamental periods of vibration are presented, performed for 11 existing buildings, with various resistance structures, located in several more important seismic cities in Romania.

The calculation of the period values was performed both using numerical methods, using an automatic calculation program (ETABS), and some of the direct methods (12 methods), Table 6.9, existing in the literature [7], suitable for each structural type. The purpose of this case study is to highlight the viability of the methods analyzed by comparison.

Table 6.9 - Direct (empirical) methods that can be used to determine the fundamental proper period of vibration of structures

Nr. crt.	Direct / empirical methods	Formula	Calculation formula Ti [s]
M1	**F.P. Ulrich and D.S. Carder:** The method is based on statistical analyzes performed on the measurements of 400 buildings, with different resistance structures.	1	T_1=0.02H
M2	**M. Takeuchi method:** The formula of the method is based on the experimental results for 60 buildings in Tokyo and Osaka and depends exclusively on the H height of the building.	2	T_2=H/60
M3	**M. Baeza method:** The formula of the method was validated only for constructions in Chile.	3	T_3=0.036n

M4	**Horia Sandi and G. Serbanescu method:** The calculation relationship was proposed for the tower blocks.	4	T_4=0.065 H/√B
M5	**J. S. Carmona and J. H. Cano method:** The formulas of the method were determined experimentally on buildings in Argentina, with reinforced concrete structures with masonry walls, with the role of stiffening.	5.1	T_{51}=0.012H+0.09
		5.2	T_{52}=0.07 H/√B
M6	**Method R. Husid, W. Pieber and J. Romo:** Formulas 6.1 and 6.2 are recommended for ordinary reinforced concrete structures.	6.1	T_{61}=0.04n
		6.2	T_{62}=n/69
M7	**M. Ifrim method:** Formula 7.1 has been validated for low constructions with masonry load-bearing wall structure and reinforced concrete frames with masonry filling walls with a height regime less than or equal to 6 levels; formula 7.2 was validated for reinforced concrete constructions with average height regime in the range of 7 ... 15 levels.	7.1	T_{71}=0.09√H
		7.2	T_{72}=0.12√H
M8	**JOINT COMMITTEE ASCE-SEA method:** The formula of the method is based on experimental measurements of 3000 buildings with various structural types and is also recommended by the U.B.C.	8	T_8=0.09 H/√B
M9	**U.S. method COAST AND GEODETIC SURVEY:** The formula was determined based on experimental investigations on 212 buildings. The formula is also recommended by the SEISMOLOGY COMMITTEE SEAOC	9	T_9=0.10n
M10	**OFFICE OF CONSTRUCTIONS VETERAN ADMINISTRATION WASHINGTON:** Formula 10.1 is recommended for load-bearing masonry constructions, and formula 10.2 has been validated for reinforced concrete constructions.	10.1	$T_{10.1}$=0.05n
		10.2	$T_{10.2}$=0.08n
M11	**Formulas according to the Romanian seismic design norm, indicative P100-81:** The Romanian seismic design norm, indicative P100-81, recommends formula 11 for constructions with a height regime less than or equal to 5 levels.	11	T_{11}=0.3+0.05n
M12	**Formulas according to the Romanian seismic design norm, indicative P100-1 / 2013:** In Annex B, B.2, for the preliminary design of buildings with a total height of up to 40 m, the following formulas are recommended: T = Ct H 2 (3/4), wherein Ct = 0.075 - for reinforced concrete space frames and metal space frames and eccentric braces; Ct = 0.05 - for other types of buildings, than those with reinforced concrete or metal structure	12.1	$T_{12.1}$=0,075* H^(3/4)
		12.2	$T_{12.2}$=0,05* H^(3/4)

In which: T - the fundamental proper period of the structure; n - the number of levels of the building; d - coefficient by which the influence of “wall density” is introduced and represents the ratio between the total length of all existing walls in the building and its total developed surface; B - the width of the structure on the ground floor; L - length of the structure on the ground floor; H - height of the building.

Modeling hypotheses considered. Each objective has been introduced in the numerical modeling program (ETABS), taking into account the following aspects of numerical modeling:

- the resistance structures of the buildings were completely introduced up to the level of embedding in the ground (load-bearing masonry walls, belts and reinforced concrete floor slabs), but except for the non-structural walls, with a partitioning role, having thicknesses less than 14 cm;
- the wooden floors were introduced as surface-type finished elements, defined with an equivalent thickness and a modulus of elasticity associated with the wood essence from which they were made, and with unloading in one direction;
- the structures were loaded at the level of floors with payloads, permanent / quasi-permanent (screeds, floors, partitions, furniture, specific equipment, installations);
- the snow load was distributed at the level of the last floor, terrace / attic support;
- the loads from the roof and roof structure were distributed linearly both on the contour of the external walls, on which it rests, and in certain concentrated points, at the level of the floor, on which the supports rest;
- both the concrete and the wooden floors were considered to satisfy satisfactorily the condition of “rigid washer”;

- the modulus of elasticity of the materials (masonry, concrete) were calculated based on the mechanical characteristics (compressive strength of concrete / masonry elements-pressed solid brick, brick with vertical gaps), determined by destructive tests, performed in an authorized laboratory, accompanied by non-destructive tests (sclerometry);
- the values of the elastic modulus of the different wood essences, from which floors were made of wooden elements, were considered the usual ones, recommended in the technical documents SR EN 338 [120] and NP 005 [121].

6.6 Case study 3. Technical expertise and consolidation of an existing historical monument construction

This case study is the subject of technical expertise of an existing construction, located in Bucharest. The technical expertise was prepared in an office of expertise and technical design in construction, a work in which the author of this doctoral thesis was directly and actively involved in all specific phases, including structural calculation.

The main objectives, pursued in this case study, are:

- exemplifying the way of classifying a building in seismic risk class;
- design and comparative study of consolidation solutions compatible both with the requirements of strength and stability, and with those imposed by the status of historical monument; the comparative study aimed to highlight important aspects such as:
  - advantages and disadvantages of structural performance, especially seismic;
  - technological advantages and disadvantages (necessary execution times and specific resources);
  - economic advantages and disadvantages, compared to structural and technological performance.

I. Classification of the building in seismic risk class. The building was built in 1921, based on a building permit. It has a height regime of D + P + 2E + Mp (partial attic) and “home” destination. It is also a historical monument, being registered in the LMI with the code B-II-m-B-18514-casa-sf. Sec. XIX-first half sec. XX.

As a result of the request of the owner (beneficiary) to change its initial destination, from “house” to “boutique hotel”, based on law 10/1995 “On quality in constructions”, article 18, paragraph (2) and H.G. 925/1995 “Regulation on verification and technical expertise of quality of projects, execution of works and constructions”, a technical expertise was prepared in 2016 [122], which resulted in the need for its structural consolidation.

The most important elements, which give the building the character of a historical monument, are: main facade - street (Figure 6.18a, Figure 6.18b); the main entrance to the building - street (Figure 6.18c) and the monumental staircase, with wooden structure (Figure 6.18d).

a) b) c) d)

Figure 6.18 - a) Main facade (street); b) balcony area; c) main entrance to the building (street); d) monumental staircase - main entrance, with wooden structure [122]

The structure of the building is made of load-bearing masonry walls (solid pressed ceramic brick - Figure 6.19a), in some areas of the walls there is inserted (completions) and with masonry made in American style - ceramic bricks with horizontal gaps (Figure 6.19b).

Within the structure there are several concrete elements with the role of “lintels”, identified at the level of the window openings of the basement. The floor above the basement is made partly with wooden structure (Figure 6.19c, Figure 6.19d) and partly with structure of metal profiles and “vaults”, made of pressed solid ceramic brick (Figure 6.20a and Figure 6.20b), and the floors above ground floor, 1st floor and 2nd floor, are entirely made of wood.

a) b) c) d)

Figure 6.19 - a) load-bearing walls made of pressed solid ceramic brick; b) inserts / completions with masonry made in American style, from ceramic bricks with horizontal gaps; c), d) floor with wooden structure; [122]

The roof of the building has a structure of softwood - fir (chair frame Figure 6.20c and Figure 6.20d), and sheet metal roofing. The structural elements of the frame are joined both by metal rods (nails) and by means of bolts / screws.

According to the actual revelations / surveys at the level of the building's foundations, it was highlighted that they are made entirely of pressed solid brick masonry, its width being variable, generally having widths of feet by ~ 15 cm, on both sides of the walls.

a) b) c) d)

Figure 6.20 - a), b) Floor area with structure of metal profiles and “*vaults*” (solid pressed brick); c), d) Roof structure [122]

The mortar used for masonry and plaster is exclusively based on lime, and the wood used for the floor elements is softwood (fir). The ceilings are made of lime plaster on fir and fir slats (Figure 6.19c, Figure 6.19d), and the floor support (wood flooring, linoleum) is made of planks. The wall thicknesses are variable, in a chaotic manner, due to the variable thickness of the plasters. Thus, the thicknesses of the partition walls are found in the values mentioned below: {14 cm; 28 cm; 42 cm; 56 cm; 70 cm; 84 cm}.

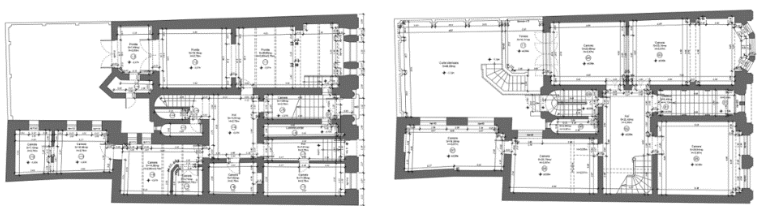
Apart from the monumental staircase with wooden structure, which provides access only between the ground floor and first floor, vertical access, on the entire height of the building, including the basement, is made mainly through a balanced reinforced concrete staircase (Figure 6.21).

a) b)

Figure 6.21 - Balanced reinforced concrete staircase, which provides access vertically, over the entire height of the building, including the basement [122]

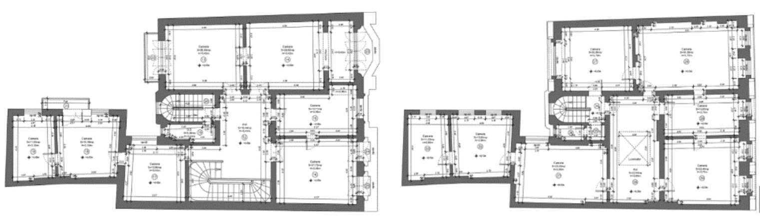
As provided by the seismic assessment code P100-3 / 2019, the technical expertise process for the situation of the present case study was carried out by going through the general steps below

Collection of technical information related to the structural conformation and the level of structural / unstructural degradation of the building. At the time of the technical expertise, there was no technical book of the building, given the fact that, being an interwar building, most likely the technical documentation was lost, and also there is no recent technical documentation, as a result of some possible structural / non-structural intervention works; in order to have a clear picture of the structural composition, a complete structural survey was drawn up (Figure 6.22, Figure 6.23); when carrying it out, in order to have a high level of confidence, extensive surveys were carried out (stripping / removal of finishes - plasters, plywood, false ceilings, etc.).



a) b)

Figure 6.22 - Structural survey: a) basement; b) ground floor [122]



a) b)

Figure 6.23 - Structural survey: a) floor 1; b) floor 2 [122]

a) b)

Figure 6.24 - a), b) Degradation of the main facade plasters [122]

a) b) c) d)

Figure 6.25 - a) Degradation of exterior plasters, caused by damaged rainwater collection and management systems; b), c) Traces of rainwater infiltration; d) Obsolete sewerage installations [122]

a) b) c) d)

Figură 6.26 – a) prezență vegetație (mușchi/licheni) crescută pe suprafețele construcției; b) Instalații invechite de canalizare; c), d) Tencuieli interioare degradate/exfoliate [122]

Establish the basic requirements of the evaluation, the associated limit states and the selected limit state. The building, which is the object of technical expertise, being a historical monument, according to P100-1 / 2013, chap. 4.4.5, Table 4.2, falls into class II of importance-exposure to earthquakes, having the correction factor γI, e = 1.2, which ensures a level of seismic hazard, higher than the basic one; in this situation, the achievement of the Basic Performance Objective (OPB) was required, which provides a hazard level corresponding to the expected seismic response, characterized by a 40% probability of exceeding in 50 years and an Average Recurrence Interval = IMR = 100 years (for ultimate limit checks - SLU) and IMR = 30 years (for service limit checks - SLS), being similar to the seismic response of a newly designed building, of class II of importance-exposure to earthquake, response characterized by 20 % exceeding in 50 years with an IMR = 225 years.

Determination of the KL level of knowledge: taking into account the definitions of the three levels of knowledge (KL1 = 1.35 - limited knowledge; KL2 = 1.25 - normal knowledge; KL3 = 1.00 - complete knowledge) and the degree of fulfillment of the three criteria: i) building geometry; ii) the composition of the detail and iii) the mechanical properties of the materials.

The level of knowledge KL1 = 1.35, was established by meeting the three criteria, as follows:

- the geometry / structural conformation of the building is known from a complete structural survey; the detailed composition was identified from a limited field inspection (local surveys / plastering, foundation unveiling, etc.) and taking into account the practice of interwar design;
- mechanical properties of materials - reinforced concrete elements - lintels, balanced ladder ramp made of reinforced concrete, for which the determination of compressive strength was made by non-destructive tests in the field (concrete, sclerometry, Figure 6.33) and limited destructive (laboratory) tests (3 specimens / bricks, Figure 6.34); the dimensional characteristics and the specific gravity are found in Table 6.11, and the compressive strengths, with standardized value, are found in Table 6.12;

a) b) c)

Figure 6.27 - a) Betonoscopie rampă demisol; b) Betonoscopie buiandrug fereastră demisol; c) sclerometrie buiandrug fereastră demisol [122]

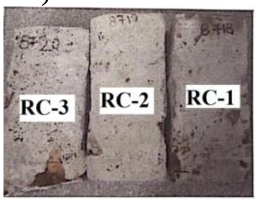


Figure 6.28 - Specimens (RC-1; RC-2; RC-3) subjected to non-destructive (determination of bulk density) and destructive (determination of standardized compressive strength) laboratory tests [122]

Table 6.10 - Dimensional characteristics and apparent density of masonry elements (pressed solid ceramic bricks, specimens RC-1; RC-2 and RC-3) [122]

Indicativ	L [mm]	W [mm]	H [mm]	M [g]	ρ_a [kg/m3]
RC-1	254,7	127,1	72,4	4220	1801
RC-2	251,4	122,5	66,2	3744	1836
RC-3	225,1	127,3	75,7	4038	1862

Table 6.11 - Standardized fb compressive strength of masonry elements (pressed solid ceramic bricks, specimens RC-1; RC-2 and RC-3) [122]

Indicativ	L [mm]	W [mm]	H [mm]	Fc [kN]	σ_c [N/mm^2]	c_{conv}	δ	f_b [N/mm^2]
RC-1	254,7	127,1	72,4	1105,8	34,2	0,8	0,828	**22,6**
RC-2	251,4	122,5	66,2	1552,8	50,4	0,8	0,810	**32,7**
RC-3	225,1	127,3	75,7	951,08	33,2	0,8	0,841	**22,3**
Notatii:	**ρ_a - densitatea aparenta a specimenelor RC-1; RC-2; RC-3**							
	σ_c - rezistenta la compresiune in stare uscata							
	f_b - rezistenta la compresiune standardizata							

Establishing the evaluation methodology: because the expert building belongs to class II of importance-exposure to earthquake (historical monument) and is located in the seismic area (Bucharest), characterized by the horizontal acceleration of the ground ag = 0.30g (> 0.15g), and having in considering the criteria for defining the three methodologies, including the definitions of knowledge levels (see above), according to P100-3/2019, in this situation, the use of assessment methodology 2 was found to be appropriate.

Determination of indicators R1 and R2 (qualitative assessment). Having established above the KL level of knowledge and the evaluation methodology, it is possible to proceed to the establishment of indicators R1 and R3, which are the object of a qualitative evaluation of the building structure. In this regard, for brick masonry constructions, Annex D provides the evaluation criteria and numerical values for establishing the values of indicators R1 (Table 6.12) and R2 (Table 6.13).

Table 6.12– $\mathbf{R_1}$ indicator values [97]

Clasa de risc seismic			
I	**II**	**III**	**IV**
Valori R_1			
<30	30-60	60-90	90-100

Table 6.13 – $\mathbf{R_2}$ indicator values [97]

Clasa de risc seismic			
I	II	III	IV
Valori R_2			
<50	50-70	70-90	90-100

As can be seen, from the point of view of the indicator $\mathbf{R_1 = 54}$ (qualitative assessment / determination of the level of structural compliance), the building falls into **the seismic risk class $\mathbf{R_sII}$**, and from the point of view of the indicator $\mathbf{R_2 = 70}$ (level assessment structural / non-structural degradation), the building falls into **the seismic risk class $\mathbf{R_sIII}$**, as no significant degradation (cracks / fissures, dislocations of structural elements, etc.) were identified, which could endanger the overall strength and stability of the building.

The finalization of the seismic risk class of the building is made only after the determination of the indicator $\mathbf{R_3}$ (evaluation by structural calculation).

Determination of indicator $\mathbf{R_3}$ (quantitative evaluation). From the point of view of the seismic action, the location of the construction, which is the object of the expertise, is in the seismic zone (according to the seismic zoning map, Seismic design code, Indicative P100-1 / 2013), characterized by $\mathbf{a_g = a_g^{225} = 0.30g}$ and the corner period Tc = 1.6 s.

According to the methodology 2, established above, for the evaluation of the seismic action the method of modal calculation with elastic response spectrum elastic $S_e(T_1)_{\xi\neq5\%}$was adopted (see below the calculation relations 6.3 and 6.4, according to P100 -1/2013), being a method that can be used without additional restrictions compared to the provisions of P100-1 / 2013, given that the structure of the expert building does not meet the minimum requirements imposed (regularity in plan and the existence of floors, which do not have sufficient rigidity to optimally ensure the role of "rigid diaphragms").

Thus, the general calculation relationship of the R_3 indicator, according to P100-3 / 2019, is:

$$R_3 = \frac{\sum V_{Rd}}{\sum \frac{V_{Ed}}{q}} \tag{6.3}$$

In which,
$\sum \mathbf{V_{Rd}}$ - the sum of the resistance capacities of the vertical elements (walls, pillars) that take over the seismic force (basic cutter);
$\sum V_{Ed}$ **-** the sum of the effective forces (shear forces) of the vertical elements (walls, columns), resulting from a structural calculation (numerical modeling);
q – behavior factor specific to the specific structural and material typology, considered according to the established methodology (P100-3 / 2019)

Table 6.14 – $\mathbf{R_3}$ indicator values [97]

Clasa de risc seismic			
I	II	III	IV
Valori R_3			
<35	35-65	65-90	90-100

Numerical modeling (ETABS). For analytical modeling, given that a calculation is made in the elastic field, the required material characteristics (program input data) are the modulus of elasticity, the specific weights and the Poisson's ratio of the materials.

In general, these characteristics can be obtained either from the literature (specific code recommendations) or by carrying out laboratory tests (limited / extensive), depending on the technical and / or economic possibilities, as the testing involves some financial costs, which are not negligible.

Masonry is made up of masonry elements (bricks), made of burnt clay, and mortar, made of aggregate (sand) and lime / cement, therefore it is a joint / collaboration between two materials of different physical and mechanical characteristics, and to could determine the above characteristics, the compressive strengths of each material (bricks, mortar) are required.

The design codes of masonry constructions [123, 124, 125, 126, 127], in the absence of destructive tests, recommend the use of empirical calculation relations of the form:

$$E_z = \alpha_z f_k \tag{6.4}$$

$$f_k = k f_b^{0.7} f_m^{0.3} \tag{6.5}$$

In which:

- α_z is a coefficient, the values of which can be 700 (recommended for old masonry) or 1000 (which will be used for the purpose of this analysis);
- f_k is the characteristic compressive strength of masonry;
- the value of the factor k depends on the type of masonry element and was considered according to Table 6.15, below;

Table 6.15 - Values of the coefficient k, depending on the type of element of masonry and mortar mark [124]

Tipul elementului pentru zidărie	Constanta K
Elemente ceramice pline (grupa 1)	0.55
Elemente ceramice cu goluri verticale (grupa 2 și 2S)	0.45
Elemente din BCA (grupa 1)	0.55

$\mathbf{f_b}$ is the standardized compressive strength of the masonry elements, which in this situation was determined by destructive laboratory tests (Table 6.11);

$\mathbf{f_m}$ is the characteristic compressive strength of mortar, the values of which could not be determined by destructive laboratory tests, but were considered, starting from usual values and according to various studies on masonry mortars [4, 6, 128 ÷ 135] , but also taking into account

the type of mortar and the age of the construction, which is the subject of technical expertise; thus, the values of compressive strength (f_m) of lime mortar, included in the 12 hypothesis combinations, were considered {0.0001; 0.10; 0.25; 0.5; 1; 2.5} N/mm^2;

Note:
The lower extreme value (hypothetical) f_m = 0.0001N /mm^2 was considered associated with the situation of a ***resistance mortar tending to 0****, situation corresponding to the hypothesis in which the resistance of the mortar was consumed in full time (crumbly mortar), as a result of the aging phenomenon, and the extreme upper value f_m = 2.50 N /mm^2, was considered as* ***the maximum credible*** *value for the situation analyzed in this article (given the type of mortar used in the 1920 being a reduced one).*

- the volumetric weight of the masonry was considered $\gamma_z = 18\ \mathrm{kN/m^3}$;

Wood. Since the assortment of wood used in the structure of the floor is fir / resinous, its modulus of elasticity (Ew), in dry layer, was considered 11300 N/mm^2, and the volumetric weight of fir wood was considered γ_w = 5 kN/m^3 (according to Table 6.16).

Table 6.16 - Values of the modulus of elasticity parallel to the direction of the fibers at the limit of proportionality (Ew = E) and transverse modulus of elasticity (G) [121]

Specia materialului lemnos	Modulul de elasticitate paralel cu direcția fibrelor la limita de proporționalitate E (N/mm^2)		Modulul de elasticitate transversal G (N/mm^2)	
	$E_{0,05}$	E	$G_{0,05}$	G
Molid, brad, larice, pin	9 000	11 300	4 000	5 000
Plop	8 000	10 000		
Stejar, gorun, cer, salcâm	9 500	11 500	8 000	10 000
Fag, mesteacăn, frasin, carpen	12 000	14 300		

Aspects and assumptions regarding numerical modeling (ETABS). The structure of the building was introduced in the automatic numerical modeling program, respecting as accurately as possible its irregular character regarding the shape in plan and elevations, variable wall thicknesses (Figure 6.29).

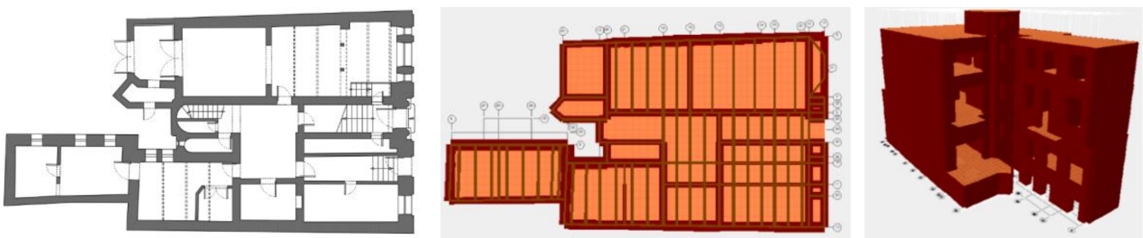


Figure 6.29- a) structural survey of the floor above the basement; b) 2D-ETABS model

By combining the parameters α_z, k, f_b and f_m, the 24 values of the elastic modulus of the masonry E_z were obtained, keeping constant the values of the other characteristics (E_w; γ_w), (E_o; γ_o) and (E_b, γ_b), was obtained the values of the periods of fundamental vibration of the building associated with the 24 theoretical hypotheses (Table 6.17, Table 6.18).

Table 6.17 - The values of the fundamental proper period of vibration of the building in hypotheses 1-12

Numerical calculation results (ETABS)							**Dynamic recordings**			
							longitudinal		**transversal**	
Assumption	**α_z**	**k**	**f_b [N/mm^2]**	**f_m [N/mm^2]**	**Ez**	**T^{ETABS} [s]**	**$T_{1,L}$ [s]**	**$T_{2,L}$ [s]**	**$T_{1,T}$ [s]**	**$T_{2,T}$ [s]**
1	700	0,55	22,3	0,0001	213	0,608	0,21	0,19	0,28	0,18
2	700	0,55	32,7	0,0001	279	0,533	0,21	0,19	0,28	0,18
3	1000	0,55	22,3	0,0001	305	0,509	0,21	0,19	0,28	0,18
4	1000	0,55	32,7	0,0001	399	0,447	0,21	0,19	0,28	0,18
5	**700**	**0,55**	**22,3**	**0,1**	**1695**	**0,218**	**0,21**	**0,19**	**0,28**	**0,18**
6	700	0,55	32,7	0,1	2216	0,191	0,21	0,19	0,28	0,18
7	700	0,55	22,3	0,25	2232	0,187	0,21	0,19	0,28	0,18
8	1000	0,55	22,3	0,1	2422	0,183	0,21	0,19	0,28	0,18
9	700	0,55	22,3	0,5	2748	0,172	0,21	0,19	0,28	0,18
10	700	0,55	32,7	0,25	2918	0,167	0,21	0,19	0,28	0,18
11	1000	0,55	32,7	0,1	3166	0,161	0,21	0,19	0,28	0,18
12	1000	0,55	22,3	0,25	3188	0,159	0,21	0,19	0,28	0,18

Table 6.18 - The values of the fundamental proper period of vibration of the building in hypotheses 13-24

Numerical calculation results (ETABS)							Dynamic recordings			
							longitudinal		transversal	
Assumption	α_z	k	f_b [N/mm^2]	f_m [N/mm^2]	Ez	T^{ETABS} [s]	$T_{1,L}$ [s]	$T_{2,L}$ [s]	$T_{1,T}$ [s]	$T_{2,T}$ [s]
13	700	0,55	22,3	1	3383	0,157	0,21	0,19	0,28	0,18
14	700	0,55	32,7	0,5	3592	0,147	0,21	0,19	0,28	0,18
15	1000	0,55	22,3	0,5	3925	0,141	0,21	0,19	0,28	0,18
16	1000	0,55	32,7	0,25	4168	0,137	0,21	0,19	0,28	0,18
17	700	0,55	32,7	1	4422	0,133	0,21	0,19	0,28	0,18
18	700	0,55	22,3	2,5	4453	0,132	0,21	0,19	0,28	0,18
19	1000	0,55	22,3	1	4833	0,127	0,21	0,19	0,28	0,18
20	1000	0,55	32,7	0,5	5131	0,125	0,21	0,19	0,28	0,18
21	700	0,55	32,7	2,5	5821	0,116	0,21	0,19	0,28	0,18
22	1000	0,55	32,7	1	6318	0,111	0,21	0,19	0,28	0,18
23	1000	0,55	22,3	2,5	6361	0,110	0,21	0,19	0,28	0,18
24	1000	0,55	32,7	2,5	8316	0,090	0,21	0,19	0,28	0,18

It was also chosen to use some of the existing direct methods in the literature [7], in order to have a broader picture of the estimated value of the leaf vibration period (Table 6.19).

Table 6.19 - The values of the fundamental natural vibration period of the determined building with direct / empirical methods

Nr. crt.	Direct/empirical methods (description)	Nr.	Calculation formula	Dimensional features of the building			
				n	B [m]	L [m]	H [m]
				4	5	27	13,7
				L_{walls} [m]	A_{walls} [m^2]	d [m^{-1}]	
				218	430	0,51	
				T [s]			
				Tmin	…	Tmax	
1	**T. Taniguchi Method:** The method is based on the results of experimental investigations performed on a large number of buildings in Tokyo and Yokohama. Formula 1.1 has been validated for all types of buildings. It is not specified for which structural types formulas 1.2 and 1.3 were determined, but were considered as alternative formulas for the study.	1.1	T=(0.12…0.40)√((2n+1)/3)=	0,21			
		1.2	T=(0.07…0.09)n =	0,28			
		1.3	T=(0.06…0.10)(n+5)=	0,54			
2	**F.P. Ulrich and D.S. Carder:** The formulas of the method were determined based on the measurements of 400 buildings with different resistance structures. Formula 2.1 describes a limit range of experimental results, and formula 2.2 is recommended as providing optimal results in practice.	2.1	T=(0.01…0.035)H=	0,14	…	0,48	
		2.2	T=~0.02H=	0,27			
3	**Method E. Rosenblu - ETH:** The method is recommended only for residential and office buildings.	3	T=(0.09…0.10)(n+1)	0,45	…	0,50	
4	**K. Nakagava method:** The formulas of the method are based on the experimental results of 53 studied buildings, according to the H / √B ratio, contained in the Californian code. Formula 4.1 describes a limit range of experimental results, taking into account the H / √B ratio, and formula 4.2, taking into account the number of levels.	4.1	T=(0.07…0.13)H/√B	0,43	…	0,80	
		4.2	T=(0.10+0.038n)…(0.20+0.064n)=	0,25	…	0,46	
5	**M. Takeuchi Method:** The formulas of the method are based on experimental results for 60 buildings in Tokyo and Osaka. The formula depends exclusively on the height H of the building.	5	T=H/60=	0,23			

6	**Method A. Arias and R. Husid:** The calculation formula was determined on constructions in Chile, with structures in reinforced concrete frames and masonry walls (closing / stiffening)	**6**	T=0.024H^(0.71d-0.14) =	**0,04**
7	**M. Baeza method:** The method was also determined on constructions in Chile, and is valid only for constructions in this country.	**7**	T=0.036n =	**0,14**
8	**Method A. Arias, R. Husid and M. Baeza:** The formulas were determined by analyzing the results obtained on 34 tall buildings, with a height regime between 4 and 17 floors, in the provinces of Santiago and Valparaiso. It is not clearly specified for which structural types Formulas 8.1 and 8.2 were determined, but they were also considered as alternative formulas for the study.	**8.1**	T=0.012n=	**0,05**
		8.2	T=0.035n=	**0,14**
9	**H. Sandi and G. Serbanescu method:** It is a Romanian method, and the formulas of the method were established experimentally, through measurements performed on some civil constructions in Bucharest. Formula 9.1 was validated for the evaluation of the vibration period, in the transverse direction, and 9.2 for tower blocks.	**9.1**	T=(0.045…0.055)n=	**0,18**
		9.2	T=0.065 H/√m=	**0,40**
10	**Method J. S. Carmona and J. H. Cano:** The formulas of method 10.1 and 10.2, were determined experimentally on buildings in Argentina, with reinforced concrete structures, and formula 10.3, for reinforced concrete structures with masonry walls, with stiffening role.	**10.1**	T=0.012H+0.09 =	**0,25**
		10.2	T=0.07 H/√B=	**0,43**
		10.3	T=H√(0.003/B)+0.0002/(1+30d)=	**0,34**
11	**Method R. Husid, W. Pieber and J. Romo:** Formulas 11.1 and 11.2 are recommended for ordinary reinforced concrete structures, and formula 11.3 has been proposed for constructions in Chile.	**11.1**	T=0.04n =	**0,16**
		11.2	T=n/69=	**0,06**
		11.3	T=0.04 H/∛B=	**0,32**
12	**M. Ifrim method:** The formula is valid for low constructions with structures made of concrete frames and masonry / load-bearing walls made of masonry, with a height regime of ≤6 levels.	**12**	T=0.09√H=	**0,33**
13	**JOINT COMMIT-TEE ASCE-SEA method:** The formula of the method is based on experimental measurements performed on 3000 buildings and is validated for all types of buildings.	**13**	T=0.09 H/√B =	**0,55**
14	**U.S. method COAST AND GEODETIC SURVEY:** The formula of the method was determined based on experimental investigations on 212 buildings.	**14**	T=0.1n =	**0,40**
15	**WASHINGTON OFFICE OF CONSTRUCTION VETERAN ADMINISTRATION Method:** The formula of the method has been validated for constructions with masonry load-bearing wall structures.	**15**	T=0.15n =	**0,60**
16	**Formulas according to the Romanian seismic design norm, indicative P100-81:** In addition to the formulas elaborated by H. Sandi and G. Serbănescu, in the Romanian seismic design norm, indicative P100-81, formula 16 was also introduced.	**16**	T=0.3+0.05n=	**0,50**
17	**Formulas according to the Romanian seismic design norm, indicative P100-1 / 2013:** In Annex B, B.2, for the preliminary design of buildings with a total height of up to 40 m, the following formulas are recommended: T = Ct H 2 (3/4), wherein Ct = 0.05 - for the other types of structures, including those with load-bearing masonry walls.	**17**	T=0.05* H^(3/4)=	**0,36**

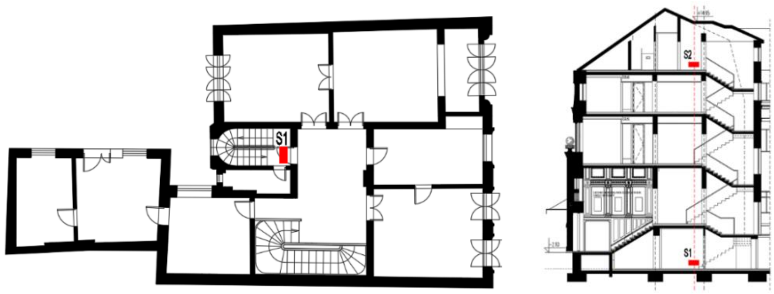
Experimental determination of its own fundamental period of vibration. Many studies [10÷45] have shown that the "*Ambient Vibration Tests*" (AVT) method or the "environmental vibration recording" method can be used successfully in the calibration of analytical modeling (ETABS) of structures, thus being able to determine issues related to possible hidden deficiencies or degradation of structures, which may be reflected in the low level of overall rigidity.

The equipment used for the dynamic instrumentation of the building is part of the props of the Research Center "*Seismic Risk Assessment*" within UTCB and consists of a GEODAS acquisition station (Figure 6.33) and sensors speeds (Figure 6.33).

These devices are produced by Buttan Service-Tokyo & Tokyo Soil Research Co., Ltd. The recordings were made together and under the careful guidance of Mr. Assoc. Dr. Eng. Alexandru Aldea.

Figure 6.32 - GEODAS purchasing station   Figure 6.33 - CR4.5-1H speed sensor

Dynamic recordings were made using two sensors (accelerometers), and their location in the building was done as follows (Figure 6.34): one sensor (S1) was placed at the basement floor, and the other sensor (S2) at the bridge ; both sensors were placed approximately vertically centered through the stairwell, and were calibrated to make recordings simultaneously; the recordings were made in both directions of the building, by successively changing their positions during the recordings.



a)   b)

Figure 6.34- Arrangement of sensors for the measurements of ambient vibrations of the analyzed building: a) positioning in the plan; b) vertical positioning

Table 6.20 - Representative spectral frequencies identified by spectral analysis, for the longitudinal direction of the building

Măsuratori dinamice		**f_1 (Hz)**	**f_2 (Hz)**	**f_3 (Hz)**
Măsuratoarea 1	Spectre Fourier de amplitudine	4.58	5.23	6.88
	Rapoarte spectrale	4.69	5.25	6.96
Măsuratoarea 2	Spectre Fourier de amplitudine	4.59	5.29	7.11
	Rapoarte spectrale	4.81	5.31	7.14
Frecvențe medii		**4.67**	**5.27**	**7.02**
Perioade medii		**$T_{1,L}$ (s)**	**T_2 (s)**	**T_3 (s)**
		0.21	**0.19**	**0.14**

Tabel 6.21 - Frecvențe spectrale reprezentative identificate prin analiză spectrală, pentru direcția transversală a clădirii

Masuratori		**f_1 (Hz)**	**f_2 (Hz)**	**f_3 (Hz)**
Masuratoarea 1	Spectre Fourier de amplitudine	3.53	5.68	7.31
	Rapoarte spectrale	3.54	5.69	7.32
Masuratoarea 2	Spectre Fourier de amplitudine	3.51	5.61	7.40
	Rapoarte spectrale	3.57	5.78	7.38
Frecvente medii		**3.54**	**5.69**	**7.35**
Perioade medii		**$T_{1,L}$ (s)**	**T_2 (s)**	**T_3 (s)**
		0.28	**0.18**	**0.14**

Conclusion. It can be seen that the fundamental vibration period obtained in hypothesis 5 resulted in $\mathbf{T_1^{ETABS} = 0.28\ s}$ (transverse translation), and from the ambient vibration (AVT) recordings, it resulted in $\mathbf{T_1^{AVT} = 0.28\ s}$ (transverse translation), can conclude that the **hypothesis 5** considered is compatible with reality (at least theoretically).

In this situation, it can be considered that the compressive strength of lime mortar can be considered as 0.10 N/mm^2, which, to some extent, the hypothesis related to the fact that the age of the mortar is reflected in the strength characteristics of this can be accepted in this situation, but the hypothesis cannot be considered generally valid.Also, it can be direct (empirical), viable (bold green) methods for this situation are those in Table 6.22

Table 6.22 - Direct (empirical) methods that proved to be viable for the present study situation

Nr. crt.	Direct/empirical methods	Formula	Calculation formula Ti [s]	General building features			
				n	B [m]	L [m]	H [m]
				3	12,21	23	13,6
				T [s]			
1	**F.P. Ulrich and D.S. Carder:** The method is based on statistical analyzes performed on the measurements of 400 buildings, with different resistance structures.	1	T_1=0.02H	0,27			
5	**J. S. Carmona and J. H.** Cano method: The formulas of the method were determined experimentally on buildings in Argentina, with reinforced concrete structures with masonry walls, with a stiffening role.	5	T_{52}=0.07 H/√B	0,27			
7	**M. Ifrim method:** Formula 7.1 has been validated for low constructions with masonry load-bearing wall structure and reinforced tone frames with masonry filling walls, with a height regime less than or equal to 6 levels; formula 7.2 was validated for reinforced concrete constructions with average height regime in the range of 7 ... 15 levels.	7	T_{71}=0.09√H	0,33			
8	**ASCE-SEA JOINT COMMITTEE Method [206]:** The formula of the method is based on experimental measurements on 3000 buildings with various structural types and is also recommended by the U.B.C.	8	T_8=0.09 H/√B	0,35			
9	**U.S. method COAST AND GEODETIC SURVEY:** The formula was determined based on experimental investigations on 212 buildings. The formula is also recommended by SEISMOLOGY COMMITTEE SEAOC	9	T_9=0.10n	0,30			
12	**Formulas according to Romanian seismic design regulations, indicative P100-1 / 2013 and P100-3 / 2008:** In Annex B, B.2, for the preliminary design of buildings with a total height of up to 40 m, the following formulas are recommended: T = Ct H 2 (3/4), wherein Ct = 0.075 - for reinforced concrete space frames and metal space frames and eccentric braces; Ct = 0.05 - for other types of buildings, than those with reinforced concrete or metal structure	12	T_{12}=0,05* H^(3/4)	0,35			

Evaluation of indicator R3 - existing situation

- **Resistance capacity of structural walls to planar forces (shear force associated with eccentric compression yielding)**

$$V_{f1} = \frac{N_d}{c_p * \lambda_p} * (1 - 1.15\nu_d) \quad (6.6)$$

in which:

$$\lambda_p = H_p/l_w \quad (6.7)$$

λ_p - the form factor of the masonry wall; H_p - the height of the wall; l_w - wall length;

c_p - coefficient that depends on the fixing conditions of the wall end: c_p=2.0 for console wall (upright); c_p=1.0 for double recessed wall at the ends (shoulder strap);

$$\sigma_0 = \frac{N_d}{A_{zi}} \tag{6.8}$$

σ_0 - the average unit compression force corresponding to the axial design force N_d and A_{zi} the cross-sectional area of the masonry wall;

$$\nu_d = \frac{\sigma_0}{f_d} \tag{6.9}$$

ν_d - compression factor;

$$f_d = \frac{f_{med}}{CF} \tag{6.10}$$

f_d - is the design value of the compressive strength of the masonry;

CF=1.35 – the confidence factor associated with the level of knowledge KL1=1.35

$$f_{med} = 1.3 * f_k \tag{6.11}$$

$$f_k = k f_b^{0.7} f_m^{0.3} \tag{6.12}$$

f_{med} - average compressive strength of masonry;

f_m – mortar strength (value determined following calibration of the ETABS analytical model, using dynamic records);

f_b – standardized strength of masonry elements, the value of which was considered as the average of the values obtained in destructive laboratory tests;

f_k - represents the characteristic compressive strength, which can be determined according to the standardized resistance of the masonry and the mortar mark, according to CR6;

- **Design value of the shear strength resistance to diagonal cracking**

$$V_{f2} = \min\{V_{f21};\ V_{f22}\} \tag{6.13}$$

V_{f21} – the design value of the shear strength resistance to the horizontal sliding of an unreinforced masonry wall, according to (6.14);

$$V_{f21} = \frac{1{,}33}{CF * \gamma_M}\left(f_{vk0}\frac{l_{ad}}{l_c} + 0{,}4\sigma_d\right) t * l_c \tag{6.14}$$

- l_c - the length of the compressed area of the section, which takes into account the alternating effect of the seismic force, determined by the relation (6.24)
- $f_{vk0} = 0{,}045\frac{N}{mm^2}$ – recommended value for old masonry made of pressed solid brick and lime mortar;

$$l_c = 1.5 * l_w - 3 * \frac{M_d}{N_d} \tag{6.15}$$

$$\sigma_d = \frac{N_d}{A_{zi}} \tag{6.16}$$

in which

- M_d – the bending moment in the verified wall; N_d – is the axial force in the checked wall;
- t - wall thickness checked; l_w – the actual length of the wall checked;
- l_{ad} – is the length on which the adhesion is active, calculated with the relation (6.17)

$$l_{ad} = 2 * l_c - l_w \tag{617}$$

if $l_{ad} \leq 0$ the design value of the breaking shear force is calculated with the relation (6.18)

$$V_{f21} = 0.53\frac{N_d}{CF\gamma_M} \tag{6.18}$$

V_{f22} – the design value of the shear strength at diagonal cracking, according to (6.28);

$\gamma_M = 2.7$ – value of partial safety coefficient for old masonry of pressed solid bricks and lime mortar, executed during 1900÷1950;

$$V_{f22} = \frac{tl_w}{b}\sqrt{1+\frac{\sigma_0}{f_{td}}} \qquad (6.19)$$

in which

- b - coefficient with values $1.0 \leq b=\lambda_p \leq 1.5$ (according CR6);
- f_{td} - the design resistance of the masonry to the main stretching efforts (6.29);

$$f_{td} = \frac{0.04 * f_m}{\gamma_M * CF} \qquad (6.20)$$

f_m, γ_M și CF – see above;

Also, if:

- $V_{f1}<V_{f2}$ → ductile walls, for which the behavior factor will be considered q=2
- $V_{f1}>V_{f2}$ → fragile walls, for which the behavioral factor will be considered q=1.5

Table 6.23 - Global indicator $R_{3,L}$

$R_{3,L}$ - global assessment on the longitudinal direction of the building				$R_{3,L}$
Storey	F_{Rd}[kN]	F_{Ed}[kN]	$R_{3,L}^{storey}$	
basment	1579	3134	0,50	0,45
1st ground floor	1420	3157	0,45	
2nd ground floor	857	1761	0,49	
3rd ground floor	331	616	0,54	

Table 6.24 - Global indicator $R_{3,T}$

$R_{3,T}$ - global assessment on the transverse direction of the building				$R_{3,T}$
Storey	F_{Rd}[kN]	F_{Ed}[kN]	$R_{3,L}^{storey}$	
basment	1045	1855	0,56	0,49
1st ground floor	212	437	0,49	
2nd ground floor	472	858	0,55	
3rd ground floor	116	195	0,60	

In which was considered:

$\mathbf{F_{Rd}} = \sum \mathbf{V_{Rd}}$ – the overall shear force on each level of the building, as the sum of the resistance capacities of the vertical elements (masonry load-bearing walls) that take over the seismic force (basic shear);

$\mathbf{F_{Ed}} = \sum \mathbf{V_{Ed}}$ - the overall shear force at each level of the building, as the sum of the actual efforts (shear forces) of the vertical elements (masonry load-bearing walls), resulting from a structural calculation (numerical modeling - ETABS);

Therefore, considering the results of the evaluation of wall densities, lateral displacements (rigidity conditions), as well as the value obtained of the indicator R3 (structural calculation / quantitative evaluation), it resulted that the structure of the building **can be classified into the seismic risk class RsII , since R3 = 0.45 <0.65**, therefore its structural consolidation is required.

II. Structural (indicator R3) and technical-economic evaluation of the consolidation solutions adopted

Solution 1 - general description

- consolidation of foundations (concrete underpinning);
- reinforced concrete jakets C25/30, applied by shotcreting, of all structural walls:
  - because the exterior walls (main facade) have monumental value, and some of the other exterior/perimeter walls are attached to the heels of the neighboring buildings, the lining on both sides is not technologically possible;
  - in this situation it was decided to be applied only on their inner faces; the concrete lining will be made 10 cm thick, and can be reinforced either with nets tied from vertical and horizontal bars Φ8/10/10, PC52 (a single network arranged in the

middle plane of the concrete lining), or with welded nets Φ8 / 10 / 10 STPB (a single network arranged in the median plane of the jacket);

- all interior walls, with thicknesses greater than 14 cm, were proposed to be lined on both sides of them (thickness 6 cm); the concrete linings will be reinforced with welded nets Φ6 / 10/10 STPB (one network arranged in the middle plane of the shirt);
- the consolidation of the existing floors with masonry structure (vaults) and metal profiles and those with wooden structure, will be done by introducing (overconcreting) new reinforced concrete slabs with tied mesh Φ8/10/10, PC52 (up / down), whose thickness will be 12 cm; the new plates will rest on the afferent walls, in a “teeth” system.

Structural assessment (indicator R3) and economic evaluation for solution 1 of consolidation

Table 6.25 - Technico-economical evaluation for solution 1- reinforced concrete jakets

Criterion/requirements	Quantities/Prices/Unit/main technical characteristics	Economical value [euro]
Estimated time	cca. 12 months	xxxxxxxxxxxxxxxxxxx
Human resources	cca.15 qulified workers: carpenters, blacksmiths, masons, locksmiths	xxxxxxxxxxxxxxxxxxx
Interventions costs (materials+ workmanship)	➢ concrete underpinning: cca. 250 euro/mp (S_{subsol}=235 mp) ➢ shotcrete: cca 50 euro/mp ($S_{total\ shotcrete}$ =1950 mp) ➢ concrete slabs: cca. 30 euro/mp ($S_{totală}$=940 mp) (upper concreting)	58.750 97.500 28.200
Structural design	➢ **design costs**: cca. 5 euro/mp (and technical consulting) ($S_{desfășurată}$=940 mp)	4.700
Level of technological design	➢ medium	xxxxxxxxxxxxxxxxxxx
Level of qualification	➢ mediu / heigh	xxxxxxxxxxxxxxxxxxx
Expected level of quality	➢ heigh	xxxxxxxxxxxxxxxxxxx
Interventions with the operation of the construction	➢ it is not possible *(Obs. Excepting the variant in different steps, but the costs and time duration are higher*	xxxxxxxxxxxxxxxxxxx
Special intervention conditions	➢ it is not necessary	xxxxxxxxxxxxxxxxxxx
Logistical necessity	➢ medium	xxxxxxxxxxxxxxxxxxx
The keep of medium level of structural/seismic ensurance	➢ 1-2 Vrancea seismic events, of medium / high intensity (with interventions after each event, for the usual normal life)	xxxxxxxxxxxxxxxxxxx
Main technical operations	➢ Plaster stripping ➢ Shredded shirts ➢ Realization of foundation subsidies	xxxxxxxxxxxxxxxxxxx

TOTAL COST OF THE INTERVENTION SOLUTION	**189.150 euro**
COST/UNIT OF THE INTERVENTION SOLUTION [EURO/mp]	**~200 euro/mp**

Solution 2 - general description

- consolidation of foundations (concrete underpinning);
- insertion of a system of reinforced concrete (class C25/30) columns (25x25 cm) and belts (25x25 cm), reinforced with vertical bars 4Φ14, PC52 and stirrups Φ8 / 10/15, PC52, and concrete belts, reinforced with longitudinal bars 2Φ14 (top) + 2Φ14 (bottom), PC52 and stirrups Φ8 / 10/15, PC52;
- consolidation of existing floors with masonry structure (vaults) and metal profiles and those with wooden structure, will be done by introducing (overconcrete) new reinforced concrete slabs with tied mesh Φ8/10/10, PC52 (up / down), whose thickness will be 15 cm; it is proposed to completely remove the existing floors.

Structural assessment (indicator R3) and economic evaluation for solution 2 of consolidation

Table 6.26 - Technico-economical evaluation solution 2 - insertion of reinforced concrete columns and belts

Criterion/requirements	Quantities/Prices/Unit/main technical characteristics	Economical value [euro]
Criterion/requirements	cca. 14 months	xxxxxxxxxxxxxxxxxxx
Estimated time	cca.15 qulified workers: carpenters, blacksmiths, masons, locksmiths	xxxxxxxxxxxxxxxxxxx
Human resources	➢ underpinning: cca. 250 euro/mp (S_{subsol}=235 mp) ➢ masonry cutting works: cca 180 euro/mp/linear cutting ($S_{totală\ necesar\ tăieri\ în\ zidărie}$ =700 mp) ➢ concrete works: cca. 150 euro/mc (V_{total}=70 mp) (columns and belts) ➢ concrete clabs: cca. 30 euro/mp ($S_{totală}$=940 mp) (upper concreting)	58.750 126.000 10.500 28.200
Interventions costs (materials+ workmanship)	➢ **design costs**: cca. 5 euro/mp (and technical consulting) ($S_{desfășurată}$=940 mp)	4.700
Structural design	➢ medium	xxxxxxxxxxxxxxxxxxx
Level of technological design	➢ mediu / heigh	xxxxxxxxxxxxxxxxxxx
Level of qualification	➢ heigh	xxxxxxxxxxxxxxxxxxx
Expected level of quality	➢ it is not possible *(Obs. Excepting the variant in different steps, but the costs and time duration are higher*	xxxxxxxxxxxxxxxxxxx

Interventions with the operation of the construction	➢ it is not necessary	xxxxxxxxxxxxxxxxxxx
Special intervention conditions	➢ medium	xxxxxxxxxxxxxxxxxxx
Logistical necessity	➢ 1-2 Vrancea seismic events, of medium / high intensity (with interventions after each event, for the usual normal life)	xxxxxxxxxxxxxxxxxxx
The keep of medium level of structural/seismic ensurance	➢ Cutting / cutting in masonry ➢ Shredded masonry shirts (if applicable) ➢ Realization of foundation subsidies	xxxxxxxxxxxxxxxxxxx
TOTAL COST OF THE INTERVENTION SOLUTION		**228.150 euro**
COST/UNIT OF THE INTERVENTION SOLUTION [EURO/mp]		**~243 euro/mp**

Solution 3 - general description

- ➢ insertion of seismic isolation (HDRB system - high damping isolators) and additional dampers, which have the role of safety/limitation of iolaters deformations;
- ➢ the insertion of the base isolation system also involves the introduction of a rigid reinforced concrete system of general screed type of 60 cm, whose stability/solidarity with the ground on the site will be ensured by providing micro-pilots, and a network of beams from reinforced concrete (lower load-bearing frame); the upper load-bearing frame will be provided by a reinforced concrete floor provided above the basement;
- ➢ the insertion of reinforced concrete floors at each level of the building.

Calculation of the HDRB isolator system. From the point of view of modeling the behavior of these types of insulators, the bilinear calculation model used [104], Figure 6.43, used to express the relationship between shear force and lateral displacement, can be defined by three parameters:

- ➢ elastic stiffness ke;
- ➢ post-elastic rigidity kp;
- ➢ the characteristic resistance Q, which is usually used to estimate the stability of the hysteretic curve when the insulator is subjected to alternating load-discharge cycles.

The three parameters used to generate the bilinear model of high damping insulators are conventionally derived from the shear modulus G and the actual damping ξeff, at a specific shear deformation.

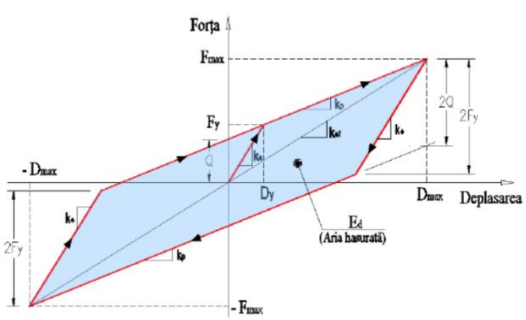


Fig. 6.35 – Modelul biliniar al izolatorului cu amortizare mare (HDRB) [104]

The shear modulus is determined from the dynamic shear tests. Also, the actual depreciation is determined from the tests on insulators and varies between 10% and 20% of the critical depreciation fraction [137, 138].

Design of isolation system for the existing construction base. Because, as a result of the quantitative assessment of the structural performance of the building structure, it was classified into the seismic risk class RsII (R3 = 0.45 <0.65), and given that by hypothetically implementing the base isolation system, the building can be included in the seismic risk class Rs IV, it is proposed as lower limit R3, iz> 0.95 (95%). In these circumstances, in order to be able to find out the necessary value of the period of the $\mathbf{T_{iz}}$ isolators, the following reasoning was adopted:

- **initial situation**: $0 \leq \mathrm{T}_1 = 0.26\ s \leq \mathrm{T}_B;\ \xi_z = 8\%$

$$\eta_z = \sqrt{\frac{10}{5+\xi_z}} = \sqrt{\frac{10}{5+8}} = 0.88 \tag{6.21}$$

$$\mathrm{S}_e(T_1)_{\xi_z=8\%} = \mathrm{a}_g^{100} + \left[\mathrm{S}_e(T_1)_{\xi=5\%} * \eta_z - \mathrm{a}_g^{100}\right]\frac{T_1}{T_B} \tag{6.22}$$

$$\mathrm{S}_e(T_1 = 0.26\ s)_{\xi_z=8\%} = 0.24g\left[1 + \left[\left(1 + \frac{2.5-1}{0.32}0.26\right)0.88 - 1\right]\frac{0.26}{0.32}\right] = 0.42g \tag{6.23}$$

$R_3 = \frac{\sum V_{Rd}}{\frac{\sum V_{Ed}}{q}} = 0.45$, and in elastic domain (q=1), $\mathrm{R}_3 = \sim 0.30$

- **situation after the implementation of insulators**:

it was proposed $\mathrm{R}_3 = \frac{\sum \mathrm{V_{Rd}}}{\frac{\sum \mathrm{V_{Ed}}}{\mathrm{q}}} = 0.95$, with q=1 (the building's structure was considered in elastic domain behavior), thus, increasing R_3 about 3.17 times, it resulted the necessity of the elastic value $\mathrm{S_e}(\mathrm{T}_1)_{\xi_z=8\%}$ reduced with ~68%
in this respect:

$$(1 - 0.68) * \mathrm{S_e}(\mathrm{T}_1)_{\xi_z=8\%} = 0.32 * 0.42g = 0.13g \tag{6.24}$$

and considering the dar considerand critical damping ratio of the isolators $\xi_{iz} = 20\%$, it resulted:

$$\eta_{\mathrm{iz}} = \sqrt{\frac{10}{5+\xi_{iz}}} = \sqrt{\frac{10}{5+20}} = 0.632 \tag{6.25}$$

thus, in this conditions, in order to obtain an important decrease of the elastic spectra compatible with the situation of a structure isolated, the isolators vibration period need to be $\mathrm{T_{iz}} > \mathrm{T_D} = 2\ s$:

$$\eta_{\mathrm{iz}}\ \mathrm{S_e}(\mathrm{T_{iz}})_{\xi_{iz}=20\%} = 0.13g = \eta_{\mathrm{iz}}\ a_g\beta(T_{iz}) = \eta_{\mathrm{iz}}\ a_g\beta_0\frac{T_C T_D}{T_{iz}^2} \tag{6.26}$$

In which $a_g = a_g^{225}$ (for seismic design based on earthquakes' mean recurancy interval IMR=225 years and 20% exeedance in 50 years), thus, the vibration period resulted:

$$\mathrm{T_{iz}} = \sqrt{\frac{\eta_{\mathrm{iz}} a_g \beta_0 T_C T_D}{0.13\mathrm{g}}} = \sqrt{\frac{0.632*0.30g*2.5*1.6*2}{0.13\mathrm{g}}} = 3.42\ s \tag{6.27}$$

Having the geometrical conformation of the building's structure, it was proposed 20 isolators HDRB.

The design of the insulator system was made considering the construction as a system with a single degree of freedom (1GLD). The displacement requirement of the isolation system at the design earthquake $D_{e,s,iz}$ is determined by the relation:

$$D_{e,s,iz} = \left(\frac{T_{iz}}{2\pi}\right)^2 . a_g . \beta(T_{iz}) . \eta_{iz} = \left(\frac{3.42}{2\pi}\right)^2 0.30 * 9.81 * 2.5 \frac{1.6*2}{3.42^2} 0.632 = 0.38\, m \tag{6.28}$$

The effective horizontal stiffness of the proposed system (20 HDRB isolators) is calculated from the relation below (6.47):

$$k_{ef,s,iz,o} = n_{iz} k_{ef,iz,o} \tag{6.29}$$

$$k_{ef,iz,o} = \frac{1}{n_{iz}} \left(\frac{2\pi}{T_{iz}}\right)^2 G_{GS} \tag{6.30}$$

în care

- $T_{iz} = T_{iz,o}$ – horizontal vibration period of the isolators;
- $n_{iz} = 20$ – numbers of isolators proposed;
- $k_{ef,iz,o}$ horizontal stiffness of the isolators;
- $T_{iz} = 3.42\ s$ – vibration period of the isolators system;
- $G_{GS} = 12484$ kN – the weight of the structure in seismic assumption

$$k_{ef,iz,o} = \frac{1}{n_{iz}} \left(\frac{2\pi}{T_{iz,o}}\right)^2 G_{GS} = \frac{1}{20} \left(\frac{2\pi}{3.42}\right)^2 \frac{12484}{9.81} = 215\ \frac{kN}{m} \tag{6.31}$$

The calculations below were made after an example of Mr. Matsutaro Seki [140], Japanese expert, who was in Romania, in a scientific collaboration within the Romanian-Japanese program, on reducing seismic risk for vulnerable buildings, program which took place in the period 2002-2008.

Structural assessment (indicator R3) and economic evaluation for solution 3 of consolidation

Table 6.27 - Technical-economic evaluation solution 3 - seismic insulation of the base

Criterion/requirements	Quantities/Prices/Unit/main technical characteristics	Economical value [euro]
Criterion/requirements	cca. 12 months	xxxxxxxxxxxxxxxxxx
Estimated time	cca.20 qulified workers: carpenters, blacksmiths, masons, locksmiths and qualified workers in seismic isolation systems montage	xxxxxxxxxxxxxxxxxx
Human resources	➢ foundations/basement: cca 200 euro/mp - general reinforced concrete screed 50 cm + isolation system (S_{subsol}=235 mp); ➢ seismic isolators: cca. 4000 euro/pieces (20 piecis); ➢ seismic dampers: cca. 7500 euro/ pieces (4 pieces); ➢ concrete slabs (upper concrete, over floors): cca. 35 euro / mp ($S_{subsol+Et1+2}$=705 mp); ➢ floor over basement (upper carrier frame required for the seismic insulation system of the base): cca. 150 euro / mp;	47.000 80.000 30.000

Interventions costs (materials+ workmanship)	➢ **structural design**: cca. 5 euro/mp (including technical advice) ($S_{desfășurată}$=940 mp) ➢ **Basic seismic insulation system design cost**	4.700 14.000
Structural design	➢ very heigh	xxxxxxxxxxxxxxxxx
Level of technological design	➢ heigh	xxxxxxxxxxxxxxxxx
Level of qualification	➢ heigh	xxxxxxxxxxxxxxxxx
Expected level of quality	➢ it is possible (different phase for possible repairs)	xxxxxxxxxxxxxxxxx
Interventions with the operation of the construction	➢ mandatory for the base insulation system	xxxxxxxxxxxxxxxxx
Special intervention conditions	➢ very high, especially for infrastructure	xxxxxxxxxxxxxxxxx
Logistical necessity	➢ minimum 3 Vrancea seismic events, of medium / high intensity (without interventions after each event)	xxxxxxxxxxxxxxxxx
The keep of medium level of structural/seismic ensurance	➢ Development of mini-pilots (eg Jet Grouting technology); ➢ General eraser (lower frame); ➢ Execution of senior staff; ➢ Cutting with diamond devices; ➢ Metal supports for supporting / mounting seismic insulators; ➢ Execution of the perimeter canal to ensure the movements of the building during the earthquake; ➢ Wrinkled shirts (if applicable);	xxxxxxxxxxxxxxxxx
TOTAL COST OF THE INTERVENTION SOLUTION		**175.000 euro**
COST/UNIT OF THE INTERVENTION SOLUTION [EURO/mp]		**~186 euro/mp**

Example of an existing building, seismically isolated Bucharest City Hall

The building where the General City Hall of Bucharest operates was built between 1906-1911, on the land in front of the Cismigiu Garden, being designed by the architect Petre Antonescu. The project of the resistance structure was drawn up by Eng. Elie Radu and Eng. Gogu Constantinescu.

According to [141], the building is on the list of historical monuments in Bucharest. It was appraised in 1995, and a consolidation project was developed in the classic solution, but which was not put into practice, as it was not agreed to interrupt the specific activities, and a possible temporary change of location meant additional costs, therefore the solution of seismic isolation of the base was adopted.

For the corresponding reduction of the efforts in the structural elements, as well as for the reduction of the values of the relative level displacements, were considered, in turn, several variants of the disposition of the insulating supports, namely:

Variant I: 223 insulating supports with an effective rigidity of 840 kN/m, resulting in values of the vibration period of: T_{iz} = 3.35 s; 3.22 s; 3.19 s;

Variant II: 305 insulating supports with an effective rigidity of 600 kN/m, resulting in values of the vibration period of: T_{iz} = 3.27; 3.25; 3.20 s.

Table 6.28 - Technical-economic evaluation of the insulation of the base for the Bucharest Capital building

Criterion/requirements	Quantities/Prices/Unit/main technical characteristics
Criterion/requirements	➢ cca. 24 months
Estimated time	➢ **stage I** - min. 35 qualified technicians for infrastructure (carpenters, blacksmiths, masons, locksmiths) ➢ **stage II** - approx. 35 technicians for superstructure, stage II - about 35 technicians for superstructure);
Human resources	➢ basement: about 1,000 euro / sqm - mini pilots + general screed 60 cm + base insulation system; ➢ (shock absorber = 36,000 euro / pc .; insulator = 12,000 euro / pc); ➢ shot: about 50 euro / sqm (8cm); ➢ concrete floors (overconcrete, over floors): ➢ about 35 euro / sqm; ➢ floor above the basement (upper load-bearing frame required for the seismic insulation system of the base): ➢ approx. 150 euro / mp;
Interventions costs (materials+ workmanship)	➢ 150.000 euro (structural elements + base insulation system);
Structural design	➢ Very heigh
Level of technological design	➢ heigh
Level of qualification	➢ heigh
Expected level of quality	➢ it is possible (different phase for shirts)
Interventions with the operation of the construction	➢ mandatory, for the base insulation system
Special intervention conditions	➢ very high (especially for infrastructure);
Logistical necessity	➢ at least 3 medium / high Vrancea earthquakes ➢ (no intervention required after each event)
The keep of medium level of structural/seismic ensurance	➢ making mini pilots (eg jet grouting technology); ➢ execution of lower / upper frame; ➢ cuts with diamond devices;

	➢ metal supports for supporting / mounting seismic insulators; ➢ execution of perimeter channel moving building; ➢ shotguns; ➢ general eraser (lower carrier frame);
TOTAL COST OF THE INTERVENTION SOLUTION	**10.000.000 euro**
COST/UNIT OF THE INTERVENTION SOLUTION [EURO/mp]	**455 euro/mp**

7. CONCLUSIONS, PERSONAL CONTRIBUTIONS AND RESEARCH DIRECTIONS

7.1 Conclusions on the specialized literature considered in the thesis

Starting from the synthesis presented in the present thesis, made based on the specific specialized literature, the following conclusions were formulated:

- the most important and oldest international professional non-governmental organizations, among whose multiple fields of activity are the field of protection of historical heritage, including the built one, are: UNESCO, ICOMOS and ICCROM;
- in Romania, international organizations have their own subsidiaries or organizations operating in the spirit of UNESCO, ICOMOS, such as UNRMI, but all of these are in direct collaboration with INP, which in turn is subordinate to the Ministry of Culture, being the central authority in the field of heritage protection historic;
- the material or spiritual good can be identified with the very concept of historical monument;
- in seismic countries, such as Romania, the practice of anti-seismic protection is often met by drastic interventionist limitations, imposed by the 1964 Venice Charter;

7.2 Conclusions on the results of the case studies approached

- the seismic calculation of the constructions requires the determination of the fundamental proper period of vibration; the use of direct (empirical) methods [7] or numerical calculations, which can be done with the help of automatic calculation programs (ETABS, ROBOT, SAP, etc.), can provide estimated values, but the susceptibility of situations in which significant differences may occur between methods, is high;
- in numerical modeling programs, for a calculation in the elastic domain, the modulus of elasticity of the materials is required; they can be determined on the basis of their own mechanical characteristics (compressive / tensile strength), which in turn can be considered with usual values, according to the literature or directly, by laboratory tests;

- determining the fundamental proper period of vibration of a structure, using the method "Ambient Vibration Tests" (AVT), as known in the literature [10 ÷ 45], can significantly reduce the relativity of structural seismic calculation ;
- of course, the technical limitations of dynamic recordings cannot be neglected either; possible errors may arise from improper handling of equipment in the field or from misinterpretation of the results;

Consolidation solution 1 - foundation foundations, reinforced concrete jakets and new reinforced concrete floors

- from a technological point of view, except for interventions at foundations (underpinning), which implies an additional increase in execution time, from the fact that the process is done in several successive steps, in essence this consolidation solution does not involve special technological operations, the most important being the removal of the existing plasters, the application of the concrete by shotcreting and the conformation / realization of the new reinforced concrete floor, by overconcreting or by classic realization, after the complete removal of the existing one;
- from the point of view of resistance, this solution successfully ensures the classification of the building in the seismic risk class RsIII ($0.65 < R3 = 0.89 < 0.91$); it can also be said that there is a noticeable increase in structural ductility;
- from an economic point of view, 200 euro / sqm represents an optimal economic value, related to the identified technological difficulties and to the advantage of ensuring the seismic risk class RsIII;
- an important advantage is that, by implementing this consolidation solution, the character of historical monument is not affected, being minimally invasive, but it should be noted that the solution is not reversible, as recommended by documents in the field of monument restoration;

Consolidation solution 2 - foundation foundations, insertion of pillars and belts, including new reinforced concrete floors

- from a technological point of view, the need for interventions at foundations (subsidies), leads to an additional increase in execution time, since the process is done in several successive stages, the stages of execution of subsidies and the system of pillars and belts, involves a considerable effort, through the extensive cuts / displacements in the masonry, in order to be able to allow the incorporation of the new reinforced concrete elements;
- the particularities of reinforcement, formwork and pouring of concrete in geometrically limited areas;
- difficulties in ensuring the stability of the masonry panels that remain in position;
- from the point of view of resistance, this solution successfully ensures the classification of the building in the seismic risk class RsIV ($R3 > 1$);
- by introducing the system of pillars and belts, practically a new structural system of confined masonry is obtained, which offers safe performance of resistance and stability, and especially a better level of ductility compared to that ensured by reinforced linings;

the system of pillars and belts ensures a level of confinement of the masonry, higher than that provided by the shirts, applied on the surfaces of the masonry walls;

- from an economic point of view, ~ 245 euro / sqm represents an optimal economic value, related to the technological difficulties;
- from the point of view of seismic risk, the solution can ensure the building in the RsIV class;
- in this situation, in which the building has the character of a monument, especially due to the main monumental facade, by implementing this consolidation solution, in this situation the architectural and historical elements are not affected; it should be noted that the option is not a reversible solution, as recommended in the documents in the field of monument restoration;
- the major disadvantage of such a consolidation solution is that in the situations of buildings where it is not allowed to affect in any way the existing elements / materials, this solution can not be practically adopted for historical monuments, because it involves too much masonry replacement;
- another disadvantage is the one related to the technological difficulties of intervention and the increased execution times.

Consolidation solution 3 - base isolation of the building

- from a technological point of view, if only the introduction of the base insulation system and the integral realization of new reinforced concrete floors is adopted, the major difficulties are those related to the realization of the specific system for the introduction of seismic insulators, which involves the introduction of a reinforced concrete screed. , with the role of lower load-bearing frame, and a network of reinforced concrete beams, arranged in the plane of the walls, which have the role of upper load-bearing frame;
- the introduction of new reinforced concrete floors, instead of wooden ones with lower rigidities, will have the role of increasing the overall rigidity of the building structure, including to respond as a "*rigid body*", placed on seismic insulators;
- from the point of view of resistance and stability, this solution ensures the successful classification of the building in the seismic risk class RsIV (R3 >> 1), as practically the structure is loaded limited and controlled to seismic load, the major efforts being taken over by insulators and considerably dissipated by them through considerable deformability capacities;
- from an economic point of view, ~ 186 euro / sqm represents an optimal economic value, related to the difficult
- in this situation, in which the building has the character of a monument, especially due to the main monumental facade, by implementing this consolidation solution, the character of historical monument and architecture is not affected in any way; In this situation, there is no question of reversibility, given the minimally invasive nature;
- although, in general, the least technical advantage, it is obvious, in this situation, from a technological point of view, the implementation of the base insulation solution is practically impossible to implement, because the building is positioned exactly between two neighboring buildings (left and right) , there are not even expansion / seismic joints; therefore, distances compatible with the required energy dissipation of the insulators could

not be ensured; a possible procedure to achieve these "distances" would involve "effective cutting" of the structure of the building, at least 30 cm of each heel, which unjustifiably complicates the consolidation technology;

7.3 Personal contributions and future research directions

- Identifying and synthesizing the important aspects regarding the rehabilitation, consolidation, conservation and protection of historical monuments.
- Identifying and synthesizing general aspects related to the seismicity of Romania, by highlighting the adverse effects of major earthquakes in the last century, emphasizing in particular the impact of the earthquakes of 1940 and 1977, and the seismic vulnerability of the built fund, including heritage constructions.
- approaching for study some aspects regarding the seismic calculation of existing buildings, emphasizing the importance of assessing the seismic response of a structure, by directly determining its fundamental period of vibration, using the method of recording environmental vibrations (AVT - Ambient Vibration Tests), whose studies began in the 1930s (in the form of forced vibrations), intensified in the 1960s and 1970s (in the form of ambient vibrations).
- articipation and direct involvement in the dynamic registration activities of two existing buildings in Bucharest, and performing specific structural calculations, using numerical modeling (ETABS finite element program), in order to perform comparative analyzes for three potential consolidation solutions, by highlighting both structural and technological performance, and especially the economic ones (estimated intervention costs).
- he main research direction, which I want to continue, is related to the dynamic evaluation of as many buildings as possible, especially historical monuments, in order to create an extensive database, with the help of which I can make a characterization. as faithful as possible to reality, in terms of the level of vulnerability of the built historical heritage.

7.4 Dissemination of results

At the time of presenting the doctoral thesis, the following articles appear in the scientific articles:

- **main author:**

  - **Daniel I. DIMA**, Pavlu TEO, Constantin BUDAN, *Dynamic response assessment of a historic building from Bucharest, Romania, using the method of ambient vibration tests*, 3rd International Conference on PROTECTION OF HISTORICAL CONSTRUCTIONS Lisbon, Portugal, 12 – 15 July, 2017
  - **Daniel I. DIMA,** *Mass-media şi predicţia cutremurelor din România. Percepţie şi efecte sociale*, Buletinul ştiinţific U.T.C.B., nr. 1/2019
  - **Daniel I. DIMA**, *Vulnerabilitatea seismică a fondului existent din România şi influenţa mass- media asupra percepţiei sociale privind siguranţa seismică*, Buletinul ştiinţific U.T.C.B., nr. 1/2019

- **Daniel Ioan DIMA**, Teodor Pavlu, *The fundamental vibration period determination for an existent historical monument building, by using numerical modelling, empirical and experimental (AVT) methods*, Mathematical Modelling in Civil Engineering, Vol. 16-No. 1: 22-33-2021 Doi: 10.2478/mmce-2021-0008
- **Daniel Ioan DIMA**, Andreea Duţu, *Traditional buildings with timber frame and various infills in Romania*, World Conference on Timber Engineering, August 22-25, 2016, Vienna, Austria

co-author:

- Alexandru ALDEA, Andreea DUTU, Sorin DEMETRIU, **Daniel I. DIMA**, *Dynamic properties identification for a timber framed masonry house*, Revista Construcţii – No. 1 / 2020, București
- Andreea Dutu, Mihai Niste, Iulian Spatarelu, **Daniel Ioan Dima**, Shoichi Kishiki, *Seismic evaluation of Romanian traditional buildings with timber frame and mud masonry infills by in-plane static cyclic tests*, Engineering Structures 167 (2018) 655–670
- Eliza Bulimar, Andreea Dutu, **Daniel Ioan Dima**, Razvan Ietan, *Seismic Analysis of Timber Frames with Infills in Romania*, Tamap Journal of Engineering Volume 2017, Article ID 13

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