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HABILITATION THESIS

Bearing structures in architecture.

Past, present and future

Structuri portante in arhitectura.

Trecut, prezent si viitor

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(a) ABSTRACT

The present habilitation thesis synthesizes the most important results of the didactic, research and professional activity of the author in order to obtain the title of doctor engineer, research series C number 0004961, issued by the Order of the Ministry of Education and Research with the number 3876 from 19.05.2004.

The author began his research activity with the elaboration of the diploma work in 1994, under the scientific coordination of Prof. Valeriu Stoian in the domain of anti-seismic design of structures with reinforced concrete walls. The PhD thesis "Contributions to the calculation and conformation of reinforced concrete structural walls" studied the seismic behaviour of reinforced concrete structural walls; it was finalized in 2003 and was publicly defended in 2004.

In 1998, the author started his collaboration with Prof. Dr. H.C. Victor Gioncu, as an assistant within the Faculty of Architecture and Urbanism from Timisoara. Under the scientific coordination of Prof. V. Gioncu, the author performed research, design and didactic activities in the design and consolidation domain of reinforced concrete, steel, masonry and timber bearing structures subjected to different types of actions. Within his studies, the author showed a great deal of interest for the satisfaction of the aesthetical and functional requirements of these bearing structures, based on the newest design, consolidation and execution philosophies. Together with architects Prof. I. Andreescu and Prof. V. Gaivoronschi, with Dr. Eng. A. Anastasiadis and other international collaborators, as well as with colleagues from H.I. STRUCT design office, the author has performed original research studies which later led to theoretical and practical contributions in the design and consolidation domain of bearing structures used in architecture.

Research domains in which the author brought innovative theoretical contributions after the completion of his PhD thesis:

- *Reinforced concrete structural walls.* Personal contributions: explanation of the brittle failure mechanisms developed by the walls with ordered vertical openings based on recordings of the acceleration component and speed of the seismic waves measured in the field and in buildings;
- *Historic masonry bearing structures*. Personal contributions: development of calculation methodologies specific for the failure modes of mosques, synagogues and orthodox churches, within the PROHITECH research program, based on the theory of failure blocks;
- *Reinforced concrete frame structures with masonry infills*. Personal contributions within the INSYSME research program: identification of the failure modes of these structures and the proposal of new technologies for the increase of the bearing capacity of these infill walls subjected to out-of-plane solicitations;
- *Timber framing systems*. Personal contributions within the COST FP1101 research program: identification of new types of timber framing systems, consolidation solutions, in-situ investigations, failure mechanisms and development of a methodology for their vulnerability assessment;
- *Steel bearing structures*. Personal contributions: investigation of the influence of the cyclic loading type and of the loading speed on the local and global ductility of steel frames as well as on the steel elements and connections;
- *Seismic vulnerability of historic centres*. Personal contributions: studies performed on individual buildings and aggregates of buildings from Timisoara;
- *Research and development of new ways of teaching structural design in architectural schools*.

Published articles by the author:103; 12 in ISI journals, 14 ISI proceedings, having 23 citations in ISI journals and 13 in ISI conferences; 1 published book, co-author for 3 international books, associate editor for 1 international book; 2 courses and 2 practical guides. The author has participated in 3 international research grants, among which 2 as coordinating director for Romania, and in 4 national research grants.

The author will continue to develop research activities in the following domains:

(i) Design of bearing structures:

- Theoretical and experimental analysis of the failure modes, ductility and rigidity developed by steel, reinforced concrete, timber and masonry structures, subjected to seismic actions;
- Experimental and theoretical research for the study of the influence of infill walls on reinforced concrete frame structures, within the INSYSME European research contract;
- Development of a calculation methodology based on rigid failure blocks, for historic orthodox churches from Banat region;
- Theoretical and experimental study of several types of composite or r. c. shear walls with staggered openings;
- Investigation of the influence of the cyclic loading type and of the loading speed on the local and global ductility of steel moment resisting and braced frames;
- Development of new details or structural systems;
- Development of a methodology to implement the ductility verification in practical design and in the corresponding design codes, thus continuing Prof. Gioncu's studies;
- Investigation of the architectural design impact on the structural design of constructions, particularly on the seismic behaviour;

(ii) Assessment and consolidation of historical bearing structures:

- Identification of new consolidation methods and technologies for modern and historic timber, reinforced concrete, steel and masonry elements and buildings, using modern materials and techniques;
- Organization of an experimental monitoring program of the behaviour of historic buildings before, during and after the consolidation of their bearing structures;
- Research with the help modern measurement and control techniques of the degradation level of buildings and historic sites with heritage value;

(iii)Vulnerability of historic buildings and historic urban areas:

- Elaboration of a rapid method for the evaluation of the vulnerability of historic masonry and timber bearing structures;
- Elaboration of vulnerability maps for historic buildings and sites, in function of the structural systems, materials used and specific earthquakes for Romania.

Within the Faculty of Architecture and Urbanism from Timisoara, the candidate will continue his collaboration with architects and engineers from didactic and scientific point of view, in regard to the future development of bearing structures which would satisfy the aesthetical and functional requirements of buildings, proposal of new consolidation methods and technologies for buildings classified as monuments, as well as the training process of future architects and engineers. The development plans and future activities of the candidate are presented in more detail in *Chapter (b-ii): Future scientific, professional and academic development plans.*

(b) ACHIEVEMENTS AND DEVELOPMENT PLANS

(b-i) SCIENTIFIC, PROFESSIONAL AND ACADEMIC ACHIEVEMENTS

1 INTRODUCTION

Starting with 1994, when I have graduated the Civil Engineering Faculty from Timişoara until the present time, I have conducted theoretical and experimental research in the anti-seismic design field of structures and consolidation of historic bearing structures.
I have started my research activity with the license diploma work and I have continued as a doctoral student under the coordination of Prof. Valeriu Stoian in the domain of anti-seismic design of structures with reinforced concrete walls. The license diploma was elaborated within the Tempus program in 1994 and it studied the seismic behaviour of a dual 9 storey dual structure subjected to various seismic solicitations. The PhD thesis was begun in 1995, having the title "Contributions to the calculation and conformation of reinforced concrete structural walls" and it was finalized in 2003. I have obtained the title of doctor engineer, series C number 004961 issued by the Order of the Ministry of Education and Research with the number 3876 from 19.05.2004.
In 1997 I have started the activity within the Faculty of Architecture and Urbanism from Timisoara, as the assistant of the late Prof. Dr. H.C. Victor Gioncu, member of the Romanian Technical Academy of Science. Under his coordination I have conducted research activities in the field of seismic engineering for steel structures, masonry structures and in the field of consolidation of historic bearing structures.

After finishing my PhD Thesis, I was an active participant in 3 European research programmes:
• FP6 PROHITECH "Seismic Protection of Historical buildings by reversible mixed Technologies" – 2004-2008, member of the research team;
• FP7-SME-2013 INSYSME "INnovative SYStems for earthquake resistant Masonry Enclosures in rc buildings" – 2013-2016, coordinating director for Romania;
• COST FP1101 action: "Assessment, Reinforcement, and Monitoring of Timber structures" – 2011-2015, coordinating director for Romania;

INNOVATIVE THEORETICAL CONTRIBUTIONS performed by the author after the completion of his PhD thesis, had a defining role in the design and strengthening strategies of bearing structures from the following research domains:
• Seismic behaviour of reinforced concrete structural walls, especially those with staggered openings after the earthquakes from 2009-2011. Personal contributions consisted of explaining the brittle failure mechanisms developed by the walls with ordered vertical openings based on new recordings of the acceleration component and speed of the seismic waves measured in the field and in buildings and explain based on these factors the good behaviour of walls with staggered openings;

• Seismic behaviour of historic masonry bearing structures within the PROHITECH research program. Personal contributions of the author led to the explanation and development of calculation methodologies specific for the failure modes of mosques and orthodox churches, based on the theory of failure blocks

• Seismic behaviour of reinforced frame structures with masonry infills within the INSYSME research program. Personal contributions of the author consisted in the identification of the failure modes of these structures based on the recordings during the L'Aquila earthquake and the proposal of new technologies of increasing the bearing capacity of these infill walls to out-of-plane solicitations;

• Wood bearing structures and especially historic ones, such as wooden churches wood framing systems. Research carried out by the author, as grant director for Romania, within the COST FP1101 research program led to the identification of new types of wood framing systems, their failure mechanisms, spatial collaboration with other building elements and their vulnerability assessment. For wood churches, the author carried out studies which brought new information on the failure modes, types of structural systems, in situ evaluation of the bearing structure and reversible consolidation solutions;

• Seismic vulnerability of historic urban centres. Research coordinated by the author, on behalf of the Faculty of Architecture and Urbanism, and it represented a premiere in Romania, seismic vulnerability maps being created for the first time for complexes of buildings. Studies began in Timisoara in 2014 in collaboration with the University of Padova, within the workshop "Seismic vulnerability of historical centres".

Concerning future activities and development plans of the candidate, there will be continued and started new studies related to the research directions of the Faculty of Architecture and Urbanism from Timisoara, in the following domains:

(i) Design of bearing structures:

• Theoretical analysis of the different failure modes developed by steel, reinforced concrete and masonry structures relative to the seismic source (far-field and near-field);

• Experimental and theoretical research for the study of the influence of infill panels on reinforced concrete frame structures, thus continuing the activity started within the European research contract INSYSME;

• Development of a calculation methodology based on failure blocks, for orthodox churches from Banat region, made from brick masonry, continuing the activity started in the European research contract PROHITECH;

• Theoretical and experimental study of several types of composite or reinforced concrete shear walls with staggered opening;

• Development of new ecological materials and technology for building construction;

(ii) Assessment and consolidation of historic bearing structures:

• Identification of new consolidation methods and technologies of wood, reinforced concrete, masonry elements and buildings, using modern materials;

• Organization of an experimental monitoring program of the behaviour of historic buildings before, during and after the consolidation of their bearing structures;

• Research with the help of drones and other modern measurement equipments, which should gather information and which can indicate degradations of buildings and historic sites of great heritage value;

(iii)Vulnerability of historic buildings and urban centres:
• Elaboration of rapid calculation methods for the vulnerability of historic buildings and complexes of historic buildings;
• Implementation of seismic vulnerability studies for archaeological sites;
• Implementation of vulnerability maps for various historic urban centres;

The most important papers were published in international certified journals, such as:
• In the field of steel constructions 4 papers: 3 papers in Journal of Constructional Steel Research (2012-2014) Ed. Elsevier, 1 paper in Bulletin of Earthquake Engineering (2014) Ed. Springer;
• In the field of reinforced concrete structures 2 articles in Engineering Failure Analysis Ed. Elsevier (2013, 2014);
• In the field of consolidation of historic bearing structures 3 articles in Journal of Cultural Heritage, Ed. Elsevier, 2012;
Taking into account the actuality of the research theme and the results produced, the following papers cited the results of the research developed by the author within research groups in the most recent period:

- P. Foraboschi, M. Mercanzin, D. Trabucco, "Sustainable structural design of tall buildings based on embodied energy" Vol. 68, Part A, January 2014, Pages 254–269, *Energy and Buildings*, DOI: 10.1016/j.enbuild.2013.09.003, Ed. Elsevier;
- Esra Mete Güneyisi , Mario D'Aniello, Raffaele Landolfo, Kasım Mermerdaş "A novel formulation of the flexural over strength factor for steel beams", Vol. 90, nov. 2013, pp. 60–71, *Journal of Constructional Steel Research*, Ed. Elsevier;
- Shokouhian, M., Shi, Y., "Classification of I-section flexural members based on member ductility", (2014) *Journal of Constructional Steel Research*, Vol. 95, April 2014, Pp. 198–210, http://dx.doi.org/10.1016/j.jcsr.2013.12.004, Ed. Elsevier;
- Mario D'Aniello, Esra Mete Güneyisi Raffaele Landolfo, Kasım Mermerdaş "Analytical prediction of available rotation capacity of cold-formed rectangular and square hollow section beams", 77(2014), pg. 141-152, *Thin Wall Structures*, Ed. Elsevier;
- Mohammad Panjehpour, Abang Abdullah Abang Ali and Farah Nora Aznieta, "Energy absorption of reinforced concrete deep beams strengthened with CFRP sheet", Vol. 16, nr.5, 2014, pp. 481-489, *Steel and Composite Structures*, , DOI: 10.12989/scs.2014.16.5.481
- Flavia De Luca, G.M. Verderame, G. Manfredi, "Eurocode-based seismic assessment of modern heritage RC structures: the case of the Tower of the Nations in Naples (Italy)", *Engineering Structures* Volume 74, 1 September 2014, Pages 96–110, DOI: 10.1016/j.engstruct.2014.05.015, Ed. Elsevier;
- Francesca Ceroni, Marisa Pecce, "Evaluation of Bond Strength in Concrete Elements Externally Reinforced with CFRP Sheets and Anchoring Devices", *Journal of Composites for Constructions*, Volume 14, Issue 5, September 2010, Pages 521-530; DOI: 10.1061/(ASCE)CC.1943-5614.0000118;

- Francesca Ceroni, Marisa Pecce, Stjin Matthys, Luc Taerwe, “Debonding strength and anchorage devices for reinforced concrete elements strengthened with FRP sheets”, *Composites Part B: Engineering*, Volume 39, Issue 3, April 2008, Pages 429-441, DOI: 10.1016/j.compositesb.2007.05.002;
- Todut C., Dan D., Stoian V., “Theoretical and experimental study on precast reinforced concrete wall panels subjected to shear force”, *Engineering structures*, Volume 80, December 01, 2014, pages 323-338; DOI**:** 10.1016/j.engstruct.2014.09.019

As a structural designer and member of a design team, I elaborated more than 150 projects that were already built it in the proximity of Timişoara, Romania.

I received recognition from the most important Romanian professional association AICPS – Romanian Association of Structural Design Engineers, where presently I am certified as a senior design engineer.

In 2008 together with the colleagues from H.I. STRUCT design office and Eng. C. Lannert, we have won 3rd national prize awarded by AICPS for the bearing structure from the office building complex City Business Centre Timişoara, for which the architects were my colleagues from the Faculty of Architecture and Urbanism, Prof. V. Gaivoronschi and Prof. I. Andreescu.

From 2009 I am certified by the Ministry of Culture and National Heritage as a specialist conductor of studies and researches, for the inventory and classification of buildings with heritage value, project manager for historic bearing structure projects, site supervisor, and responsible for monitoring the in time behaviour of historical buildings.

Presentations within national and international Conferences:

The scientific activity in the mentioned period increased constantly reaching the following level:

- ISI journals – 12
- ISI proceedings – 14
- BDI journals – 3
- BDI proceedings - 10
- Papers in International conferences – 30
- Citations in the international literature – 36
- Books – 3 as author, 1 as associate editor
- Chapters in books internationally published – 3
- Coordinator for Romania in European Research Grants – 2
- Active member in European Research Grants – 1
- Participation at international conferences with communications – 25

It has to be mentioned that during 2004 - 2014, I elaborated in total 101 papers (33 as corresponding author).

As recognition for the quality of the performed research activity, I was asked to be a reviewer for prestigious publishing houses for international Journals:

- Ed. Elsevier: *Engineering Structures* (2013); *Thin walled structures* (2013); *Journal of Constructional Steel Research* (2013), *Engineering Failure Analysis* (2012),
- Ed. Taylor and Francis: *International Journal of Architectural Heritage*(2014); *European journal of Environmental and Civil Engineering* (2014)
- Ed. Springer: *Bulletin of Earthquake Engineering* (2014),
- Ed. Ernst & Sohn Whiley : *Structural Concrete* (2013)

For my contributions in the research domain of buildings situated in seismic zones, I was invited as a member in the international scientific committees at the following conferences:

1. PROHITECH '14, *"2nd International conference on protection of historical constructions*", 2-7 May 2014, Antalya, Turkey;
2. VANEQS 2013, "*International Van earthquake symposium",* 23-27 October, 2013, Van, Turkey;
3. RICH 2014, *"2nd International Conference Robotics: Innovation for Cultural Heritage",* 6th-7th October, 2014, Venice, Italy;
4. SHATIS'15, "*3rd International Conference on Structural Health Assessment of Timber Structures*", September 9-11, Wroclaw, 2015, Poland;

Published articles constituting the habilitation thesis:

The habilitation thesis presents the results of the research performed by the author and published in the following ISI Journals:

1) **MOSOARCA M.**, Seismic behaviour of reinforced concrete shear walls with regular and staggered openings after the strong earthquakes between 2009 and 2011, *Engineering Failure Analysis* (2013) Vol. 34, pp. 537-565, Ed. Elsevier, DOI: 10.1016/j.engfailanal.2013.05.014, ISSN: 13506307;

2) **MOSOARCA M.**, Failure analysis of RC shear walls with staggered openings under seismic loads, *Engineering Failure Analysis* (2014) Vol. 41, pp. 48-64, Ed. Elsevier, DOI: 10.1016/j.engfailanal.2013.07.037, ISSN: 135-6307, WOS: 000334511500006;

3) **MOSOARCA M.,** Gioncu V., Failure mechanisms for historical religious buildings in Romanian seismic areas, *Journal of Cultural Heritage* (2013), Vol. 14, Issue 3 SUPPL, pp. e65-e72, Ed. Elsevier, DOI: 10.1016/j.culher.2012.11.018, ISSN: 1296-2074, WOS: 000327013800011;

4) **MOSOARCA M.**, Gioncu V., Historical wooden churches from Banat region, Romania. Damages. Modern consolidation solutions, *Journal of Cultural Heritage* (2013) Vol. 14, Issue 3 SUPPL, pp: e45 – e59, Ed. Elsevier, DOI: 10.1016/j.culher.2012.11.020, ISSN: 1296-2074, WOS: 000327013800009;

5) **MOSOARCA M**, Gioncu V., Structural safety of historical buildings made of reinforced concrete, from Banat region - Romania, *Journal of Cultural Heritage* (2013) Vol. 14, Issue 3 SUPPL, pp. e29 – e34, Ed. Elsevier, DOI: 10.1016/j.culher.2012.11.015, ISSN: 1296-2074, WOS: 000327013800006;

6) Gioncu V.**, MOSOARCA M**, Anastasiadis A., Prediction of available rotation capacity and ductility of wide-flange beams: Part 1: DUCTROT-M computer program, *Journal of Constructional Steel Research* 69 (2012) pp. 8-19, Ed. Elsevier, DOI: 10.1016/j.jcsr.2011.06.014, ISSN: 0143-974X, WOS: 000297894100002;

7) Anastasiadis A., **MOSOARCA M.**, Gioncu V., Prediction of available rotation capacity and ductility of wide-flange beams: Part 2: Applications, *Journal of Constructional Steel Research* 69 (2012) pp. 176-191, Ed. Elsevier, DOI: 10.1016/j.jcsr.2011.08.007, ISSN: 0143-974X, WOS: 000296171200017;

8) Gioncu V.**, MOSOARCA M**, Anastasiadis A**.,** Local ductility of steel elements under near-field earthquake loading, *Journal of Constructional Steel Research* 101 (2014) pp. 33-52, Ed. Elsevier, DOI: 10.1016/j.jcsr.2014.05.001, ISSN: 0143-974X, WOS: 000340336200004;

9) Gioncu V., **MOSOARCA M**., Anastasiadis A., Investigation of the cyclic inelastic capacity of steel beams through the use of the plastic collapse mechanism, *Bulletin of Earthquake Engineering* (2014) Ed Springer, DOI: 10.1007/s10518-014-9665-2, ISSN: 1570761X;
10) Andreescu I., Gaivoronschi V., **MOSOARCA M**, The hidden gem, *Advances Material Research* (2013) Vol. 778, pp. 880-887, Trans Tech Publications, Switzerland DOI:10.4028/www.scientific.net /AMR.778.880, ISSN: 1022-6680, WOS: 000336185300111;

2 REINFORCED CONCRETE BEARING STRUCTURES: FROM SEISMIC BEHAVIOUR TO CONSOLIDATION SOLUTIONS

2.1 Seismic behaviour of reinforced concrete shear walls with regular and staggered openings after the strong earthquakes between 2009 and 2011

The strong earthquakes recorded worldwide, between 2009 and 2011, have shown that the damages and the failure mechanisms of the reinforced concrete structural walls depend on a series of factors, such as: the shape in plan and elevation, the dimensions of the walls and openings, the reinforcement and the openings layout, the site conditions, the type of earthquake and the strain rates. Even if failure modes have been extensively researched, there are still certain failure modes we know little about. This is the case of the walls with staggered openings, whose rigidity, bearing capacity, high ductility were highlighted after the earthquakes of 1985 and 2010. Within the PhD thesis, the author has performed theoretical and experimental studies on three types of walls with vertical staggered openings, one with regular openings and a solid wall. The models were loaded until failure and provided information on: the forces, the horizontal displacements, and the maximum stresses and strains recorded in the concrete and in the reinforcement. We compared the sequence of the yielding of the reinforcement and the crushing of the concrete, for models with the same amount of reinforcement and the same physical and mechanical properties of the concrete. The innovations brought by this study are: to present the failure mechanisms recorded after the earthquakes between 2009 and 2011, to explain their failure modes based on the latest recordings of seismic wave characteristics, to present the recordings made at the ground level and on the bearing elements of the constructions and to analyse the advantages of the reinforced concrete structural walls with staggered openings subjected to seismic loads function to the position of the openings.

2.1.1 Introduction

During the period between 2009 and 2011, there were recorded a series of strong earthquakes which caused a lot of deaths and significant damages. Thus, for example, in Europe: L’Aquila, Italy, earthquake, April 6, 2009, (6.3 magnitude) **[2.1]** , in South America: earthquake in Chile, February 2, 2010, (8.8 magnitude) **[2.2]** , Christchurch, New Zealand, February 22, 2011, strong quakes (6.3 magnitude) **[2.3]** , Tohoku, Japan, March 11, (9.0 magnitude) **[2.4]** , Ercis-Van, Turkey, October 23, (7.1 magnitude) **[2.5]** .
The buildings with reinforced concrete structural walls have recorded several failure modes of the walls after these earthquakes, which confirmed the results of the theoretical and experimental tests. The research based on the failure modes developed by the reinforced concrete buildings, after the strong earthquakes between 2009 and 2011, identified new factors which had led to the collapse and damage of the buildings.

After the devastating earthquakes of Northridge and Kobe, there was a continuous development of the systems and of the technologies to record the displacements, speeds and velocities produced by the seismic waves in the foundation ground and in the bearing elements on the full height of the structures. Consequently, new factors which had led to the collapse of the buildings were discovered in seismic engineering.

2.1.2 Failure modes of reinforced concrete shear walls after the earthquakes in: Italy, Chile, New Zealand, Japan and Turkey

Despite all the failure modes known and studied in this domain, there is also a series of new factors, which can explain some less known failure modes of reinforced concrete structural walls. We are going to discuss the factors affecting the failure of the reinforced concrete structural walls, resulting from the research of the earthquakes between 2009 and 2011 **[2.14] [2.17]**.

The principles of the structural failure developed in the last years, revealed the following new factors for the collapse of the buildings:

(i) City-soil factor. This shows the influence that buildings have on the foundation ground and on the seismic forces that are transmitted to the neighbouring structures, in densely built areas. Due to the weight of the buildings, the properties of the ground are changed and the acceleration and frequency are modified. This phenomenon is referred to as seismic soil-structure interaction (SSI), and induces non-classically damped systems. The importance of SSI on the failure modes of the buildings is presented by Gioncu and Mazzolani (2010) **[2.7]**: "due to the vibration introduced in the soil, each building produces a perturbation of the ground motion, being a secondary seismic source."

(ii) The distance from the building to the seismic source. Similar bearing structures develop different failure modes function to the distance to the seismic source. This difference is made by the seismic waves which act on the building, i.e.: buildings that are close the seismic source (near field) are subjected to R and L waves, while P and S surface waves act on the buildings farther from the seismic source (far field). Buildings that are near the field, recorded high values of the velocities of the seismic waves **[2.8]**, very large values of the vertical components of the seismic acceleration **[2.1] - [2.5]** , a reduced number of cycles and short periods of action. All these building particular characteristics, near the seismic source, prevent the development of the plastic hinges, lead to the occurrence of an increase of the strain-rate effects **[2.9]** and to brittle failure modes of the buildings.

Regarding the effects of these high values of the vertical components of the seismic acceleration on the failure modes, Saragoni et al (2010) **[2.10]** state: "The effect of these unusually high vertical components may have greatly affected the response of some tall buildings and it requires more research in the future in order to know that it is necessary to include their effect in buildings code".

(iii) Local site conditions. It is a well-known fact that weak soils amplify the seismic forces and increase the damage of the buildings **[2.11]**.

In conclusion we can say that the 2009 to 2011 earthquakes provided new research directions for the failure modes of the buildings in seismic zones, and led to the improvement of the current design codes.

2.1.3 Factors which influence less known failure mechanisms of RC shear walls

A. Site factors

(i) The influence of the great values of the vertical components of the seismic acceleration of the ground. The EERI reports show great values for these components: 1g at a recording station from Abruzzo [2.1], 0.58g Conception **[2.2]** , between 1.8 and 2.2g Christchurch **[2.15]**. These high values of the vertical components of the acceleration determined the formation of large compression and tension forces in the walls that produced specific deformations in the concrete and reinforcement; values well above those determined by the experimental testing made according to the design codes.

Since high values of the vertical components of the seismic accelerations are difficult to reproduce or simulate in experimental trial laboratories, we cannot know precisely the types of failure modes these reinforced concrete walls develop, and obviously we cannot suggest appropriate reinforcement rules.

Regarding this effect Kam and Pampanin **[2.15]** state: “the effects of the high vertical acceleration of the 22 February 2011 earthquake could have also amplified the compression force demand on RC walls with already non-negligible axial load”. These large values of the vertical components of the acceleration, together with other specific factors can even explain the particular failure, i.e.: the buckling of the structural walls. After the earthquake from Chile, Wallace and Moehle **[2.6]**: describe the phenomenon “a second global buckling mode begins with the spalling of cover concrete, leaving a relatively thin core with longitudinal reinforcement that tends to buckle laterally, displacing the remainder of the wall. As noted in the section on recent earthquake reconnaissance, the latter mode was widely observed in the 2010 Chile earthquake. This latter buckling mode has not been studied previously.” Another possible effect of these vertical components is the different zones in which plastic hinges occur. Generally, the plastic hinges and the failures occur at the base of the reinforced concrete shear walls, but, in figure 2.1 **[2.2]** it can be seen that, in Conception, the failures occurred at the upper storeys of the buildings.

Fig. 2.1 Failures suffered by upper storeys of buildings: Conception, Chile [2.16]

(ii) The influence of the speed of the seismic waves that are transmitted in the bearing structure. The recordings of several buildings made during earthquakes have shown speeds up to 1000m/sec **[2.8]**. Todorovksa and Rahmani **[2.8]** identify a connection between the damages and these speeds: “our analyses provide specific quantitative knowledge about the changes of the wave velocities associated with damage, and about their variability due to factors other than damage“. These large values for the speed of seismic waves trigger the strain-rate effect which is very important for the failure modes of the reinforced concrete shear walls. Shimazaki and Wada **[2.18]** state: “the behaviour during severe earthquake is affected by the concrete component, and the strain rate effect is not negligible”. The effects of the strain rate on the failure mode of the shear walls are hard to clarify and require further studies due to the fact that the high loading speed is difficult to simulate in laboratories. Regarding these failure modes generated by the strain-rate, Carillo **[2.19]** says: when the seismic behaviour of an element or system is studied using the quasi-static method, imprecise interpretations of results can be generated when the governing failure mode is strongly affected by the strain rates”. Information regarding the influence of the strain rate on the failure modes of the reinforced concrete shear walls was obtained by experimental and theoretical testing by Xu et al. **[2.20]**, **[2.21]**. Due to the high loading speed and the reduced number of cycles, the compression capacity of the concrete and the tensile capacity of the reinforcement increase very much so that the walls cannot develop plastic hinges on a large area. Localized cracks in the concrete are recorded, that lead to brittle failure modes. According to Kam and Pampanin **[2.15]**, this strain-rate effect can be the main cause for a less known failure mode, recorded by the shear walls after the Christchurch earthquake, i.e.:”the lack of a distributed cracking pattern in the plastic hinge zone of the RC walls is also an unexpected observation that requires further research”.

B. The conformation mode of the reinforced concrete shear walls. The influence of the vertical disposition of the openings

The failure modes of the walls with ordered vertical openings have been studied, both theoretically and experimentally **[2.12] [2.13]**. The results indicated clearly the types of the failure modes developed by these shear walls subjected to earthquakes. However, a particular failure mode less studied and less known is the one developed by the walls with staggered openings.

The good behaviour of the reinforced concrete shear walls with staggered openings was observed on the high rise buildings in Chile, after the 1985 earthquake. The research report made, after inspecting a number of 13 buildings built with reinforced concrete walls, indicated a high bearing capacity and stiffness of the walls with staggered openings **[2.22]**. Thus, at the 23 floors building Torre del Amendral, built in 1972, in Valparaiso (Fig.2.2), these structural walls have recorded minor damages, while at the Hanga Roa building, built with reinforced concrete shear walls with staggered openings and orderly arranged openings, brittle failures appeared at the coupled beams above the vertically ordered openings. In the staggered openings there were recorded only inclined cracks without the crush of the concrete in these areas (Fig.2.3).

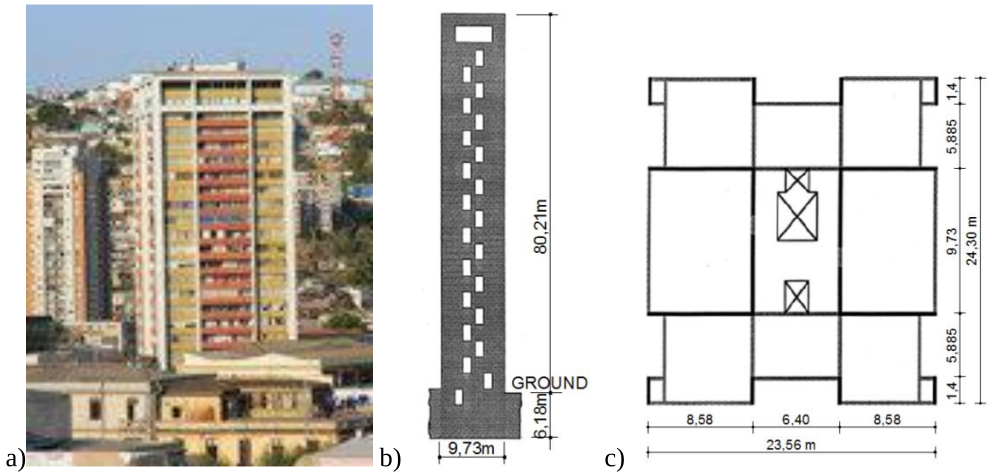


Fig. 2.2 Torre del Amendral building in Valparaiso Chile [2.22]: a) photo b) vertical section, c) horizontal section

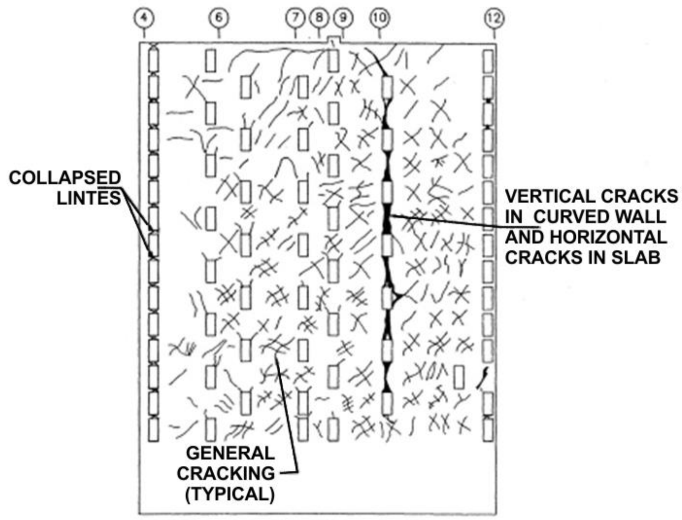


Fig. 2.3 Brittle failures of coupled beams at the shear wall with staggered openings from Hanga-Roa building, Chile, 1985 [2.22]

The very good seismic behaviour of these walls was confirmed after the earthquake of 2010, even though these walls recorded damages after the 1985 earthquake. There were no reports of brittle failures at shear walls with staggered openings after the earthquakes of 2010. Taking into account that the design codes do not have a lot of provisions for reinforcing and for the computation mode for these walls, it is necessary to continue the studies of using the advantages given by these types of walls, in seismic areas.

It seems worth noting that opposite findings have been reached for masonry walls with an irregular layout of openings. In such a case, damage tends to concentrate in some parts of masonry walls, causing a significant reduction in lateral strength and displacement ductility **[2.23][2.24][2.25]**.

In conclusion, we can say that as far as the RC shear walls are concerned they protect the high rise buildings. That is why it is necessary to identify the failure mechanisms that can be developed, since these walls did not reach the failure stage after the earthquakes of 1985 and 2010.

2.1.4 Conclusions

The study performed by the author present a synthesis of the failure modes of the reinforced concrete structural walls after the strong earthquakes in Italy (2009), Chile (2010), New Zealand (2011), Japan (2011) and Turkey (2011).

There are presented failure modes with known causes, but also failure modes that need further studies. Reinforced concrete structural walls with staggered openings represent a special case. These special reinforced concrete walls, obtained as the results of the vertical disposition of the openings, have developed a bearing capacity, rigidity and high ductility and did not record any failure after the earthquakes of 1985 and 2010. Nevertheless, we have to mention that the extraordinary seismic behaviour in which these reinforced concrete walls behaved, were obtained without special tools from the existent design codes.

Reports made by experts after the two earthquakes (1985, 2010) do not mention any local failure of the structural walls with staggered openings. However, they underline the advantages of these walls designed according to rules which are not found in the existing design codes, as far as reinforcement, dissipation of seismic energy and transmission of the seismic forces are concerned.

The research performed by the author provides new information on the seismic behaviour of the reinforced concrete walls with staggered openings by comparing it with the well-known behaviour of the regular vertical opening walls and provides new international research directions for these types of walls.

2.2 Seismic behaviour of shear walls with staggered openings consolidated with FRP

After 2004 when the author presented his PhD thesis, his studies were continued in the CCI department from the Faculty of Constructions from Timisoara through researches performed by Assoc. Prof. T. Nagy-Gyorgy under the coordination of Prof. V. Stoian, focusing on the seismic behaviour of shear walls with staggered openings consolidated with FRP.

2.2.1 Experimental models and the testing methodology of reinforced concrete shear walls

The experimental models were denoted with SW and were designed and experimentally tested by the author. The models had a height of 2600mm, 1250mm width and a storey height of 650mm. In order to avoid the failure of the models due to stability, as a result of the absence of the slabs and bulbs, a thickness of 80mm was chosen. The openings had a dimension of 250 x 500 mm. The experimental models were fixed in foundation blocks having a 400mm height, 35mm width and 1750mm length (Fig.2.4). The concrete from the foundations was casted simultaneously with the concrete from the walls. The dimensions of the walls resulted from size restrictions.

The experimental models were reinforced based on constructive prescriptions from P85/96 code and not from design. In order to ensure bonding between concrete and reinforcement, 6mm ribbed reinforcement of PC52 type was used, having the characteristic resistance of f_{sk}=355N/mm2. Around the openings, steel casings formed from 4 bars of 6mm were placed, having stirrups of the same diameter made from OB37 steel type. The compressive resistance of the concrete used was f_{cm}=50 N/mm^2.

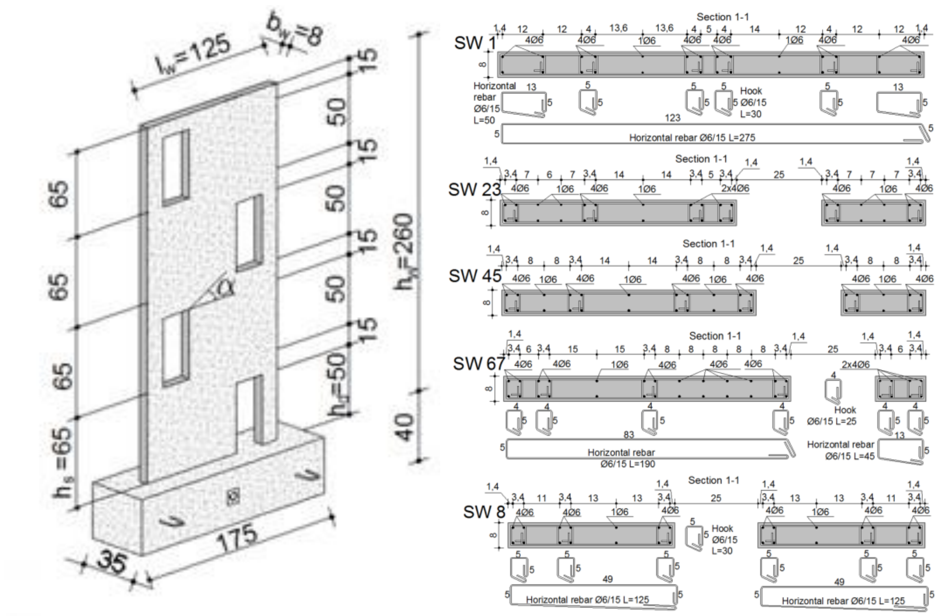


Fig. 2.4 Geometry and reinforcement layout of the test models

The walls were loaded at the superior part with a vertical force of F=50kN. The horizontal load for the solid wall SW1, consisted of a increasing monotonic load until the failure stage, and for the walls with openings, cyclic alternant forces were applied (Fig.2.5). The load was introduced in displacement control (horizontal displacement of the superior part of the experimental models) (Fig.2.6).

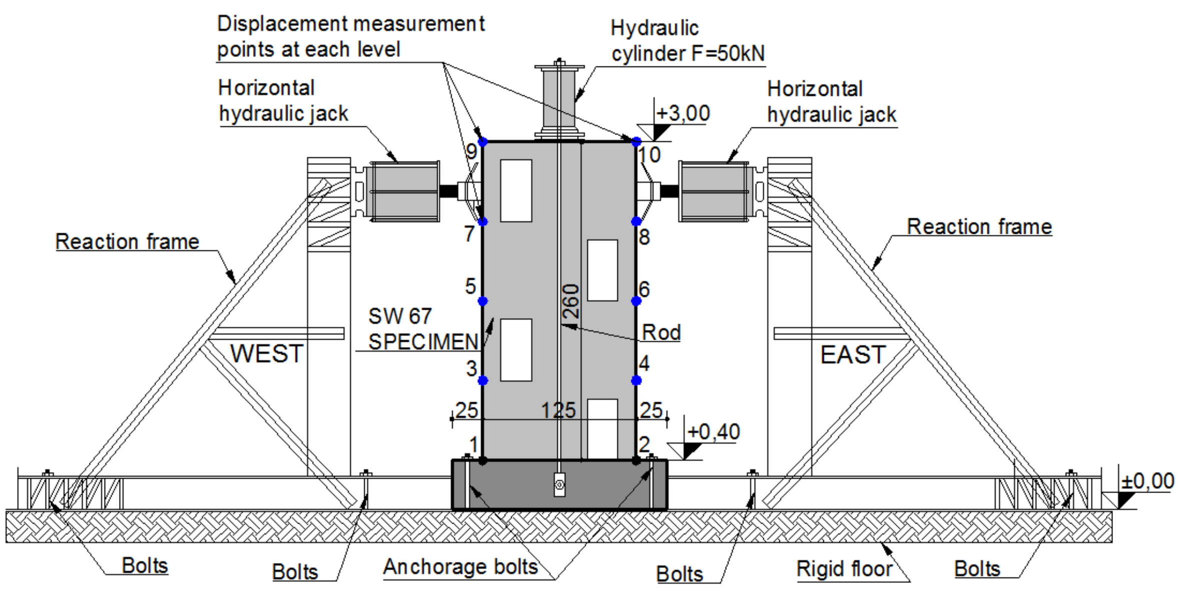


Fig. 2.5 Testing methodology

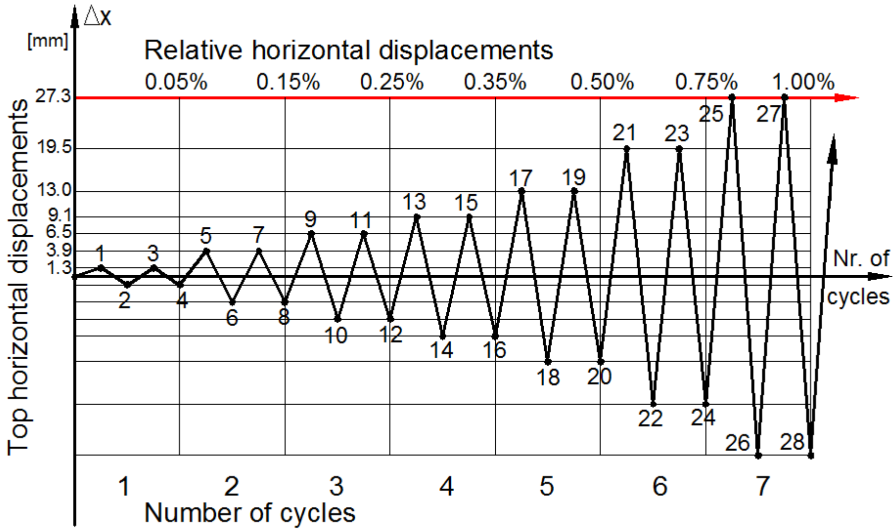


Fig. 2.6 Testing protocol

The following observations were drawn after the load application on the reinforced concrete walls:

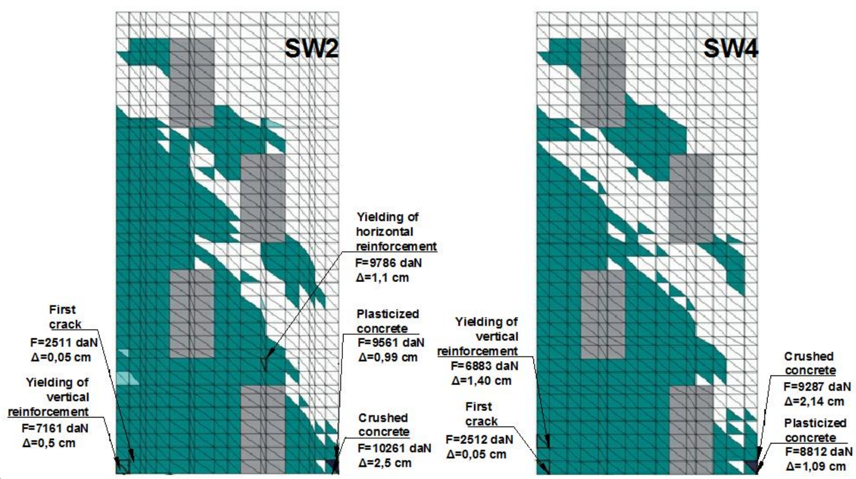
• Solid wall SW1 had a ductile bending failure;

• The wall SW8, having ordered vertical openings, the failure was produced by plastic hinges formed at the level of the coupling beams, followed by the base of the pillars;

• The walls with staggered openings SW23, SW45 and SW67 failed by crushing of the concrete at the base of the small pillar, followed by cantilever behaviour of the larger pillar.

The failure modes studied by the author with theoretical testing (push-over analysis) and the experiments are presented in figure 2.7.

a)



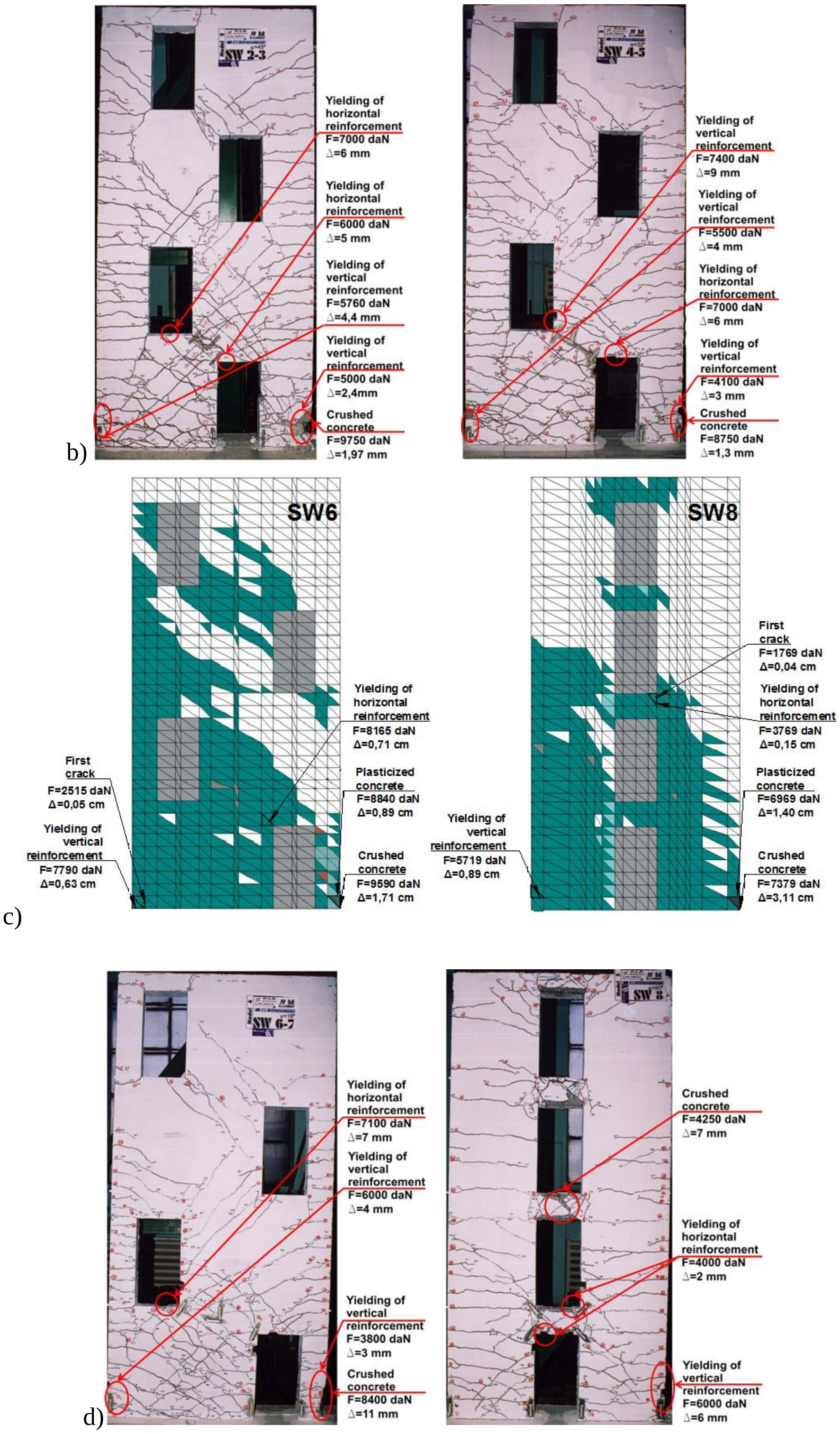


Fig. 2.7 a,b) Theoretical and experimental failure modes of shear walls SW2-3 and SW4-5
c,d) Theoretical and experimental failure modes of shear walls SW6-7 and SW8

The final conclusions were that shear walls with staggered openings, in function of certain values of the α angle, have a similar behaviour to solid shear walls and thus no special measures of reinforcement are necessary for the ductilization of the potential plastic zones **[2.26] [2.27]**.

2.2.2 *Study of the seismic behaviour of the walls consolidated with FRP*

The experimental tests performed on the RC shear walls consolidated with FRP underwent the same procedure as the unconsolidated walls. After testing of the unconsolidated walls, the following procedures were followed:

- Preparation of the walls for the FRP consolidation by cleaning the surfaces to be consolidated, filling of the existing cracks and repairing of the zones with crushed and exfoliated concrete
- Preparation and execution of the anchorage zone of the FRP
- Consolidation with layers of carbon FRP
- Processing and compare of the results before and after consolidation

The selected data that were recorded during the testing were:

- Horizontal load at the top of the model
- Horizontal displacements
- Failure modes of the elements
- Strains in the composite material.

Due to the fact that very little experimental tests were done on RC shear walls consolidated with composite materials, the results obtained are with certainty of great importance and useful. From the experimental tests, it was determined the contribution of the composite material in overtaking bending and shear stress, the efficiency of laying out the composite material and there were checked some recommendations regarding the calculation of this type of consolidation.

The shear walls were consolidated and tested by Assoc. Prof. T. Nagy-Gyorgy **[2.28]**, and they were denoted with RW. The consolidation of these walls was done with unidirectional weaving with carbon FRP on one side of the walls (Fig.2.8). The carbon strands had a mean tensile resistance of f_{frp} = 3900N/mm2, modulus of elasticity E_{frp} = 231000 N/mm2 and an ultimate strain of ε_{frp}=1.7%.

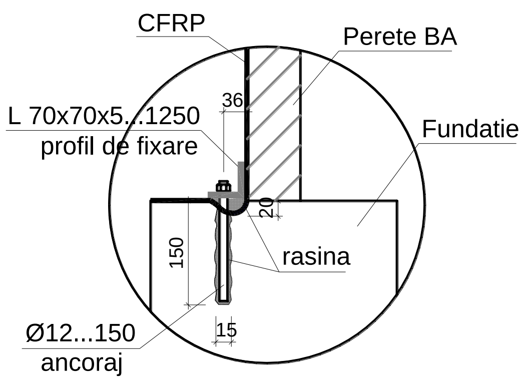


Fig. 2.8 Anchorage system of the carbon fibre strips

Shear wall SW1, after consolidation was denoted as RW1 and the testing of the consolidated element was performed in the same conditions as the initial one. Because initially it was not clear what the capacity of the consolidated element would be, 4 strips of 150mm width of unidirectional carbon fibre weaving were applied on one side of the wall. Later it was confirmed that this quantity of carbon fibre strips increased the capacity of the shear wall with 35% with respect to the initial unconsolidated element. For the next shear walls, 3 vertical strips of 150mm width were used, together with 4 horizontal strips of 150mm width. The vertical strips were placed at the extremities of the walls and in the central zone, and the horizontal strips were placed at the superior part of each level. In order to record the behaviour of the composite material during testing, all the elements had strain gauges on the composite materials in the most stressed zones, oriented along the fibres.

2.2.3 Results of the theoretical and experimental tests

Based on the comparison of results obtained from the original shear walls with staggered openings and the consolidated ones, the following conclusions can be drawn **[2.28]**:

• Consolidation with composite materials of the RC shear walls determine a significant increase of ultimate bearing capacity of these walls (basically, the initial bearing capacity was negligible);

• The recorded strains from the composite material proves its contribution to the bearing capacity of the consolidated walls and the collaboration with the RC element, having values between 0.54 ÷ 0.84%;

• Failure of the consolidated elements occurred by a gradual opening of the existing cracks, by detachment of the composite material in the compressed zone, then in the tensioned zone at the base of the pillar, followed by a tensile and sometimes a compressive fracture of the composite material;

• The maximum horizontal deformations of the consolidated walls were usually larger, or at least identical with the deformations of the unconsolidated walls;

• The results depend mainly on the initial condition of the consolidated element (number and opening of cracks, quantity of yielding reinforcement, the method and materials used for rehabilitation) and the evaluation method used. With the chosen method for evaluating the mechanical characteristics the following observations were made:

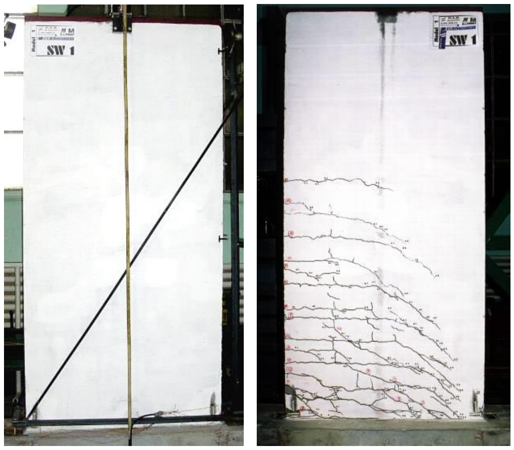
- the rigidity of the elements has decreased with 54%;
- the ductility of the elements decreased with 61%;
- the value of the elastic limit of the walls increased with 48%;
- the value of the maximum load of the walls increased with 46%;
- the composite material strains were between 0.54 - 0.84%;

• The chosen anchorage system had a good performance, without degradations and local failures;

• The RC walls subjected to seismic forces have a ductile behaviour. By consolidating these types of ductile elements with composite materials having non-ductile behaviour, the overall ductility of the element is maintained, but at maximum load a brittle failure is expected.

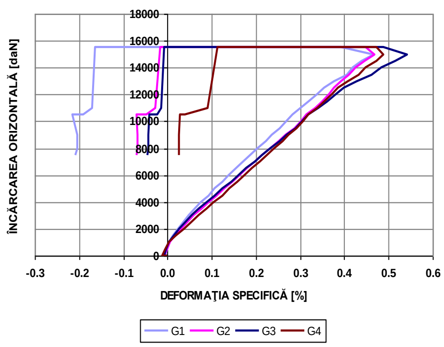
In tables 2.1 and 2.2 the results of the experimental tests are presented for monolith walls and FRP consolidated ones.

In figures 2.9a, 2.11a, 2.13a, 2.15a and 2.17a there are presented the test models prior and after testing (noted with SW) and their failure modes developed at ultimate limit stage. In figures 2.9b, 2.11b, 2.13b, 2.15b and 2.17b, there are presented the RC walls consolidated with FRP (noted with RW) and their failure mode at ultimate limit stage. The strains from the composite materials are presented in figures 2.10a, 2.12c, 2.14c, 2.16c and 2.18c. The force-displacement curves for the SW and RW walls are presented in figures 2.10b, 2.12d, 2.14d, 2.16d and 2.18d.

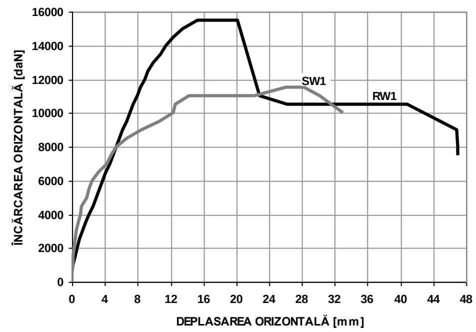
a) 

b) 

Fig. 2.9 Test models a) before and after testing (SW1); b) after consolidation (RW1)



a)



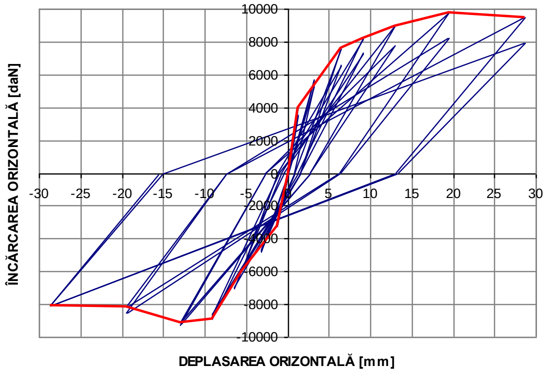
b)

Fig. 2.10 a) Strains in the composite material; b) Force-displacement diagrams of models SW1 and RW1

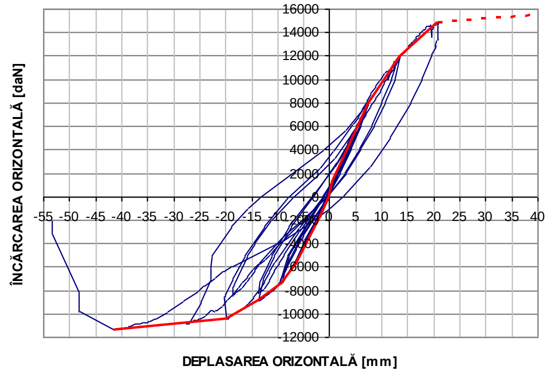
a) 

b) 

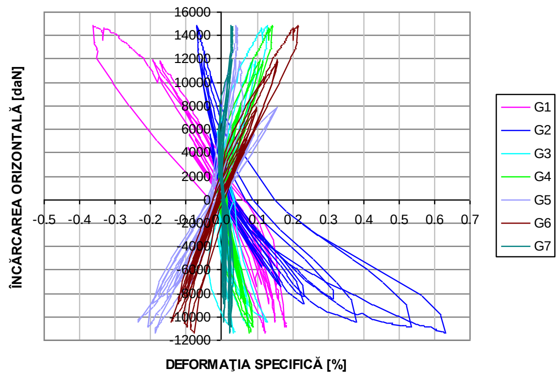
Fig. 2.11 Test models a) before and after testing (SW23); b) after consolidation (RW23)



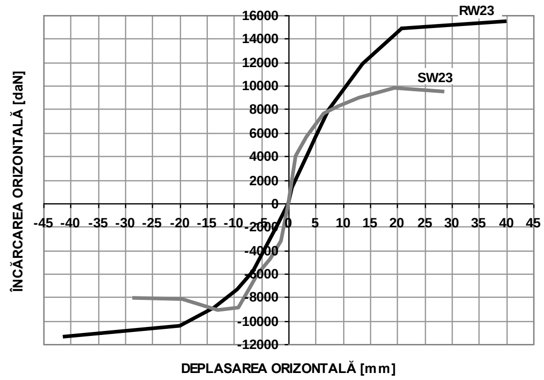
a) Force-displacement diagram of SW23



b) Force-displacement diagram of RW23



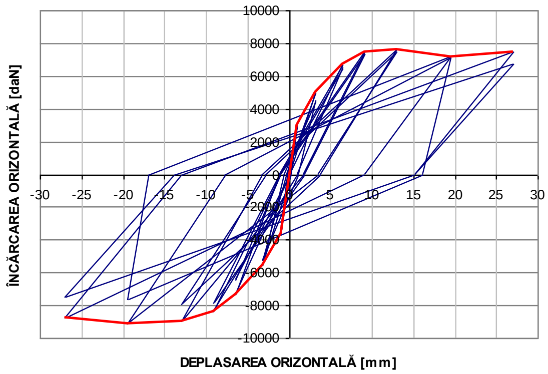
c) Strains in the composite material



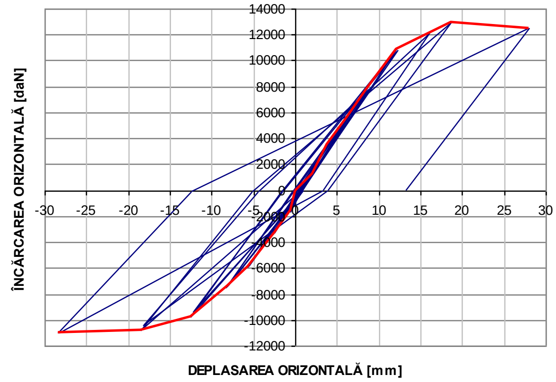
d) Envelope diagrams for models SW23 and RW23

Fig. 2.12 Behaviour diagrams of models SW23 and RW23

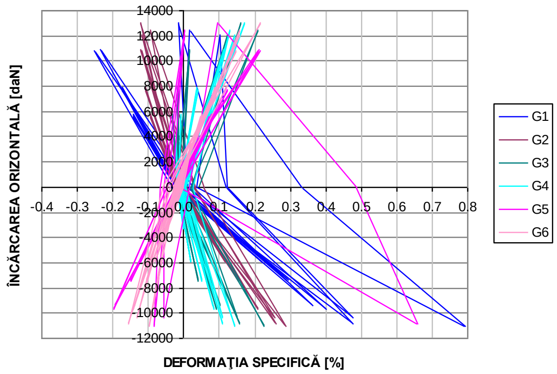
Fig. 2.13 Test models a) before and after testing (SW45); b) after consolidation (RW45)



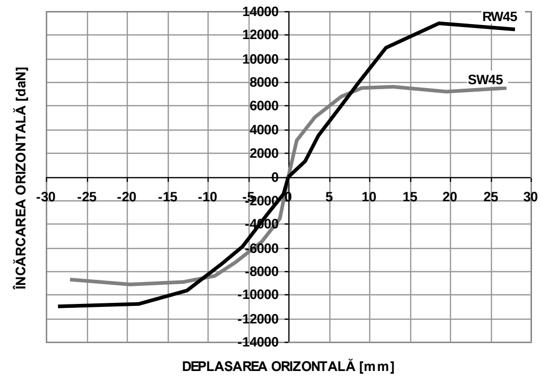
a) Force-displacement diagram of SW45



b) Force-displacement diagram of RW45



c) Strains in the composite material



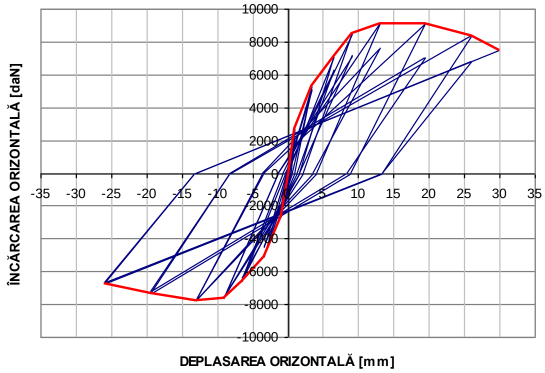
d) Envelope diagrams for models SW45 and RW45

Fig. 2.14 Behaviour diagrams of models SW45 and RW45

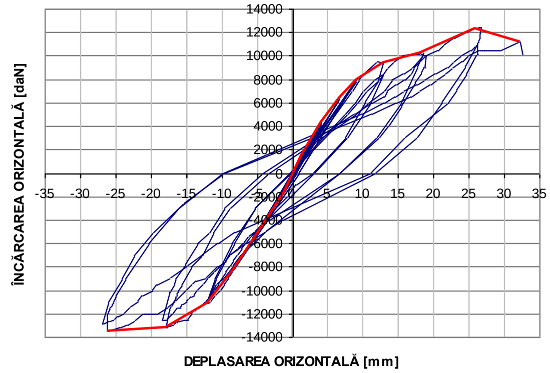
a)

b)

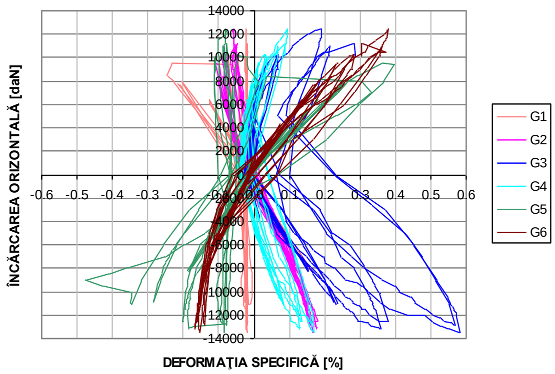
Fig. 2.15 Test models a) before and after testing (SW67); b) after consolidation (RW67)



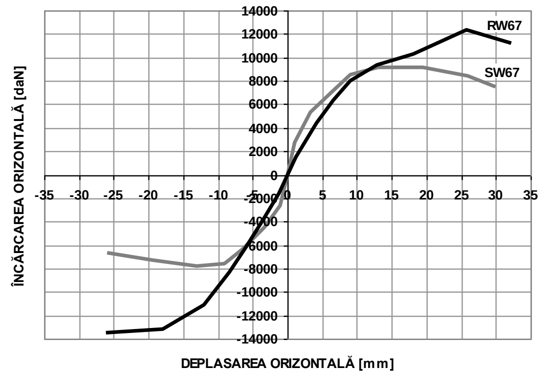
a) Force-displacement diagram of SW67



b) Force-displacement diagram of RW67

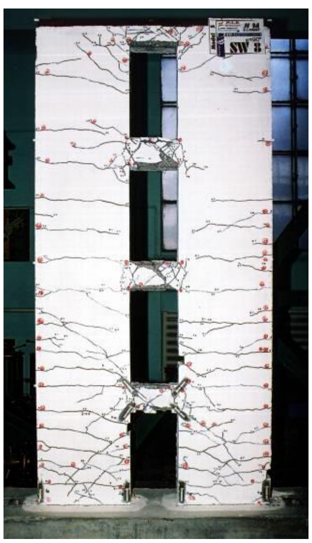


c) Strains in the composite material



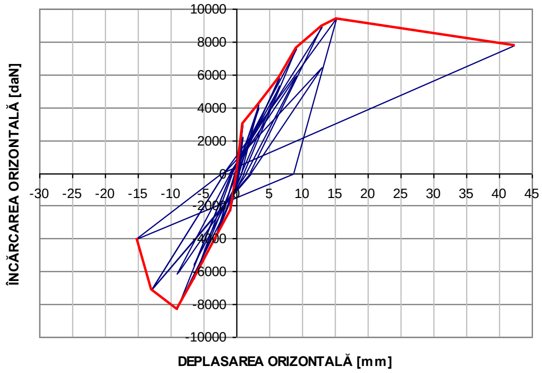
d) Envelope diagrams for models SW67 and RW67

Fig. 2.16 Behaviour diagrams of models SW67 and RW67

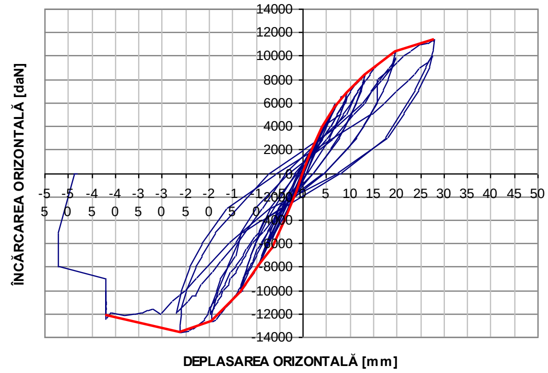
a) 

b) 

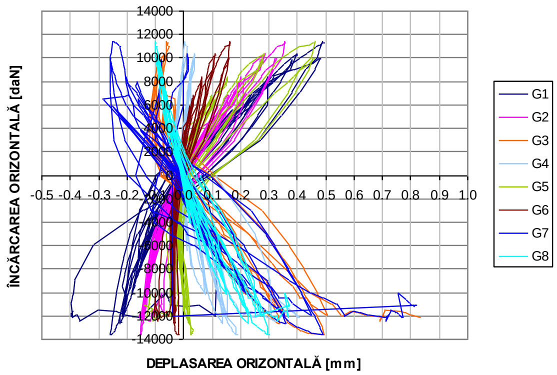
Fig. 2.17 Test models a) SW8 after testing; b) RW8 after consolidation



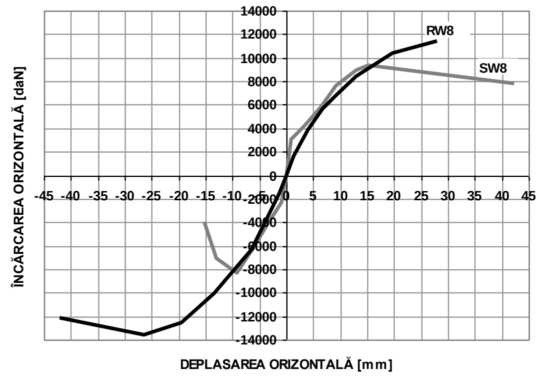
a) Force-displacement diagram of SW8



b) Force-displacement diagram of RW8



c) Strains in the composite material



d) Envelope diagrams for models SW8 and RW8

Fig. 2.18 Behaviour diagrams of models SW8 and RW8

Table 2.1 Results of the experimental tests

Element	Cycle	ke [daN/cm]	H_{el} [daN]	Δ_{el} [mm]	Hmax [daN]	Δmax [mm]	Hu [mm]	Δu [mm]	D [-]	Hel / Hmax	ε_{frp} [%]
SW1	1 (+)	348.5	10250	2.9	11500	28.2	10000	33.0	11.2	0.89	-
RW1	1 (+)	159.0	12400	7.8	15500	20.1	10500	37.5	4.8	0.80	0.54
SW23 (1+)	1 (+)	309.5	8620	2.8	9750	19.5	9500	28.6	10.2	0.88	-
SW23 (1-)	1 (-)	184.8	-8070	-4.4	-9100	-13.0	-8100	-28.6	6.6	0.89	
SW23 (2+)	2 (+)	270.5	7370	2.7	8250	19.5	7900	28.6	10.5	0.89	
SW23 (2-)	2 (-)	182.3	-8200	-4.5	-9250	-13.0	-8050	-28.6	6.4	0.89	
RW23 (1+)	1 (+)	109.5	14400	13.2	15500	40.0	15500	40.0	3.0	0.93	0.63
RW23 (1-)	1 (-)	90.3	-10320	-11.4	-11400	-41.4	-11400	-41.4	3.6	0.91	
RW23 (2+)	2 (+)	111.4	9980	9.0	11700	12.9	11700	12.9	1.4	0.85	
RW23 (2-)	2 (-)	88.5	-7920	-8.9	-8500	-18.6	-8500	-18.6	2.1	0.93	
SW45 (1+)	1 (+)	298.0	7000	2.3	7600	13.0	7500	27.0	11.5	0.92	-
SW45 (1-)	1 (-)	343.4	-8250	-2.4	-9100	-19.5	-8750	-27.0	11.3	0.91	
SW45 (2+)	2 (+)	280.4	6730	2.4	7500	13.0	6730	27.0	11.2	0.90	
SW45 (2-)	2 (-)	312.9	-7200	-2.3	-7900	-13.0	-7450	-27.0	11.7	0.91	
RW45 (1+)	1 (+)	91.7	12360	13.5	13000	18.6	12360	28.0	2.1	0.95	0.79
RW45 (1-)	1 (-)	112.8	-10140	-9.0	-10800	-18.5	-11000	-28.5	3.2	0.94	
RW45 (2+)	2 (+)	90.9	11250	12.5	12100	16.0	12100	16.0	1.3	0.93	
RW45 (2-)	2 (-)	115.2	-8978	-7.8	-10400	-18.2	-10400	-18.2	2.3	0.86	

SW67 (1+)	1 (+)	168.6	6900	4.1	7800	13.0	6900	26.0	6.3	0.88	-
SW67 (1-)	1 (-)	198.9	-8340	-4.2	-9100	-19.5	-7450	-30.0	7.1	0.92	
SW67 (2+)	2 (+)	170.5	7090	4.2	7800	13.0	6625	26.0	6.3	0.91	
SW67 (2-)	2 (-)	176.7	-6935	-3.9	-7600	-13.0	-6935	-26.0	6.6	0.91	
RW67 (1+)	1 (+)	102.3	10640	10.4	12300	25.8	10640	32.3	3.1	0.87	0.58
RW67 (1-)	1 (-)	105.1	-12540	-12.0	-13500	-26.1	-13500	-26.1	2.2	0.93	
RW67 (2+)	2 (+)	100.9	9423	9.3	11000	26.3	11000	26.3	2.8	0.86	
RW67 (2-)	2 (-)	100.8	-12200	-12.1	-12900	-26.8	-12900	-26.8	2.2	0.95	
SW8 (1+)	1 (+)	154.7	8300	5.4	9400	15.2	7800	42.4	7.9	0.88	-
SW8 (1-)	1 (-)	143.7	-6746	-4.7	-8300	-9.1	-4000	-15.2	3.2	0.81	
SW8 (2+)	2 (+)	174.3	5395	3.1	6450	13.0	6450	13.0	4.2	0.84	
SW8 (2-)	2 (-)	156.9	-5850	-3.7	-7100	-13.0	-7100	-13.0	3.5	0.82	
RW8 (1+)	1 (+)	88.9	9660	10.9	11400	27.9	11400	27.9	2.6	0.85	0.84
RW8 (1-)	1 (-)	99.5	-12250	-12.3	-13600	-26.3	-12250	-42.1	3.4	0.90	
RW8 (2+)	2 (+)	92.7	8840	9.5	9900	27.6	9900	27.6	2.9	0.89	
RW8 (2-)	2 (-)	92.1	-11040	-12.0	-12000	-19.7	-11900	-27.1	2.3	0.92	

Table 2.2 Experimental results comparison

Specimen		SW1	RW1	SW23	RW23	SW45	RW45	SW67	RW67	SW8	RW8
Maximum horizontal load Hmax [kN]	W	115	155	98	155	76	130	91	135	94	114
	E	–	–	93	114	91	108	78	123	83	136
Difference in capacity [%]	W	+ 35		+ 58		+ 71		+ 48		+ 21	
	E	–		+ 22		+ 19		+ 57		+ 63	
Maximum horizontal deformation Δu [mm]	W	33	47	28	40	27	27.5	30	32	42	28
	E	–	–	28	41	27	27.5	26	26	15	42
Difference in displacement [%]	W	+ 42		+ 42		+ 1		+ 6		- 33	
	E	-		+ 46		+ 1		+ 0		+ 180	
Rigidity ke [N/mm]	W	348	159	309	109	298	91	199	105	154	89
	E	–	–	184	90	343	112	168	102	143	99
Difference in rigidity [%]	W	- 54		- 65		- 69		- 47		- 42	
	E	–		- 51		- 67		- 39		- 31	
Ductility D [–]	W	11.2	4.8	10.2	3.0	11.5	2.1	7.1	2.2	7.9	2.6
	E	–	–	6.6	3.6	11.3	3.2	6.3	3.1	3.2	3.4
Difference in ductility [%]	W	- 57		- 71		- 81		- 69		- 67	
	E	–		- 45		- 72		- 51		- 6	
Maximum strain in the composite material [%]		–	0.54	–	0.63	–	0.79	–	0.58	–	0.83

2.3 Strengthening of building by modification of structural system

Lately, many of the RC buildings, built in Timisoara in the years 1960–1977, began to change their destination based on the applications on market economy. Since these buildings were not designed to seismic actions or for actions smaller than required by current design codes, results these do not satisfy the actual requirements for strength and ductility. In this situation it is required the identification of general measures for aesthetic strengthening, which satisfy the new functional requirements (modern technologies for ventilated facades and the realization of steel penthouses) and the modification of structural system by including walls and tubes which must carry out the seismic actions, remaining for existing structure to support only the vertical loads.

The principle of strengthening by modification of structural system was applied for many buildings where the changing of function was required **[2.29]-[2.32]**.
One of the best examples of the application of this principle is the CFR Marfa building **[2.50]**. This building was built in period 1972–1973 and during the communist period worked as a factory canteen. Few years after the 1989 Revolution it was bought by a private investor, and for some years, it was out of any activities. This period has contributed to the advance of physical degradation, mainly due to rainwater infiltrations. The building was bought in 1999 by CFR Marfa Society, aiming to turn it into an office building. The building's structure was designed in 1970 in accordance with the structural concepts of that period. It was designed for seismic forces, according to the provisions of old codes, much lower than those required by current codes.
Therefore, the design problems were related to the reconversion from factory canteen in an office building, extension of building with a penthouse and building strengthening to correspond to provisions of the new seismic code. Initially the building was two levels one, having the dimensions in plan of 42.90m×24.40 m. Heights were 3.35m and 4.40m for ground floor and first storey, respectively. Image of the initial building is presented in figure 2.19.

Fig. 2.19 Initial state of the building

It is made from RC precast foundations, beams and columns. Precast beams have the dimensions of 25 × 55 cm (Fig.2.20). The columns with the section of 40 × 40 cm, are made of precast RC, being extended on two levels (Fig. 2.22). The concrete used in the precast elements was C16/20. The main structural problem was the beam-column connections. Particularity of the used precast framed system consists in solving these connections using welding of steel pieces placed at the beam ends and in the columns at the floor level (Fig. 2.21). Examining this connection type one arrived at the conclusion that they correspond to a hinged connection. So, the structure works for lateral action as a hinged system with cantilever columns, extended on two levels, without any capability to redistribution of internal forces. The floors were made of precast panel type, having length of 6.0 m and thickness of 22 cm.

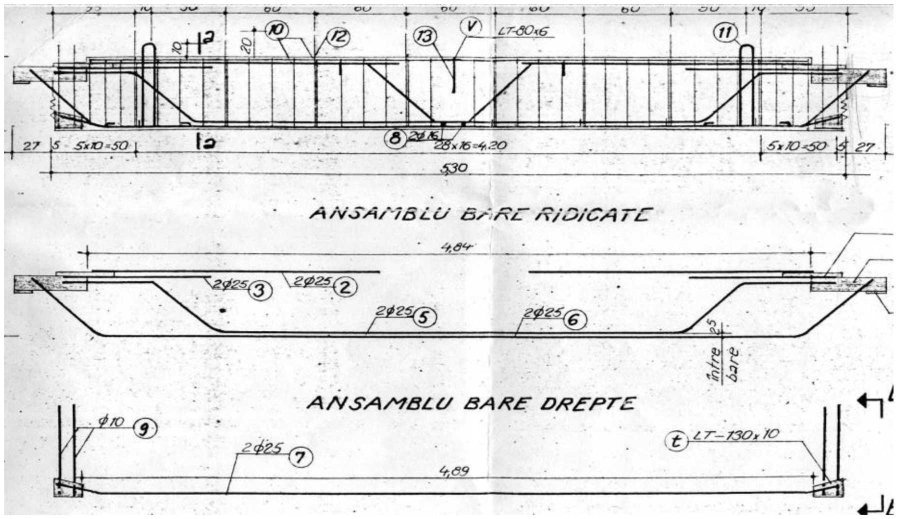


Fig. 2.20 RC precast beam


Fig. 2.21 Beam to column connection

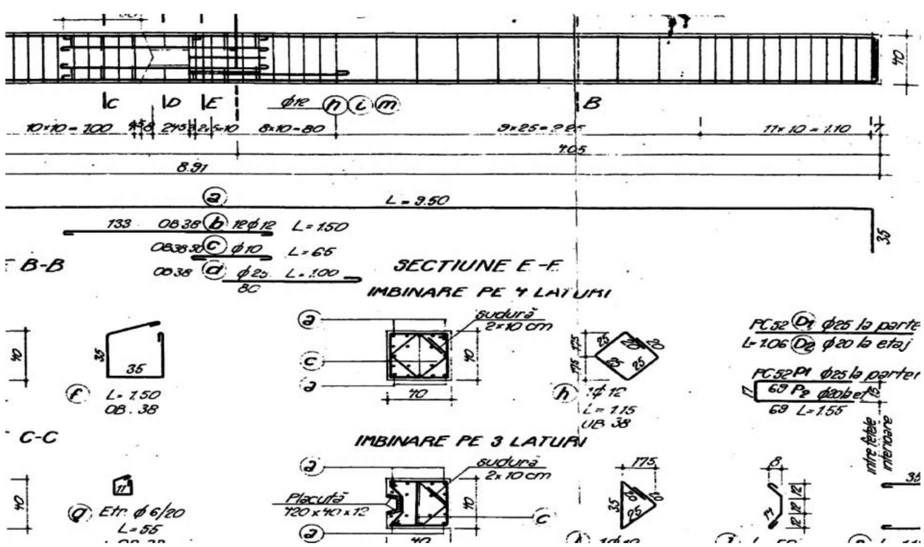


Fig. 2.22 RC precast column

The exterior walls and interior partitions were made of masonry with thicknesses of 20–30 cm.

The expertise of the existing building was made by the author together with Prof. Gioncu and the structural design office H.I. STRUCT, based on the architectural solutions proposed by the architecture design office ANDREESCU & GAIVORONSCHI. After inspections and measurements carried out during the building expertise the following deficiencies were identified:

i) *Design deficiencies:*

- Design deficiencies are caused by adopting in structure design a behaviour q = 5. This value was a wrong one because of the hinged beam column connections, structure working as a cantilever with possible plastic hinges only at column base. Therefore it had to be chosen a higher value for q behaviour factor (recommended value q = 2–3);
- The existing structure develops a brittle failure for the actual seismic actions. The actual structural elements can support only vertical loads.

ii) *Execution deficiencies*:

- Deviations from the verticality of the columns;
- Bad joint welding of steel piece;
- Levelling deviation up to 15 cm of the concrete floor between different corners of the building;
- Classes of concrete cast in place are lower resistances than those proposed in the project;
- Corrosion of joint steel pieces of joints;
- Concrete damage due to rain water infiltration;
- Corrosion of reinforcement bars in floors and beams;

Dynamic analysis of the existing building, led to obtain the following results:

- Period vibration for the longitudinal translation 0.608 sec;
- Period vibration for transversal translation: 0596 sec;
- Period vibration for general torsion movement: 0.526 sec.

Drifts for the level were 4.7‰ at the ground floor and 6.5‰ at the storey did not exceed maximum value of 7‰.

2.3.1 Methods of consolidation

The innovative consolidation measures have been imposed by the author together with Prof. V. Gioncu, and consists in a changing of the functional destination of the building, the need to construct a new penthouse floor, (Fig. 2.23, 2.24), and to satisfy the requirements of the new Romanian seismic design code. Consolidation solutions were established on the basis of interpretation of the results achieved with dynamic analysis program AXIS. The discretization model of the structure is presented in figure 2.25 and the results are presented in figures 2.26 and 2.27. The structure of the existing building has been strengthened by modification of structural system, through the changing from a precast structure frames in a dual structure: frames & central steel tube and corner RC tubes and walls.

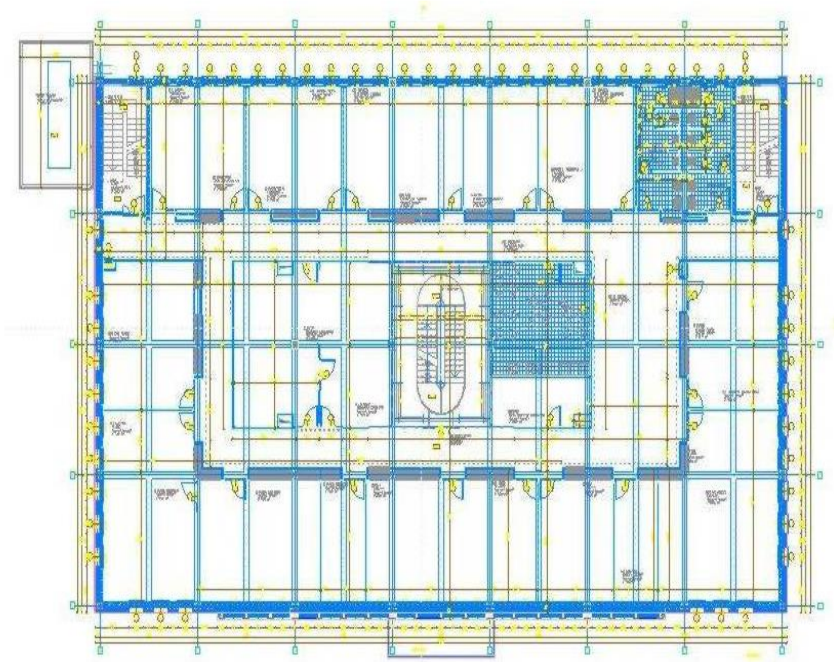


Fig. 2.23 Horizontal section of penthouse

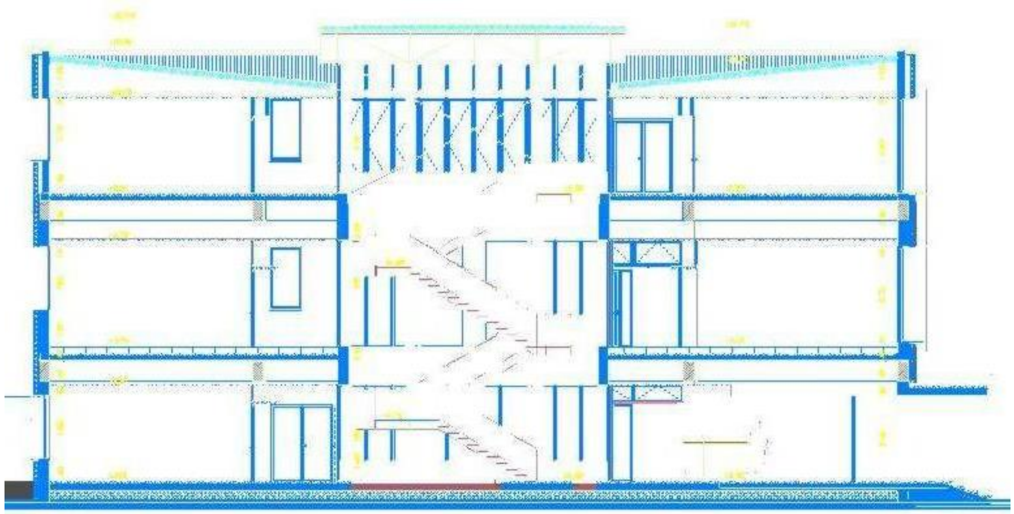


Fig. 2.24 Transversal section

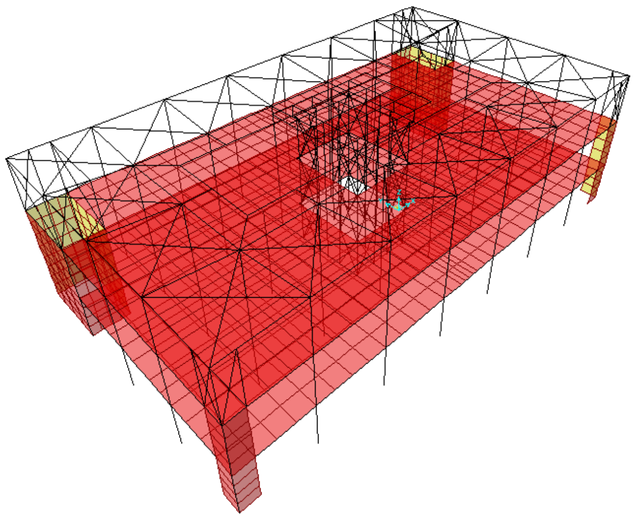


Fig. 2.25 Discretization of structure

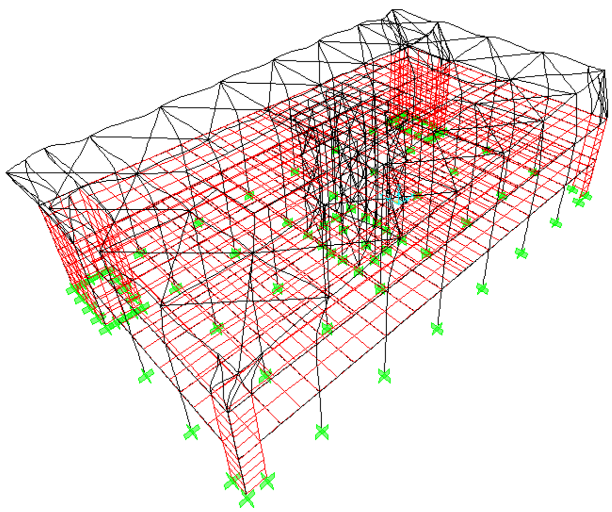


Fig. 2.26 First vibration mode

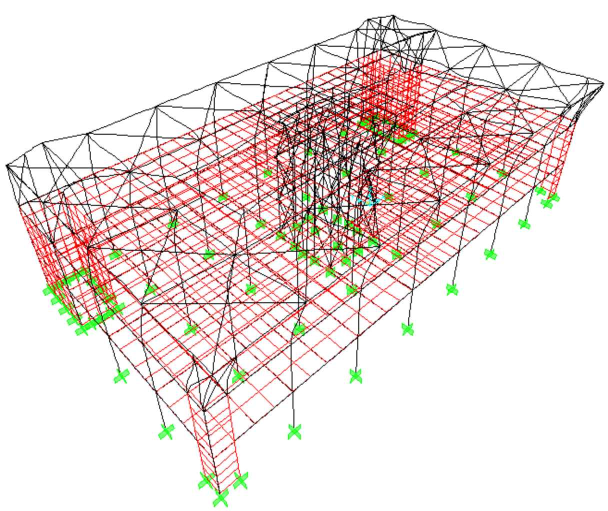


Fig. 2.27 Second vibration mode

In this solution the seismic forces were carried out only by the new introduced steel and concrete tubes and the existing frames must take only the vertical loads. For this structure modification were introduced one central steel tube, two RC tubes in the two corners were exist staircases and two RC structural walls in the other corners of the building. Concrete walls were 25 cm thick and were made only at ground floor and first storey. The concrete tubes were continued till the roof level through steel tubes with X bracing. For the first two levels the RC tubes resulted by the introduction of RC walls between precast columns and beams. The connections between these elements have been achieved through steel connectors, fixed mechanical on this columns and beams. Details are shown in figures 2.28-2.31.

Fig. 2.28 Connection shear wall and column

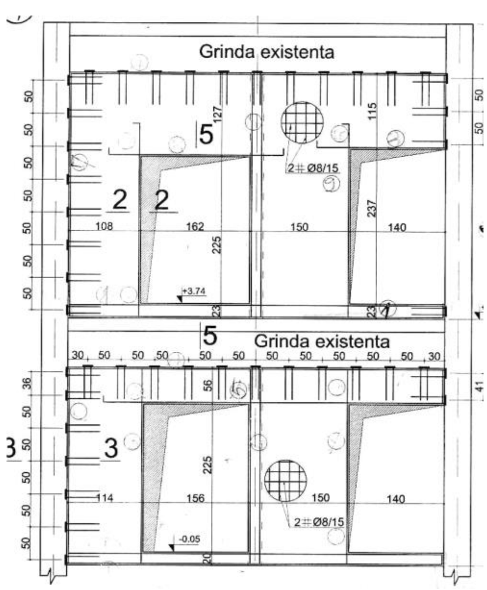


Fig. 2.29 Reinforcement of the shear walls


Fig. 2.30 Shear wall and last storey steel bracing


Fig. 2.31 Steel central tube bracing

The new function as office required a new staircase in the central part of building. Therefore it was needed to create an opening in the existing floors. So, a central part of floor was demolished. As a result of this functional intervention, a very nice steel braced tube was introduced in this area. This steel braced tube, beside the structural contribution, has a remarkable contribution to the improving the internal architectural aspect.

In figure 2.32 is presented an axonometric image of the tube as a staircase, and in figures 2.33 and 2.34, some images with the central braced steel tube are shown. The tube is made of X bracing steel tubes, enforced on the upper part with lattice beams, and it is fixed on the rest of the building through boarding beams from reinforced concrete.

This steel central tube, together with the other two corner RC tubes and corner RC structural walls, has provided the rigidity requirements, strength and ductility of the building. Following the modification of structural system were recorded the following important changes:

- The efforts of the existing columns and beams have dropped and have not exceeded its bearing capacities;
- The rigidity of the building has increased significantly;
- The stress the nodes of the structure were reduced.

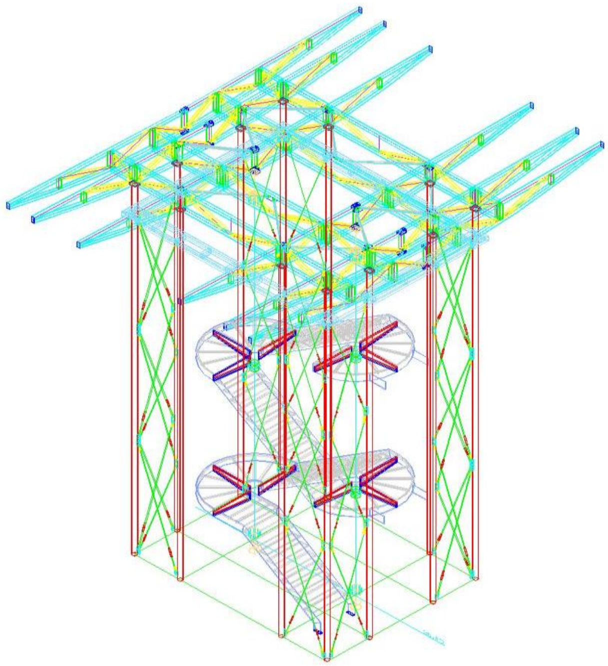


Fig. 2.32 Steel structure of central tube

Fig. 2.33 Bracing of central tube

Fig. 2.34 Central staircase

Figures 2.35, 2.36, 2.37 show the images during erection of penthouse and finished building, respectively.

Fig. 2.35 Central staircase during erection

Fig. 2.36 Building during erection

Fig. 2.37 Finished building

2.3.2 Conclusions

The author presents an original method which brings new and important data at an international level concerning of strengthening the buildings located in seismic zones, by modification of structural system. The advantages of this method are:

• The local structural interventions (concentrated only in some building parts) are able to satisfy the strength and ductility demands required by current codes for seismic design of the entire building, without strengthening all elements of the structure. In another variants, each beam, column or connection must be strengthened, which is a more expensive and complicated solution;

• The presented consolidation solutions allow introducing, in a common structure without architectural value, some aesthetic architectural solutions.

2.4 Reconversion of a damaged industrial building using FRP

2.4.1 Introduction

Due to the city expansion, many industrial building, erected at outskirts of the town, are now situated in the centre of Timisoara City. Therefore, a very important design activity is to eliminate the industrial activities from these zones and the reconversion of these buildings in shopping centres, exposition halls, or office buildings **[2.51]**. To solve the structural problems of these reconversions, there are mainly two problems: (i) for a long period the Banat region (Timisoara is the capital of this district) was not considered as a seismic area. After the moderate earthquakes occurred in the last decades, the new design code includes the verification of buildings to the seismic actions. So, for the each intervention on the existing buildings the structure strengthening begin to be the main problem; (ii) due to a bad exploitation during the active period, and a long period in which the industrial buildings were not used and maintained, the structures presents important damage.

The Galeria 1 Shopping Centre is one very good example about the problems of such reconversions. The building is situated in an old industrial zone, which now undergoes a strong conversion in a commercial and entertaining zone. Some existing buildings were proposed to be demolished and others to be modernized.

The presented building was analyzed in the light of this situation. Using the changing of structural system (by introducing four RC tubes around the staircases in order to improve the seismic resistance), and the strengthening of damaged structure (by means of FRP technology), a building condemned to be demolished was converted, from an industrial building, in a modern and successful shopping centre. Figure 2.38 shows the images of Galeria 1 building before and after interventions.

a) 

b) 

Fig. 2.38 View of building: a) Before reconversion; b) After reconversion

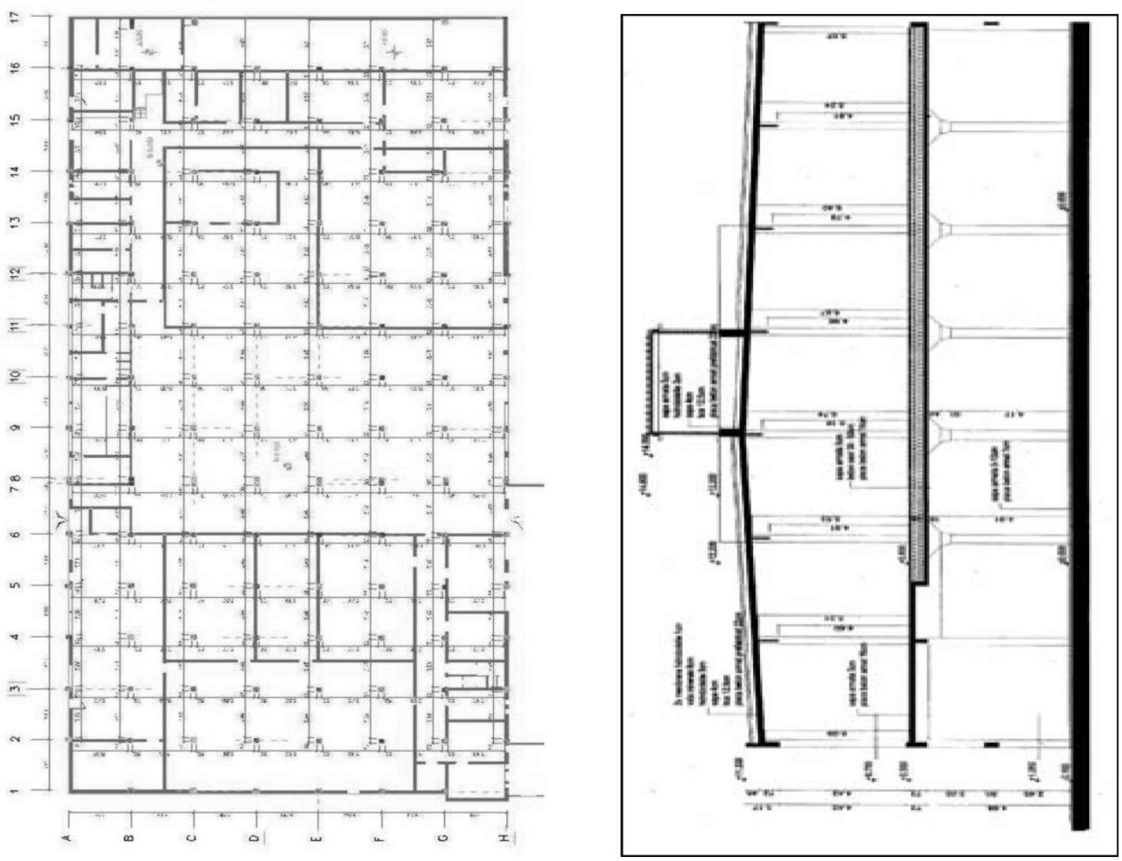


Fig. 2.39 Sections of the existing building

The existing building has two levels, having a mixed RC structure (Fig. 2.39). The structure consists in spread shallow individual foundations, monolithic first ground level with RC floors, precast haunch columns, beams and roof panels for the first storey.

The building was erected during the period 1965 – 1970 when the Banat district was not considered as seismic zone. Due to this fact the existing structure is not prepared to resist to

seismic actions, consisting in weak lateral resistance for the ground level and inadequate connections for the first storey.
It was used as a meat smoking at the ground level and as meat refrigeration and depositing at the first storey. After 1990 the building was bought by a company which changed its destination, selling the existing equipments. Therefore, many years the building has remained out of the use, years contributing to the structure strong damage (Fig. 2.40).

Fig. 2.40 Damage of the existing building

At the ground level, due to smoke and high temperature, the protection layers of RC columns were destroyed, increasing the corrosion of the steel corner bars (Fig. 2.41).

Fig. 2.41 Damage of columns

Due to an incorrect operation, required by the technological process, the water vapours from the ground level were condensed in the layer of thermal insulation between the two levels. Therefore, the permanent high humidity and the destroyed protection layer of floor reinforcement due to the smoke, the reinforcement area was reduced by approximately 50% (Fig. 2.42).

Fig. 2.42 Damage at the first floor

At the first level, the main problems are related to the inadequate connections between precast elements (Fig. 2.43).

Fig. 2.43 Inadequate connections

2.4.2 Strengthening for seismic resistance

To increase the seismic building resistance four RC tubes were introduced around the staircases, rigid connected to existent structure (Fig. 2.44). Due to this changing of structural system, the behaviour of building for horizontal loads was very much improved, by reducing the storey drifts (Fig.2.45).

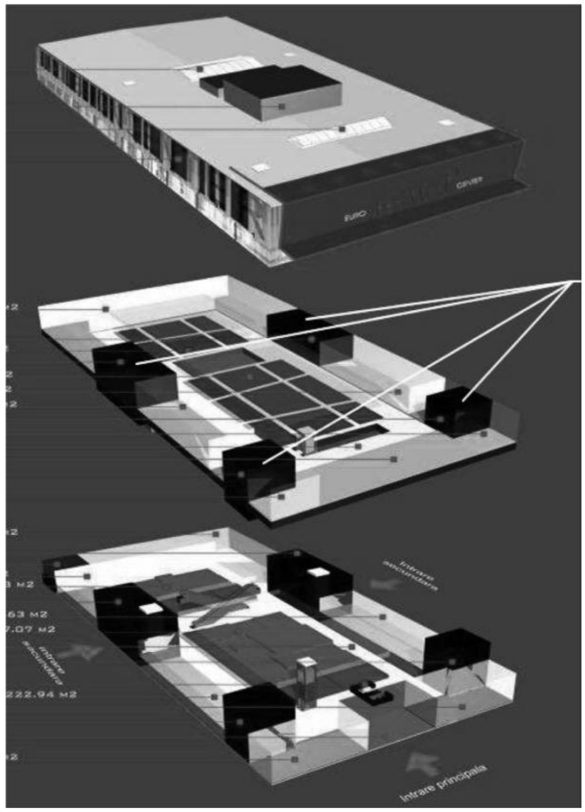


Fig. 2.44 Introduction of 4 RC tubes

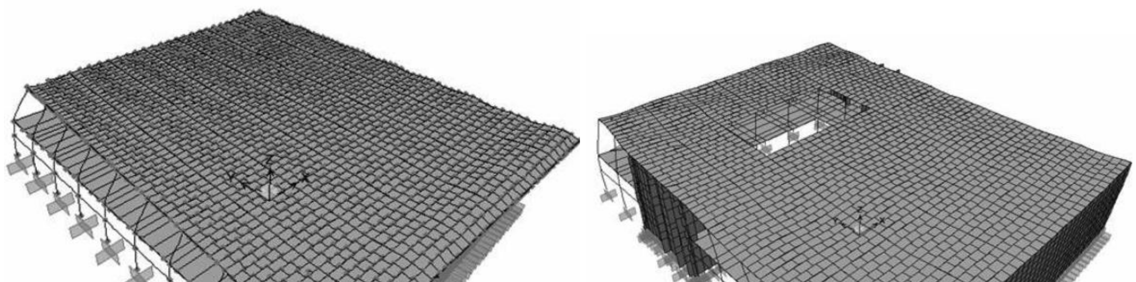


Fig. 2.45 Effects of structural changing

All bearing elements have been reinforced with materials made of carbon fibre and the elements were verified using the methodology provided by SIKA Company, using special adhesive and mortars. The strengthening of columns and floors used different methodology.

The columns and their capitals were strengthened using angles at the corners to supply the steel corner corroded bars and carbon fibre reinforced polymer sheets as stirrups (Fig.2.46).

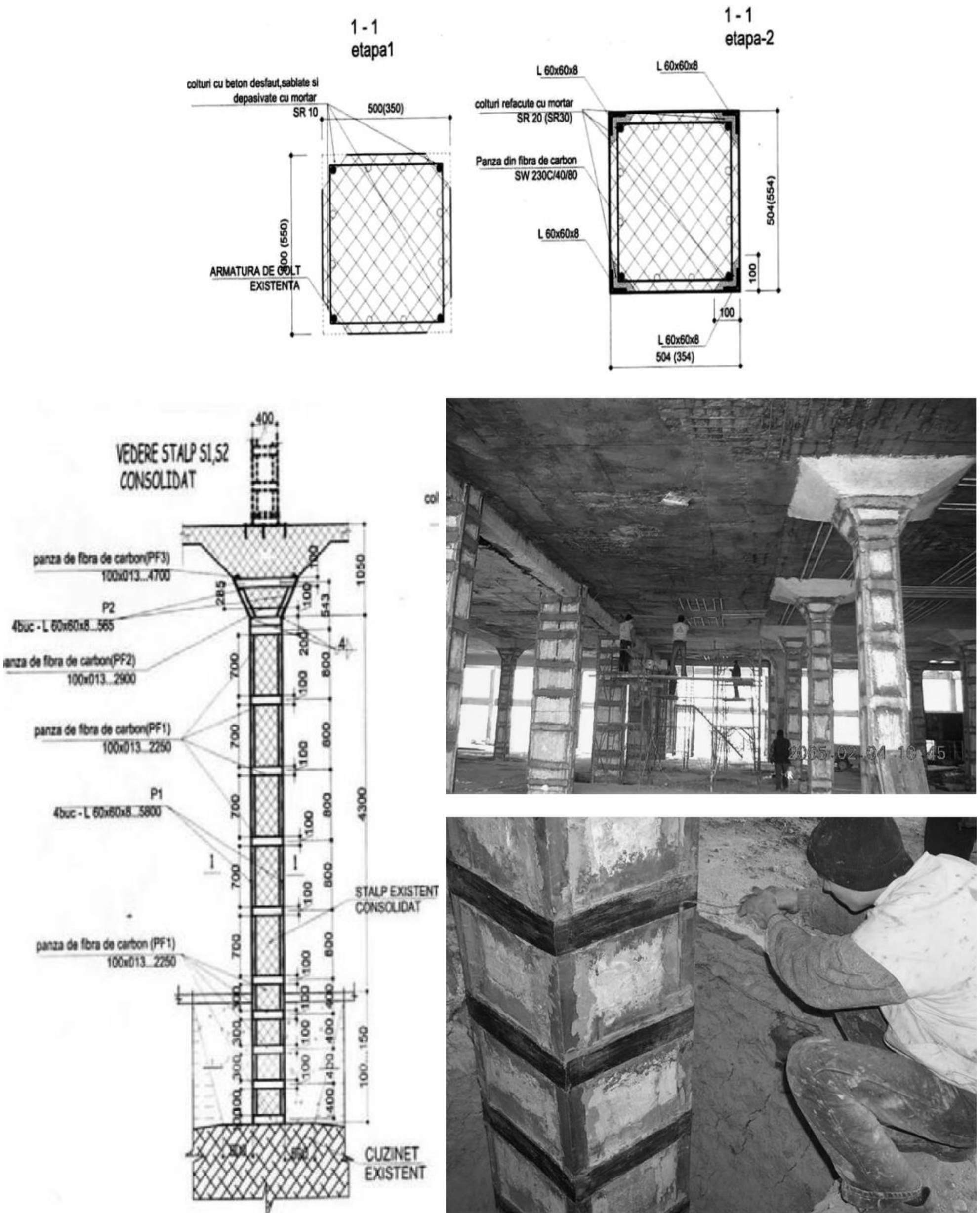


Fig. 2.46 Strengthening of columns

The strengthening of first floor was made using strips of carbon fibre reinforced polymer, applied only at the inferior face of the floor (Fig. 2.47), because the steel bars at the superior face of the floor were proved to not be corroded.

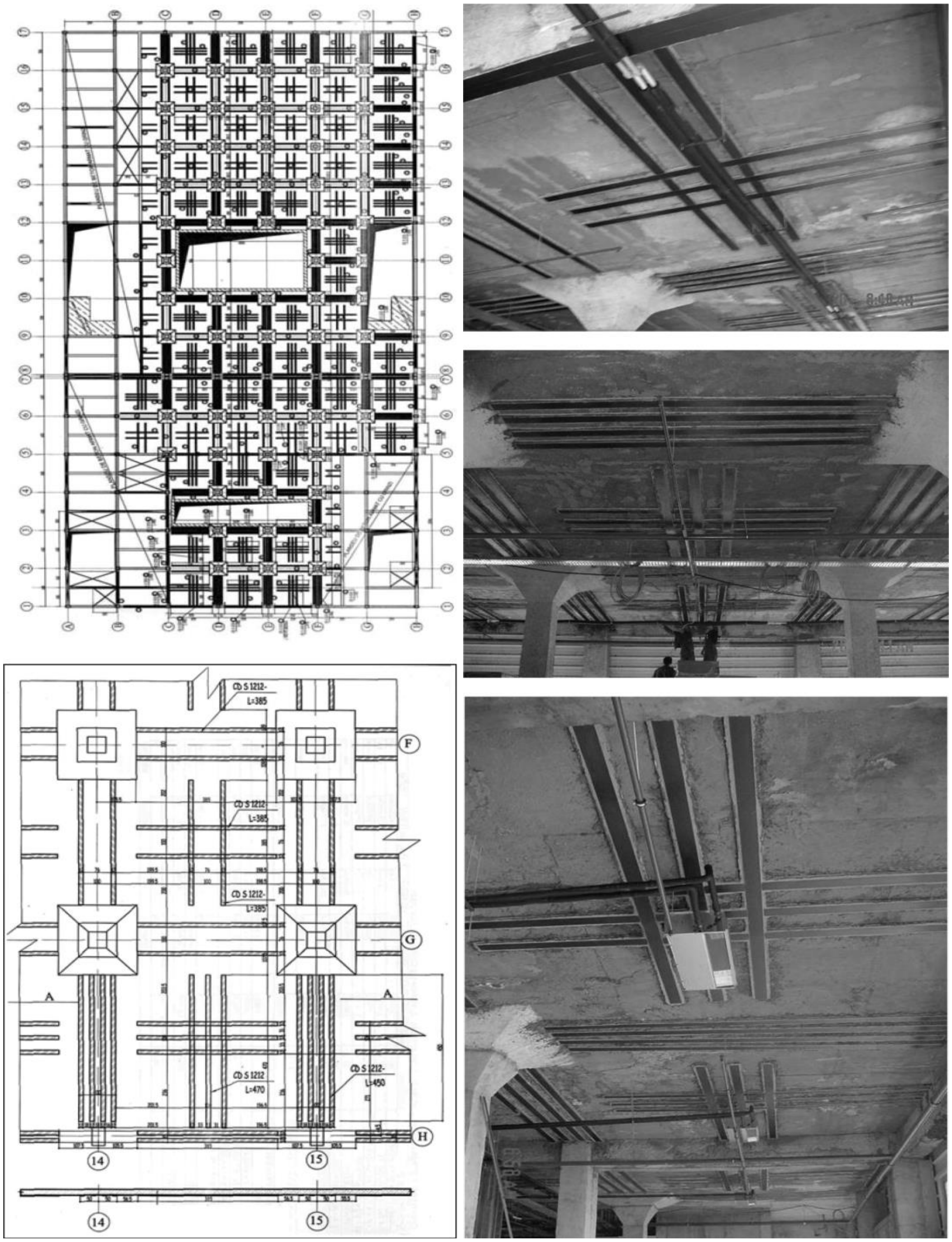


Fig. 2.47 Floor strengthening

2.4.3 Conclusions

Studies performed by the author shows that using innovative solutions for strengthening can save a building condemned to be demolished and to transform it in a successful building. This consolidation solution performed in 2005 was among the first consolidations in Romania using FRP for columns, slabs and beams, for buildings of this size, bringing some new elements on an international level.

2.5 Innovative systems for earthquake resistant masonry enclosures in RC buildings

2.5.1 Introduction. INSYSME research contract FP7-SME-2013

According to the research studies performed within the European Research Contract INSYSME **[2.52]**, the use of masonry infill walls and, to a certain extent, veneer walls, especially in reinforced concrete (RC) framed structures, is widespread in many countries. This practice derives from the natural evolution of the traditional building technique, which was based on masonry walls. The exceptionally rapid growth in the use of RC elements for creating the bearing structure, transformed the latter into a "wire frame" of negligible volume, mass and stiffness, when compared to traditional masonry walls.

With reference to structural problems, it has to be underlined that they have only recently started calling additional attention, and still no suitable solutions, acceptable under all possible points of view (compliance with code required performance, safety, economy, aesthetics, durability, adequacy of design procedures, etc.), have been investigated and proposed **[2.33]**. This fact is particularly pronounced when the so-called non-structural elements are subjected to actions that forces them to behave structurally, as in the case of earthquakes, strong winds, etc (Fig. 2.48, 2.49).

Fig. 2.48 Examples of in-plane and out-of-plane seismic damage to clay unit masonry infill walls

Fig. 2.49 Examples of in-plane and out-of-plane seismic damage to clay brick masonry veneers

The 2009 earthquake in L'Aquila (Italy, Mw = 6.3) **[2.34]** produced around 300 casualties and more than 1500 injuries. The highest number of casualties (around 200) was concentrated in the town of L'Aquila, with dominant damage type causing fatalities equally subdivided between masonry houses collapse and poorly designed/built RC frame failure and infill walls failure (Fig.2.50). However, widespread extensive damage to masonry infill and internal partition walls was detected, and caused the highest losses in RC buildings **[2.35]**.

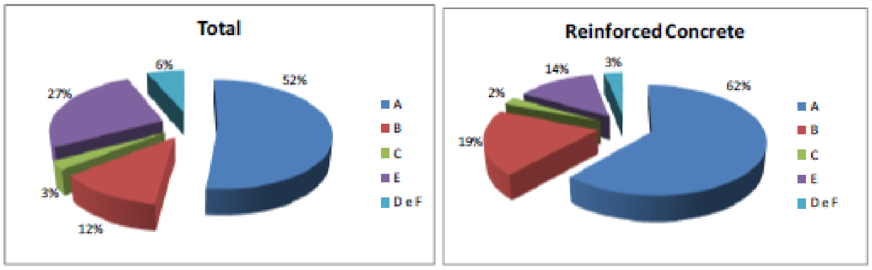


Fig. 2.50 Results of inspection on 73521 buildings after L'Aquila 06/04/2009 earthquake **[2.36]**

Detailed economic analyses have been carried out during the post-earthquake reconstruction to evaluate the cost of repair, including repair works on clay unit infill walls, equipment and interior finishing. Concerning RC buildings with moderate to high non-structural damage, that were 21% of the inspected RC buildings , a cost equal to about 318 €/m2 has been estimated. For buildings having high non-structural damage and structural damage, 14% of the inspected RC buildings), the cost increases to about 400 €/m^2 **[2.37]**. In economic terms, it is evident that the impact of enclosure walls repair, even in a severe earthquake, can be more relevant than the cost related to purely structural interventions.

The 2011 earthquake in Van (Turkey, Mw=7.1) once again demonstrated the highly variable nature of the seismic damage to infill walls in RC frame buildings. In some cases (Fig. 2.51a, b, c), the infill walls contributed significantly to strength and hence helped in the survival of the building. In some other situations (Fig. 2.51d), masonry infills detached from the structure and/or collapsed due to a combination of in and out-of-plane demand. This type of non-structural damage can be extremely dangerous for occupants **[2.38]**, emphasising the importance of masonry infills in RC buildings and calling for the development of new systems for their improved performance (Fig.2.52).

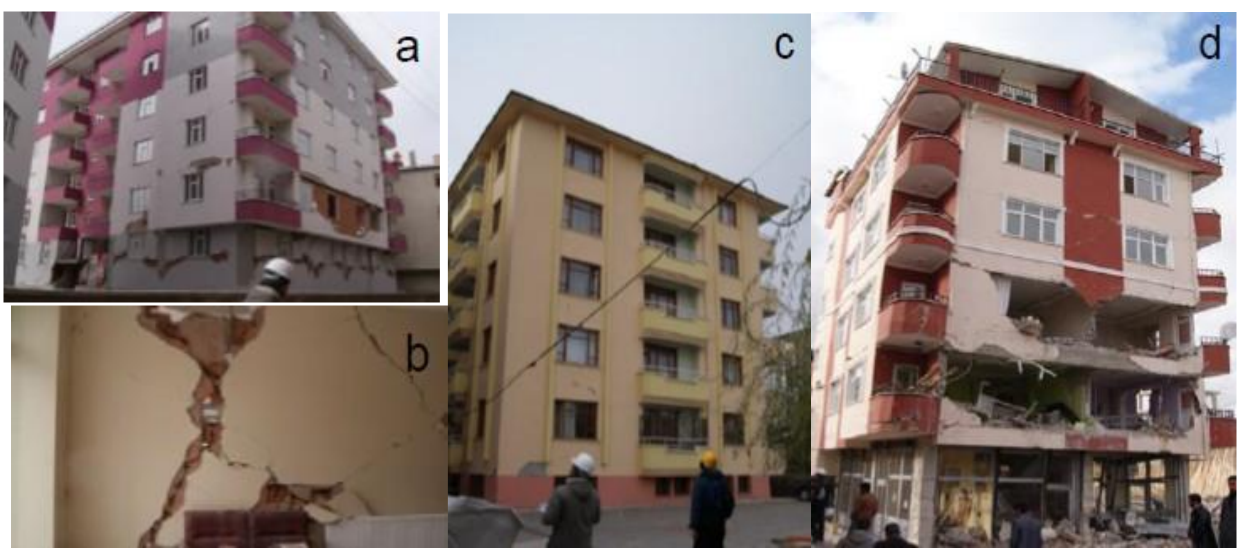


Fig. 2.51 Damage to infills in 2011 Van Earthquake: a) in-plane damage, b) damage inside building in a, c) moderate damage, d) heavy combined damage

Fig. 2.52 Masonry infill and veneer damage: Emilia, Italy, 2012

In this context, most of the codes have recognized that also non-structural elements need to be designed for earthquake actions, in relation to different performance levels **[2.38]**. However, sound design procedures still do not exist **[2.39] [2.40]**. The European structural codes for design of reinforced concrete and masonry buildings do not specify details, performance requirements, or compliance criteria for the safe use of masonry enclosures, nor for the behaviour at serviceability and ultimate limit states.

In the current design practice in Europe, referring to the design of new buildings, RC frame structures subjected to seismic loads are usually examined using linear elastic structural models on which equivalent static or multimodal dynamic response spectrum analyses are performed. The design of infilled RC structures is usually performed on bare frame elastic structural models; where the masonry infill panels are considered in terms of masses and vertical loads only. In this context, the safety verification of RC frames at the ultimate limit state, according to Eurocode 8, has to be accomplished in terms of resistance to seismic action effects for both structural and non-structural elements. In particular, for non-structural elements like interior and exterior walls, partitions and facades that might, in case of failure, cause risk for human life or affect the main structure of the building or services of critical facilities, the verification of resistance for the design seismic action is foreseen and a simplified procedure is proposed for the evaluation of the horizontal seismic force acting on the non-structural element in the out-of-plane direction. Nevertheless, in Eurocode 8 no recommendation for the calculation of the corresponding resistance of the building enclosures is provided. Moreover, according to Eurocode 8 – Part 1, the damage limitation requirements for buildings with non-structural elements are considered satisfied when the induced inter-storey drifts do not exceed certain limits in each storey of the building, defined only as a function of the "ductility" of the infills and on the connection with the surrounding structure, without any reference to the type of masonry enclosure and to the dimensions and amount of infills. Hence, the further development of presently existing code requirements for seismic design of infilled RC structures, as well as the introduction of practical solutions which allow the compliance with the code, in order to achieve satisfactory levels of damage limitation and life safety, are of primary interest and presents one of the important objectives of this research. Furthermore, the empirical solutions proposed by the code are not accompanied by rationales for design, applicable to the various types of masonry enclosures, and not even rules for the use of connectors in composite systems such as masonry veneers are given. The absence of clear performance requirements and design methods, and the lack of some practical (even if theoretically stated into the code) measures, can hinder in the long term the further development of masonry construction systems for enclosure walls and, even worst, could turn out to be ineffective, causing a greater damage to the industrial sector **[2.41][2.42][2.43]**.

2.5.2 Research objectives

Enclosure "specialization" for adequate earthquake resistance is the core of the project and will consist in defining integrated and innovative systems, such as to:

- Optimize (maximize) the local structural performance, by limiting damage under the most frequent (and less intense) earthquakes and minimizing the probability of detachment and out-of-plane collapse under the effects of the most intense, i.e. the "design", earthquakes (for verification at the ultimate limit state);
- Minimize the negative effects that inadequate design and construction of enclosures walls cause on the global structural behaviour of the RC framed structure under the effect of the design earthquake, i.e. at the ultimate limit state;
- Enhance and exploit the non-structural performance of the infill/veneer walls, i.e., from one side, all the properties related to environmental, energy saving and comfort aspects, but also, on the other side, those related to the capacity of limiting damage under serviceability limit states, thanks to the use of innovative smart solutions.

2.5.3 Targeted development of materials and technologies

Possible types of innovative masonry enclosure systems to be developed, with reference to their main conceptual characteristics and details described as follows, may be divided in three major groups: (i) systems built of conventional material components, following original design methods, (ii) systems built of conventional material components and applying sophisticated enhancement techniques, following original design methods, and (iii) systems built of innovative material components, following original design methods.

The resistance in the out-of-plane direction of such simple system can be ensured due to the possibility to fully exploit the arch resisting mechanism. However, due to the significant drop of the out-of-plane resistance due to previous or contemporary in-plane damage, as shown in previous studies on other types of infill, the limitation of in-plane damage in the design through original design criteria is indispensable and has to be quantified.

The solutions of thick (~0.30m), single leaf and self-insulated enclosures are very interesting because only clay units are used. When the unit geometry is properly designed and the composition has proper additives/pores, this type of enclosure can fulfil the internal environment requirements alone. This solution leads to great energy saving and reduced environmental impact, as it does not require the use of any insulating material, and it is likely that it would be sufficient (without any reinforcement) in low to moderate seismic risk areas.

Further improvement can be achieved by using novel dry stack semi-interlocking units, which further economize the construction process (as they are mortarless), but enhance the possibility of exploiting interlocking and friction to increase energy dissipation and, hence, the behaviour of the entire structure. Indeed, mortarless joints or specific grooves allow for relative in-plane sliding of portions of the masonry panels and increase structural ductility, while the semi-interlocking units are locked against relative out-of-plane movements, solving at once the two combined problems related to the behaviour of infill walls. Another alternative, still for low to moderate earthquake countries, is using single leaf walls made of hollow clay masonry units with thin layer mortar.

Specifically, the envisaged approach for optimization is the design of the front face of the units (head joints), as to provide an improvement of the contact surface in the vertical joints. In this case, also, the head joints are not filled with mortar. Top and side connections of the enclosure

will be improved by special corner units and optimized anchors, with the aim to positively affect the physical properties of the units.
Further possible solutions can be derived from the previous through the application of different enhancement techniques and are related to the improvement of serviceability performance. Indeed, we currently accept extensive damage to infills, with high economic losses, even in the case of medium earthquakes. However, there are two interrelated problems: firstly, the acceptable extent of damage depends on the performance level adopted for the design of the entire structure; e.g., in case of hospitals or other strategic structures, an almost elastic overall behaviour of the building is required to keep full functionality also immediately after earthquake events. Secondly, also for ordinary structures, limiting damage and enhancing the ductility of the infills, thus allowing for more evenly distributed cracks that could protect embedded installations and be more easily repaired without the need of special facilities, would be of great advantage. Indeed, as demonstrated on load-bearing masonry, this condition can be reached by adding normal or prefabricated bed joint reinforcements (Fig. 2.53). This has been also proved for infill walls, but only in the case of systems with thin elements **[2.44] [2.45]**.

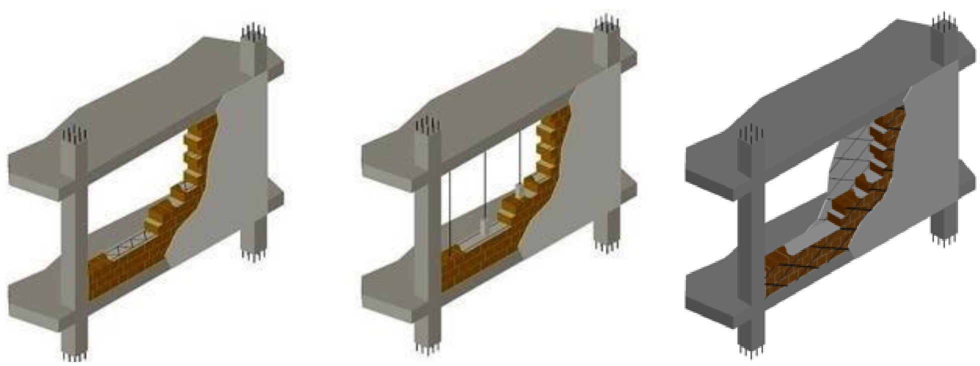


Fig. 2.53 Solutions: horizontal reinforcement; horizontal and vertical reinforcement; new types of plastered wire meshes (from left to right)

Further developments consider, in addition, the use of vertical reinforcement, providing a system that is more similar to load-bearing reinforced masonry (Fig. 2.53). It has to be highlighted that these solutions improve serviceability performance but also enhance the response to out-of-plane loads at the ultimate limit state. Indeed, for “design” earthquakes (most intense and occurring with long return periods), in high seismic risk areas, it is foreseen that the masonry infill walls will undergo severe damage, but thanks to the presence of reinforcement, the cracked portion of masonry will not fall, ensuring the fulfilment of life safety requirements. Related to the practical implementation of these solutions, it will be necessary to study the unit geometry, so as to make it possible to lay the units after vertical reinforcement has been already put in place, as recently proposed for load bearing masonry **[2.46] [2.47]**, thus resulting in a system that can be constructed within an already built frame. In these systems, there will also be the need of designing mortar and reinforcement depending on the exposure conditions (durability), keeping in mind, for mortar, also the need for developing adequate bond (mechanical properties), properly filling the unit recesses (workability and constructability), and keeping good insulation properties.
Other solutions that rely on the enhancement of the ductility of enclosure walls are based on the use of wire meshes inserted in the plaster, on both wall sides. However, instead of the traditional steel based meshes, or those relying on expensive FRP, today a new generation of cheap and

very light materials (nylon, polyester, polypropylene, etc) with engineered cementitious matrix and cruciform inter-support systems, are available. These new composite materials have the advantage of being very cost-effective and easy to be applied, as the process is similar to common plastering; in addition, they can be used to limit damage related to serviceability states. To improve the ductility of enclosure walls, another approach may be to take advantage of the composite action of RC concrete and masonry in confined masonry typologies. This can be achieved through the insertion of lightly reinforced ties embedded in the masonry wall, combining the advantages of very light concrete belt and post solutions, allowing displacements in the wall to occur.

Further solutions rely on the separation of infill walls from the surrounding frames, as already proposed in practice. In this case, several advantages can be pointed out: serviceability performance is dramatically increased in these solutions, as the frames can sustain relative in-plane relative displacements without interacting with the infills. Also, despite the fact that the positive stiffening effect of infills might be lost while keeping their mass, there is a significant advantage when infills are not regularly located in a building **[2.48]**.

The use of external shelf angles for connection to the upper frame beam should be avoided (Fig. 2.54), and replaced with advanced connectors and fasteners in the masonry mid-plane, in order to create a technological ‘sliding joint’ between masonry and frame (Fig. 2.54). The innovative connectors that will be developed in the project will solve the problem of allowing in-plane relative displacements of frame and infill, while counteracting the out-of-plane failure of the latter.

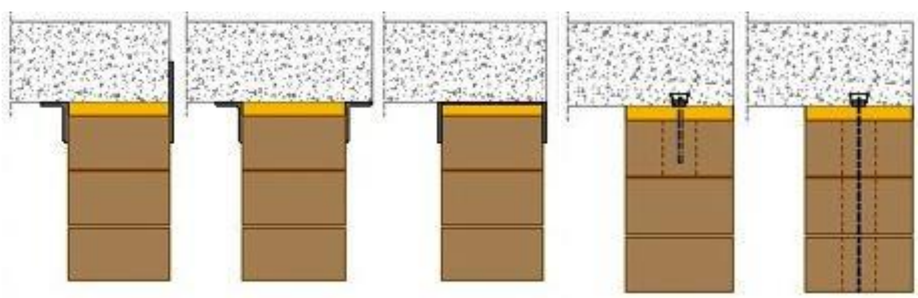


Fig. 2.54 Horizontal displacement joint: current shelf angle connection and new solution with sliding connectors

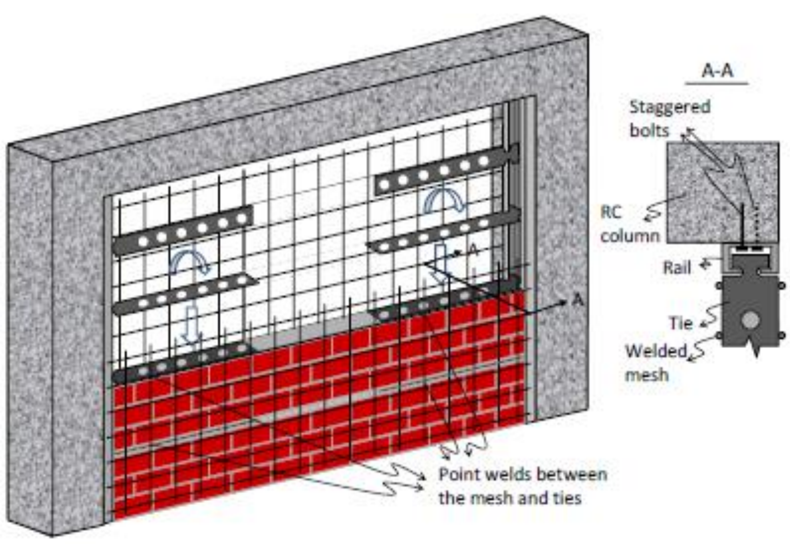


Fig. 2.55 Solutions with special vertical joints and internal tie mesh system

Besides solutions based on horizontal displacement joints at the top, innovative solutions based on special vertical joints will be developed (Fig. 2.55). The envisaged tie system is designed to ensure the stability of infill walls under both in- and out-of-plane forces, contributing to the lateral load resistance of the frame. The system makes use of two shaped steel members connected to the columns using driven bolts. A simple connector shaped like a dog bone at one end is easily locked in the rail, simply by inserting and rotating, and laid flat on a course of the masonry wall to tie the wall to the column. The connection only works in the horizontal direction and enables free movement in the vertical direction, which allows flexibility during construction. The steel plate includes punched holes to promote bonding and interlocking as well as to save on material. Design parameters for all elements must be determined through experimental and numerical studies.
Attention will be devoted to the development of another innovative infill typology that allows the implementation of new materials in close cooperation with the producers. In particular, the introduction of a smart masonry infill system is foreseen, allowing the infill to follow the in-plane deformation of the RC frame through special sliding bed joints, controlling at the same time out-of plane displacements.
For the development of such system, possible adequate innovative mortar types need to be studied in detail and brick shapes capable to restrain the out-of-plane expulsion have to be identified. Alternatively, elastomeric strips may be used in the mortar joints. As demonstrated in **[2.49]**, the deformation capacity can be increased by 5 to 10 times in comparison to conventional mortar.

2.5.4 Directions of research within the INSYSME research program

The development of a masonry unit with double interlocking in the vertical and horizontal joints is pursued, together with the development of a new fastening method. Also, a new expansive mortar is to be used in this construction system at the interface between the RC column and the masonry panel. This system is characterized by dry joints; hence the resistance of the system is relying on the interlocking system. The connector is fixed to the RC frame and the masonry unit, preventing out of plane movement of the infill (Fig.2.56, 2.57).

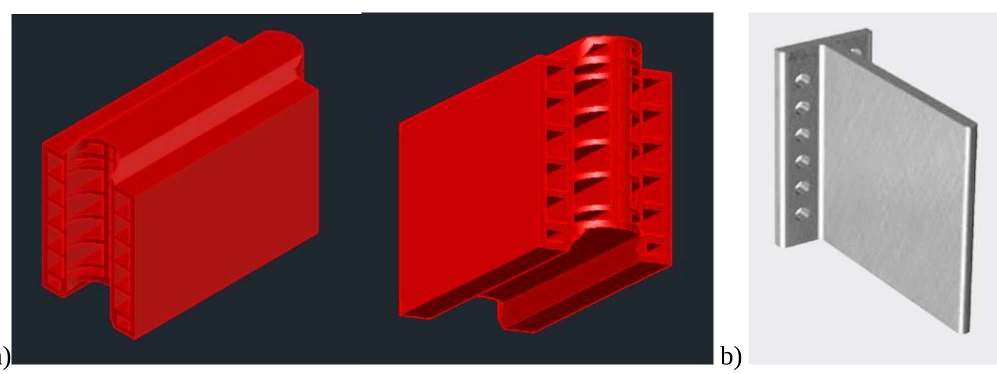
a)  b)

Fig. 2.56 a) Prototype masonry block; b) T-shaped connector

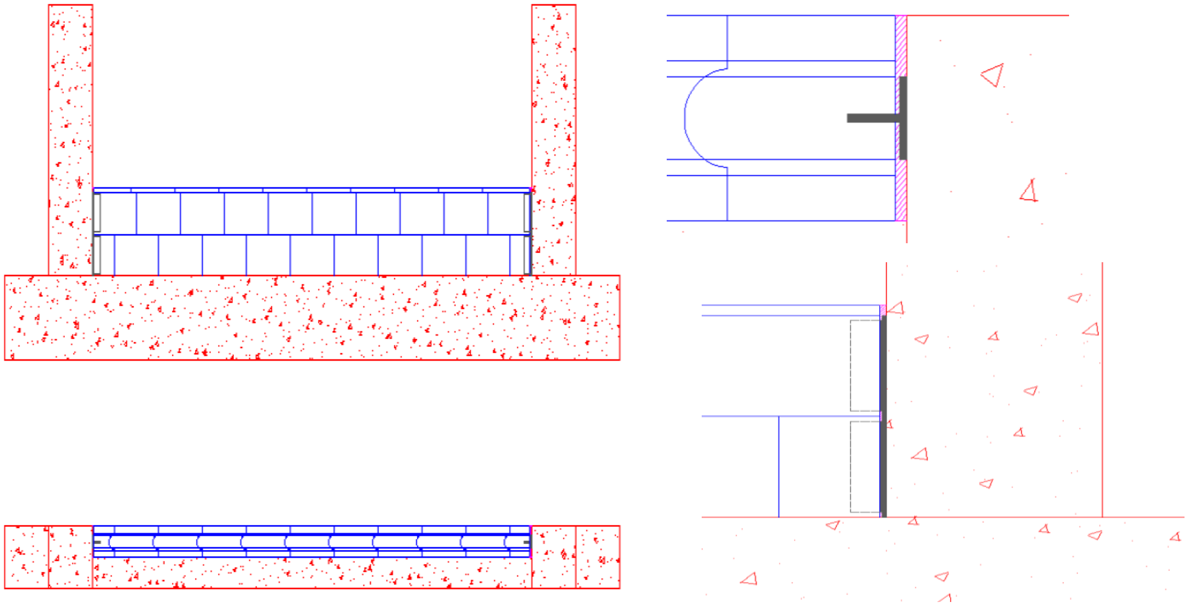


Fig.2.57 Construction system and details

Multi-scale experimental testing are to be performed in order to obtain the characterization of the developed materials, for the masonry units there are to be perform compression, tension and shear tests, for mortars compression and flexure and tension tests for the reinforcement. The interaction between masonry and reinforcements are to be studied by means of pull-out tests, while the interaction with the fasteners are tested with the help of pull-out and/or shear tests.

Fig. 2.58 Experimental tests

In order to define the main constitutive laws relevant for the numerical simulation of walls and to obtain experimental results under cyclic and dynamic loading on structural sub-assemblies and entire structures for model calibration, shaking table tests and combined in-plane and out-of-plane tests must be performed (Fig.2.58).

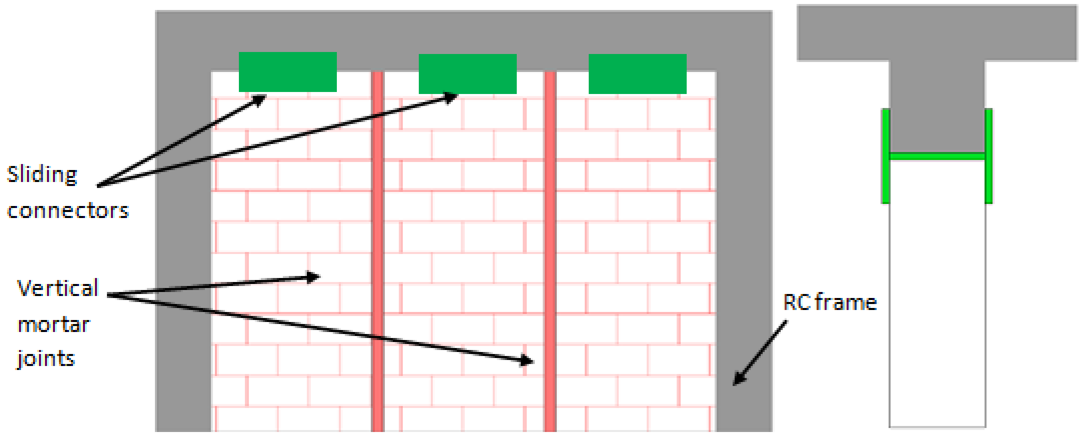


Fig. 2.59 Infill masonry walls divided by vertical mortar joints and sliding connectors

Various types of reinforcements can be used in order to connect the masonry panel and the RC frame. Several connections with special connection profiles with slip abilities are to be studied (Fig.2.59). The presence of reinforcement in the bed joint represents another feasible solution together with various materials reinforcement, placed on the face of the masonry infill panel (Fig.2.60).

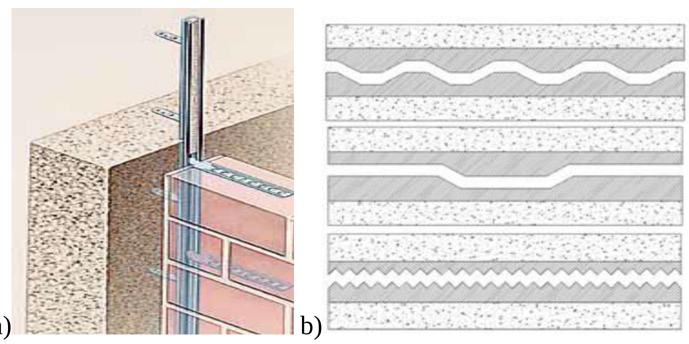


Fig. 2.60 a) Reinforcement connection; b) various sliding joints configurations

Testing of sub-structure represents an important step in the analysis of RC frames with infill masonry panels. The types of structures that are to be analyzed are bare RC frames, infilled frames using different types of materials, infilled frames having internal reinforcement or external geo-textile reinforcements.

2.5.5 Description of experimental stand din UP Timisoara

Within the European Research Program INSYSME FP7-SME-2013, the author will perform some experimental and theoretical studies. The proposed experimental stand constructed in the laboratory of the CCI Department within the Civil Engineering Faculty from Timisoara allows for full-scale out of plane testing of masonry infill panels (Fig.2.61) and brings new directions of international research of the consolidation of these types of walls. The loading type chosen can be cyclic or monotonic, with the help of some steel tie rods that pass through the masonry panel. This experimental setup can be used for studying the effect of a polypropylene mesh on the surface of the infill panel. This material can improve the behaviour of the infill, subjected to out-of-plane loading. The versatility of the test stand is given by the possibility to apply in-plane, out-of-plane and torsional loads in the infill panel.

Given the current interest in carbon dioxide emissions, a great effort is focused in the thermal insulation of buildings. A recent trend in this aspect is represented by the 30cm polystyrene applied on the exterior part of the building, covering the veneer panels. The effect of the polystyrene can be studied, for green (passive) houses, when subject to out-of-plane loads.
Using this experimental stand, a parameterization can be obtained with the mobile columns which can be moved along the length of the opening. Various specimens can be tested and observed in function of materials used, type and presence of reinforcements, presence of openings, length, height and loading method.
Optimization of the materials used for constructing the experimental stand was achieved by implementing a composite solution with reinforced concrete and steel profiles. The possibility to dismantle and transport the structure is given by the high level of the parts prefabrication.

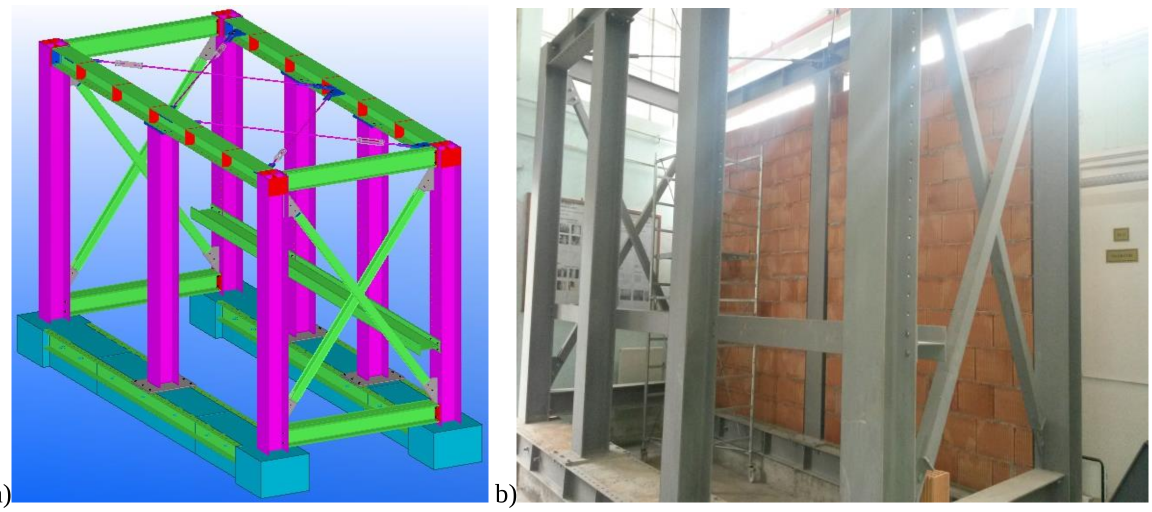
a) b)


Fig. 2.61 Experimental test stand a) Project phase, b) Complete phase with infill panel

3 STEEL BEARING STRUCTURES IN SEISMIC ZONES

3.1 Introduction

The severe Northridge, USA, 1994, and Kobe, Japan, 1995, earthquakes unveil the deficiencies in the design of steel framed structures that was considered, until then, as invulnerable. Moreover, fractures at the joint region without any sign of ductile behaviour were observed. As a consequence the issue of ductility and particularly the local ductility regained a leading role in the seismic design, not only based on the material ductility but also to the section, joint and member level of inelastic deformation. A great research effort was performed all over the world (e.g. FEMA/SAC, USA, RECOS Project, Europe, E-Defense, Japan) focused on the investigation of the unexpected brittle damage and further providing methodologies to predict the ductility; however the issue of the local ductility is not clearly specified in the design codes. In each case the implementation of the non-linear design requires the direct verification of the ductility, based on the capacity-demand ratio.

In the framework of the multilevel-based design, under a specific ground motion, a structure should be sized in order to behave within code prescribed bounds. To achieve these levels of verification, the seismic design is laid out through required available formulation:

REQUIRED CAPACITY < AVAILABLE CAPACITY

Currently, the required-available pairs of three mechanical characteristics are considered in earthquake resistant design, namely rigidity, strength and ductility:

REQUERED RIGIDITY < AVAILABLE RIGIDITY
REQUIRED STRENGTH < AVAILABLE STRENGTH
REQUIRED DUCTILITY < AVAILABLE DUCTILITY

In a coherent earthquake design strategy, the structure must be verified for rigidity at serviceability level, for strength at the damageability level and for ductility at ultimate limit state. According to these verifications, one can remark that, in the first two ones there are no difficulties to perform these verifications, but for the last one, referring to ductility, the verification is far to be satisfactory. The main reason is the difficulty to define the required and available ductility which are based on the very vague codes provisions **[3.1]** In addition, the new design philosophy considers that the required ductility must by defined in function of earthquake characteristics:

(i) Reduced ductility but high strength (reduced *q* factor) for earthquakes with short duration and reduced number of cycles;
(ii) Moderate ductility and moderate strength (medium *q* factor) for earthquakes with moderate duration and number of cycles;
(iii) High ductility but reduced strength (high *q* factor) for earthquakes with long duration and large number of cycles, where *q* is the behaviour factor.

With regard to current Eurocode 8 (2004) **[3.2]** , chapter 6-Specific rules for steel buildings, it prescribes some vague limits in order to verify the local and global ductility, although does not specify a clear methodology. Thus, sufficient local ductility is assured by limiting flange and web width-to-thickness ratios, however taken from Eurocode 3 **[3.3]**, which is mainly a structural code for the design of structures under static loading conditions. But neither this classification is well specified due to the fact that in the classification parameters the influence of the span was not taken into account **[3.4]**. Therefore, the local ductility classes should be redefined. In a more advanced step, the correlation between local and global ductility (ductility class – q factor – local ductility class) should also consider the differences in the seismic action (near-source vs. far-source earthquakes) for both the local and global level as well as the available and required capacities of inelastic behaviour **[3.1]**, **[3.5]**. Moreover, concerning the prediction of the local ductility of beam-to-column connection, only some limits of plastic rotation of the potential plastic hinge were specified, without providing a methodology for the calculation of those limits. Finally, the dissipative-non dissipative concept should be updated with current trends allowing for the implementation of the strengthening (use of cover plates, ribs, etc) or the weakening (´´dog-bone´´connection) joint detailing that moves the plastic hinge away from the column face; in this case member ductility is critical and not the connection ductility. In order to develop a ductile design for steel structures, it is obvious that a discrete process considering all the levels of influence (material, cross-section, connection, joint, and member) should be defined.

3.2 Prediction of the local available ductility of steel members

3.2.1 Investigation of the mechanical behaviour and the proposed local plastic mechanisms

During the experimental tests on the steel wide-flange beams one can observe that the plastic deformations are produced only in a limited zone. The rest part of the beam remains in elastic field. In this plastic zone large rotations are concentrated, working as plastic hinges. The inelastic rotations are amplified if in these zones plastic buckling of flange and web occurs. Two main buckling types were distinguished during the experimental tests, the in-plane and out-of-plane buckling modes (Fig. 3.1) **[3.1]**, **[3.6]** .

Different models were proposed from various researches such as the Climenhaga and Johnson **[3.7]** as well as Ivanyi's collapse models **[3.8]** which are based on a perfect symmetry in relation with vertical axis, and consider the length of flange and web plastic zone equal with the flange width. Kuhlmann's model **[3.9]** is based only on yield lines, for symmetric local plastic mechanisms, formed by plastic buckling of compressed flange and compressed part of web. Feldmann's model considers only the collapse mechanism formed in compressed flange **[3.10]**. In any case the proposed models herein are refined ones completely describing the behaviour of the plastic hinge and are also experimentally calibrated **[3.6]** .

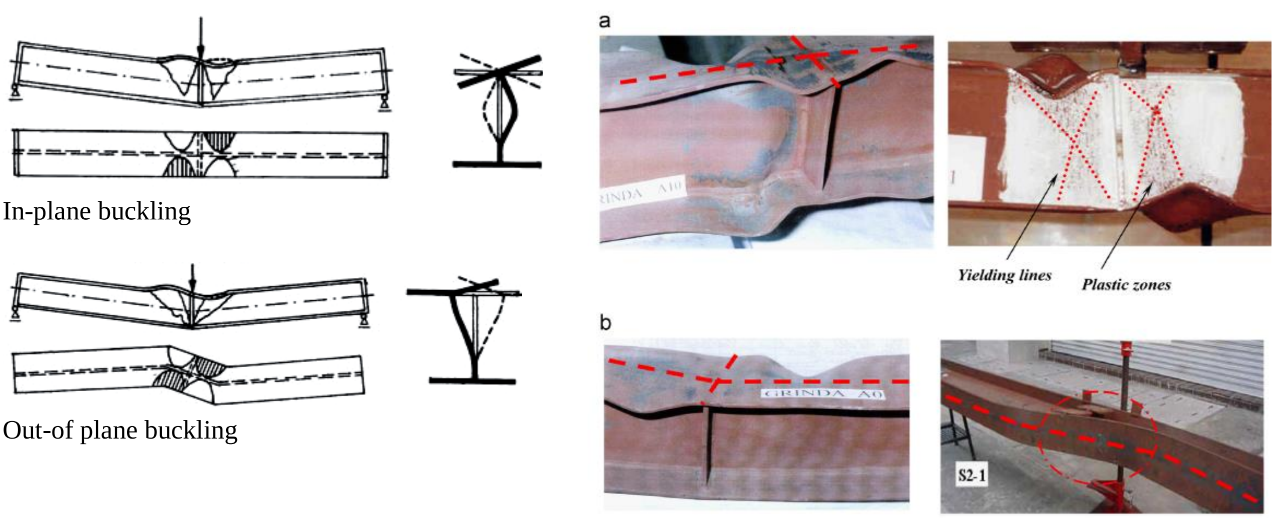


Fig. 3.1 Plastic buckling types for standard beam SB 1: a) In-plane buckling, b) Out-of-plane buckling

However, the available rotation capacity must be determined taking into account that the member belongs to a structure with a complex behaviour. But this is a very difficult task, due to the great number of factors influencing the behaviour of the actual member. Thus, it is important to simplify the analysis by using for the actual member a simple substitute element with a very similar behaviour. This member is so named *standard beam,* to determine the rotation capacity **[3.1]**, **[3.6]** .

Figure 3.2 shows the behaviour of a framed structure, where the *inflexion points* divide the beams into two portions, with positive and negative bending moments. The rotation capacity of the beam ends must be determined in different conditions. For positive moments, the plastic hinge works in a zone with quasi-constant gradient, while for negative moments, under important moment gradient (Fig. 3.2). Therefore, the actual behaviour of a member in a structure can be replaced with the similar behaviour of two standard beam types: the SB 1, with a central concentrated load for the zone under quasi-linear moment gradient, and the SB2, with a distributed load for the zone with weak moment gradient (Fig. 3.2). Considering that the inflexion point is situated at (0.2-0.3) Lb, the relation between standard beam span, L, and real beam in a structure, Lb, is:

Standard beam span, mm	Beam span, mm
2000	3500…5000
3000	5000…7500
4000	6500…10000
5000	8000…12500

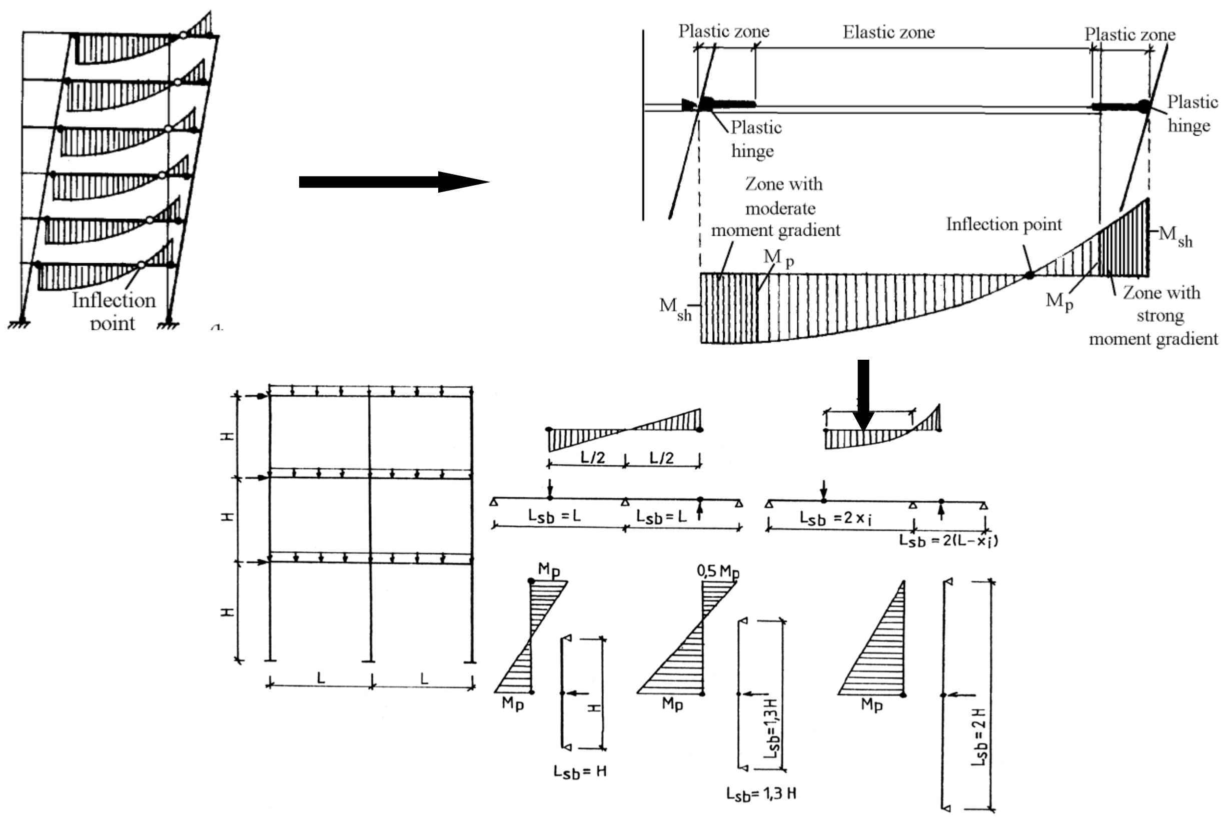


Fig. 3.2 The standard beam concept introducing the frame effect

3.2.2 In plane local plastic mechanism

This local plastic mechanism is characterized by deformation situated in the beam plan, without lateral displacements of flanges (Fig. 3.1a). The shape of collapse mechanisms observed during the experimental tests is composed by yield lines (true mechanism) or by combination of yield lines and plastic zones (quasi-mechanism). The last mechanism type involves a large amount of energy than the first type, due to the membrane yielding of plastic zone. For the standard beam SB 1 (Fig. 3.3a), the global plastic mechanism is presented in figure 3.3b. It is composed by two local plastic mechanisms. The mechanisms rotate around the rotation centre O. An experimental moment-rotation curve is presented in figure 3.3c. There are some important characteristic points which mark some significant changes regarding beam behaviour. The first point A refers to the reaching of the flange yielding; the second one B is defined by the occurrence of fully plastic moment. Moreover, in order to be developed the plastic hinge, the rotation should be increased. At this stage an important observation is that the increasing of bending moment over the fully plastic moment is due to the strain-hardening behaviour. The maximum value for moment is reached in point C, when plastic buckling occurs in the yielding zone of the compression flange and web. In this step, the local plastic mechanism is formed. After point C the bending begins to decrease with the increasing of rotation and the equilibrium of the beam becomes unstable. The ultimate rotation capacity is determined in the lowering post-buckling curve at the intersection with the theoretical fully plastic moment (point O”).

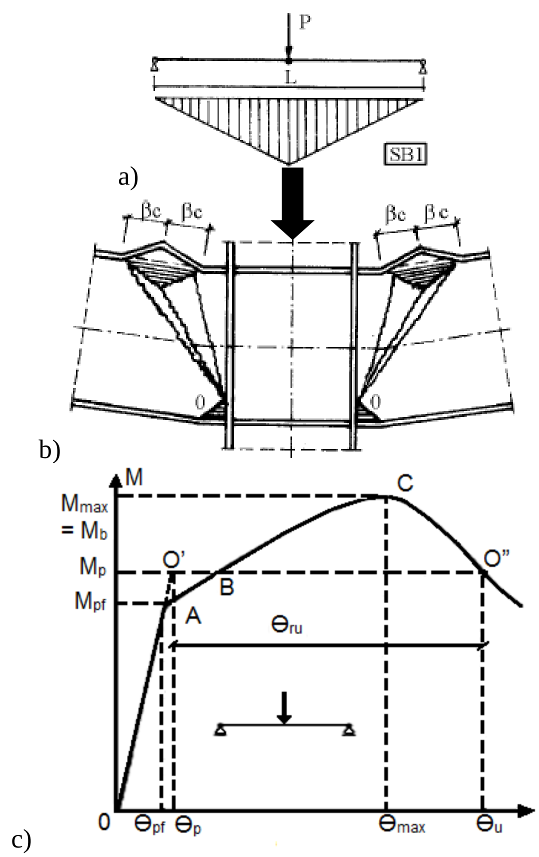


Fig. 3.3 Local plastic mechanism for gradient moment: a) SB 1 standard beam, b) Local plastic mechanism, c) Actual moment-rotation curve

The work of a plastic mechanism implies that a larger amount of energy is absorbed in the small area of plastic hinges and zones, therefore the other parts can be neglected. The rigid-plastic analysis is based on the principle of the minimum of total potential energy. The result of this analysis, after some mathematical operations presented in Gioncu & Mazzolani, 2002 **[3.1]**, is the post-critical curve of the local plastic mechanism, described by the relationship:

$$M/Mp = A_1 + A_2\,\theta^{-1/2} \qquad (3.1)$$

The coefficients A_1 and A_2 contain the mechanical and geometrical characteristics of beams and the shape of plastic mechanism. They are given in Gioncu & Mazzolani, 2002 **[3.1]**. The post-critical curve depends of geometrical parameters of local plastic mechanism. The length of mechanism is examined in **[3.1]**, based on theoretical studies and experimental data. For the prediction of the in-plane local available ductility the DuctRot-M computer program was developed **[3.10]** and experimentally calibrated **[3.1]**, **[3.6]** .

3.2.3 Out- of plane local plastic mechanism

This plastic mechanism is characterized by lateral displacements of flanges (Fig. 3.1b, 3.4). This mechanism type is produced by a free lateral rotation around the vertical axis (Fig. 3.1b, 3.4). The majority of research works do not take into account the fact that the joint could be restrained by elements to prevent the rotation. However, in practical cases the formation of plastic mechanism also involves the participation of the adjacent column. The plastic mechanism is composed by two plastic zones and yielding lines. The post-critical curve is described by the relationship:

$$M/Mp = B1 + B2\,\theta\text{-}1/2 + B3\,\theta\text{-}3/4 \qquad (3.2)$$

The coefficients B_1 , B_2 and B_3 are given in Gioncu & Mazzolani, 2002 **[3.1]**. One can remark that, comparing the equation (3.1), for in-plane mechanism, with equation (3.2), for out-of-plane mechanism, an additional term appears, producing a supplementary degradation in the post-buckling range. Figure 3.5a shows the determination of out-of-plane rotation capacity, for the same profile as the one of figure 3.5b. Obviously the rotation capacity for out-of-plane is larger than the in-plane one, but the degradation is higher.

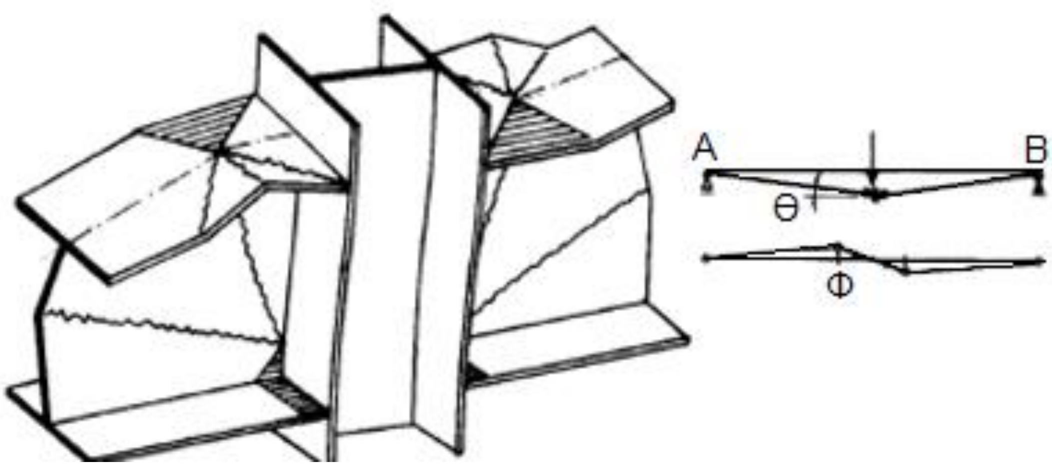


Fig. 3.4 Out of plane plastic mechanism

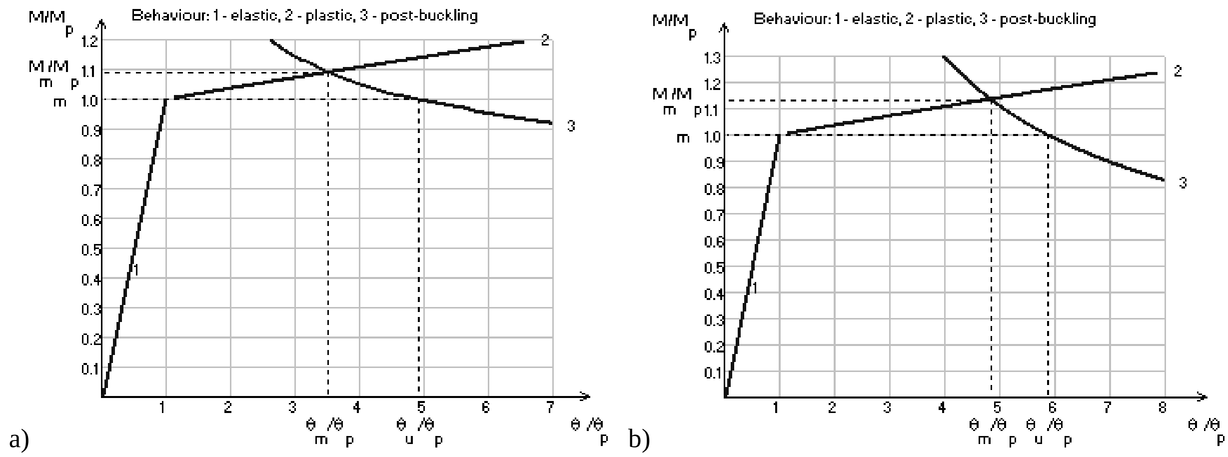


Fig. 3.5 M-θ a) for in plane plastic mechanism, b) for out of plane plastic mechanism

3.2.4 Interaction between in-plane and out-of-plane local plastic mechanisms

The experimental evidences **[3.11]** show that in majority of tests the first formed mechanism is the in-plane one, and only in the post-buckling range the beam buckles in out-of-plane, due to considerably weakened of flange rigidity, caused by the plastic deformations. Two cases of interaction were distinguished (Fig. 3.6):

(i) The intersection of two post-buckling curves takes place under the line M/Mp = 1, when the rotation capacity is defined by in-plane mechanism.

(ii) The intersection occurs over this line, and the rotation capacity must be determined taking into account the interaction of these two buckling modes and the rotation capacity is defined by out-of-plane mechanism.

It is very well known that the coupling of two buckling forms can increase the influence of imperfections **[3.12]**. But this form of coupling belongs to the category of weak interaction in post-critical range. In this case the interaction could be neglected, being covered from the scatter caused by other factors with higher influence on rotation capacity than this interaction.

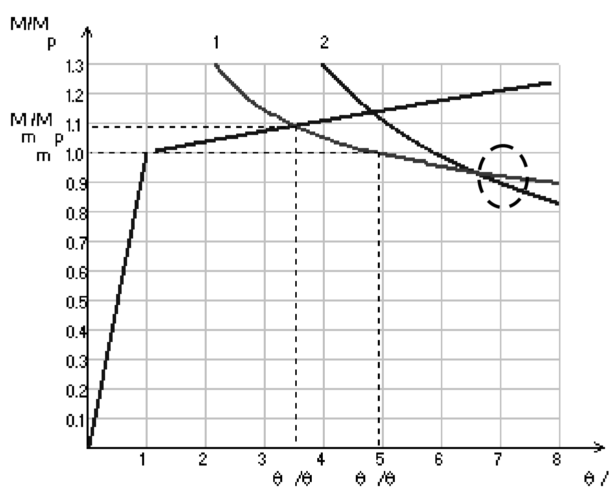


Fig. 3.6 Interaction between in-plane and out-of-plane local plastic mechanisms

3.3 Definition of the rotation capacity

In order to decide if a structural member has or not sufficient ductility and to ensure a suitable response under different loading conditions, the practice requires defining some indicators. As such can be the ductility index or rotation capacity. Concerning these indicators one must recognize that there is no standard definition which is universally accepted by all the specialists. However the most rational definition is related to ultimate rotation, figure 3.7. The member ductility is based on the determination of rotation capacity parameter R, defined as the ratio between plastic rotation at collapse θ_u and the elastic limit one θ_p ($\theta_u = \theta_u - \theta_p$):

$$R = (\theta_u/\theta_p) - 1 \tag{3.3}$$

This definition requires the calculation of ultimate rotation. With the aid of DUCTROT-M **[3.10]** it is possible to determine the ultimate rotation at the intersection of post-critical curve with the theoretical fully plastic moment (for the case of in plane or out plane plastic mechanisms). The software facilitates the evaluation of the available ultimate rotation or alternatively the non-dimensional available rotation capacity under various geometrical, mechanical and loading conditions (monotonic, cyclic and strain-rate).

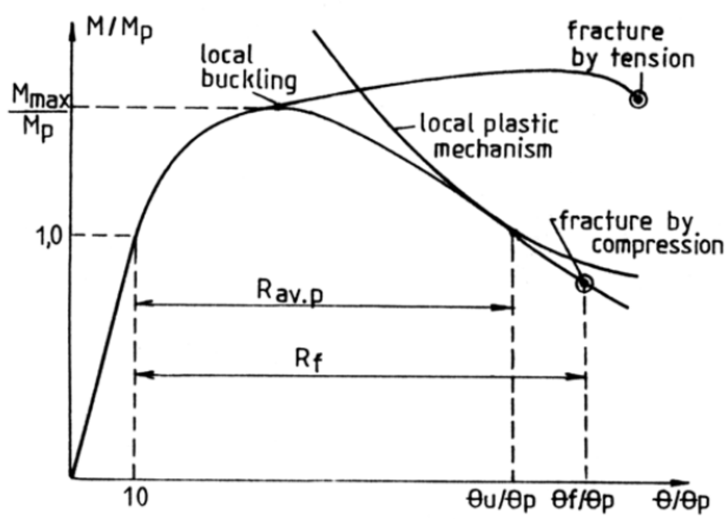


Fig. 3.7 Definition of the rotation capacity of a steel member

Considering the definition of ultimate rotation capacity, focused on local ductility under basically monotonic loading conditions **[3.1]**, the classification criteria are:

- HD (DCH) $R > 7.5$
- MD (DCM), $4.5 < R < 7.5$
- LD (DCL), $1.5 < R < 4.5$

Members having $R < 1.5$ are considered non-ductile.

3.4 Investigation of the monotonic local available ductility of steel beams

3.4.1 Influence of junction

Hot rolled I-section members widely used for beams in multi-storey buildings have different available rotation capacity than the welded sections due to a rigid zone, named junction, which connects the web with the flange. This effect was introduced in the draft version of Eurocode 3 **[3.3]** whereas the final code provisions prescribe conservatively the same slenderness limits for both rolled and welded steel sections (see Section 3.2). The junction, r, creates a condition under which the flange buckles around the rigid zone, (Fig.3.8) thus reducing the flange width and as a consequence increasing the rotation capacity of the element. In order to evaluate the available ductility under real constructional circumstances an improved plastic collapse mechanism was proposed **[3.13]**, **[3.14]** (Fig. 3.8). In this mechanism the increased plastic zone due to the junction is introduced in local plastic zone.

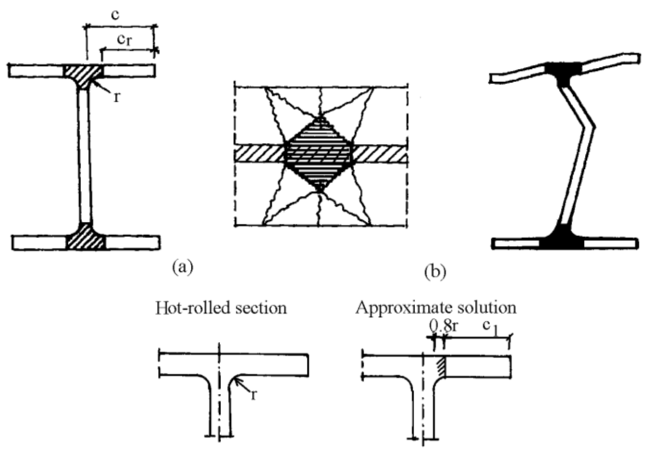


Fig. 3.8 Plastic collapse mechanism for hot-rolled sections

Benefiting from the facilities of the DUCTROT-M computer program, which implements the usual constructional details of I-wide flange sections, a parametrical analysis took place. The main target is to demonstrate the contribution of the junction on the plastic rotation capacity. Concerning the contribution of the junction on the plastic rotation capacity, one can remark a very significant increasing of available ductility attaining approximately 50% and 85% for HEA and IPE sections respectively as compared with the same sections without junctions (Fig. 3.9).

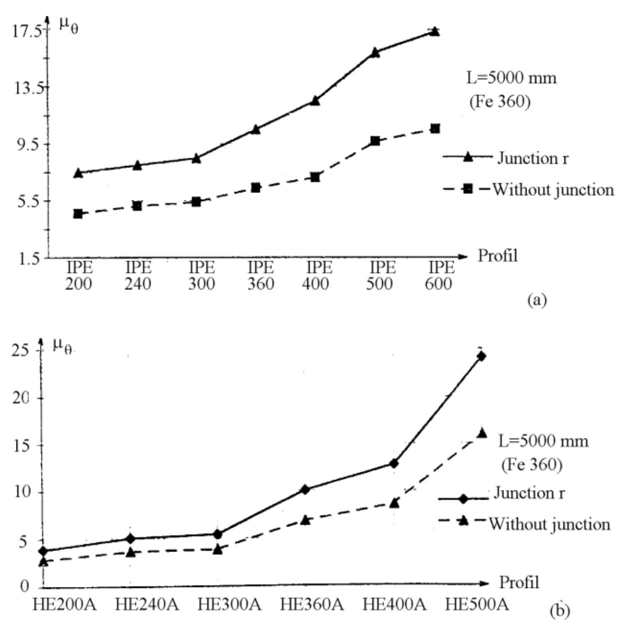


Fig. 3.9 Influence of the junction on the rotation capacity of the steel hot-rolled I-beams

The correlation between the values obtained using the modified collapse mechanism and the ones obtained using equation $R_j = r_j R$ with $r_j = (c/c_r)^2$; $c_r = c - 0.5t_w - 0.8r$ **[3.8]**, is presented in figure 3.10, showing that this simple procedure allows to determine the improved values of the rotation capacity of rolled sections. One must mention that the influence of junctions is much higher than the approaches given in code. This fact was not taken into consideration in the mandatory version of the Eurocode 3, Part 1, excluding completely the contribution offered by the interaction between flange and web.

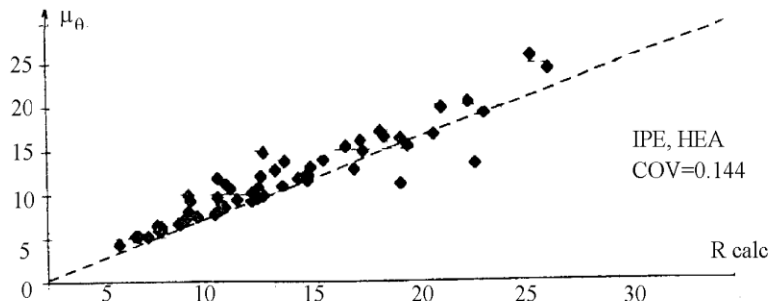


Fig. 3.10 Correlation between theoretical values and simplified approach

3.4.2 Ductility of reduced beam section (RBS)

An idea of the weakening of flanges at the beam ends has been proved to be very effective solution, among others, safeguarding the brittle joints **[3.15]**. By cutting the beam flange near the joint in a specific zone (dog-bone), the moment capacity is reduced, thus allowing the formation of plastic hinges away from the beam-to-column connection. In this way the flange welds are protected generating stable conditions for beam yielding. A numerical study investigating the ductility of European IPE and HEA RBS connections was presented in Anastasiadis A, Mosoarca M, Gioncu V., 2005 **[3.13]**, providing a classification according to member ductility criterion depending on section reduction, Table 3.1. It is important to notice that the treatment of reduced beam section zone with respect to cross-section ductility is doubtful; therefore the member ductility is a promising option.

Figure 3.11 presents the example of IPE profile, for which the rotation capacity is determined in function of the flange reduction for in-plane and out-of-plane plastic mechanisms. One can remark that an important increasing of in-plane rotation capacity is obtained, if the measure to prevent the joint twisting is assured. During the experimental tests performed on standard beams, it is evidenced that always the plastic buckling starts with an in-plane mechanism, followed by out-of-plane mechanism due to the weakening of buckled flange. Moreover, experimental tests performed in Chi, B., Uang, C.M., 2002 **[3.16]**, have shown that, due to flange reduction this phenomenon is accelerated. A result of this behaviour is the reduction of lateral rigidity. Because the plastic buckling occurs in the reduced beam section, away from the joint, the measures to increase the joint rigidity have no effects. This phenomenon is mainly connected with the low twisting rigidity of a deep column as compared with the shallow one. Practically the collapse mechanism occurs by the out-of-plane plastic mechanism and, in consequence, a reduction of rotation capacity must be considered. In addition, due to the presence of out-of-plane forces the degradation of lateral rigidity during cyclic loading is more accelerated. A recommendation for the application of an extra lateral bracing near the RBS region, to avoid the out-of-plane buckling, is proposed in Chi, B., Uang, C.M.,2002 **[3.16]**. In any case a simple constructional solution is to use shallow columns and deep beams (e.g. HEB/HEM columns and IPE/HEA beams).

Table 3.1 – Available ductility of RBS sections

Profile	L	Unreduced Beam	g (mm) $0.20b_f$	Ductility Level	Perform. Level	g (mm) $0.25b_f$	Ductility Level	Perform Level
IPE 330	6000	M	32	M	SD	40	H	NC
	8000	L		M	SD		M	SD
	10000	L		L	RD		M	SD
IPE 360	6000	M	34	M	SD	42	H	NC
	8000	M		M	SD		M	SD
	10000	L		L	RD		M	SD
IPE 400	6000	M	36	M	SD	45	H	NC
	8000	M		M	SD		H	NC
	10000	M		M	SD		M	SD
IPE 450	6000	M	38	M	SD	47	H	NC
	8000	M		M	SD		H	NC
	10000	L		M	SD		M	SD
IPE 500	6000	H	40	H	NC	50	H	NC
	8000	M		M	SD		H	NC
	10000	M		M	SD		M	SD
IPE 550	6000	H	42	H	NC	52	H	NC
	8000	M		M	SD		H	NC
	10000	M		M	SD		H	NC
IPE 600	6000	H	44	H	NC	55	H	NC
	8000	H		H	NC		H	NC
	10000	M		M	SD		H	NC

Profile	L	Unreduced Beam	g (mm) $0.20b_f$	Ductility Level	Perform. Level	g (mm) $0.25b_f$	Ductility Level	Perform Level
HEA 240	6000	L	48	M	SD	60	M	SD
	8000	L		L	RD		M	SD
	10000	L		L	RD		L	RD
HEA 260	6000	L	52	M	SD	65	M	NC
	8000	L		M	SD		M	SD
	10000	L		L	RD		M	SD

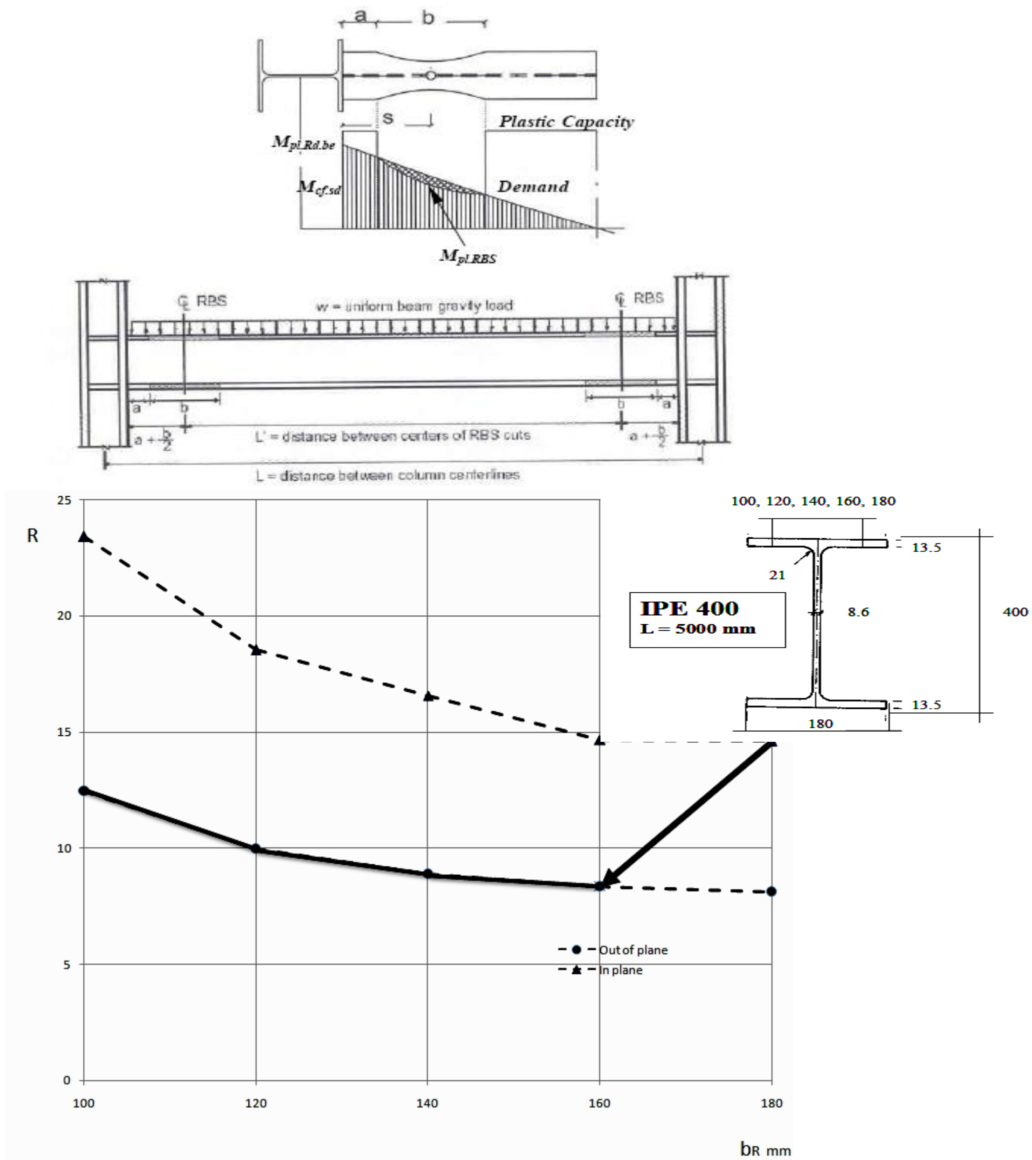


Fig. 11 Rotation capacity for RBS

3.4.3 Member ductility of IPE and HEA beams

It has shown in Gioncu V, Mosoarca M, Anastasidis A, 2012a,b **[3.4]**, **[3.5]** that the classification based on the section concept, used from Eurocode 3, does not cover the real deformational behaviour of the element due to the fact that does not consider some very important factors. Based on the validated methodology of plastic collapse mechanism a new member ductility classification was proposed. Table 3.2 and Table 3.3 provide the monotonic available member ductility of IPE and HEA European I-sections. For the elaboration of the tables, using the DUCTROT-M computer program, all the crucial factors affecting the local ductility (section and member characteristics, steel quality, constructional details, type of loading) was taken into account.

A first remark is that all the European profiles reveal an out-of-plane mechanism due to the fact that the thickness ratio of web and flange, t_w/t_f, varies between 0.63-0.66 for IPE sections and

0.52-0.63 for HEA sections. This is a confirmation of the numerical results presented in Gioncu V, Mosoarca M, Anastasiadis A., 2009 **[3.17]**. A rule in order to avoid the out-of plane mechanism is that this ratio must be higher than 0.7-0.8.

Secondly, comparing the section classification according to the Eurocode 3, with the proposed classification it is evident that the available ductility changes as a function of member span, Table 3.3. For IPE beams a lowering plastic capacity could be observed as the member span increases and the steel quality becomes higher. Generally, the same conclusions could be observed for the case of HEA beams. One must mention that the increasing of rotation capacity has two opposite effects:

(i) a favourable one with respect to structural behaviour, considering the large capacity to dissipate seismic energy induced by the earthquakes.

(ii) an unfavourable one, with respect to the fact that disappear the filter against large strains in tension flanges. Hence in case of ground motions inducing high strain-rate the danger of brittle cracking is increased. Due to this fact all the connections tested in the research program of SAC, 2000, collapsed by the brittle fracture.

Current codes do not prescribe direct criteria for the evaluation of local available ductility. An important advancement of the Eurocodes will be the implementation of the ductility based design together with the strength based design. Exploiting the current knowledge such an achievement could be obtained considering that the basic components of a frame (beams, columns, connections, slabs) interact between them always taking into account the constructional details of the structural system as well as the fact that exceptional earthquakes (cyclic vs. strain-rate loading) induce different inelastic requirements in structural components.

Table 3.2 – Classification of IPE beams based on the member criterion of local ductility

Member classification of IPE.

Steel Section	Buckling mode	L = 2000mm		L = 3000mm		L = 4000mm		L = 5000mm	
		S235	S355	S235	S355	S235	S355	S235	S355
IPE 300	IP	HD	HD	HD	MD	- -	- -	- -	- -
IPE 330	IP	HD	HD	HD	MD	HD	MD	- -	- -
IPE 360	IP	HD	HD	HD	HD	HD	MD	MD	LD
IPE 400	IP	HD	HD	HD	HD	HD	MD	MD	LD
IPE 450	IP	- -	- -	HD	HD	HD	MD	MD	MD
IPE 500	IP	- -	- -	HD	HD	HD	MD	MD	MD
IPE 550	IP	- -	- -	- -	- -	HD	HD	HD	MD
IPE 600	IP	- -	- -	- -	- -	- -	- -	HD	MD

IP - In plane post elastic buckling mechanism obtained with measures to increase the torsional rigidity of the nodes.

- - Sizing of the member would be other than ductility limit state. For instance serviciability limit state would be the predominant criteria for member sizing.

Table 3.3 – Classification of HEA beams based on the member criterion of local ductility

Member classification of HEA profiles.

Steel section	Buckling mode	L = 4000mm		L = 5000 m	
		S235	S355	S235	S355
HEA 320	IP	HD	MD	HD	MD
HEA 340	IP	HD	HD	HD	MD
HEA 360	IP	HD	HD	HD	HD
HEA 400	IP	HD	HD	HD	HD
HEA 450	IP	HD	HD	HD	HD
HEA 500	IP	HD	HD	HD	HD
HEA 550	IP	HD	HD	HD	HD
HEA 600	IP	HD	HD	HD	HD

IP - In plane post elastic buckling mechanism obtained with measures to increase the torsionalrigidity of the nodes.

3.4.4 Available rotation capacity of welded sections

3.4.4.1 Influence of welding type

For practical design an issue connected with ductility of welded beams is the definition of flange width as a function of welding type. Using the same method as for rolled sections, in the first step the rotation capacity is determined without the influence of the junction and after that a correction factor is applied using for the effective widths, where:

-for filled welds (Fig. 3.12a):

$$c_r = c - 0.5\, t_w - 1.1a \tag{3.4a}$$

-for penetrated welds (Fig. 3.12b):

$$c_r = c - 0.5\,(t_w + a) \tag{3.4b}$$

a denoting the weld thickness. By a numerical analysis with DUCTROT-M for the welded section (with L = 5000 mm) (Fig. 12) the influence of welding type is presented in Table 3.4:

Table 3.4 – Rotation capacity of welded sections

Welding Type	**a , mm**			
	4	5	6	7
Without welding	3.14			
Fillet welds	3.71	3.78	3.86	3.91
Penetrated welds	3.55	3.58	3.62	3.65

As it can be noticed, the influence of welding thickness and the welding type are not very important on the inelastic deformation of such sections.

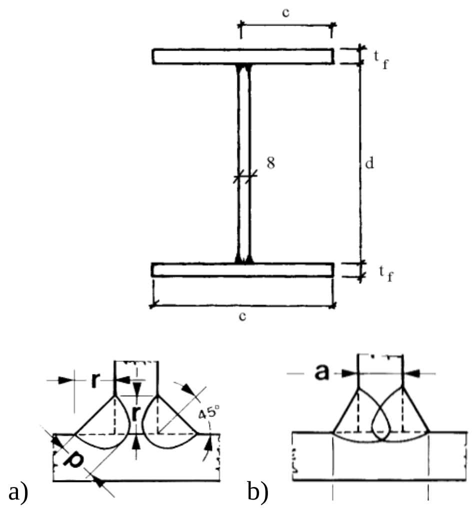


Fig. 3.12 Welded I sections with a) fillet weld type, b) penetrated welds

3.4.4.2 Influence of steel grade and yield stress random variability

Figure 3.13 presents the influence of steel grade on rotation capacity. One can see that the ductility decreases with the increase of yield stress. The decreasing is more important than the method proposed in the code, in which the results must be multiplied with the factor $(235/fy)^{1/2}$.Due to this fact, for the strength verifications the minimum of yield stress must be used, whereas for ductility verifications must be performed with the maximum yield stress. Considering the random variability of yield stresses, an approximate relation between $f_{y,max}$ and $f_{y,min}$ is further proposed:

$$f_{ymax} = f_{ymin} + 50...70 \qquad (3.5)$$

Figure 3.14 shows the decreasing of rotation capacity taking account of random variability. Obviously it is very important; therefore the calculation of rotation capacity in the DUCTROT-M computer program is determined considering the random variability of yield stresses.

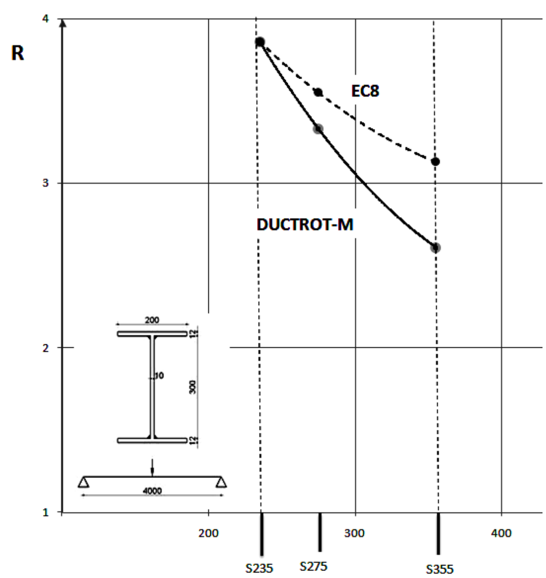


Fig. 3.13 Influence of yield stress and steel grade on rotation capacity

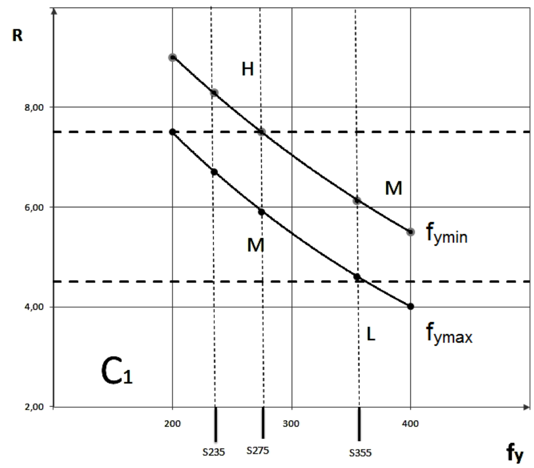


Fig. 3.14 Influence of random variability on rotation capacity

3.4.4.3 Parametric studies on member rotation capacity

In order to establish the optimum solution providing the maximum member ductility the influence of decisive geometrical cross-section dimensions is examined.

Influence of flange thickness. Figure 3.15 shows the influence of flange thickness on the member ductility. The increasing of rotation capacity is very high with the increasing of flange thickness. A comparison between cross-section classes and member classes is also presented in this figure. The observation referring to the critics about the discrete subdivision in classes is confirmed. Member ductility of medium class, MD, superposed over the cross-section class C3. Additionally, the rotation capacity resulted from in plane mechanism is limited by the out-of-plane mechanism.

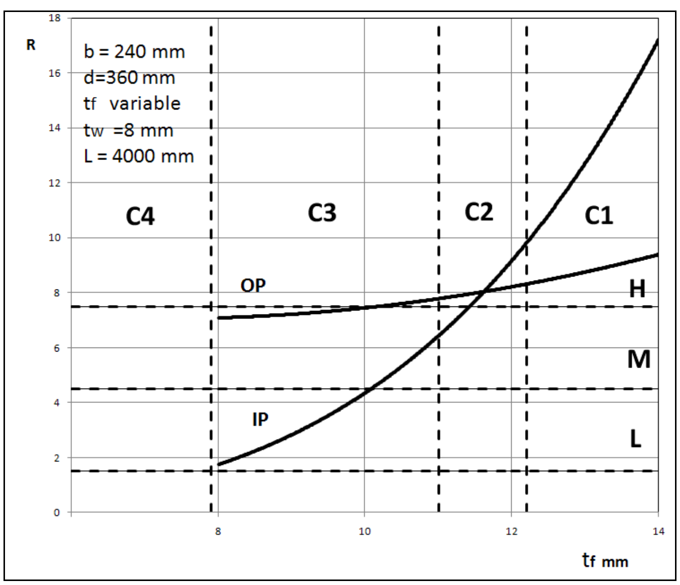


Fig. 3.15 Influence of flange thickness

Influence of flange width. The profile can be classified in C1 class reported to cross-section classification. However, the member classification introduces the profile in medium ductility class, MD, the variation of flange width having no significant influence on the rotation capacity (Fig. 3.16).

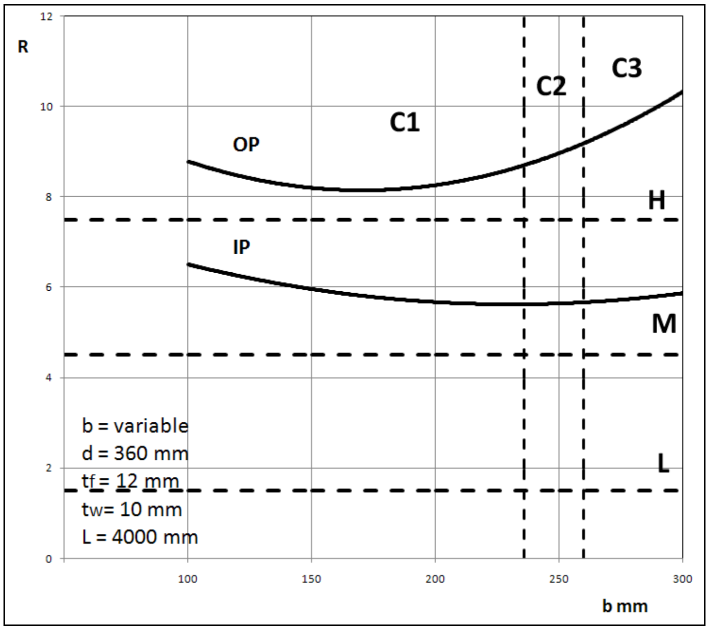


Fig. 3.16 Influence of flange width

Influence of web thickness. Contrary to the influence of flange thickness, the decreasing of rotation capacity with the increasing of web thickness is very important (Fig. 3.17). The profile is framed in class C1, while in relation with the member ductility criterion can achieve a medium, MD, or low, LD, inelastic capacity. The out-of-plane mechanism limits the increasing of rotation capacity with the decreasing of web thickness.

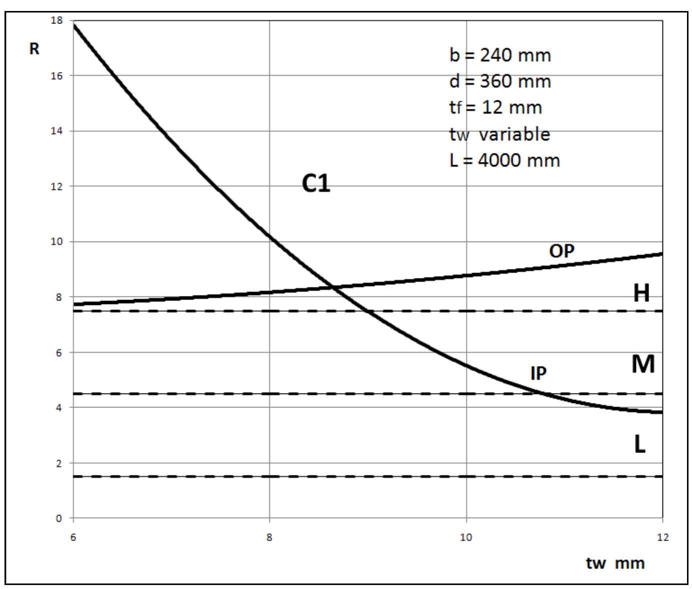


Fig. 3.17 Influence of web thickness

Influence of web height. Like the flange width, neither the web height has a very important effect in increasing of rotation capacity (Fig. 3.18). The profile belonging to class C1 provides a medium level, MD, of rotation capacity.

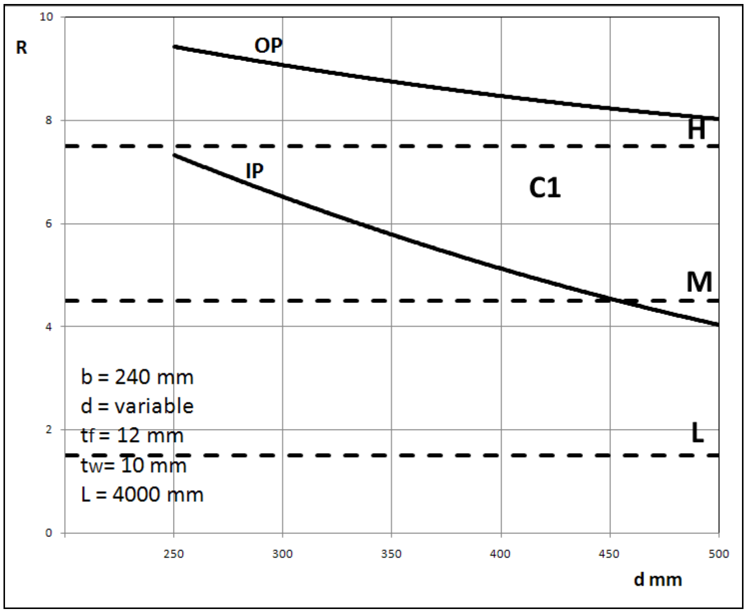


Fig. 3.18 Influence of web height

Influence of beam span. The inability of cross-section classification to deal with the real deformational capacity of steel element is showed in figure 3.19, where the rotation capacity of a profile framing in class C1, presents medium, MD, and low, LD, ductility, in function of the beam span.

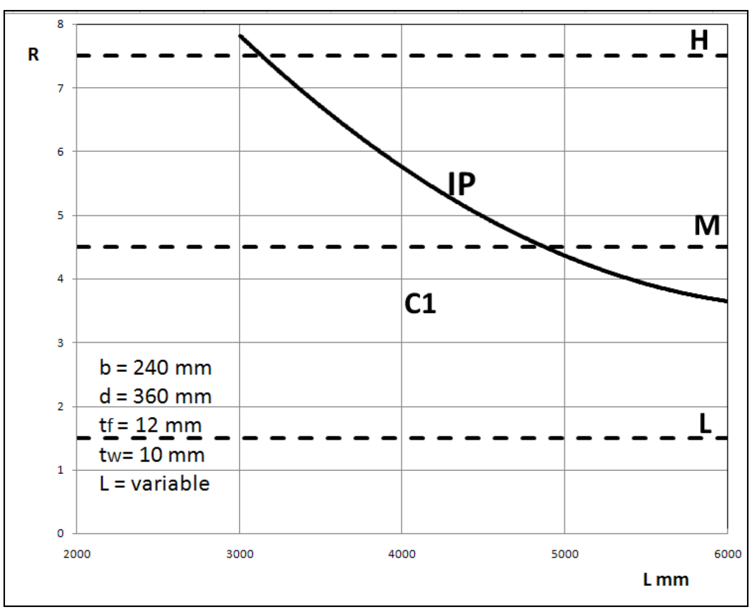


Fig. 3.19 Influence of beam span

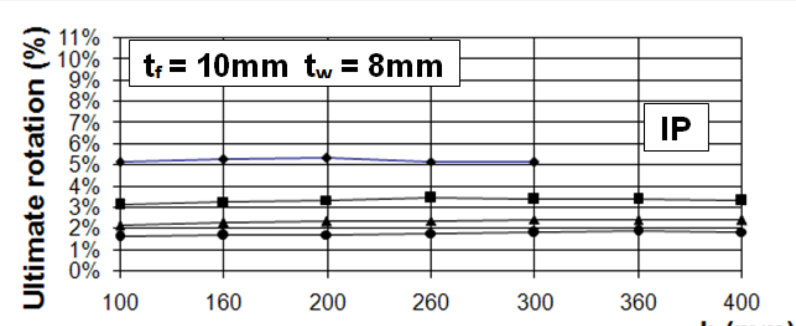
3.4.4.4 Conclusions for a proper selection of profile dimensions

Generally, due to the functional reasons the beam span is not a parameter which can be modified. Therefore, only cross-section dimensions like thickness and width of flanges as well as thickness and height of web should be varied in order to obtain a proper rotation capacity. The parametrical studies have shown that the flange width and web height have no important effects on increasing the rotation capacity, and that the increasing of flange and web thickness has contrary effects. The reasons of these conclusions are related to the wave-length of plastic mechanism. For in-plane mechanism the rotation capacity is function of this length, the capacity of dissipating energy being multiplied by increasing of yielding line lengths. The length of plastic mechanism is given in Gioncu V, Mazzolani FM, 2002 **[3.1]**, based on theoretical researches and experimental data as:

$$Lm = 0.6\ (t_f/t_w)^{3/4}\ (d/b)^{1/4}\ b \qquad (3.6)$$

Examining the relationship (3.6) it is obviously that the ratio t_f/t_w has a great influence on the plastic mechanism length, while the ratio d/b has a reduced influence. These observations are confirmed by the numerical tests on rotation capacity. The main factor affecting the rotation capacity is the increasing of flanges thickness. In addition, the factor producing the most important decreasing of rotation capacity is the web thickness.

Special attention should be paid to eliminate the possibility to occur the out-of-plane mechanism. This deformational condition drastically reduces the rotation capacity and also accelerates the degradation during cycle loading. Figure 3.20 examines the influence of the flange width and web height. Keeping constant the web thickness, t_w, for instance taken equal of 8 mm and varying the flange thickness, the following observations could be done: for flange thickness, tf, equal of 10 mm, the dominant mechanism is the in-plane one (Fig. 3.20a), for t_f equal to 12 mm, the both plastic mechanisms are presents (Fig. 3.20b), while for t_f equal of 14 mm only out-of-plane mechanism is present (Fig. 20c). Therefore, in order to eliminate the out-of-plane plastic mechanism and fulfil other structural properties such as stiffness and strength, the ratio of flange to web thickness, t_f/t_w, is recommended to be taken between 1.10–1.35. Moreover, as a design rule of thumb or for preliminary design purposes it is proposed that the thickness of flange and web could be approximately of the same order.



(a)

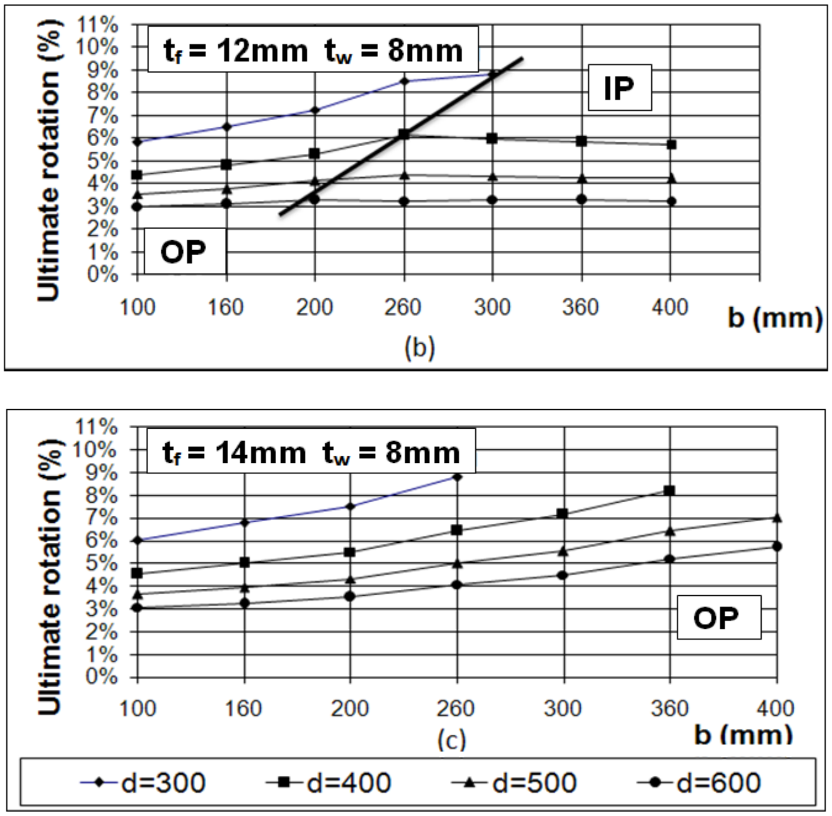


Fig. 3.20 Optimum adjustment of welded profiles

3.5 Considerations to the contradictive issues related to the prediction of the local ductility

3.5.1 Cross-section ductility vs. member ductility

Generally, in order to characterize a beam into a frame we distinguish two ductility types, widely used in structural community, (Fig. 3.21):

(i) Cross-section ductility, or curvature ductility, which refers to the plastic deformations of cross-section, considering the independent behaviour of the parts composing the cross-section itself;

(ii) Member ductility, or rotation ductility, when the properties of members (interaction between cross-section parts, influence of beam span and loading system) are considered.

The choice of structural ductility by one of these two types gives rise to many discussions in the context of specialists. The first definition mainly used in code provisions, is based on cross-section behavioural classes (Fig. 3.22c):

- C1, class 1, plastic sections; sections are characterized by the capacity to develop a plastic hinge with high rotation capacity;
- C2, class 2, compact sections; sections are able to provide their maximum plastic flexural strength, but they have a limited rotation capacity due to some local effects;
- C3, class 3, semi-compact sections; the bending moment capacity for the first yielding can be attained, without reaching the plastic moment;
- C4, class 4, slender sections; sections are not able to develop their total flexural resistance due to the premature occurrence of local buckling in their compression parts.

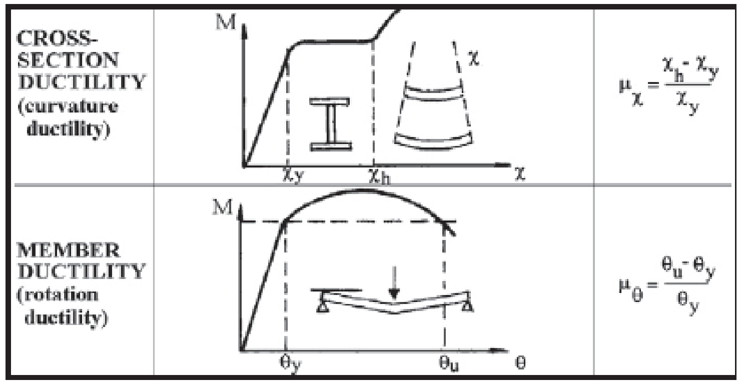


Fig. 3.21 Ductility types: cross-section ductility, member ductility

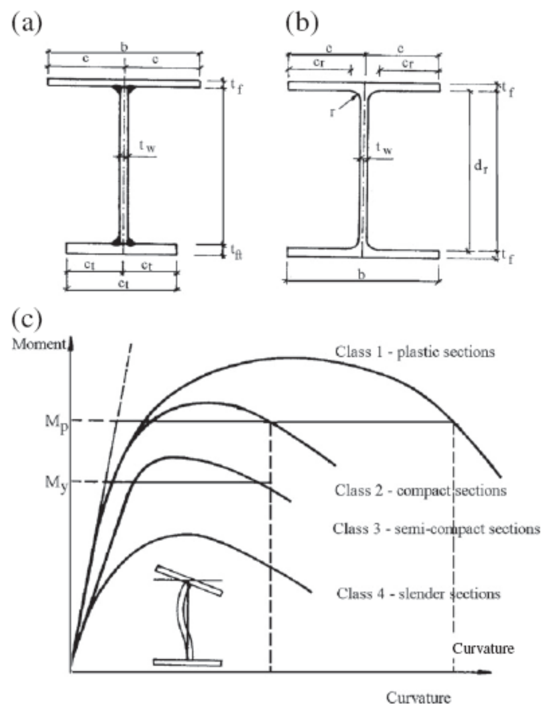


Fig. 3.22 Cross-section classification according to current design codes (i.e. Eurocode 3)

For the wide-flange sections, the flange and web slenderness are defined by the following ratios **[3.3]** (Fig. 3.22 a, b) (e.g. for steel S235) from Table 3.5:

Table 3.5 – Rotation capacity of welded sections

Section	Flange	Web
Type	c/tf	d/tw
prENV 1993		
Rolled	C1<10	<72
	C2<11	<83
	C3<15	<124
EN 1993, Part1:2005		
Welded	C1<9	<72
	C2<10	<83
	C3<14	<124

All the geometrical parameters regarding the hot-rolled sections are presented in figure 3.22b.
The classification limited only at the cross-section level has many deficiencies:
(i) independent limitations between flanges and web ratios are unreasonable because, obviously, the flange is restrained by the web and the web by the flange;
(ii) the local ductility depends not only on cross-section dimensions, but also on the ratio between width of flange and web, member length, and loading type;
(iii) the subdivision in different classes does not correspond to the actual behaviour of beams. The elements behaviour is continuous and the given discrete values of slenderness to define different classes seem to be very arbitrary taken from the independent calculation of plate buckling with some adjustments.
In spite of these recognized deficiencies, this classification is used in codes, due to its simplicity in design practice.
Another more effective classification at the level of member ductility for design has been proposed in **[3.1]** (Fig. 3.23). Following the conceptual spirit of Eurocodes could be defined as:
- HD (DCH), high ductility class, corresponding to members designed, dimensioned and detailed such that they ensure the development of large plastic rotations, HD (DCH) R > 7.5
- MD (DCM), medium ductility class, corresponding to members designed, dimensioned and detailed such that they ensure the development of moderate plastic rotations, MD (DCM), 4.5 < R < 7.5
- LD (DCL), low ductility class, corresponding to members designed, dimensioned and detailed such that they ensure the development of low plastic rotation only, LD (DCL) 1.5 < R < 4.5

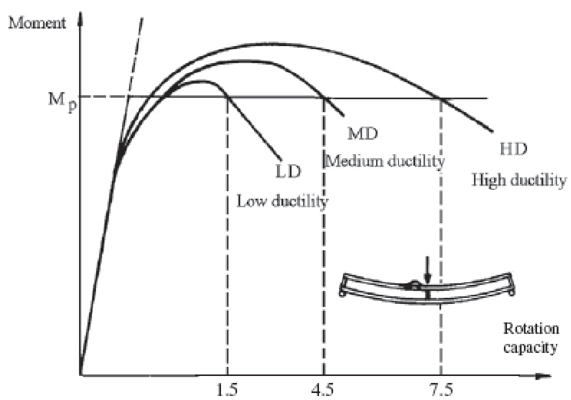


Fig. 3.23 Member ductility classes based on moment-rotation curve

The comparison between the results obtained using DUCTROT- M computer program for member ductility and the code provisions for cross-section ductility is presented in figures 3.24 and 3.25 for welded beams. Only in-plane plastic mechanisms are considered (see Section 3.3).
In these figures the hachured areas show the cases when the two classifications are coincidental. One can see that the coincidence is very poor. In figure 3.24, according to Eurocode 3: Part 1, the build up section has web slenderness belonging to C1 class. In a general case when the flange thickness, tf , is equal to 8, 10, 12 mm on can remark that, while in member ductility the profile belongs to LD, MD, HD classes, respectively, independently of flange width, in cross-section classification concept the classes change in function of flange width. For instance, the profiles with flange thickness, t_f = 12 mm can be framed in HD class for the entire field of flange width, b, of 120 – 280 mm. Conform to cross-section ductility criterion for b > 216 mm the section

belongs to C2 class, while for b > 240 mm to C3 class. Consequently, it is noticed that interrelating both the concepts, the classification in different classes as a function of flange slenderness is contestable.
Figure 3.25 considers a profile with such web dimensions belonging to C2 class due to the high web slenderness. In spite of this, the section having flange thickness, t_f, of 12 mm, can be classified in HD class. For this reason, even the sorting in different classes in function of web slenderness is contestable. Therefore, examining these Figures, one can observe great differences between the two concepts. The classification in cross-section ductility and member ductility lead to very different results. If the output provided by the DUCTROR-M software is considered reliable as revealed in Gioncu V., Mazzolani FM, 2002, **[3.1]** and Anastasiadis A, 1999 **[3.14]** then the cross-section ductility cannot be used in design practice and must be replaced by member ductility.

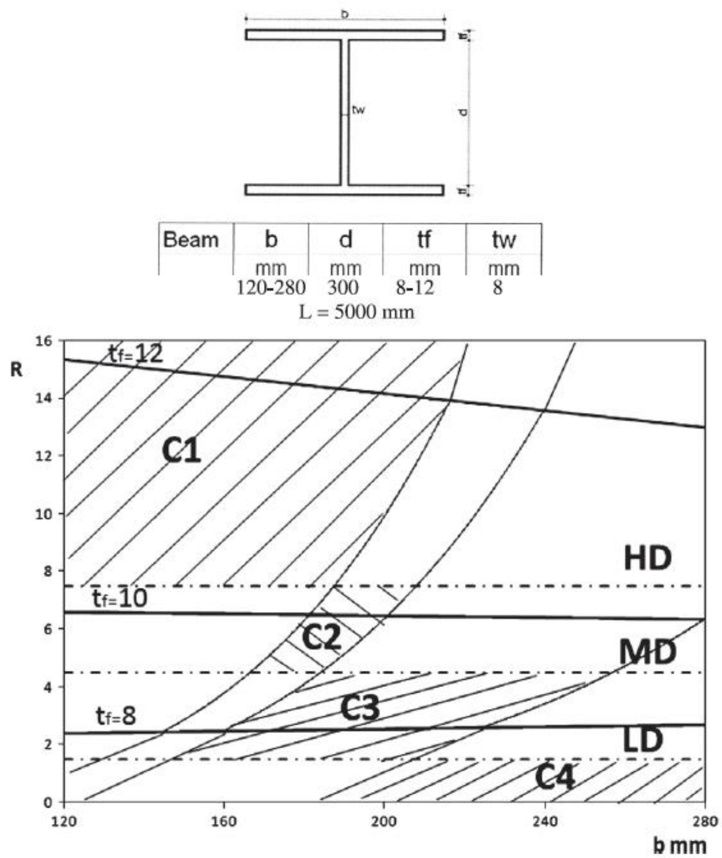
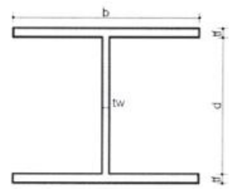


Fig. 3.24 Member ductility vs. cross section ductility for a profile with C1 class for the web



Beam	b	d	tf	tw
	mm	mm	mm	mm
	120-360	450	8-12	6

L = 6000 mm

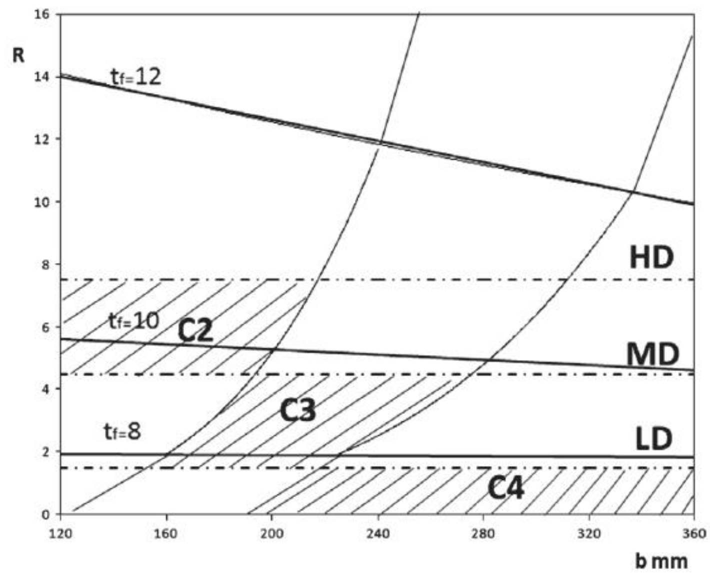


Fig. 3.25 Member ductility vs. cross section ductility for a profile with C2 class for the web

3.5.2 Gradient vs. quasi-constant moment

The methodology of the plastic collapse mechanism considers that in order to be evaluated the member ductility the element that belongs to a structure must be modelled by replacing it in two effective beams, namely the standard beam, SB1, loaded by a gradient moment of a short span beam and the second one, SB2, loaded by a quasi-constant moment of a long span beam (Fig. 3.26a). Hence, the ductility of the beam is given by the minimum of the values determined for these beams:

$$R = \min(RSB1, RSB2) \tag{3.7}$$

This approach neglects the presence of the slab, assuming that the formation of local plastic mechanism is free. In reality, the interaction of beam with the floor slab does not allow the plastic buckling of the upper beam flange. Hence, the formation of plastic mechanism will be exhibited in the compression zone of the steel section of SB 2 standard beam (Fig. 3.26b). The plastic hinge in this zone with sagging moment is developed with concrete crushing only. Therefore, the first plastic hinge is formed in SB 1 standard beam, for hogging moment, where the formation of plastic mechanism in steel beam is free. Accordingly, in practical design, for determining the available beam ductility, it is sufficient to consider only the SB 1 standard beam characterized by gradient moment:

$$R = RSB1 \tag{3.8}$$

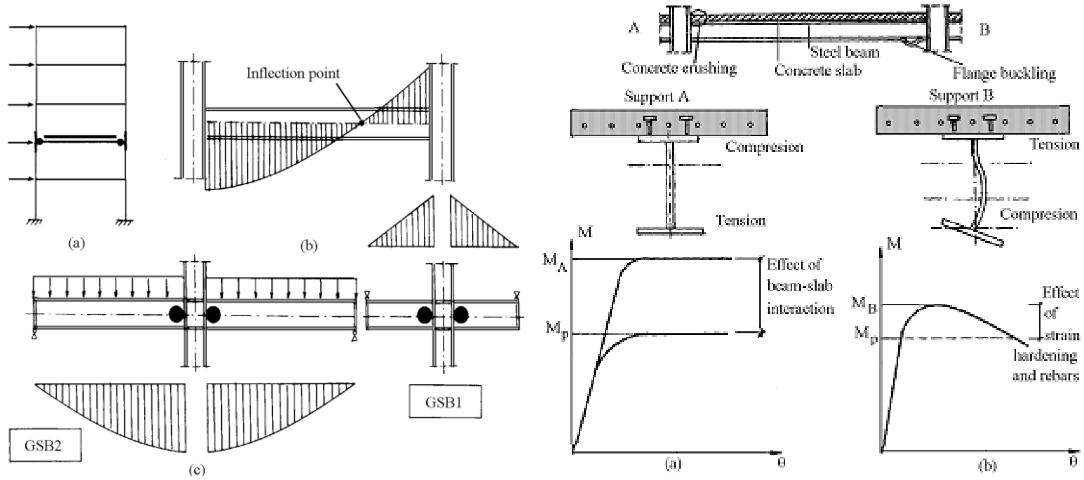


Fig. 3.26 Behaviour of steel beams under real conditions and the use of the standard beam concept

3.5.3 In plane vs. out-of plane mechanism

The plastic buckling for SB 1 standard beam occurs for hogging moment in compressed part by in-plane or out-of-plane plastic mechanism. The validation of DUCTROT-M is based on a mock-up that is fabricated with only a simple reinforcement under the force, which cannot prevents the lateral rotation of beam during plastic buckling. The examination of theoretical and experimental results has provided that the great majority of tested beams (especially for rolled sections) lose their carrying capacity by out-of-plane plastic mechanism. In contrast, in practical situations, the beam ends are connected to a complex joint, composed by a column reinforced with horizontal stiffeners and transversal beams which inhibits the free lateral rotation. Therefore, the results obtained during the experimental tests on the standard beams must be used only with a careful examination regarding the plastic mechanism type. In addition, it is essential to mention that during the experimental tests provided from SAC and RECOS research programs, where the beam ends were connected in columns, there are no collapse mechanisms denoting out-of-plane deformations. After the Northridge earthquake, in order to protect the beam-column connections, several experimental tests were executed using the reduced beam section concept, RBS or dog-bone. From the experimental evidence one can remark that, in case of deep columns, out-of-plane plastic mechanisms associated with an accelerated degradation of connection rigidity are observed. This degradation is due to the lateral-torsional buckling, produced from out-of-plane force components. This is an additional reason to eliminate in practical design the out-of-plane buckling mechanisms.

It is necessary to underline that the DUCTROT-M is the single one in which the user can differentiate the cases when the ultimate carrying capacity of beam is defined by in plane or out-of-plane plastic buckling. A parametrical analysis performed on the European sections IPE and HEA have shown that for these profiles the dominant local plastic mechanism is the out-of-plane one, with an important reduction of rotation capacity in comparison with the in-plane mechanism, in Table 3.6.

Table 3.6 – Rotation capacity of European hot-rolled sections

Rotation capacity of European hot-rolled sections						
Rolled sections	IPE 300	IPE 400	IPE 500	HEA 400	HEA 500	HEA 600
		L = 5000 mm			L = 6000 mm	
In-plane	9.03	13.41	15	21.13	35.12	40.59
Out-of-plane	6.27	8.12	8.83	9.75	10.29	10.51

Therefore, in order to obtain an increased local ductility of hot-rolled sections, it is very important to solve the beam-column details in a way to avoid the out-of-plane plastic buckling. An option is to increase the joint twisting rigidity which can be greater than the lateral rigidity of beam.

For welded sections, there are two ways to eliminate the out-of-plane plastic mechanism:

(i) To increase the joint torsional rigidity, which can be greater than the beam lateral rigidity.

(ii) To chose proper section dimensions preventing out-of-plane deformations.

In order to eliminate the out-of plane plastic mechanism, the ratio between web and flange thickness must be chosen in interval of 0.7 to 0.8. Because the European sections do not respect this condition, those ones have the tendency to lose the carrying capacity by out-of-plane plastic mechanisms **[3.4]**, **[3.5]**.

3.6 Investigation of the seismic available local ductility of steel beams

3.6.1 General remarks

Seismic-resistant structures are usually designed relying on their ability to sustain high plastic deformations. The design philosophy considers that the earthquake input energy is dissipated through the hysteretic behaviour of a member; plastic hinges are formed in predetermined positions due to a number of cycles of seismic loading. This concept is based on the assumption that the plastic hinge must show a stable hysteretic behaviour with a sufficient rotational capacity to allow for dissipating this input energy. According to current design trends, the structure may be designed for lower forces than those it has to resist, taking into account the inelastic reserves of the structural system. Therefore, the evaluation of the available ductility is of primary importance. The use of the monotonic ductility for seismic actions has provided to be a valuable concept, corresponding to the methodology included in the modern codes; however it is not possible to capture the deformational effects of a dynamic action on a structure. For instance, the Eurocode 8 **[3.2]** specifies the use of the ductility classes which is associated with a section classification given by the Eurocode 3 **[3.3]**, mainly determined for static loading, as well as with a behaviour factor, q, empirically determined. This approach is based on the observation that for many cases the load-deformation skeleton curves (constructed using the cyclic curve) correspond very well with the monotonic curves.

Lessons learned from past 40 years of real earthquake excitations reveal that in seismic design the following should be considered:

(i) the ground motions characteristics in function of source-site distance (far-field and near-field earthquakes);

(ii) the source types (inter-plate, intra-plate and intra-slab) with very different rupture characteristics.

As a consequence, the seismic actions are determined with great uncertainty. But the structure must be endowed by design with the ability to develop and maintain its bearing capacity, even when the considered seismic action exceeds the design limits. A measure of this ability is the ductility, which is the structural performance to sustain these exceeding by large deformations in plastic range without significant loss of resistance. The basic ductility design criterion, where any earthquake-resistant structure must satisfy, is the following:

Required ductility < Available ductility

Generally, the prediction of the available ductility should take into account the fundamental differences between ground motions. In this context, the simplest methodology is to evaluate the ductility for monotonic loads and accordingly to correct the determined values considering the specific characteristics of each earthquake type:

Seismic ductility = {Correction factor} x Monotonic ductility

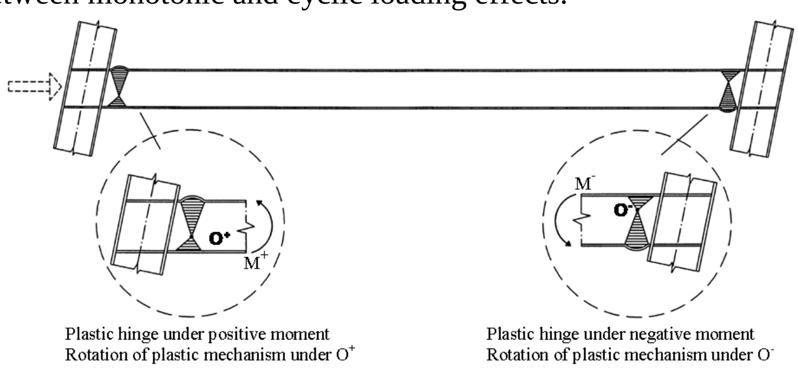
3.6.2 Local available ductility under cyclic loading

3.6.2.1 Plastic collapse mechanism of I-shaped steel beams under cyclic loading

At the member level, the plastic failure mechanism is introduced in the position where the plastic hinges are intended to be formed (adjacent to the column face or at a distance from the column face as a function of the joint detailing). Hence, this mechanism is a tool that permits to describe the inelastic behaviour of the beam that belongs to a frame. Under cyclic action the following response is observed (Fig. 3.27a):

- The positive moment, M+, causes buckling to the compressed upper flange while the section rotates around the point O+ situated near the tensioned flange;
- The negative moment, M-, causes buckling to the lower compressed flange while the section rotates around the point O^- located near the opposite flange.

Under the reversals of the seismic action the process continues in the same way to the next cycles. The buckled flanges and the rotation point are always situated in a different position while the tension strains are not able to straighten the buckled flanges. As a result the collapse mechanism is developed by the superposition of the two local plastic mechanisms. Moreover, during each of the next cycles, the element works with an initial geometrical imperfection as resulted from the previous cycle. Consequently, an accumulation of plastic deformations in the buckled flanges, associated with the number of cycles, could be observed (Fig. 3.27b). Physically the aforementioned process produces a gradual degradation which creates the condition for a differentiation between monotonic and cyclic loading effects.



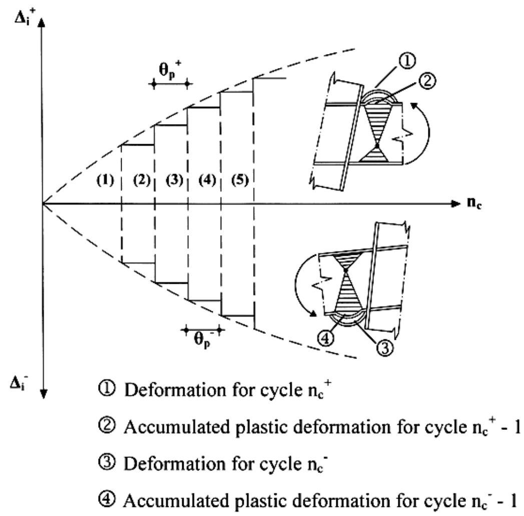
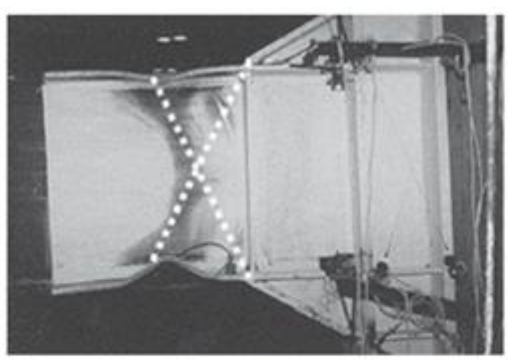


Fig. 3.27 Plastic collapse mechanism at the member level

The proposed plastic collapse mechanism, developed under monotonic loading, further extended, introducing the effect of the cyclic bending; under cyclic action describes the process of plastic accumulation with a main consequence, i.e.: the gradual strength and deformation deterioration. At the member level, due to the repetitive character of the action, a steel beam works under positive and negative bending moments, thus accumulating plastic deformations. From the experimental observations and the analytical works, the following issues could be remarked:

- under cyclic loading, during experimental tests, a superposition of two plastic mechanisms for positive and negative moments was clearly observed, (Fig. 28a);
- the first semi-cycle buckling occurred in an upper compressed flange while the section rotates around a point in or near the opposite flange. The tensile forces in the opposite flange are very small (Fig. 3.28b);
- for the reversal semi-cycle the bottom compressed flange buckles too, but due to the fact that the tensile forces are small the reversal action is not able to straighten the upper buckled flange;
- during the next cycles, n_c^+ (for upper flange) and n_c^- (for bottom flange), the section works with an initial geometrical deformation, Δ_i,, resulted from the previous cycle**,** n_c^+ -1 and n_c^- -1, respectively. Therefore, after each cycle an additional deformation is superimposed on the previous one and in this way an accumulation of plastic deformations is achieved.
- the initial deformation increases with the increasing of cycles.



(a)

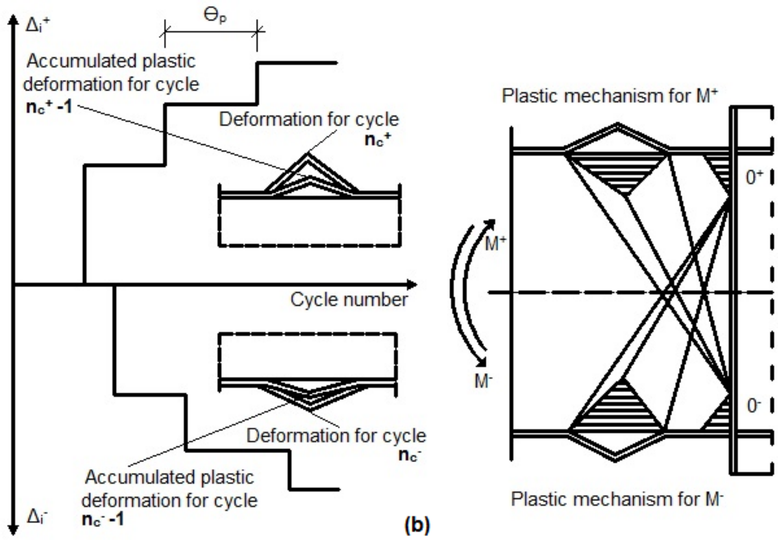


Fig. 3.28 Plastic local mechanism for cyclic loading

After the occurrence of the flange local buckling a deviation from the characteristic monotonic behaviour is generally observed; some yielding lines and plastic zones became ineffective, thus reducing the ultimate available rotation capacity. There are no differences between the monotonic and the cyclic loading until the buckling of compressed flange. This observation allows for an extension of the stable part of the moment rotation curve to its unstable part. The difference in behaviour begins to be significant only after plastic buckling occurs in cycle n_b. It is obvious that the level of degradation is directly associated with the number of the cycles. Thus, the accumulation of the residual deformations, in flange and the web, deteriorates the load carrying capacity and reduces the rotation capacity of a steel element. Hence, the proposed model for cyclic loadings is based on the concept of accumulated initial deformations, described by the shape of the local plastic mechanism (Fig. 3.29), in the same manner as the initial geometrical imperfections act in the elastic field for stability problems.

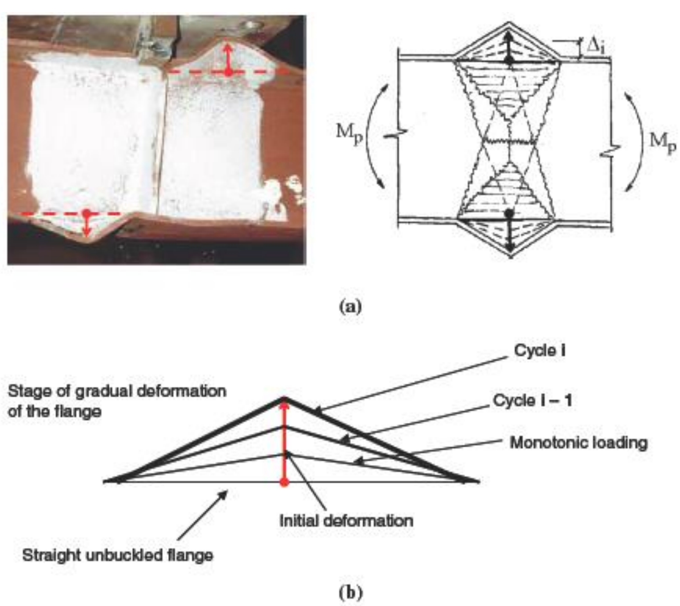


Fig. 3.29 Accumulation of plastic deformation under cyclic loading

The DUCTROT-M computer program **[3.10]** is developed with the aim of predicting the inelastic rotation capacity of the steel members (Fig. 3.30), validated by the verification with experimental and numerical results **[3.18]**, **[3.6]** . Beyond the prediction of the rotation capacity under monotonic load, DUCTROT-M software contains a series of functions for the calculation of the rotation capacity under cyclic load.

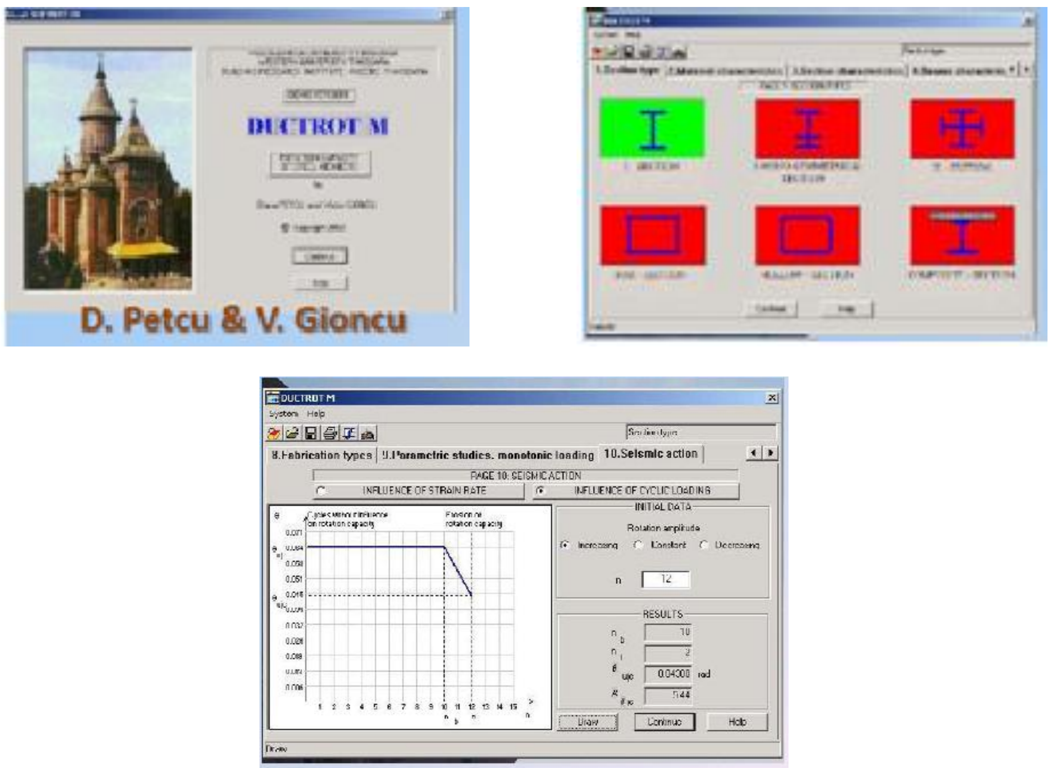


Fig. 3.30 DuctRot-M. Prediction of the local ductility under cyclic loading

3.6.2.2 Influencing factors affecting the available cyclic ductility

With the aid of the DUCTROT-M computer program an analysis was performed which focused on the main influence parameters which affect the cyclic available rotation capacity **[3.26]**. In this study European I-sections, IPE and HEA widely applied as beams, were used. The choice to apply mainly mild steel in the parametrical analysis is based on the fact that, generally, in the capacity design the beams are made of low yield steel (S235) and the columns of higher yield strength (S355, S460) in order to obtain a strong column-weak beam mechanism for ductile frames.

(i) *Loading type.* The effect of cyclic loading conditions, with constant or increased amplitude, on the rotation capacity is presented in figure 3.31. For the case of increasing amplitude, taking into account the number of the cycles that produce a gradual deterioration, it is pointed out that after several cycles the rotation capacity was exhausted, as compared with the monotonic one. In addition, for constant amplitude, one can observe a dramatic erosion of the available rotation capacity that approach more than 50% of the monotonic one. Consequently, due to great uncertainties with regard to the ground motion the available ductility could be determined as an envelope which is defined: a) by an upper bound given by the increasing loading amplitude b) by a lower bound described by the constant loading amplitude.

Looking from a different point of view and assuming that the increasing loading type could be directly related to the soft soil conditions, whereas the constant one with the stiff soil conditions, it could be remarked that the same member has a different available inelastic rotation capacity, due to a different process of the accumulation of plastic deformations. This was revealed by real earthquake events **[3.19]**. This concept can introduce the effect of local soil conditions on the member available ductility and further the effect of local soil conditions on the member available ductility is achieved. Therefore, function to the earthquake type (for instance far-source seismic action) and to the local soil conditions, not only the required ductility, but also the available one is influenced.

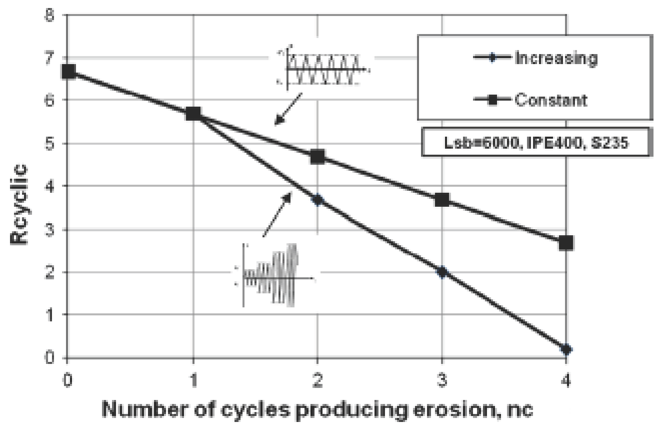


Fig. 3.31 Influence of the loading type on the cyclic ductility

(ii) *Cross section conformation*. The influence of the section slenderness, as a function of the cycle producing erosion, after flange buckling, is plotted in figure 3.32. By comparing two different types of hot-rolled sections, with approximately the same flange slenderness ratios (i.e. for IPE400, $c/t_f = 5.11$, for HEA450, $c/t_f = 5.85$) and different web slenderness (i.e. for IPE, $d/t_w = 38.48$, for HEA450, $d/t_w = 29.91$), as well as with different load carrying capacity, one can observe that HEA450 keeps the cyclic rotation capacity constant; which is the same with monotonic one, while for IPE a gradual degradation of the cyclic rotation capacity is finally remarked. With the help of the DUCTROT-M computer program an extensive analysis for the whole range of HEA and IPE was carried out and it led to the same conclusion. HEA sections have a superior behaviour, due to greater web thickness and to lower web height, and greater intersection zone, between flange and web.

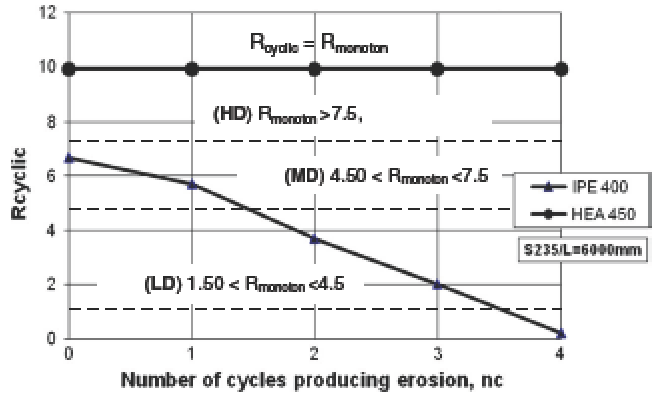


Fig. 3.32 Influence of the cross-section conformation on the cyclic ductility

In order to demonstrate the effect of the two aforementioned critical parameters a sensitivity analysis was carried out. By using an IPE section it was varied the web thickness and height, while keeping the flange width and thickness constant. From figures 3.33a, b, comparing a hot rolled IPE 400 with a modified one, it can be observed that the increase of the web thickness leads to the increase of the rotation capacity, while the reducing of web height also leads to the increasing of the rotation capacity. However, the effect of the web thickness is more pronounced than that of the web height; thicker web plates increase the number of cycles until buckling, n_b.

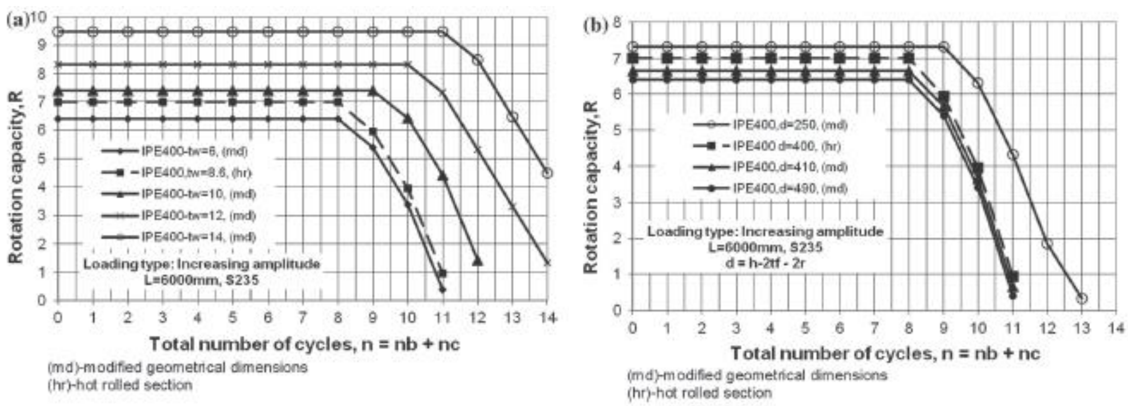


Fig. 3.33 Influence of the web slenderness (height vs. thickness) on the cyclic ductility

A certain web thickness provides the possibility for a stable deformation of the flange supported by the web. Therefore, for design purposes, the following recommendations are proposed:

- The use of HEA sections for the conformation of the moment resisting frames; for instance in the case of frames constructed on soft soils.
- A structural detail improving the inelastic capacity of a plastic hinge as is presented in figure 3.34. According to the column tree concept, the stub could be shop welded from HEA section and the remaining part of the beam from IPE. In this manner, in a conventional way, both economical and behavioural requirements could be achieved. Alternatively, the HEA section could be fabricated as a reduced beam section. This is more advantageous than IPE section, due to a better adjustment of the flange reduction and to an easier fabrication. However, using a welded I-section, with properly selected slenderness ratio, it is possible to obtain the desired available ductility which will be greater than the required one.

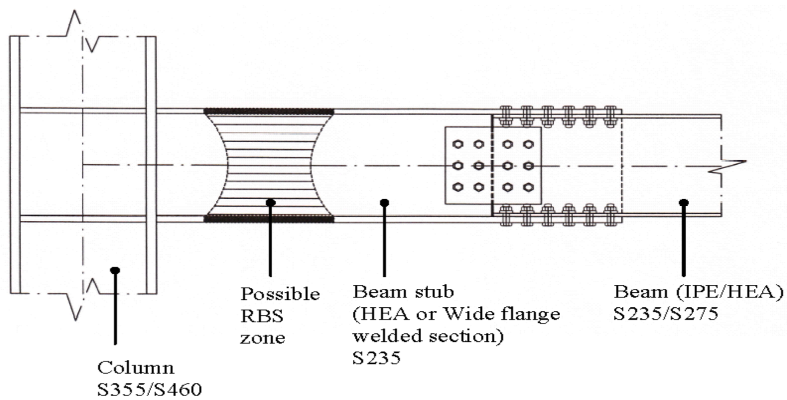


Fig. 3.34 Structural detail enhancing the local cyclic ductility

(iii) *Yield to Ultimate stress ratio.* The investigation of the influence of yielding ratio, $\rho_y = f_y/f_u$, on the cyclic available ductility is directly related to the number of cycles, n_c, producing erosion of the local available ductility. Figures 3.35a,b plotted for IPE sections, show the accumulation effect, introduced by a certain number of cycles, comparatively with the variation of the yielding ratio. However, one can observe the following three main issues: firstly, the effect of the cyclic cumulative action has a more pronounced character than the increasing of the yield to ultimate stress ratio, secondly, the effect of the aforementioned ratio becomes noticeable for values greater than 0.80, where the reduction is approximately 20%-30% as compared to the one of a ratio equal to 0.65 and as a function of loading type (increasing vs. constant amplitude). Finally, the reduction is more severe in case of increasing amplitude than in case of constant one, but in any case the dramatic reduction is due to the number of cycles producing ductility degradation. For HEA sections and for low values of ρ_y equal to 0.65 the effect of the cyclic action disappears, however with the increasing of ρ_y in the range 0.75-0.95 the rotation capacity is reduced, due to the predominately effect of the cyclic deterioration (Fig 3.36a,b). The divergence in the behaviour of HEA and IPE sections is because of the different sectional conformation (e.g. web and flange width-to-thickness ratio). Current Eurocode 3, with regard to the yielding ratio, prescribes a limit value of 0.90 (in the text of the code the ratio presented inverse as f_u/f_y >1.1). The previous mentioned limit seems to be proper for static conditions, while for cyclic and especially for strain-rate conditions is unsuitable. A proposal, for the cyclic far-field actions, to limit the yielding ratio to a value of 0.85 (or in the code format a value of 1.20) is considered to be convenient, taking into account the capabilities of the producers. Moreover, Eurocode 8 specifies only measures regarding the assurance of the yielding strength, f_y (clauses 6.2 and 6.11). In this direction, in order to fill the gap for material control it is necessary the introduction of such a ratio, $\rho_y = f_y/f_u$, thus creating more stable conditions for the development of the capacity design. However, a level of conservatism should be considered, due to a great number of factors affecting the local ductility (e.g. detailing, loading, workmanship, etc), as well as to the difficulty and the differentiation in experimental conditions to capture the critical deformations. After an extensive parametrical study, it is observed that the above mentioned conclusions are valid for the whole range of IPE and HEA sections. It is a very important observation when one tries to associate the monotonic and the cyclic local ductility with the ductility classification. Hence, for design conditions within the range of the material variability between ρ_y = 0.65-0.80, the influence of the yielding ratio does not affect the cyclic available rotation capacity; the number of the cycles being of primary importance. For values larger than 0.85, it is necessary for the designer to proceed in the prediction of the cyclic rotation capacity further, by reducing the rotation due to the material variability.

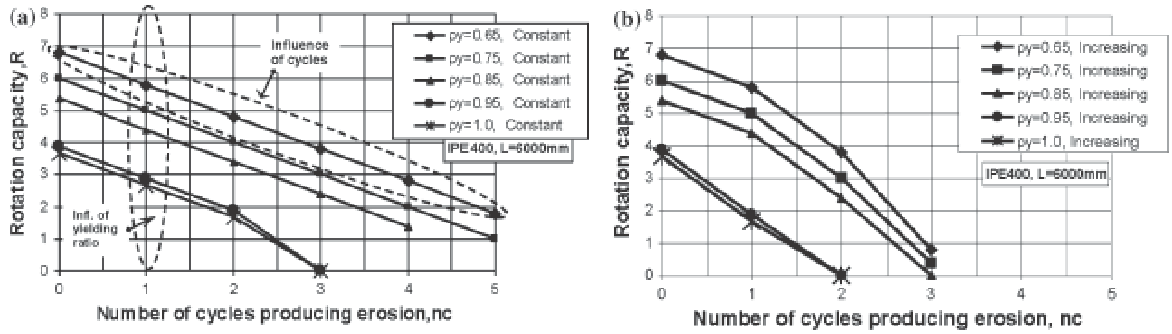


Fig. 3.35 Influence of the yielding ratio on the cyclic rotation capacity

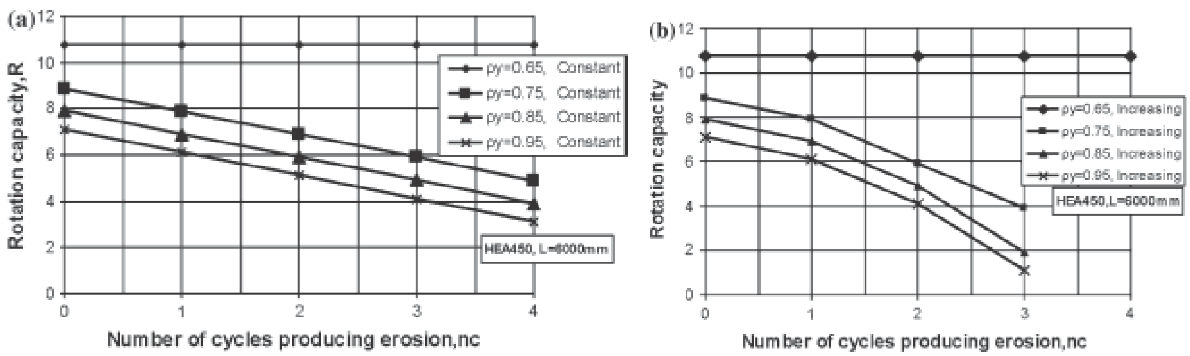


Fig. 3.36 Influence of the yielding ratio on the rotation capacity of HEA beams

(iv) *Yield strength limit.* Dealing with the influence of the yield strength, at a first glance, it appears to be decisive regarding the rotation capacity under cyclic action, figure 3.37. Nevertheless, the severe ductility reduction is associated with the number of the cycles as compared to the influence of the increasing yielding strength, especially for IPE sections. Moreover, the same is true for the HEA sections made by S355 steel quality. For lower qualities of HEA sections the effect of the cycles seems to be ineffective. In fact, the stable behaviour of this type of cross-section derives from the cross section conformation (Fig. 3.31, 3.32). As a conclusion, it could be pointed out that the influence of the material variability and the steel quality is covered by the effective action of a certain number of cycles.

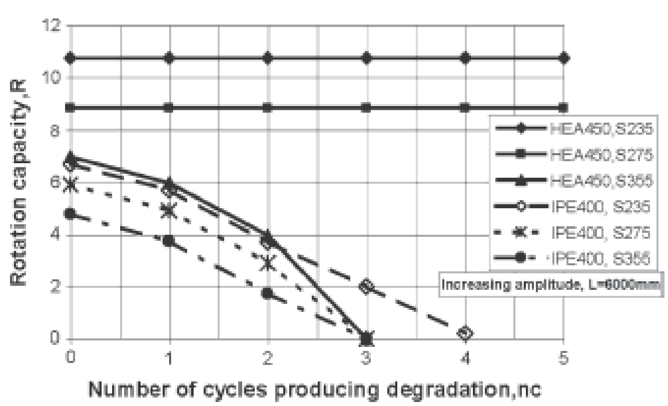


Fig. 3.37 Influence of the steel quality on the cyclic rotation capacity

(v) *Strength degradation.* In order to study the strength degradation of hot rolled profiles, the loading history proposed from ECCS was applied in combination with the concept of the cumulated initial deformation presented in this paper. Figure 3.38 shows comparatively the gradual strength degradation of I shaped beams conformed by IPE and HEA sections. It is clear that HEA sections have a better strength capacity than IPE sections with about 30%-35% as revealed by a further parametrical analysis. Furthermore, the experimental findings also validated the concept of the initial cumulated deformation. Therefore, taking into account all the aforementioned influences that affect the cyclic available ductility, it appears to be safe the application of HEA sections for the conformation of high ductile moment resisting frames.

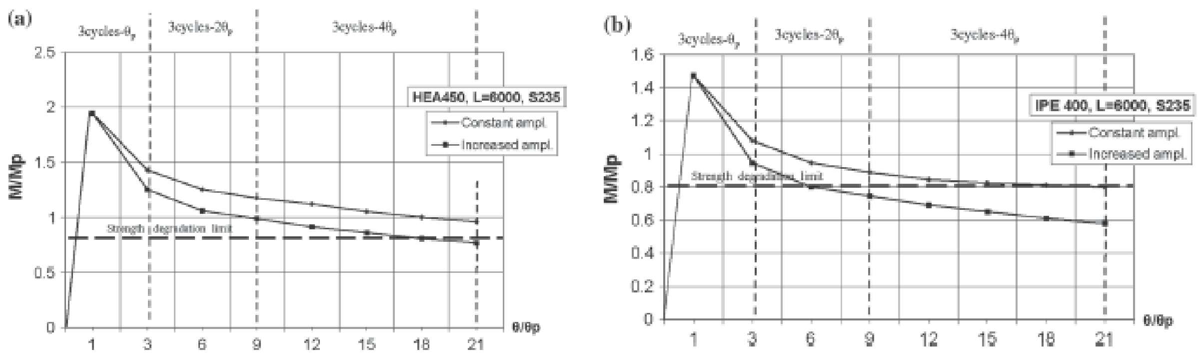


Fig. 3.38 Moment-rotation curve according to ECCS loading protocol

3.6.3 Local available ductility under strain-rate conditions

The inelastic behaviour of structures strongly depends on the type of earthquake excitation. Moreover the ductility, both the local and global one, as well as the associated strength is depending on the loading history and the rate of loading **[3.19]**, **[3.20]**. The engineering community, starting from the San Fernando earthquake, USA 1971, and further after the Northridge, 1994 USA, and Kobe, 1995, Japan, earthquakes, well recognized and classified the differences between the far source and near source seismic excitations **[3.1]**, [**3.21]**. Generally, after a review of research papers in the field of geotechnical and structural engineering, it was demonstrated that the far source earthquakes were related to a cyclic action and low rate of loading, while in case of near source earthquakes the load rating is high, developing brittle failures to the base material. Therefore, a series of studies was focused in the investigation of the inelastic behaviour of the steel elements under strain-rate loading conditions. However, the best way to tackle this topic is to relate engineering seismology with the earthquake design of structures.

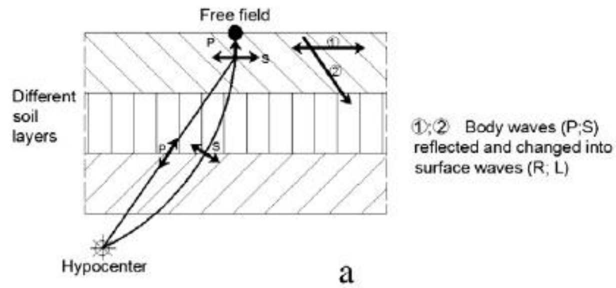
Conversely to far-field earthquakes, where the ground motions are mainly produced by the surface waves R and L, the near-field earthquakes are directly influenced by the body waves P and S; the surface one producing only secondary effects. As a matter of fact, all the characteristics are different. The main difference is focused on the transmission of the seismic forces to the structure. For far-field earthquakes the transmission of the waves is made by inducing vibration in structure, due to horizontal seismic actions, while for the case of near-field earthquakes the seismic wave propagation is made by the vertical seismic actions. In the case of a structure situated over the source, due to the last ball effect (Newton's cradle), the seismic action is dominated by direct P and S seismic waves which are propagated with very high velocity along the structure. Because of this action with a long pulse, the effect of the strain rate is of paramount importance as it causes brittle fractures on members and connections. For this reason, besides the fulfilling of the ductility requirements, the strength capacity it is, also, of equal importance in order to avoid sudden failures.

Further on, approaching the first issue associated with the seismological and geotechnical conditions, a very important question arises for the design of the structures to the action of P and S body waves, namely: could the ground motions recorded on free field be used or not?

During an earthquake the rupture of a fault initiates P and S seismic waves, with the tendency to assume a vertical direction towards the Earth surface, as far as the surface layers are concerned, (Fig. 3.39). Then, P-wave motions are thought to be dominant on the vertical direction, while the S-wave motions on the horizontal one. P-waves always are characterized by: higher frequencies, more rapidly attenuation with the horizontal distance, as compared with S-waves. Consequently, the near-field ground motions are described by important vertical components and high frequency energy.

The wave propagation phenomenon gives rise to an important aspect, which is not relevant for far-field recorded ground motions. When the surface is free (Fig. 3.39a), the recorded ground motions represent only a part of site movement, because when the waves reach the surface they are reflected back into the Earth, or at the Earth's surface, changing into R and L surface waves. However, if there is a multi-story structure on the surface, the waves continue to propagate into the structure until the top of it (Fig. 3.39b). Examining the amplification of the waves, two cases should be considered, a surface layer for a free site and a structure on the surface. Both of them could represent the last object in the direction of wave propagation.

Therefore, it is possible to draw a parallel with the very well known of Newton's cradle, figure 3.40; it is a device that demonstrates conservation of kinetic energy via a series of swinging balls. If one lateral ball is pulled away and it is left to fall, it strikes the first ball in the series and comes to nearly dead stop. The impact produces a shock wave that propagates through the intermediate balls. The ball on the opposite side acquires most of the velocity and almost instantly swings in an arc almost as high as the released height of the first ball. This shows that the last ball receives most of the energy that was in the first ball **[3.25]**.



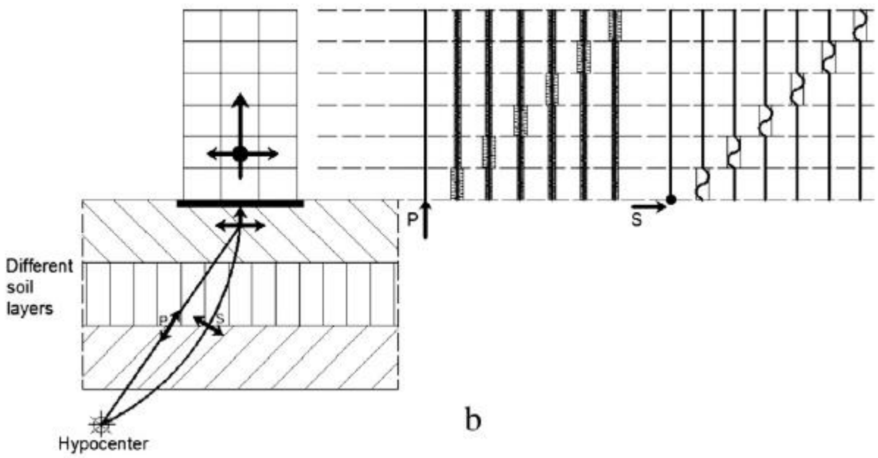


Fig. 3.39 Ground motion types: a) free site, b) site with structures

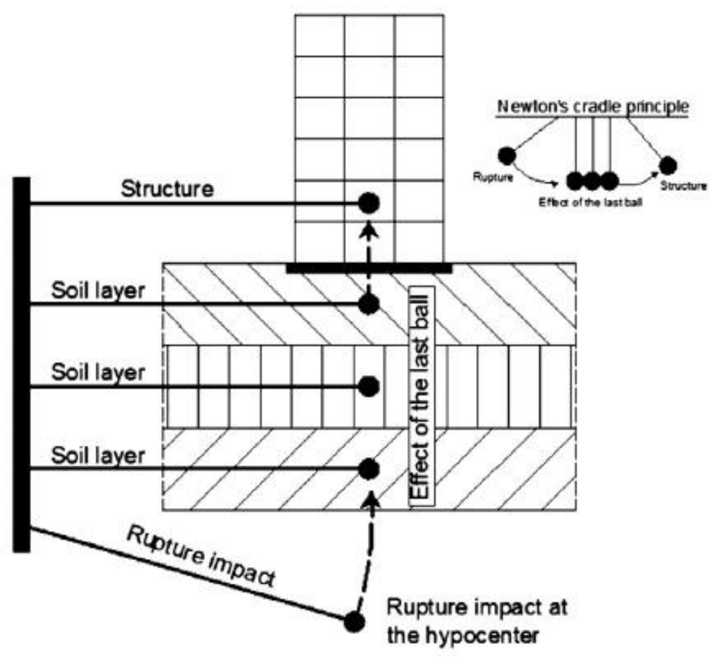


Fig. 3.40 Soil-Structure interaction and the analogy with Newton's cradle

During the Kobe earthquake an evolution of the peak acceleration function to depth was recorded (Fig. 3.41). For the horizontal component it was observed a de-amplification. At a certain depth an increased value of 0.6g was recorded, while at the surface a decreased one of only 0.3g was registered. On the contrary, for the vertical component a very important amplification was remarked. The surface value of 0.55g was strongly reduced in comparison with the bottom level where a value of 0.2g was recorded. The variation of the measurements is showing that this shock amplification is because of passing through the superficial soil deposits, which is the last layer. Although, this amplification disappears after short time, it has pulse characteristics.

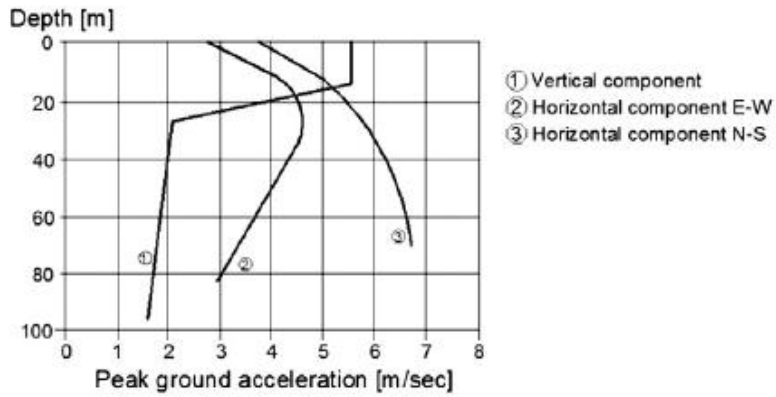


Fig. 3.41 Measured acceleration on free site **[3.22]**, **[3.23]**

The same process occurs during the propagation of the seismic waves in a soil-structure system. The system is composed of a series of different layers, playing the role of balls; the first one being the source and the last one the structure. Due to the sudden rupture, the wave of the first layer strikes the second layer, which in turns sends its energy to the series of layers, where the last layer (the structure) receives the total kinetic energy of the seismic wave. Therefore, a structure situated in a near-field site captures an important part of kinetic energy developed by the source rupture process; its behaviour is very different from the one of the structures situated in far-field sites. P and S waves arrive at the structure at some time interval. Unfortunately, the only wave velocities recorded, related in literature, corresponds to P waves. So, there is not sufficient information about the recorded velocities of S propagation waves. It is rational if we make an analogy, between P and S velocities transmitted in soil, to consider that the S velocities are more reduced than P velocity, by a relation of $V_s \sim 0.6\ V_p$. The main characteristics of P and S waves correspond to the pulse actions.
In conclusion, there are great differences in the seismic waves measured at the soil surface. If the site is free and the wave has to accommodate to this situation, the last ball in the Newton's cradle is the last layer, and accordingly, an amplification of the accelerations is observed. If the site is occupied by some structures, they are the last object in the Newton's cradle and a great part of the seismic energy is concentrated on them. Thus, the Newton's cradle states that the last storey of a structure is where the seismic waves propagate with great velocities. This is the reason why, in many cases, the damage is concentrated at the upper levels of a structure. It is well known that the maximum recorded velocities on free sites during the near-field earthquakes do not exceed 4 m/sec (e.g. during Taiwan earthquake). For sites with structures, the situation is totally different; the wave propagation time, measured during some recent earthquakes, and the resulting velocities for five instrumented buildings, located in California, pointing out that the velocities for all the buildings exceed 100-200 m/sec, many times the values obtained on the free field, even if there was a different material, structure, and a different general conformation of each building. For this reason, the recorded free-field velocities do not explain the unexpected connection fractures. Therefore, free-field ground motions could not be applied for the design of

the structures situated in near-field areas. At the same time, the methodologies used for the design of the structures should be replaced by approaches considering the seismic wave propagation along the structure.

Approaching the second issues associated with the loading velocity, the influence of the strain rate on fracture rotation was studied by using the DUCTROT-M which contains the relationships presented in the paper Gioncu V., Mosoarca M., Anastasiadis A., 2014, consulting this thesis. It is of paramount importance to parametrically investigate the correlation between the yield ratio ρ_y and the strain rate $\dot{\epsilon}$. The diagrams from figure 3.25 were performed for the rolled and for the welded profiles, in conditions of room and low temperature. The rotation capacity and the fracture rotation are examined function to the strain rate, for far-field and near-field earthquakes. Examining these diagrams, the following observations resulted:

- important reduction of rotation capacities occurs due to strain rate, for far and near-field earthquakes. Experimental tests and numerical analysis performed by El Hassouni et al, 2011 **[3.24]**, also pointed out such reductions;
- in both cases, far and near-field earthquakes, the influence of the temperature with respect to ductile fracture is relevant, while for the ductile rotation is not very significant;
- for hot-rolled sections (Fig. 3.42a), in conditions of room temperature (protected structure), available capacity results from the ultimate plastic rotation, even for far and near-field earthquakes. On the contrary, for the welded section (Fig. 3.42b) the ultimate available capacity is given by the fracture rotation;
- for both fabrication types (hot-rolled, welded) and in the case of low temperature the available capacity is given by the fracture rotation. Furthermore, the welded sections are very sensible to fracture; therefore they must be used taking the necessary precautions to avoid brittle failures.
- in any case, the ductile fracture has a steeper variation than the plastic ductility, thus revealing the desirable effect of a controlled local flange buckling for both far and near field earthquakes.

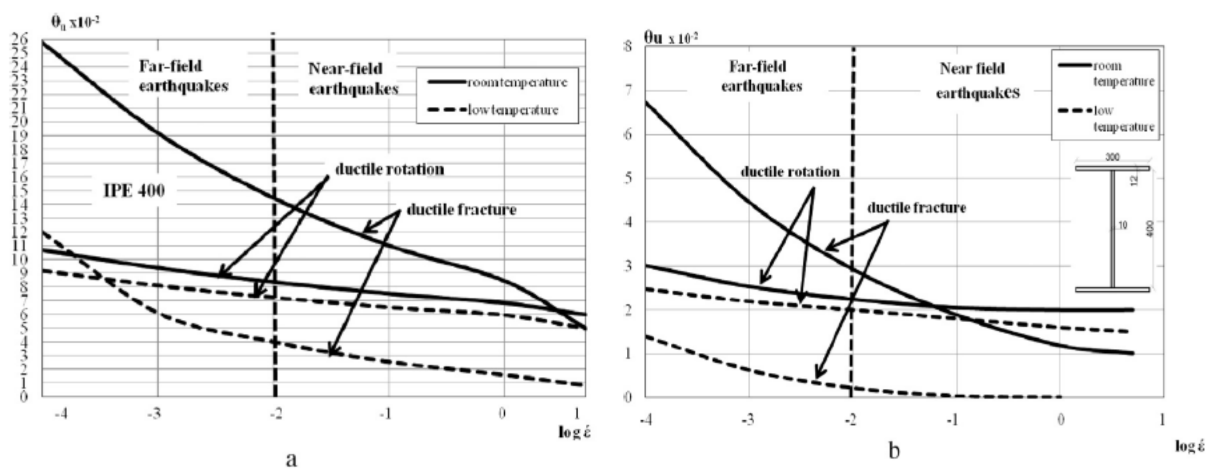
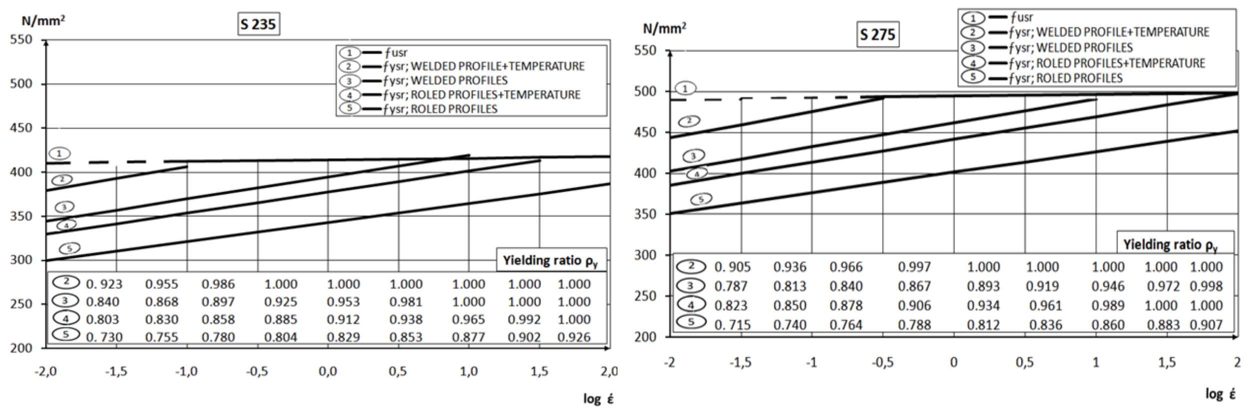


Fig. 3.42 Influence of strain rate on the ultimate rotation capacity

Moreover, it is well recognized that the increase of the yield ratio due to a high strain-rate has a detrimental effect on the available ductility. It significantly reduces the capacity of the seismic energy dissipation, especially in the strain-rate of 10^{-1} to 10^{1} sec^{-1} where there is the field of the velocities of near-field earthquakes. Taking into account the steel quality (S235 and S275, mainly used in beams due to capacity design reasons), the conformation type (rolled and welded sections), the temperature conditions (in room conditions and at low temperature), and the strain rate level figure 3.43 plots the influence of the loading rate on the yield strength. The main conclusions that can be drawn from these figures are the following: (i) Higher steel quality (S275) has a more favourable behaviour in condition of high strain-rate than the steel with lower quality (S235); (ii) Rolled sections have a better behaviour than the welded ones, (iii) in the case of welded profiles, the poorest behaviour during strong earthquakes in the near-field area is reached in low temperature conditions.



$$\rho_{ysr} = \{\varphi_{ysr}/\varphi_{usr}\}\rho_y = \{c_T c_W(1.46 + 0.925\ log\dot{\varepsilon})/(1.15 + 0.00496\ log\dot{\varepsilon})\}\rho_y$$

Fig. 3.43 The effects of the strain-rate on steel qualities a) S235, b) S275

The influence of the loading rate on the rotation capacity is plotted in figure 3.44. One can observe that the effect of the strain-rate strongly reduces the rotation capacity and further on this reduction affecting in a more pronounced way the IPE sections.

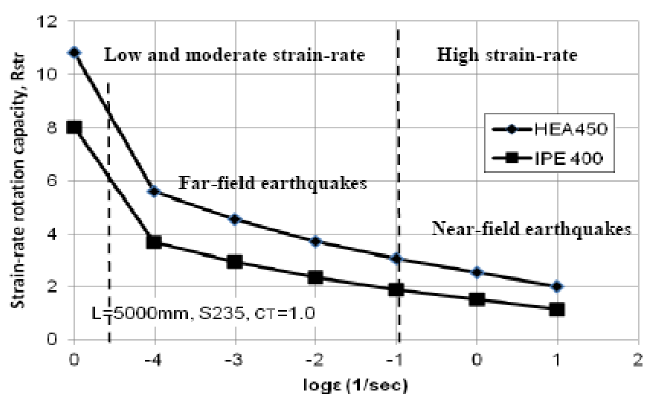


Fig. 3.44 The effects of the strain-rate on the rotation capacity

Attempting to compare the different ductility types, it is obvious that the static-monotonic ductility should be considered only as a reference value (Fig. 3.45). For design purposes, the reference value could be properly reduced introducing the main influencing parameters as previously discussed. Therefore, using as the basis the monotonic rotation capacity, R_{mon}, the available rotation capacity under strain-rate, R_{str}, and cyclic conditions, R_{cyclic}, is possible to be obtained. Under exceptional seismic actions steel I sections are very difficult to achieve a high ductility level. Therefore, the plastic hinge should be moved away from the joint high stressed region by using weakening (reduced beam section) or strengthening solutions (ribs, haunch, cover plates, etc). Despite the difficulties to select the most suitable loading type and the complexness of the seismic phenomena a step forward in practical design should be the direct implementation of a comprehensive ductility methodology. Only in this way the results of inelastic analysis, like pushover and time-history, could be considered stable and reliable, because there is an interrelation between the available rotation capacity and the target displacement. Each member of a structure should dispose sufficient local ductility otherwise the structure's target displacement is not achieved. Therefore, the prediction of the available ductility is of crucial importance.

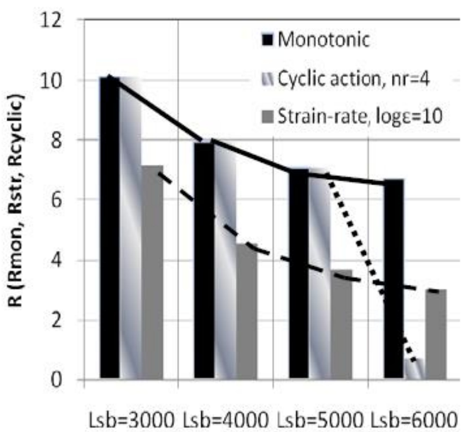


Fig. 3.45 The effects of the strain-rate on the rotation capacity

Research in the far and near field were started under the coordination of Prof. Gioncu and were continued and published by the authors in prestigious journals, as a sign of respect for the prestigious activity of nearly 40 years in the research domain of historic structures.

3.6.4 Final remarks

Concluding this series of studies focused on the inelastic behaviour of steel structures we realize that there is a need for a multi-level earthquake design which is a multidisciplinary approach due to great variety of factors affecting the final response of a structure under the probable seismic action. In figure 3.46 the inter-disciplinary factors for an integrated design is presented. One can observe that in this process the core is the clients' need and expectations. Furthermore, safety levels, social requirements as laws and customs, market values of the buildings, maintenance costs, repair costs after earthquakes, insurance premiums can be well prescribed in order to take the real value of the multi-level earthquake design concept. A research in Japan standing on questionnaires demonstrates that the owners want to suppress hazard severity to a small level, being more interested about "hazard of human life" and "loss of property" than for "function of building". Such researches are useful driving the social policy and the design performance objectives. Once we have resolved the social system framework it is necessary to evaluate the design earthquake. An important factor represents the reliable consideration of probable sources of potential seismic hazard, their selection and representation, in correlation with local site conditions. The contribution of engineering seismology and geotechnical engineering is valuable through macro and micro zonation studies, evaluating the seismic behaviour at the construction site. Unfortunately, there is a lack of such studies. Having all the aforementioned information conceptual conformation design, preliminary design and final design and detailing should be performed. As a function of considered performance targets a more or less sophisticated analysis and design must be made. So, it is easy to understand that the process of multi-level earthquake designs is an iterative one with so many uncertainties, cumbersome and time-consumable for current structural design offices. A solution should be the development of a new multi-level engineering framework composed by laws, open structural codes, insurance and banking support, proper fees for engineering services. Until to obtain such a system in the current design practice, it is important to introduce the spirit of multi-level structural performance exploiting the existing codes and methodologies.

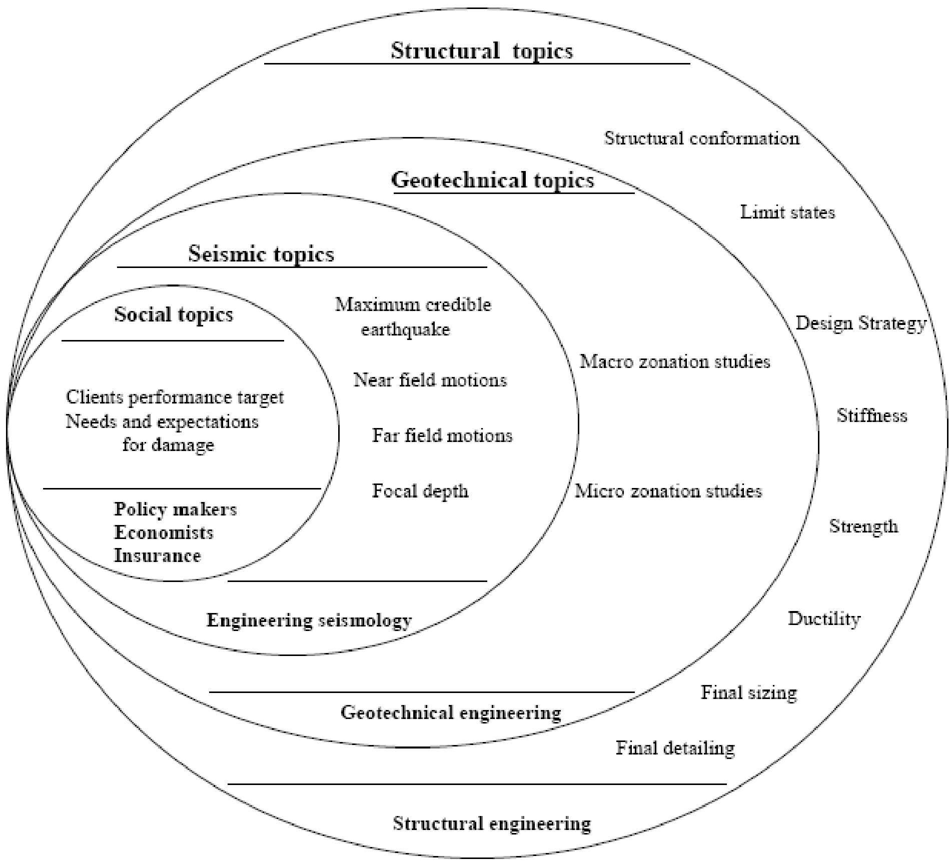


Fig. 3.46 The multidisciplinary approach of the earthquake design of steel structures

4 HISTORIC REINFORCED CONCRETE BEARING STRUCTURES

4.1 Structural safety of reinforced concrete historic buildings

4.1.1 Structural safety of historical buildings made of RC, from Banat region – Romania

In the last 150 years reinforced concrete was recognized worldwide for its high strength and its ease to take any shape. It is due to these properties that it is used on large scale for special civil buildings as high-rise or modern free shaped buildings. Although reinforced concrete is a rather new material in comparison with wood and bricks, it has begun for more and more reinforced concrete buildings to be declared monuments. The paradox consists in the fact that in current practice, the use of reinforced concrete in monuments consolidations is slightly used because it doesn't respect the intervention reversibility principle. Research studies have been prepared by author in the field of risk and vulnerability of historic buildings.
At the end of the nineteenth and early twentieth century, in western part of Romania important buildings of reinforced concrete were built such as: water towers, bridges, industrial buildings. There are some buildings that used only reinforced concrete elements such as slabs, beams, walls and framing. Currently, these elements have low bearing capacity, putting at risk the security of buildings and their historical value. The main reasons are: low grade concrete, reinforcements without ductility that are highly damaged, low percentages of reinforcement. Different types of reinforcements do not provide the necessary ductility for buildings located in the seismic zone Banat, Romania.
Studies performed by the author are original and present some representative types of reinforced concrete structures from Banat region which are declared historic monuments **[4.1]** – **[4.5]**, **[2.51]** the state of degradation of these constructions and different ways to strengthen these buildings **[4.6]**.

4.1.1.1 Structural damages and consolidation solutions for reinforced concrete monuments from the Banat region, Romania

With the channelling of the Bega river, the history of reinforced concrete buildings begun. In 1793, the Dutch engineer Fermaut continues the channelling implementing dams for the regulation of the Begej watercourse. Between 1900 and 1916 on the 144 km Bega channel there was built a system of locks. Reinforced concrete was then used for the first time. In figure 4.1 it is presented a photo with the lock from Sanmihaiul Roman in the Timis County. In total, six locks were built, two of which are on Romanian territory and four on Serbian terrain. Figure 4.2 presents the original project of the lock. Today, all hydro technical nodes are considered monuments and the reinforced concrete buildings were not consolidated. Due to poor maintenance, the concrete and reinforcement are much degraded.

Fig. 4.1 Sanmihai lock

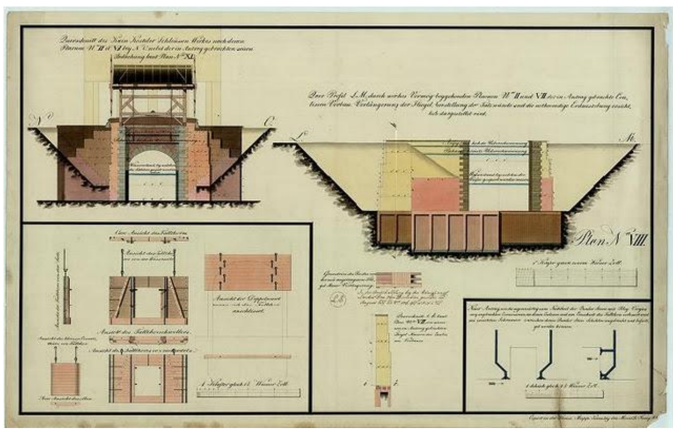


Fig. 4.2 Original project of Sanmihai lock

As a consequence of the construction of the Bega waterway, in Timisoara were built a series of reinforced concrete and steel bridges. The most famous bridge is the Decebal Bridge which was built in 1909 and it was, at that time, the bridge on reinforced concrete beams with the largest opening in Europe. It has a total length of 58m with a span of 39m and a width of 9.0m (Figure 4.3). The project obtained its honour diploma, at the International Exhibition, Paris 1910. Today, the bridge is declared a monument and it is fully functional. The structural frame didn't need major consolidations until today, but experimental tests performed on the reinforced concrete beams of similar bridges built in Timisoara have indicated the fact that the reinforcement is severely corroded and as a consequence the reinforcement area is no longer insured.

Fig. 4.3 Decebal Bridge in Timisoara

For the water supply of Timisoara, there were built two water towers, (Fig. 4.4, 4.5) between 1912 and 1914. The structural frame was designed at Budapest, figure 4.5, according to the plans of Laszlo Szekely architect from Timisoara. Today, the towers are declared monuments, they are not functional and they are not private property. The structural frame is made of reinforced concrete walls with perimeter circular columns. The concrete was prepared and cast in situ. The walls have reinforcing rebars of maximum 12mm with grooves to insure a good bond with the concrete.

Fig. 4.4 Water tower in Timisoara

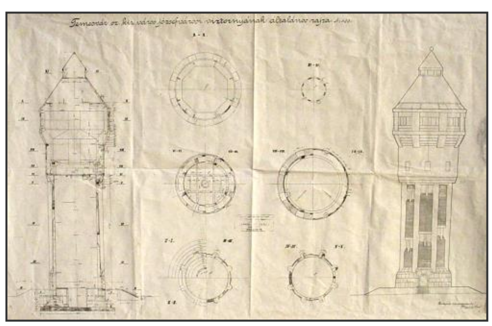


Fig. 4.5 Water tower project in Timisoara made in Budapest in 1913

The reservoir area is composed of reinforced concrete walls and it separated from the staircase through a reinforced concrete slab supported on beams. The stairs are made of precast concrete. The roof is made from reinforced concrete and covered with ceramic tiles. The water reservoirs are made of steel and have been disabled since the 70’s. Poor maintenance is the main cause of

the damages in the structural frame. Due to rainwater infiltrations through the roof and windows, the reinforcement has corroded and the concrete cracked as a consequence of the freezing and thawing process. On a height of 1.50m from the base, the walls are permanently moist due to the lack of exterior waterproofing and between the foundation and walls. There are areas where the concrete cover is missing and the reinforcement is much corroded. Major cracking occurred due to shrinkage of the concrete caused by the reduced area of reinforcement in the walls cross section. Damages were reported after earthquakes in the Banat seismic zone. Thus, diagonal cracks appeared in the walls between the windows at the reservoir level. No local crushing of the concrete occurred, nor buckling of the compressed reinforcement in the walls and columns.
Based on the recommendation from the Chart of Venice, regarding the conversion of industrial buildings for reuse, the author studied two reversible consolidation solutions for the structural frame. Because the largest efforts in the structure can be induced by seismic loads, the spatial response of the building was modelled before and after consolidation, in the elastic domain with the help of AxisVM software. In both solutions, the consolidation was made inside the building in order to affect as little as possible the exterior image which became a symbol of towers. The circular staircase was proposed for consolidation with steel profiles by introducing a central core made of steel profiles. The perimeter walls are consolidated with beams and steel columns on the inside perimeter of the building. Steel beams make the central core and perimeter walls work together as a whole. The perimeter columns secure the reinforced concrete columns, and the central core increases the rigidity of the building by reducing the maximum horizontal displacements. In order to increase the commercial area the elimination of the stairs was proposed and execution of seven composite slabs made from steel sheeting and reinforced concrete up to the reservoir area. The slabs can be easily removed because they are supported on the new steel structure. Where the concrete's resistance was significantly reduced as a consequence of water infiltrations, the concrete section will be locally treated with epoxy resin materials. The reservoir area has two solutions of consolidation based on the simulation of the seismic response and identified structural damages during the technical expertise. The elements with the highest vulnerability to seismic action are the cracked concrete walls between the windows of the tower as a result of the general torsion phenomena and the reinforced concrete cantilevers which support the tower as a result of the high shear forces and bending moments recorded in them. To reduce the vertical loads from the slabs which act on the reinforced cantilever beams, the introduction of a steel cantilever fixed in the central core were proposed at the level of each intermediary slab. On the edges of the cantilevers are fixed continuous steel columns over the entire area of the reservoir. These steel columns don't discharge the loads on the existent reinforced concrete elements. The main difference between the two consolidation solutions consist in the introduction of steel perimeter bracings between the steel columns in the zones with the window gaps in order to reduce the shear forces, figure 4.6. The seismic behaviour simulated on the computer, before and after the consolidation, in both solutions is presented in figure 4.7. It can be observed that by introducing steel bracings, the general torsion movement of the tower is cancelled in the second solution of consolidation. Without these bracings the danger of failure exists due to general torsion of the entire structure and through shearing of the reinforced concrete posts in the area of the reservoir. Also the steel braces reduce the relative displacements recorded between the base and the top of the reinforced concrete tower. As a conclusion, the consolidation solution preserves the exterior surface and it is reversible by increasing the bearing capacity of the historical reinforced concrete with the help of

steel profiles and epoxy resins and by introducing a central core which overtakes vertical loads and satisfies the rigidity, stability and ductility requirements imposed by design codes.

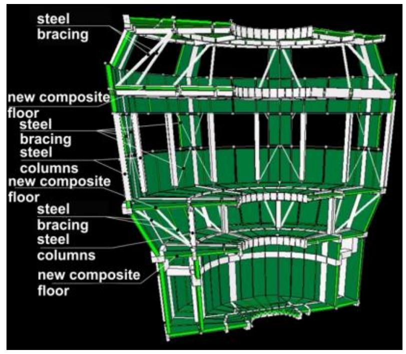


Fig. 4.6 Consolidation solution 2

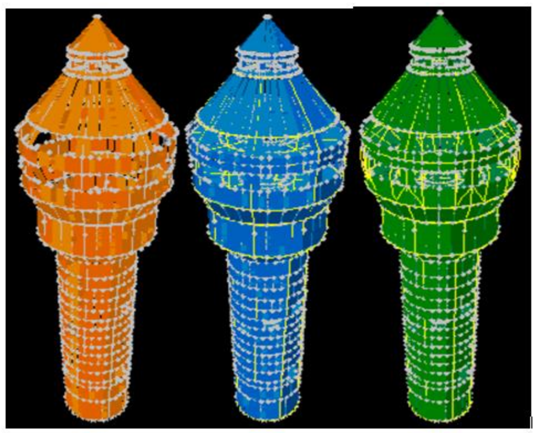


Fig. 4.7 Seismic Deformed shape of unconsolidated tower and solution 1 and 2 of consolidation

Another remarkable building studied by the author from the Banat seismic zone, which is a historical monument and which is composed of reinforced concrete load-bearing elements made in the early twentieth century is the church of St. Mary's Monastery in the city of Lipova. The church was rebuilt several times due to damages caused by war and fire. It was originally built of wood by Franciscan monks in 1520 but burned in 1695. The present church was built in Baroque style between 1756 and1782, as it can be seen in figure 4.8. Over the years has suffered several fires, but the largest in 1910 led to the complete burning of the wooden towers. To avoid the repetition of fires, in 1911 the towers were raised 30m and were made completely of concrete, figure 4.9. There have been made concrete columns, walls, beams and rafters. The roof remade with reinforced concrete elements which mimic the roof framing of the towers is spectacular and it respects the classical load discharge static scheme.

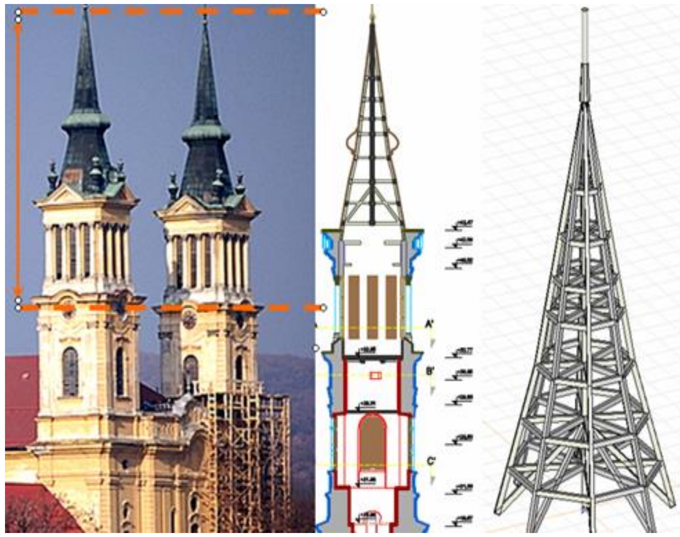


Fig. 4.8 Towers of Saint Mary's Monastery, Radna - Lipova

Fig. 4.9 Tower and roof structure photos

The historic study of the archive reveals the fact that the concrete was made by locals at the base of the Monastery and it was raised and casted in wooden formwork. Laboratory tests showed a low compressive strength of concrete. The reinforcing consisted of bars without yielding plateau with maximum diameters of 12mm in the columns and walls, and 10mm in the elements of the roof. Horizontal bars were made of the same type of steel and had a maximum diameter of 8 mm and do not insure the concrete confinement. In the present time, due to the degradation of the bars, the active section is maximum 50% of the initial section of the reinforcing rebars. Because of this the reinforced concrete sections do not meet the strength and ductility requirements. The casting procedure led to segregations of the concrete and allowed the reduction of its initial strength and corrosion of the reinforcement, figure 4.9. Load bearing of the structure is not provided due to the incorrect placement of the rebars in the nodes and to the very small overlapping of the rebars. Now, after 100 years from the construction of the towers, they do not meet the minimum strength, stiffness, ductility and stability requirements for such a monument. In order to find the most effective reversible solutions for the consolidation, the spatial response of the tower was studied by the author in the elastic domain with the help of AxisVM software. The deformed existing structure and the efforts are presented in figure 4.10. Consolidation solutions proposed by the author and H.I. STRUCT design office team from Timisoara increase the bearing capacity, rigidity, stability and ductility of the structure by means of steel bracing tubes inside the towers, fixed in the brick masonry. These provide a good support for the concrete towers and increase the bearing capacity, connecting the columns and the concrete walls to the steel tube. This reduces the buckling length of the columns and provides another discharge path for the vertical and horizontal loads. A reduction of the loads and efforts in the reinforced concrete elements will appear. The historic concrete will be consolidated by means of cleaning and treatment with epoxy resins. Confinement of the concrete at the base of the towers will be made with FRP materials. The roof elements will be consolidated with angle profiles fixed on each corner of the elements. These angle profiles are fixed between them by steel plates as stirrups which provide concrete confinement.

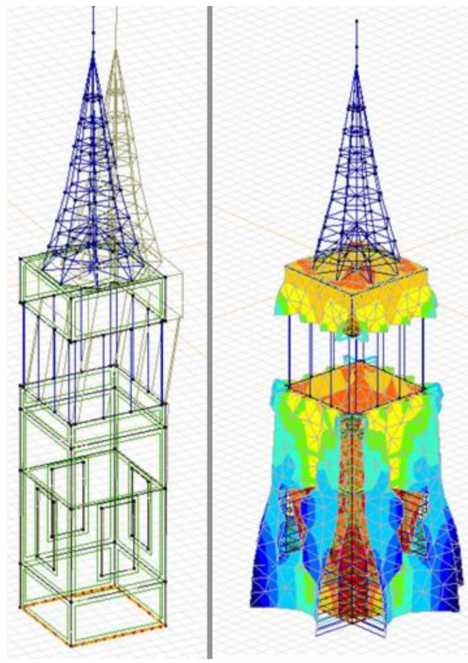


Fig. 4.10 The deformed shape and state of efforts in the tower

4.1.1.2 Conclusions

In the western part of Romania, as throughout Europe, there are a lot of historic buildings such as dams, water towers, bridges and churches which need reconditioning according the recommendations of the Chart of Venice. Special attention must be given to these buildings by the historic building community because some of them are older than 150 years **[4.6]**. It is necessary that design regulations contain special sections on strengthening of reinforced concrete monuments in order not to affect their historical value. The author performed a static and dynamic analysis of such type of historic buildings made of reinforced concrete and he has presented the state of efforts and the deformed shapes of these buildings, and also proposed some minimal consolidation measures which are reversible and can be further developed in the future. In conclusion, the author, by the original studies performed has underlined the fact that it is time that reinforced concrete should be given the same attention as a historic building material like wood, stone and masonry, and brought information which can open new international research directions on unique reinforced concrete structures. The construction techniques and materials used are particular for Romania and should be included in an international data base regarding these types of structures.

5 HISTORIC MASONRY BEARING STRUCTURES

5.1 Churches

5.1.1 Failure mechanism of orthodox churches situated in seismic areas in Romania

5.1.1.1 Introduction

The latest earthquakes in Romania showed that the resistance structure of historical Orthodox churches located in seismic areas, develop specific failure mechanisms. Currently, there are simplified methods for calculating the maximum seismic force, were the structure achieves the ultimate limit state, based on the theory of rigid block failure, only for Roman Catholic churches type. These methods are not developed for Orthodox churches because of the differences of vertical and horizontal shape and especially of the original way of transmitting the loads from the vaults trough the pendantives. Seismic forces produce longitudinal and transversal cracks in arches, vaults and walls and cause failure blocks which cancels the spatial character of the structure.

Within the PROHITECH research program **[5.11] [5.12]**, the author has studied specific failure mechanisms for several types of masonry at Orthodox churches situated in seismic areas in Romania, resulting in an analysis performed on Orthodox churches affected by earthquakes from 1977 in Vrancea and 1991 in Banat.

The seismic behaviour of historic buildings in this period in majority of cases is studied with computer software. They simulate very well the seismic response of the buildings in elastic lineal domain and in nonlinear domain. The numeric simulation of the masonry does not respect any hypothesis (isotropy, elastic behaviour, homogeneity) assumed for other materials. In these conditions, elastic models considering a homogenized continuum, can give an indication on the mechanical behaviour in the undamaged range and can only be used to detect the weak parts of the structure and the positions of to come cracks. For ultimate state, nonlinear models, using complex finite elements, based on plasticity theory and considering the joint and interface elements to model the planes of weaknesses, can be used only for simple masonry elements, being inadequate to model a full structure. A rapid and efficient method for simulation of the response of buildings with complex shapes in ultimate limit state was developed after examining the cracks after the earthquake. The conclusion is that often failures occur by formation of collapse mechanisms involving all the buildings or only some part of them. In this case computational models use some rigid body macro elements and the discontinuities are concentrated only along the borders of these elements **[5.1]**.

5.1.1.2 Calculation models of masonry buildings for seismic design

A current trend in seismic design is the incorporation of performance-based design methodology. In this methodology, every building is designed to have the desired levels of seismic performances corresponding to different specific earthquake ground motion. To achieve this goal, elastic analysis is insufficient, because this cannot realistically predict the forces and deformations during earthquakes. Inelastic analytical procedures become necessary to identify the mode of failure. Inelastic time-history analysis is the most realistic approach for evaluating the building performances. However, this inelastic analysis is too complex and time-consuming in the design of most buildings, especially if the spatial behaviour is considered. As a

compromise, a simplified procedure commonly accepted is the pushover analysis, where a sequence of inelastic static analysis is performed for a set of monotonically increasing lateral loads. For the historical masonry buildings, the pushover methodology is complicated by the definition of mechanical properties of the materials, definition of constitutive laws for decayed materials and structure rigidity degradation due to the cracks formation **[5.2]**, **[5.3]**. The behaviour of a masonry building is presented in figure 5.1. In the first stage, the building works as a compact element until the first fissures. In this field, the building's masonry can be characterized as an elastic medium with heterogeneous properties. The first fissures produce a reducing of building's rigidity, but the elastic behaviour is not modified very much. At superior level of load, the fissures are turned in a system of cracks, which began to affect very much the building behaviour. The increasing of load produces a local failure, where the first very important damage of building occurs as seen in figure 5.1. Behaviour of masonry arch for lateral load in the ultimate limit, a collapse mechanism is formed, which, finally, generates the building failure. In the frame of performance-based design philosophy, until the formation of crack system, the building works without important damage and the safe occupancy and operational usage can be considered. In this field the damage control is the main task of design. The field until the local damage is the precursory phase of the structure failure, while the formation of a global collapse mechanism represents the ultimate limit state. In many cases, between local failure and the limit states considered in EUROCODE 8, the masonry structure behaviour until the formation of crack system can be considered as the range of damage limitation stage, while the behaviour near to formation of collapse mechanism as the ultimate limit state.
It is well known that masonry structure analysis requires nonlinear modelling accounting for the low tensile capacity and the consequent cracking phenomena. It is well known though that such models cannot be used in the case of very complex structural systems characterized by large number of degree of freedom. In the same time, it is very clearly that the methodologies required for the two limit states differ very much. While for damage limitation state on can use the elastic analysis (with or without considering some cracking effects), for the ultimate limit state, the methodology must be very different. Therefore, analyzing the behaviour of historical buildings, it is need to be adopted a two step procedure.

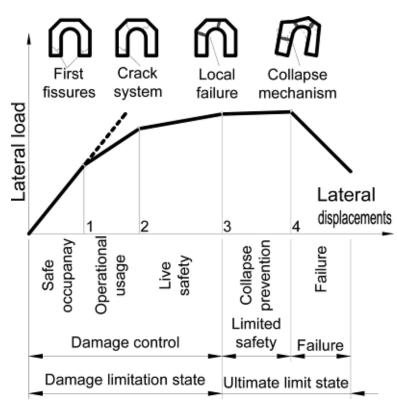


Fig. 5.1 Behaviour of a masonry triumphal arch **[5.5]**

Global behaviour analyses for damage limit state, in the linear elastic range, through a complete and refined FE Method-3D model. This analysis can give indications on the global behaviour in the undamaged range and can only detect the weak part of the structure and the position of future cracks. In the same time, it can be use to have indication about the efficiency of some strengthening methods **[5.4]**, **[5.5]**.

Global behaviour analyses in the ultimate limit state, using the Collapse Mechanisms Method, considering the structure composed by some rigid body macro-elements with discontinuities concentrates only along the borders of these elements, resulted due to seismic action. This methodology is based on the observation in situ or on the models concerning the cracks system of a damaged building, leading to a collapse mechanism **[5.6]**.

The use of theory of rigid blocks to determine the limit of historical building has the potential to become a powerful tool in engineering practice. In particular, this approach avoids the use of sophisticated and time-consuming nonlinear finite element technique. The applicability of this theory determined amplification factors. This methodology was successfully applied for determining the collapse mechanisms and ultimate limit state forces for buildings **[5.7]**, **[5.8]**, **[5.9]**, Romanesque churches presented in figure 5.2**, [5.10**]-**[5.14**] and for Orthodox Churches in Romania **[5.2]**, **[5.3]**, **[5.6]**, **[5.15]**, **[5.16]**. To masonry structures modelled as an assemblage of rigid blocks interacting through joints depends on some basic hypothesis, confirmed by in-site observations and experimental results: limit loads occur at small displacements, so the linear theory can be used; masonry has no tensile strength; compression and shear failure at the joints are perfectly plastic; hinging failure at joints does not consider the effects of local crushing. The seismic collapse load corresponding to the ultimate load is determined using a cinematic method.

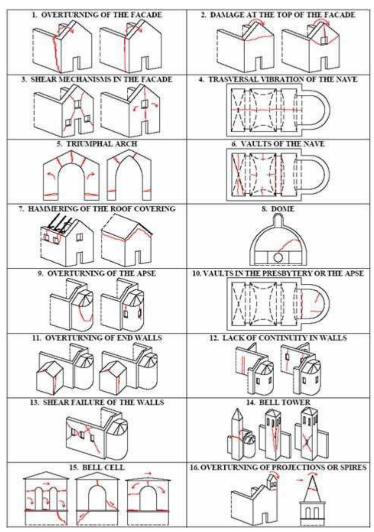


Fig. 5.2 Behaviour of a masonry Romanesque church. Masonry rigid block **[5.10]**

5.1.1.3 Seismic behaviour of Romanian orthodox churches

The buildings of Romanian Orthodox churches are based on the Byzantine style, being characterized by the using pendentives and dome on pendentives as in figure 5.3. It must be pointed that the main structural characteristic of Byzantine architecture is the use of pendentive domes, dome on pendentives, and tower on pendentives. This is unique way of adjusting the circular form of a dome or tower to a square plan. The pendentive dome is derived by trimmer the sides of a circular dome over a square plan as in figure 5.3, and it enables to transfer the total load of the roof to the four corners of the building. Additionally, the top dome enables to transfer the total load of the roof to the four corners of the building. The top of the pendentive can be trim to introduce another dome on top. The additional dome can further be raised to introduce a cylindrical tower between the pendentive dome and the additional dome. Windows can then be introduced in the cylindrical tower enabling architects to create interior light effects. The surrounding infilled and exterior masonry walls also contribute to carry out the loads, forming very rigid corner pendentives **[5.1]**, **[5.2]**, **[5.3]**.

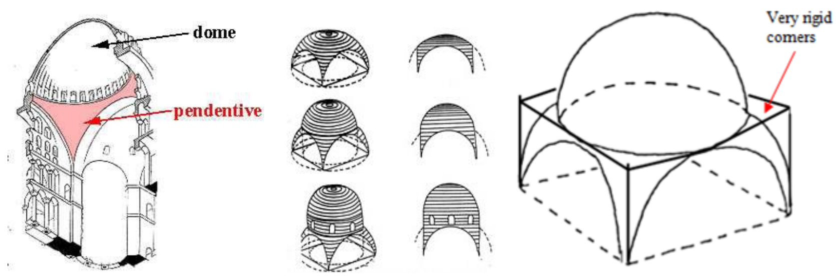


Fig. 5.3 Pendentives, domes and towers on pendentives

The typical plans of Orthodox churches are presented in figure 5.4a, rectangular nave with one lob and three-lobed nave. Unlike the Catholic churches, the Romanian Orthodox churches are relative small in size. The main typical Romanian orthodox churches are the three-lobed plan. This form plays a crucial role in the improving the church behaviour during the earthquakes, because it reduces the distance between stiffness centre and centre of gravity on the longitudinal axis of symmetry **[5.15]**. In some cases, some churches were provided with buttresses in order to reduce the distance between these two centres.
Very many damaged churches were recorded during 1977 and 1986 Vrancea earthquakes. Among the hundreds of damaged churches during these earthquakes, figure 5.4b presents the damages from the Borzesti (rectangular plan) and Cozia (three-lobed plan) churches **[5.16]**.

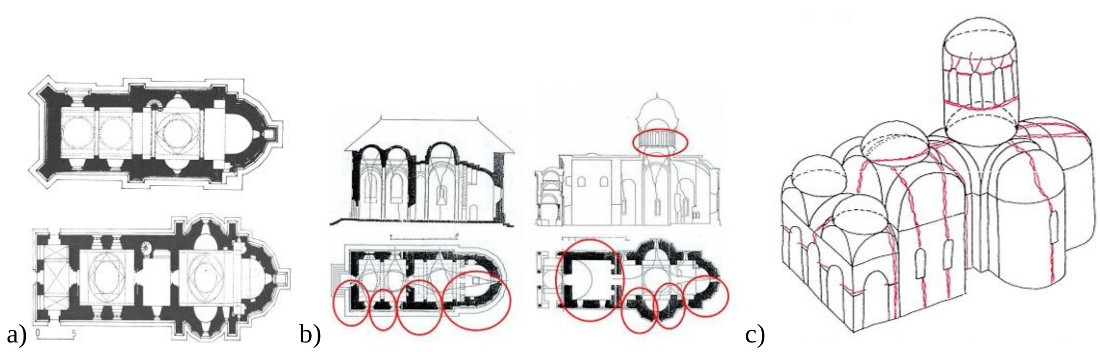


Fig. 5.4 a) typical plans of Romanian orthodox churches; b) damage of Borzesti and Cozia churches due to 1977 Vrancea earthquake; c) failure blocks of Romanian orthodox churches **[5.16]**

Analyzing the occurred system of fissures and cracks it is very clear that the spatial collapse mechanism is formed by a longitudinal fracture and multiple transversal fractures which round the pendentives, due to the great rigidity of these ones. In addition, cracks occurred at the base of tower. The cracks start always from the windows, due to reduced rigidity in these zones. Considering the system of fractures, it is very clear that, in the ultimate limit state, the churches form a block system, working independently each other's as in figure 5.4c. The blocks are formed by the wall delimited by two windows and the corresponding corner pendentives. Due to the seismic actions, the blocks rotate around a basis axis. This rotation is equilibrated by the gravity loads, mainly due to the masonry weight. The ration between overturning and stabilizing forces gives the possibility to determine the ultimate limit loads, this one being the minimum of all the values, determined for each block.

Earthquake of 1991 has produced significant damages to some churches made of brick masonry. Among these damaged churches is the one of St. George Monastery in the village Manastirea, Timis County. It is attested in the sixteenth century and was originally built as a Byzantine church. The present form dates from the years 1795-1796 when the church was transformed into a baroque one. Currently it is declared a historical monument. The length of the church is 21m and the width 9,80m (fig. 5.5a,b). Walls are 75 cm thick, reinforced with brick pillars in front of the roof arches. Hall-type church building is covered with brick arches and wood framing. The church has two towers: the West Tower and East Tower. The East tower is circular and it is supported by the pendants of the middle dome. Two domes on brick pendants supported by arches cover the middle area. The West tower is the bell tower and it has a rectangular section, being supported on one side by the church walls and on the opposite side by a brick arch with cracks and no tie. The masonry structure presents a longitudinal symmetry, except on the West side where the presence of stairs introduces an asymmetry; hence the centre of rigidity of the structure is shifted to the West. At the same time, taking into account the different stiffness of the stair area on the opposite side, the centre of rigidity is moved in North. Rotation of the church from the centre of rigidity is shown in figure 5.5c. It can be noticed that the round wall next to altar is the most stressed, being the farthest from the centre of rigidity.

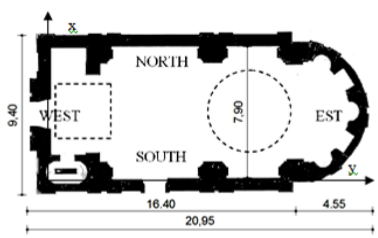
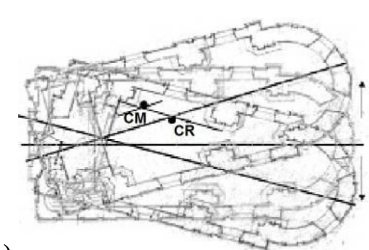
a)  b)  c) 

Fig. 5.5 a) view of the towers; b) horizontal section; c) position of centre of mass and rigidity

Because the distances to the two poles are 15 km and 10 km, earthquake fits the category of "epicentral earthquake" which is characterized by: very short periods of vibration (below 0.2 to 0.3 sec) in which case the massive masonry structures are most affected; pulse action, the first cycle is the most powerful, next alleviating considerably; components perpendicular to the fault rupture are the most important and have vertical components are the same size as the horizontal ones.

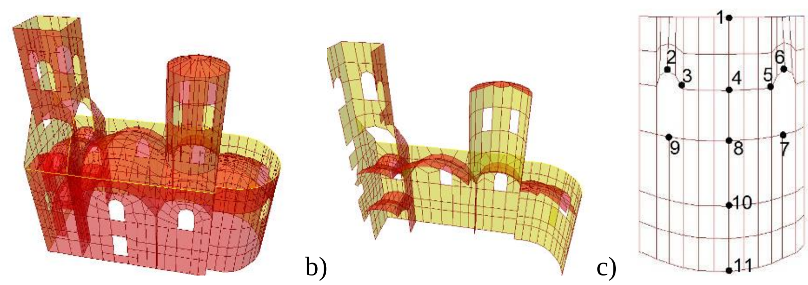
Examining the sizes and positions of the cracks after the earthquake in 1991 the followings are found: the East tower has the upper closure in a vault made of bricks and has no significant cracks; the West tower presents X cracks due to seismic actions and horizontal cracks due to torsion at the level of the windows (fig. 5.6a). The largest cracks are developed in the eastern wall next to the altar because it is situated furthest away from the centre of rigidity. In figure 5.6b are visible cracks and vertical ruptures of the masonry of 3-4cm wide which start from the attic, across the windows and reach the ground level. These ruptures delimit the failure blocks from the altar zone. Asymmetrical turning tendency can be seen on the walls: the South wall is much more cracked than the North wall. Fewer cracks can be seen in the North and South walls than in the eastern wall. Inside the church there are cracks at the top of the vaults because the arches don’t have ties for taking vertical loads. Unveiling of the foundations in the altar area and in the North facade revealed that there are no cracks in the foundation.

a) b)

Fig.5.6 Cracks in the a) Western wall; b) Eastern wall

(i) Spatial seismic analysis of the unconsolidated building in the elastic domain

To analyze the behaviour of the resistance structure, finite element method was used. Structural model shows the original geometry of the building before producing of the damages. To understand the overall behaviour of the resistance structure is sufficient to consider linear elastic behaviour of materials. Walls and vaults modelling were done with two-dimensional elements of the "shell" type and the arch elements of the "frame" type (fig. 5.7a,b). Walls were considered embedded in the foundations. The roof, wind, snow, service load and seismic excitation have been introduced as external actions. Since the maximum damages have been recorded in the eastern wall, after the earthquake, comparative values of the maximum efforts have been studied in 11 points in this wall (fig. 5.7c). In Table 5.1 are presented the values of the maximum unitary efforts for the seismic action on transversal direction denoted by "Comb1" and on longitudinal direction denoted by "Comb2".



a) b) c)

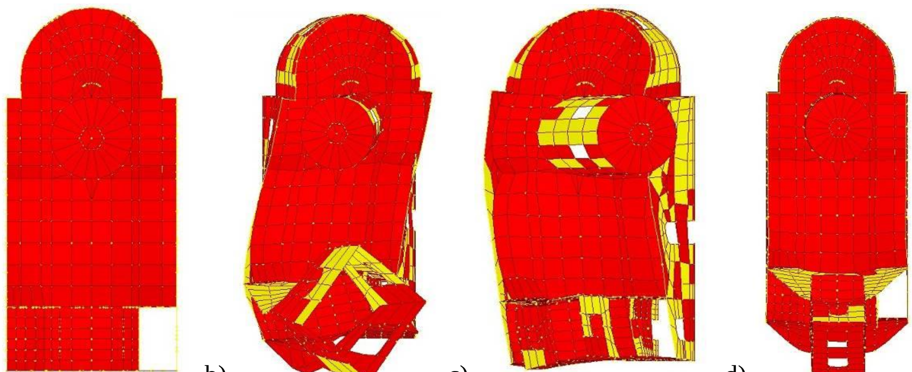
Fig. 5.7 a) meshed structure; b) longitudinal section; c) measurement points of the unitary efforts

Characteristic modes of vibration analysis have revealed the weak areas of the building, where damages occurred. The spatial dynamic analysis performed with finite elements has shown:

- In the first mode of vibration, the church has a general torsion movement (fig. 5.8a) having a period T1=0.21 sec, in the second mode of vibration is characterized by a translation movement on transversal direction (fig. 5.9b) with T2=0.34 sec and in the third mode of vibration a translation movement on longitudinal direction (fig. 5.9c) with T3=0.48 sec. It can be observed that the first two periods of vibration are within 0.21 and 0.34sec, being in the zone of maximum seismic acceleration. The damages recorded by the church from general torsion and transversal translations are confirmed by the dynamic analysis. The general torsion of the building has led to the appearance of horizontal cracks in the western tower (fig. 5.6a) and the rupture of the wall from the altar area (fig. 5.6b);
- Important vertical displacements of the eastern tower produced by large vertical components of the earthquake. Large efforts are transmitted by the tower to the transversal interior arches, inducing cracks in the brick arches. The failure of the arches hasn't been due to the rigidity and the bearing capacity of the pendentives. The North and South walls have been consolidated by

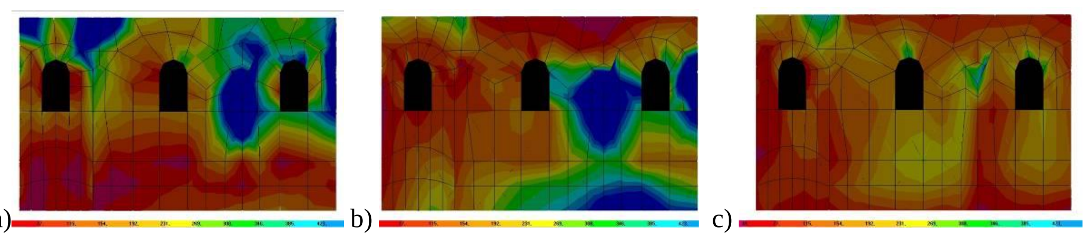
means of columns disposed on transversal direction, on which the arches discharge the loads. The relative few cracks recorded in the northern and southern wall after the 1990 earthquake, confirm the results given by the computation software;

• The largest values of the unitary efforts are recorded in the eastern wall in the windows area in points 2, 6, 7 and 9 from figure 5.7c, regardless of the direction of the seismic action. The values of the unitary efforts are presented in Table 5.1. A good correspondence between the efforts and the cracks is shown in figure 5.6b. The separation of the building in failure blocks, failure mechanism theory, is confirmed by the manner in which the efforts are concentrated as shown in figures 5.9, 5.10 and 5.11.



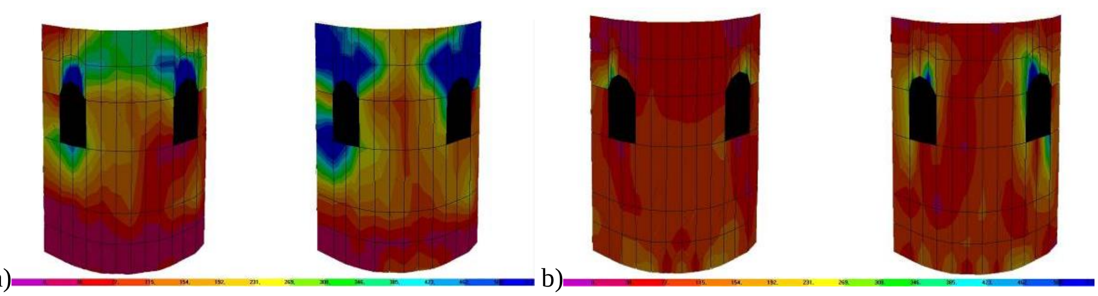
a) b) c) d)

Fig. 5.8 Modes of vibration a) undeformed; b) 1st shape mode; c) 2nd shape mode; d) 3rd shape mode



a) b) c)

Fig. 5.9 Stress diagram for North wall a) Tension stress S11; b) Compression stress S22 c) Shear stress S12



a) b)

Fig. 5.10 Stress diagram for East wall a) Tension stress S11; b) Compression stress S22

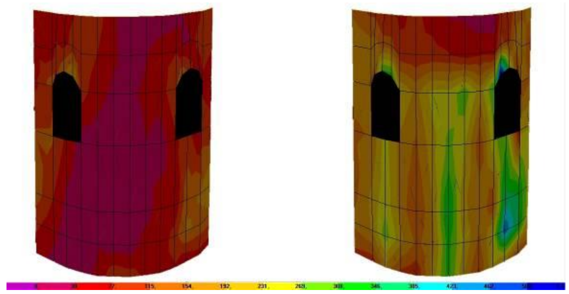


Fig. 5.11 Shear stress diagram S12 for East wall

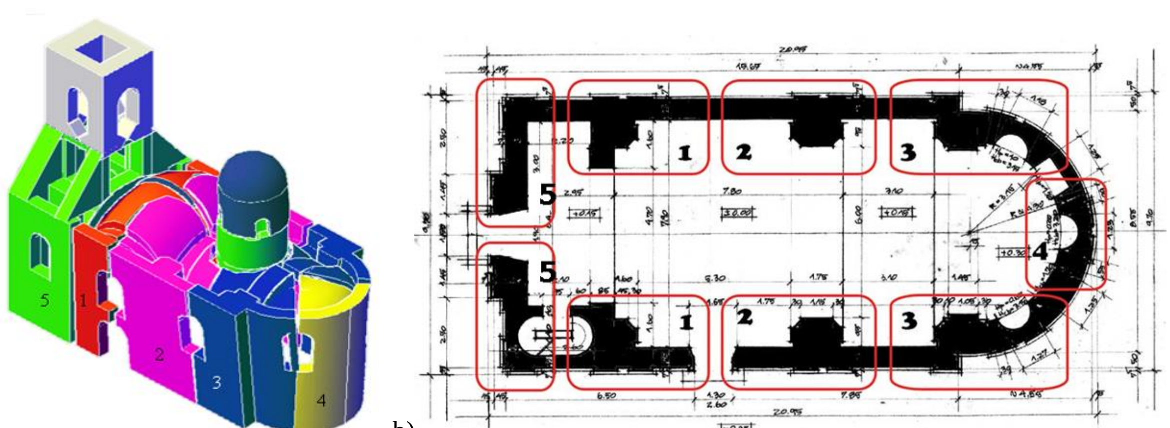
Table 5.1 - Values for recorded and allowable stresses in eastern wall

Point number	Tension stress S11 [N/mm^2]		Shear stress S12 [N/mm^2]			Compression stress S22 [N/mm^2]		
	Comb 1	Comb 2	Comb 1	Comb 2	Allowable	Comb 1	Comb 2	Allowable
1	0.18	0.39	0.04	0.01		0.04	0.04	
2	1.34	0.76	0.31	0.15		0.21	0.09	
3	0.27	0.19	0.23	0.10		0.2	0.03	
4	0.10	0.14	0.25	0.03		0.08	0.06	
5	0.44	0.28	0.31	0.12		0.38	0.04	
6	1.76	1.06	0.30	0.10	0.12	0.34	0.15	0.31
7	0.12	0.04	0.30	0.10		0.05	0.02	
8	0.11	0.16	0.28	0.01		0.09	0.07	
9	0.52	0.47	0.25	0.07		0.05	0.02	
10	0.13	0.11	0.36	0.01		0.11	0.07	
11	0.10	0.05	0.33	0.13		0.15	0.17	

(ii)Modeling masonry building by rigid blocks

Based on the modelling of seismic behaviour of Church of St. George Birda, Timis County, Romania, based on the theory of transfer mechanisms. The following assumptions were made **[5.6]**:

• The church was divided into 5 rigid blocks based on observations recorded after earthquake cracks in the Banat area since 1991. Transfer blocks are shown in figure 5.12. Because the greatest damages were recorded in the nave and apse altar, a special attention was given to these areas;



a) b)

Fig. 5.12 a) blocks system in ultimate limit state; b) in plan dimensions and positions of blocks

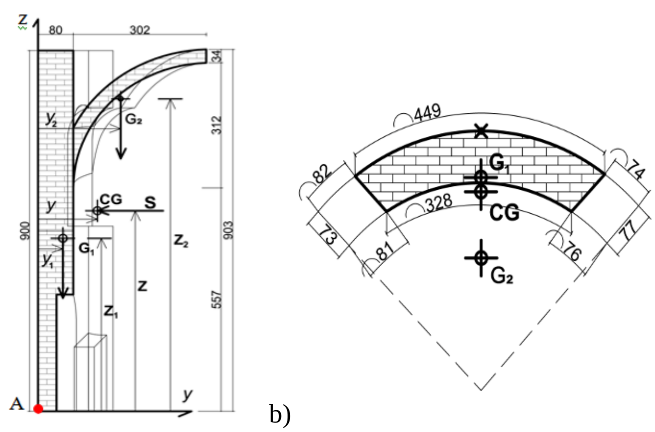
• There were not made any calculations to determine the peak ground acceleration for the tower named block 5 because the vertical stabilization forces are bigger in this area and tower has developed cracks in the plan of the masonry after earthquakes;

• In its calculations the roof weight was neglected because it introduced very low values;

• In the calculations there was not assigned any weight to the circular tower of the nave area because it wasn't participating in the failure mechanism. At failure blocks 2 and 3 the weight of the arches was taken into consideration, along with the weight of the walls and the pendants;

• The church has no metal tie to prevent the overthrow of the outside walls;

The author separately calculated the weight, the position of the centre of gravity for each wall from a failure block. It was determined the position of the centre of gravity and the centre of mass for the entire failure block on a 3D model drawing. The overturning seismic force outside the wall was applied in the centre of gravity of the failure block. The equilibrium relation between overturning moment and stabilisation moment was written for each failure block in relation to point A (fig. 5.13a). It is located at the outer end wall. According to **[5.11]** the proportion ratio between the overturning moment and the stabilization moment of the transfer block is called seismic coefficient and is noted by λ. It has subunit value and represents the maximum value of ground acceleration for which the building collapse that occurs outside of the plan of the failure block. Maximum seismic acceleration of ground in the Monastery of St. George is 0.20g, so λ_{max} field= 0,16g/g = 0,16. λ values calculated by theoretical modelling for each failure block were compared with this value of λ_{max} field. Blocks with a value of $\lambda < \lambda_{max}$ field are most vulnerable because they will fail and will cause the collapse of the structure. For St. George Monastery was calculated for blocks 2: λ_2 = 0,264, for block 3: λ_3 = 0,338. In figure 5.13 are presented the dimensions and position of the centre of gravity and the point of application of seismic force. For failure block 4, which was the most damaged after the earthquake, it has been calculated λ_4 = 0,168.



a) b)

Fig. 5.13 Failure block no. 4; a) vertical section b) in plan dimensions and positions of centres of gravity

Comparing relations it is observed that: $\lambda_4<\lambda_2<\lambda_3$ although the theoretical modelling, predicts that the limit state will be achieved by the collapse of block 4, that will yield before blocks 2 and 3 for the same value of the seismic force. This damage is confirmed by the biggest masonry rupture of the apse area and results provided by theoretical modelling are confirmed with the damage and collapse mechanisms developed by the resistant structure of the church after the 1991 Banat earthquakes **[5.6]**.

Examining the formed collapse mechanisms of Orthodox churches it is clearly that the restoration must considers solutions to prevent the division of church's walls in blocks, by corseting the exterior walls. This is possible to perform this task only introducing a horizontal diaphragm at the roof level, or including also the reinforcement of vertical walls. In the first case, the rigidity of structure is not significantly modified, while in the second solution, the structural rigidity increase very much.
The Romanian territory is shaken by two earthquake types, the Eastern parts being characterized by long periods, while the Western parts, by short periods. In these conditions, the rigidity increasing by strengthening procedures has different effects on seismic actions. In the case of long periods, the increasing the structural rigidity reduces the seismic actions. Contrary, for short periods, the tendency is to increase the seismic forces (fig. 5.14). Therefore, two different philosophies in restoration of Orthodox churches exist:

• For the Eastern Vrancea earthquakes, the designers prefer the completely corseting restoration (fig. 5.15);

• For the Western Banat earthquakes, the designers consider that the introducing only a horizontal diaphragm under the roof is the best solution.

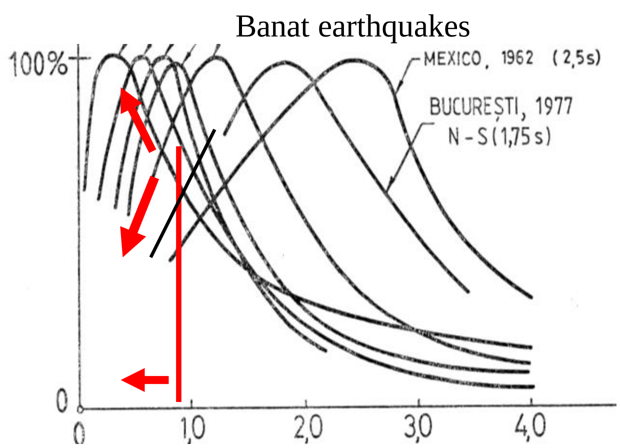


Fig. 5.14 Effects of rigidity increasing by strengthening

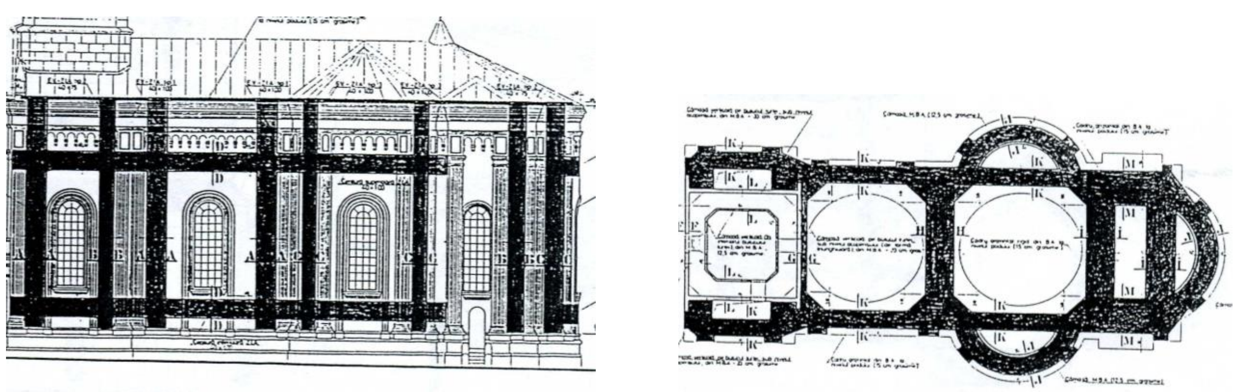


Fig. 5.15 Corseting the churches for Vrancea earthquakes (Crisan, 2003) **[5.7]**

5.1.1.4 Conclusions

Using structural modelling of brick with finite elements in the elastic linear domain, determines the main modes of vibration, which detected the weak areas of the structure.

Modelling of failures of buildings through blocks is a method which estimates with good accuracy the maximum ground acceleration at witch structure collapse occurred. In the same time it simulates vet well the seismic response of building in the ultimate limit state. With this information, you can easily determine building solutions to be taken before an earthquake emergency only in the most vulnerable areas of the building to avoid collapse of the historic building.
The theory of mechanisms of failure developed a simple and fast calculation method that has been verified by numerical analysis for the Catholic Church of Roman type. By applying the same principles of modelling and calculation, the theory of yielding mechanisms was verified and developed by the author for the first time for orthodox churches which have different plan conformation than catholic churches. The method was developed and verified by the author for the first time in Romania on the baroque orthodox church of St. George from Birda, Banat region, Romania **[5.18]**. The study performed by the author showed a good concordance between finite element method, theory of failure mechanisms and the damaged structure after the Banat earthquake of 1991 and opens new international research directions in the domain of seismic vulnerability of orthodox churches.

5.2 Apartment buildings

5.2.1 Innovative system for consolidation of historical few storey masonry buildings situated in seismic areas

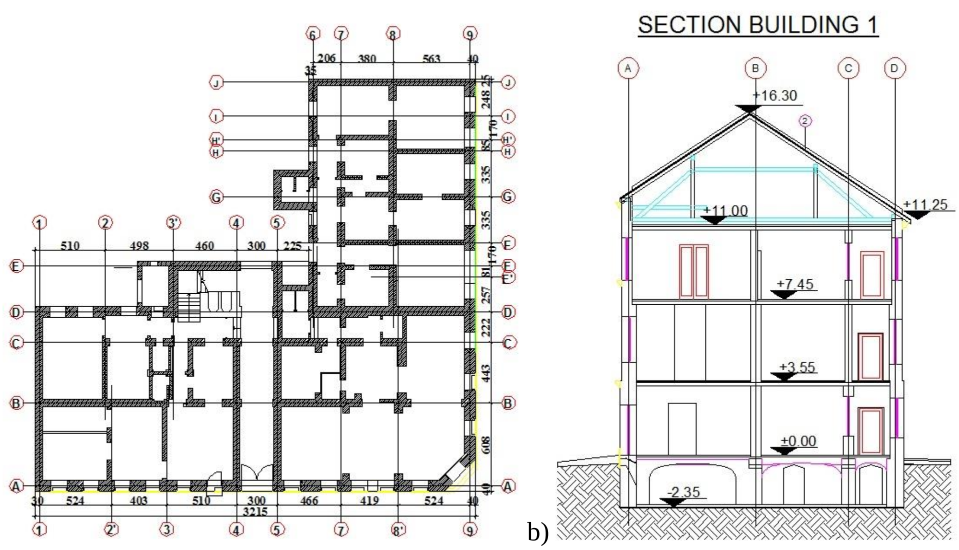
5.2.1.1 Introduction

Discoveries in the constructions field have led to the development of spatial static, dynamic and dimensioning computations for cross sections with a higher degree of accuracy as well as to the development of new methods and advanced consolidation technologies of the bearing structures for exceptional loading caused by earthquakes, fire, explosions, floods, landslides etc. In the conservation and restoration field of the cultural heritage, the consolidation solutions are harder to identify because they have to respect the reversibility principle of the intervention as stated in the Chart of Venice in order to maintain the historic value of the building. First, the architects coordinate a team of specialists which analyze the impact and destination of the restored building in the future time. The design and consolidation stage of historical buildings is an important field of structural engineer’s activity. The main problem of restoration of these buildings is that they were designed without taking into account the constructive rules of the actual seismic codes.
Until now, the majority of them managed to remain without any important damages after a series of earthquakes. It is due to the fact that the location of these buildings are in areas with low to moderate earthquakes and because during erection some constructive rules, based especially on intuition and observations, were adopted. However, at many buildings, the time left marks on the materials by their degradation, and the buildings have suffered major structural interventions, by changing of their functionality. This paper presents some conclusions about the seismic behaviour of historical buildings: conformation deficiencies and the effects of structural interventions brought to them in time. For exemplification, it is presented a corner building in Timisoara, for which the spatial seismic behaviour was analyzed using computer programs based on FE method, both for simple and reinforced building.
Generally, in restoration of historical buildings, there are two traditional solutions:
i) the strengthening all the structure or only some damaged or weak structural elements;
ii) preserving only the masonry building façade and changing the interior masonry structure with a new steel structure;

The chosen solution is an innovative and mixed one, preserving all the main elements of the existing building, strengthening them, and introducing in the inside of the building a new steel structure **[5.18] – [5.22]**.

5.2.1.2 Description of the building

The building is located in the city of Timisoara and is part of a historic building complex from the protected area Josefin, outside the old city stronghold. It is a corner building, having an L shape in plane and it was built in two stages: building 1, bounded by the axes 1, 9-A, E having the in plane dimensions of 13.00m by 32.15m and it was built in 1899, and building 2 bounded by the axes 6, 9-D, J with the in plane dimensions of 17.20m by 11.80m being erected later on (fig. 5.16a). The two main buildings had initially a basement of 1.80m height, ground floor and two storeys, having a maximum height of 16.35m (fig. 5.16b). The bearing structure of the building was executed using historic manufacturing technologies for burnt bricks, wood and steel brought by German and Italian builders which came in Timisoara after 1718, the year of conquest of the citadel by the Habsburg army.



a) b)

Fig. 5.16 Vertical and horizontal section of the building

At the acquisition of the building by the current owners in 2007, the building was abandoned by the previous owners and it was in an advanced state of degradation. The architectural solution proposed the transformation of the building in a hotel of deluxe apartments. In this way, a restaurant was made in the basement, increasing the height of the basement by excavating about 1.50m of earth from beneath the entire building thus enhancing the bearing structure of the basement, transforming the ground floor with plenty compartment walls into an elastic floor for shops, cafes and turning the rooms from the first and second floors into hotel rooms. The height of the building remained the same, the façade and colours used were chosen based on old pictures made at the inauguration of surrounding buildings. In the inner courtyard a panoramic new elevator was mounted. Today the hotel is called “Josefin apartments” and it is one of the best rated international hotels from Timisoara. In figure 5.17 there are presented pictures of the building before and after the change of functionality and restoration works.

Fig. 5.17 The building in 2007 before and after the change of functionality

The resistance structure of the building can be recognized as the most commonly used type in Timisoara in that time and is part of a historic building complex because it was erected in different stages. Due to the lack of provisions in the Romanian design codes for evaluating the seismic vulnerability of complexes of buildings as developed by Mazzolani **[5.23]**, Langomarsino **[5.24]**, Modena **[5.5**], during the expertise the bearing capacity of the most stressed structural elements was checked, as well as the maximum displacements resulted from the seismic action. The bearing historic structure was made from walls and brick foundations. The thickness of the walls varies on height: 80cm at the basement and ground floor, 70cm at the first floor and 50cm at the second floor. The slabs were made from brick arches and vaults at level +0.00m and from wooden beams supported on the exterior walls and inner longitudinal walls.

As all historic buildings, the bearing structure of the building was made from structural sets and subsets based on manufacturing techniques and technologies specific for that time. Generally the buildings were made of brick masonry with lime mortar, wood for the slabs and roof framing, steel beams for the slabs and steel rods encased in the masonry. The conductive system of the hotel was made from bearing exterior walls, disposed on the perimeter and inside on longitudinal direction, without bearing transversal walls with bracing role to overtake the seismic actions. Moreover, building 2 was erected afterwards and doesn't have a transversal wall in the intersection zone of the two buildings. The slabs were made from wood, unbraced in their plan at the superior levels, and from brick arches and vaults over the basement, without horizontal rigidity that can insure a rigid diaphragm. The building doesn't have steel rods in the masonry which can insure a good transmission of tension loads from the arches, vaults and walls. Insufficient depth of the foundation system was observed, the walls of the basement being embedded only 30cm in the foundation ground, resulting in settlements of the foundations due to the discharge of the soil inside the building under the seismic actions. At the intersection of the walls, openings for chimneys were disposed which prevented a good conformation of the walls with that perpendicular to them.

The recorded degradations in the historic bearing structure in 2007 were due to the former owners, but mainly due to poor maintenance and unauthorized interventions at the bearing structure. Through the broken covering rainwater infiltrated and damaged the wooden elements from the roof framing and floors. Missing gutters and broken water installations from inside the buildings facilitated the infiltration of the water in the walls producing large areas of mould and the rotting of the wooden beams from the floor system. Unauthorized interventions on the structure consisted in breaking of some walls, creating of new openings in the existing walls, execution of large installation holes in the arches and vaults, breaking of lintels made from brick arches in order to introduce new windows and doors, demolishing of brick vaults, casting new reinforced concrete slabs over the historic floor system with wooden beams. A second important source which generated damages was the systems which acted on the structure in time, leading to inclined cracks in the walls, arches and lintels. No breakings of the walls due to seismic actions were recorded, but the perimeter masonry at the attic level presented characteristics of out of plane failures. The damages statistics recorded in time for the Banat earthquakes, point out the out of plane failure mechanism as the most spread failure type in Timisoara for 2 or 3 storey buildings.

5.2.1.3 Consolidation solutions for the historic bearing structures

The resistance structure deficiencies were analyzed by a bearing capacity calculus based on the computation methods from the seismic design code from Romania P100/1-2006 and was compared with the results obtained following a spatial seismic analysis of the building with the aid of finite element method software. In figure 5.18 are presented the results for the computations performed in the walls at the level +0.00m subjected to bending with axial force (fig. 5.18a) and shear force (fig. 5.18b).

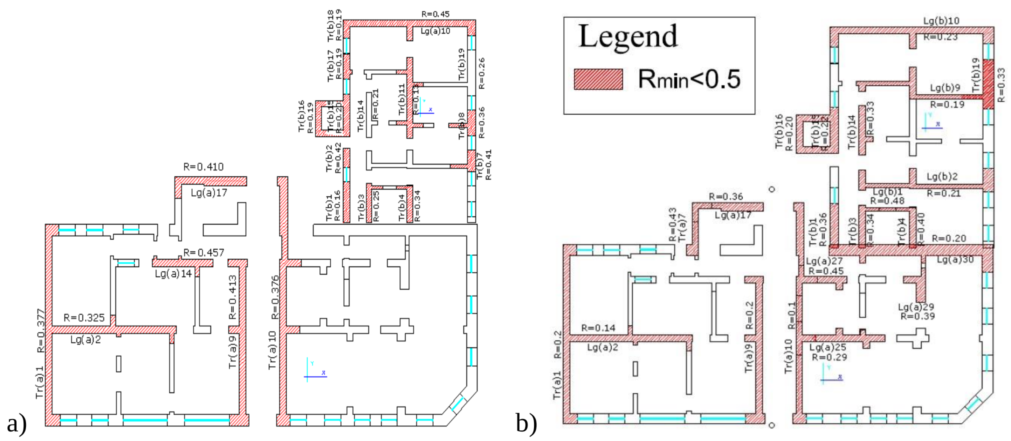


Fig. 5.18 Values for the minimum insurance degree a) bending and axial force; b) shear force

In general the transversal walls and the interior longitudinal walls and a part of the masonry columns from the façade don't have even 50% from the necessary bearing capacity for seismic loads, meaning R_{min}<0.5. The results were checked and confirmed by the spatial computation of the building made in the elastic domain, with the help of finite element software. The deformed shape of the unconsolidated structure for the first two vibration modes are presented in figure 5.19.

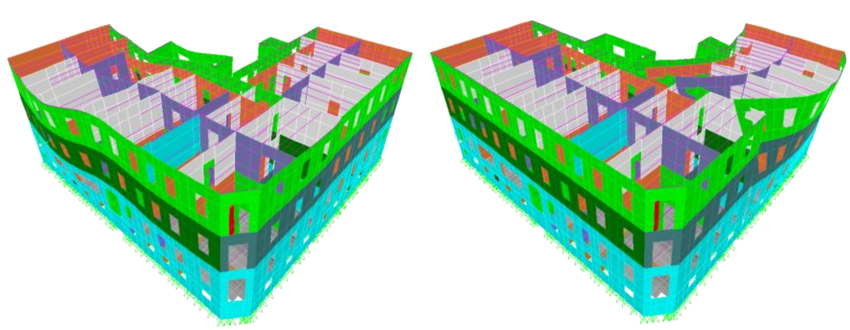


Fig. 5.19 The vibration modes 1and 2 of the unconsolidated building

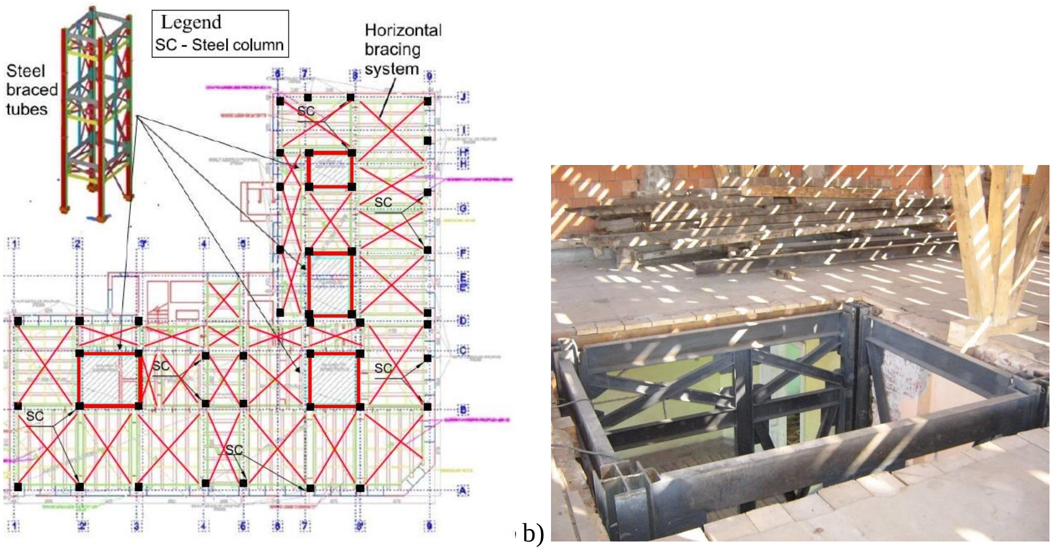
High periods of vibration indicate a high rigidity of the structures, from the spatial dynamic calculus (tab. 5.2). For the first 2 modes of vibration, the periods varied between 0.31 – 0.36 seconds for building 1 and 0.28 – 0.33 seconds for building 2. These values indicate that the most rigid structure is also the most vulnerable and stressed in the Banat seismic zone. Reduced values of the horizontal maximum displacements calculated for each storey indicates a high rigidity. Computations revealed the fact that the floors made from brick arches and vaults, the walls and the continuous foundation system isn't a rigid box and doesn't insure a good embedment of the walls at the ground floor level.

The base for establishing the methods and consolidation technologies of the historic bearing structure was to respect the reversibility principle of the intervention, as stated in the Chart of Venice and in articles published in the domain of consolidations **[5.25]**, **[5.26]**, **[5.27]**. They have permitted to raise the height of the basement, execution of short term works, turning the ground floor into an elastic floor for retail space, repartitioning of first and second floors, building an attic room and local interventions on the historic bearing structure. These demands of the beneficiary were satisfied by building an innovative structural system based on turning the basement intro a rigid box and local reversible interventions at the superstructure with steel elements. The rigid box was done with the help of a rigid foundation plate of reinforced concrete of 40cm thickness and a reversed foundation beam (fig. 5.20), executed approximately 1.50m lower than the level of the existing foundations. At the level +0.00m the rigid box was insured by a reinforced concrete slab with the thickness of 25cm fitted with a system of reinforced concrete beams supported on the walls between the brick vaults. Connecting reinforcement was provided into the basement walls in order to insure a good collaboration with the network of reinforced concrete beams.

a)  b) 

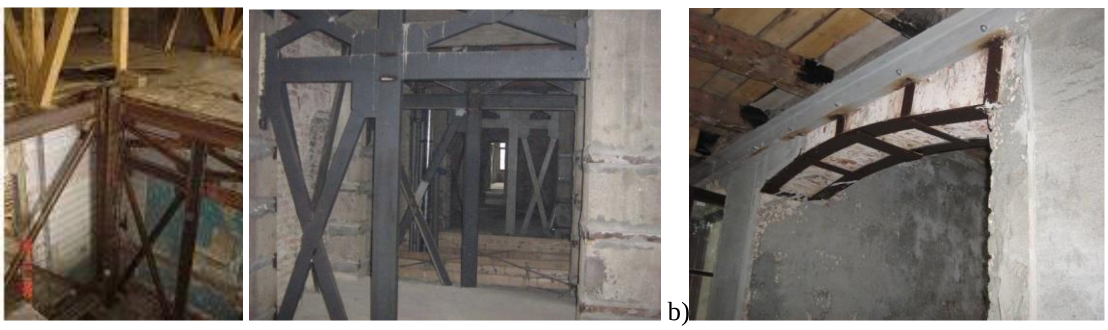
Fig. 5.20 Basement view a) after increasing the height with 1.50m; b) foundation plate with reversed beam

The innovation of this structural system developed by the author consists in four steel tubes introduced inside the building, properly connected with the perimeter brick masonry walls by means of steel beams (fig. 5.21). Thus, the seismic load is overtaken by the steel tubes, and the masonry walls remain subjected only to vertical loads. Steel columns were introduced in the masonry walls and the masonry columns from the façade in order to overtake vertical loads and to transmit the seismic loads to the steel central tubes. The section of the façade columns and walls was redone by pouring concrete between the steel columns and the brick masonry. The vaults, lintels and historic arches were consolidated with steel profiles (fig. 5.22, 5.23).



a) b)

Fig. 5.21 Steel tubes a) positioning; b) execution details of the steel tubes



a) b)

Fig. 5.22 Execution details of the consolidations

Fig. 5.23 Positioning of the steel tubes and the bracing system of the wooden slabs

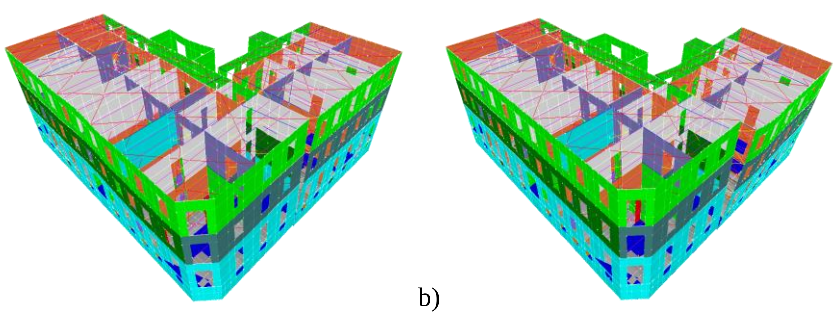
The spatial collaboration between the steel tubes and the brick walls was insured by the execution of rigid diaphragms at the level of the wooden slabs at each level and by mounting bracing walls in line with the steel tubes (fig. 5.24). The rigid diaphragms were made by steel profiles fixed to all the room walls, consolidating steel profiles for the wooden beams and horizontal X braces fixed to these steel beams. This spatial system of consolidation didn't pierce the historic façade although major interventions were made to the historic bearing structure.

Fig. 5.24 Steel bracings a) for floors; b) vertical steel walls

The advantages of this new system of consolidation were checked by a dynamic spatial analysis (fig. 5.25). There were compared the maximum unitary efforts (fig. 5.26, fig. 5.27), maximum horizontal displacements (fig. 5.28) and periods of vibration recorded for the unconsolidated and consolidated buildings, having the enlarged basement and an extra storey by execution of the attic room. From table 5.2 it can be observed that the interventions do not increase the rigidity of the buildings.

Table 5.2 Periods of vibration of the building before and after consolidation

Mode of vibration	**Unconsolidated building [sec]**		**Consolidated building [sec]**	
	Building 1	**Building 2**	**Building 1**	**Building 2**
1	0.36	0.33	0.31	0.33
2	0.31	0.28	0.28	0.26



a) b)

Fig. 5.25 The vibration modes 1 and 2 of the consolidated building

As a result of the utilisation of some light consolidation materials and keeping the seismic loads unchanged, in figure 5.27 it can be observed a great reduction of the unitary stresses, avoiding the failure due to crushing of the brick and brittle failures due to shearing produced by the seismic actions. In figure 5.28 it can be observed that also the maximum horizontal displacements are reduced compared to the unconsolidated version of the building. There are a few exceptions, but the values recorded do not exceed the maximum admissible values from the Romanian design codes. The efficiency of the consolidation was confirmed by the checks performed according to the tracking programs in time. In the two years since the consolidation and finishing works were done, there weren’t recorded any cracking in the walls, settlements of the foundations and rotations of the elements of the bearing structure.

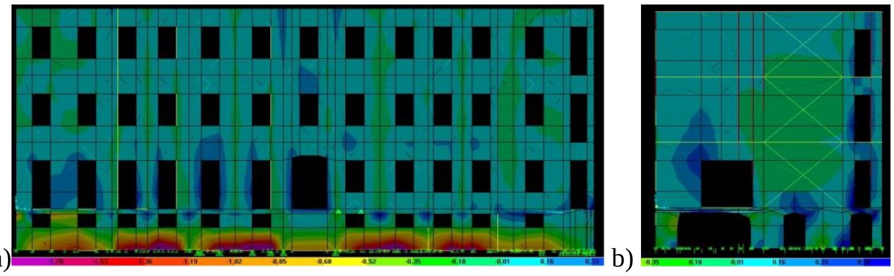


Fig. 5.26 Normal stresses in a) external longitudinal wall - axis A; b) internal transversal wall – axis 5

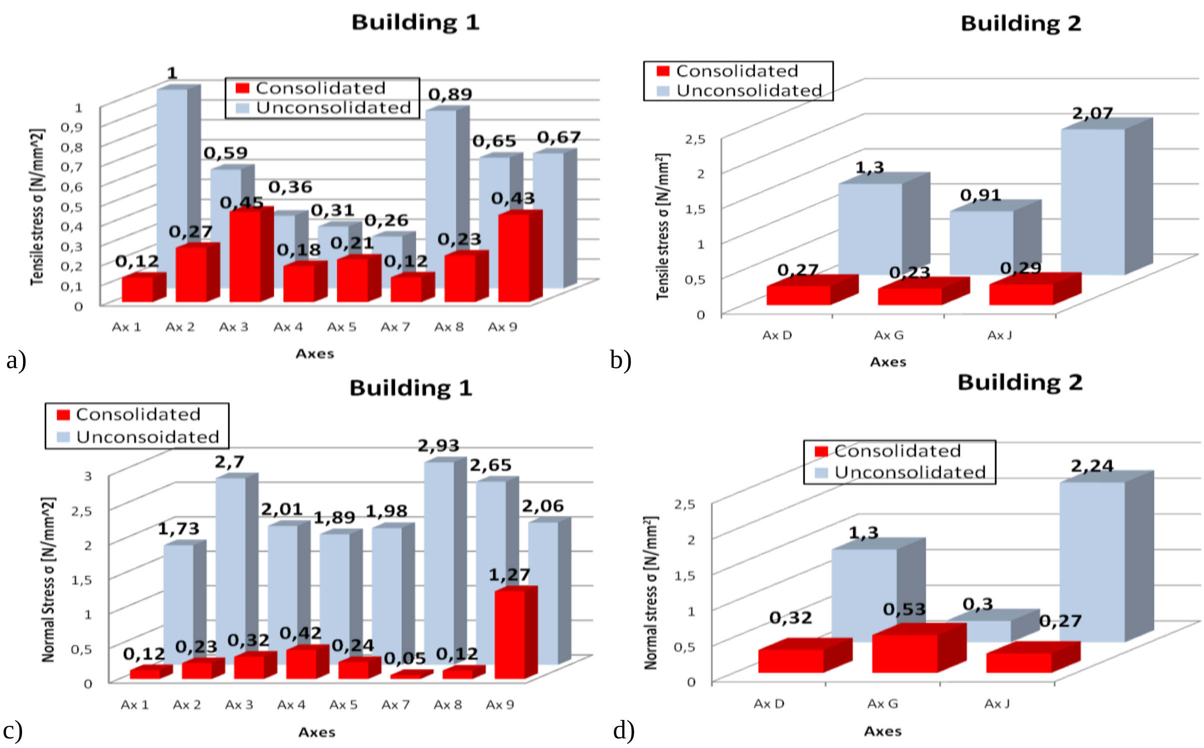
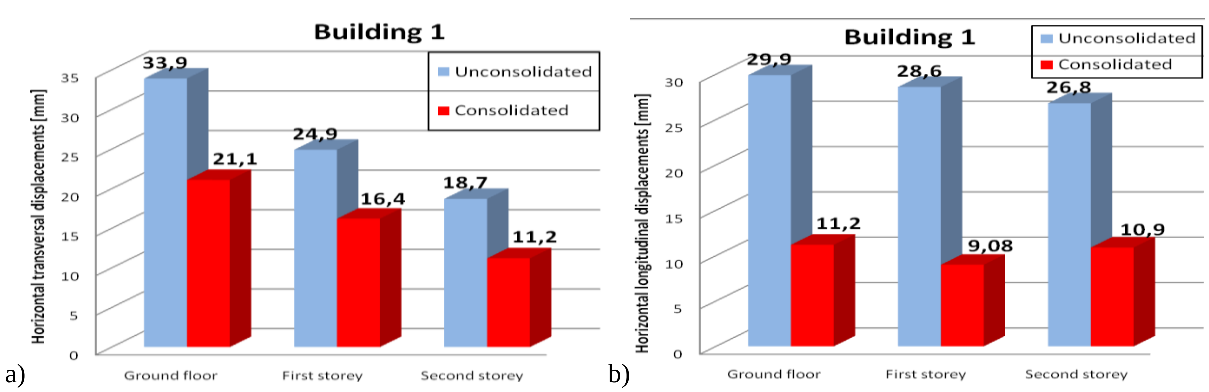
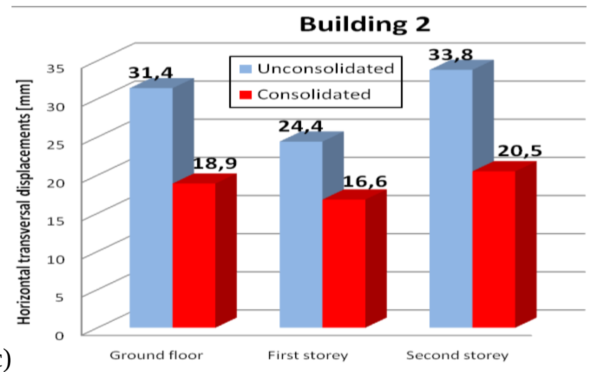
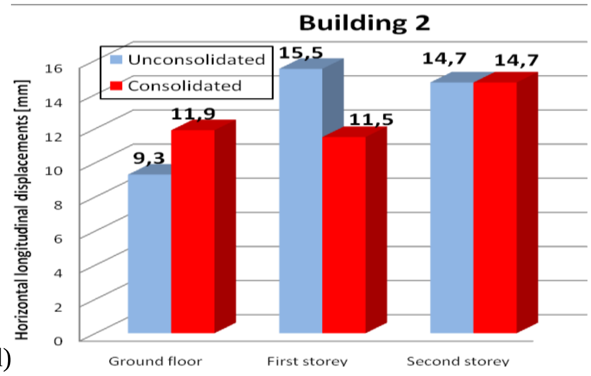


Fig. 5.27 Comparative values of maximum tensile and compressive stresses before and after strengthening of the buildings a, b) tensile stress, c-d) normal stress





c)



d)

Fig. 5.28 Maximum comparative values of horizontal displacements on transversal and longitudinal directions before and after strengthening of the buildings, a- b) building 1, c-d) building 2

5.2.1.4 Conclusions

• The author presented an innovative consolidation method for historic brick masonry bearing structures, for buildings of 3-4 storeys from seismic areas.

• The innovative solution developed by the author consists in the introducing of some steel tubes inside the buildings, connected to steel columns introduced in the façade masonry by means of steel beams. This solution changes the static scheme of taking over the loads from the building: the rigid steel tubes overtake most of the seismic forces, thus reducing the horizontal displacements and the bearing masonry will be subjected mainly to vertical loads, reducing the maximum unitary compressive and shear stresses from the walls.

• Multiple advantages of the innovative solution proposed by the author and introduced at the building of Josefin Apartments from Timisoara:

(i) it is reversible, according to the requirements presented in the Chart of Venice;

(ii) it consolidates the bearing structure of the façade without piercing it and affecting the decorations which make up the historic exterior surface.

(iii) it allows the ground floor to be used for retail space, with large openings, eliminating the danger of elastic floor failure,

(iv) it reduces the execution time by rapid assembly of the steel parts on site and it requires the consolidation of reduced areas of the building without heavy lifting and transport equipment.

6 HISTORIC TIMBER BEARING STRUCTURES

6.1 Churches

6.1.1 Historical wooden churches from Banat region, Romania. Damages. Modern consolidation solutions

6.1.1.1 Introduction

Historical wooden churches are spread in several countries in the Balkans and are considered priceless World Heritage values, because of the traditional manufacturing techniques, religious paintings and in plane and elevation forms specific to each geographic region. Over the years, some have disappeared, others have been moved and some have remained present in the same location. Depending on external factors that acted on them, they have recorded various failures and degradations. Among the most important factors we can mention: fire, floods, landslides, earthquakes, biological attacks. In the western part of Romania as in neighbouring countries such as Serbia and Hungary, there are many historical wooden churches. Generally they were built between 1650 and 1850 with some exceptions. In the present time they are extremely degraded their historical and cultural value is given by religious paintings made directly on the wooden walls.

The author tries to focus on these monuments of great cultural heritage importance unknown in this region and present the main damages of these churches and some reversible consolidation and strengthening solutions according to the Chart of Venice. This research presents reversible particular strengthening solutions for these historical buildings

During the last years, the Romanian Orthodox Church from Banat, together with the author started developing studies in the vulnerability domain of these buildings for strengthening them. The program includes 6 wooden churches: Dobresti, Romanesti, Crivina de Sus, Povergina, Curtea, and Cosevita, all from Timis County (fig. 6.1).

a) b) c)

Fig. 6.1 The wooden church: a) Povergina de Sus; b) Romanesti; c) Dobresti, Banat, Romania

The studies focus on the consolidation of the wooden bearing structures of these churches, paintings conservation, traditional manufacturing techniques and all architectural elements of heritage value. The project team was made up of expert architects, biologists, geologists, restorers and constructors.

Due to the fact that in the Banat region the resistance structure of wooden churches were not consolidated, the author used the results from the latest researches in the consolidation domain of wooden bearing elements, published at conferences and journals by Mazzolani **[6.1] [6.2] [6.3] [6.4] [6.8] [6.9]**, A. Borri **[6.5][6.6]**, Kappos **[6.7]**, Jasienko **[6.10][6.11]**. At the base of these consolidation solutions developed by the authors from the "Restauro" Foundation **[6.15]** were several studies made on historical wooden buildings within the research contract PROHITECH **[6.9]-[6.14]**. The author participated along with researchers from universities and laboratories from all the countries surrounding the Mediterranean Sea at theoretical and experimental studies in historical bearing structures field.

6.1.1.2 Theoretical and experimental studies

Wooden churches of Banat are made of oak wood carved with an axe. They are situated in an area in which the average annual temperature varies between +9ºC and +11ºC and the average annual rainfall is between 700-800 mm/m^2. The freezing depth is situated at 75cm from the ground level and the peak ground acceleration is 0.08g.

In terms of construction techniques and systems there are two types. Blockbau system (fig. 6.2) is made by wood beams (mostly oak) carved on the four sides and placed one over the other. The corners are joined by carving for stiffening. The entire scaffolding is placed on some large wooden base. Fackwerk system (fig. 6.3) has wooden columns from place to place, but with poles along the walls; the exterior walls are plastered with a mixture of earth, chaff and chopped straw. This system allows larger openings. From the Fackwerk system resulted other structural systems.

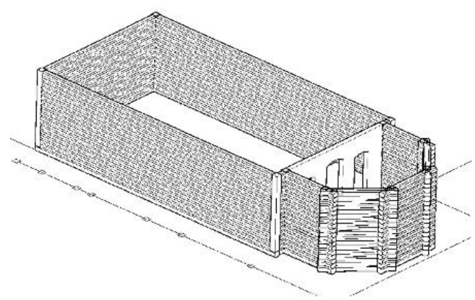


Fig. 6.2 Historic building system Blockbau

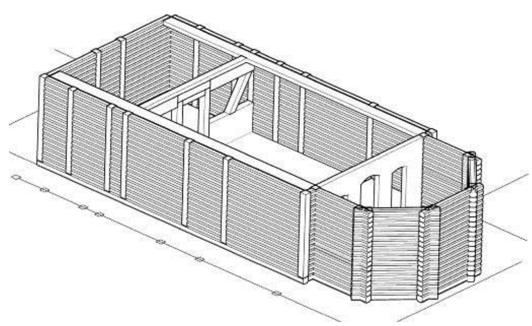


Fig. 6.3 Historic building system Fackwerk

From the roofing system or interior walls point of view, all these types show common elements: narthex is covered with ceiling planks, usually lower height than the nave; nave shows a semicircular arch on the longitudinal direction also the ceiling planks are decorated with paintings; painted tympanum; veil on the east, 4 roof slopes, follow the form and path of the altar, according to the constructive typology. All churches have a tower. The bell is located in the narthex. Maximum length of the churches is 17.0 m, the maximum width is 7.20 m and the maximum height of the tower is around 12.0 m. The wall thickness is 25 cm.

There are many causes for structural degradation:

(i) Poor maintenance

The first cause is poor maintenance of the churches during the communist period from 1945 to 1989 and lack of financial resources after 1990. There weren't made coherent conservation and consolidation works due to the lack of experience in this domain. Also the absence of modern technologies and analysis laboratories contributed to the further degradations. The roof framing presents areas with rotten rafters due to rainwater penetration; the buildings have no drains or sidewalks; churches generally have no pipes and gutters, in some cases there is no tight joint between them. Unauthorized works were made such as: casting of concrete under foundation or replacement of elements with other material and dimension elements, introduction of metal ties through painted walls (fig. 6.4a) and wooden columns for supporting of horizontal wooden beams in the Church (fig. 6.4b).

a)  b) 

Fig. 6.4 Unauthorized strengthening interventions

(ii) Degradation of paintings

The paintings inside the Church are made on wooden surface and are much deteriorated (fig. 6.5). They were executed in two ways: directly on oak wood, or a concoction of primer on a canvas with tempera colours. This canvas was used for the aesthetical purpose of hiding the plank edges **[6.16]**. Mural painting in general shows inconsistent deposits of dirt (dust, smoke, traces of insects, spider nets); painting layer separations, separation of the canvas which cover the interstitial spaces, signs of infiltration and erosion, important separations of coat and colour layer, marks of nails, superficial or large gaps, cracks oriented on the direction of the support fibre; traces of inappropriate washing interventions on pictorial surfaces; inscriptions of inventory numbers directly on the painting layer with blue oil paint.

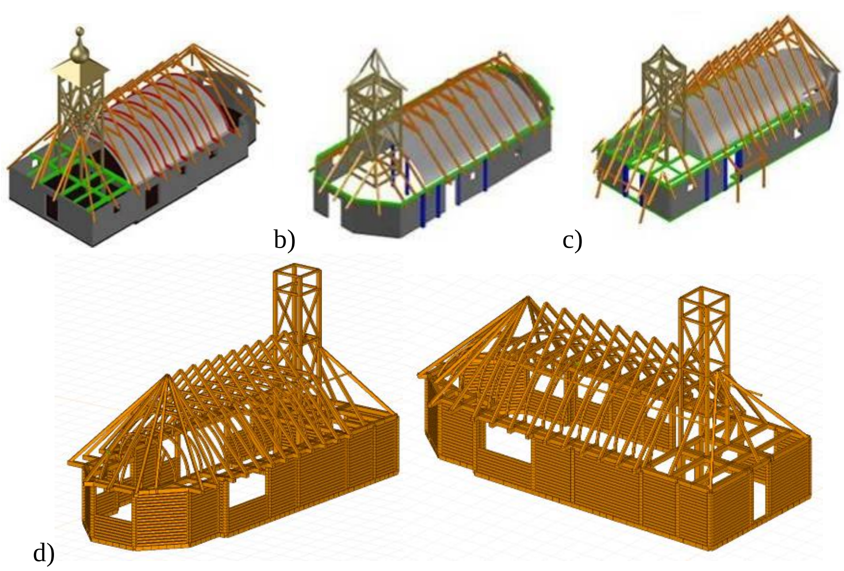
Fig. 6.5 Paintings on wooden ceiling inside the church

(iii) Biological degradations

Biological degradations are the result of an attack produced by xylophages' insects of the species Xestobium Rufovillosum. The samples collected and analysed under a microscope revealed that the columns and beams were made from oak wood (Quercus robur) **[6.17]**. The attack is seen in the form of fly holes and insects galleries that led to the weakening of the wood. Laboratory analysis showed that the attack is not active and the durament kept its mechanical resistance. As result of these degradations the base support has visible biological attack and mechanical strength of wood is decreasing.

(iv) Structural degradations

Structural degradations at the foundation level come from the building system. In figure 6.6 are shown the 3D bearing structures of Churches from Curtea, Crivina de Sus, Romanesti and Dobresti. The foundation is made of stone masonry with clay mortar and is only 50 cm deep and doesn't reach the minimum freezing depth of 75cm **[6.18]**. They have deteriorated over time due to moisture and small depth of foundation, smaller than the depth of freezing. The binder of stone blocks comes off easily and does not ensure their cohesion. Wooden beam over the stone foundation are rotting due to moisture in the ground (fig. 6.7a) and are rotated and broken due to different settlements of the foundation (fig. 6.7b). In these wooden beams are fixed the columns. Because of the settlements, the joints between beams and columns are damaged both at the bottom and at the top (fig. 6.8).



a) b) c) d)

Fig. 6.6 Bearing structure 3D model of churches a) Curtea, b) Crivina de Sus, c) Romanesti, d) Dobresti

a)  b)  c) 

Fig. 6.7 a), b), c) Degradations and breaks of the horizontal wooden beams over the stone foundation

Fig. 6.8 Joint between beams and columns damaged

At nave and at the altar the arches have three joints and are realized of rectangular section wood. The dimensions of the arch section are 10x10 cm and because of their poor stiffness, can be seen significant vertical deflections which lead to degradation of the ceiling paintings. Due to settlements and deformations of the perimeter bearing structure, the arches have broken elements in the joint area and do not ensure the continuity of the elements (fig. 6.9). There were also observed breaks of the joints between horizontal wooden beams disposed perimeter which support the roof framing. The wooden rafters of roof framing are deformed because of their small section of 12x14 cm. Some wooden rafters are rotten because of rainwater penetration (fig. 6.10). The bell tower is supported by four wooden beams of small rigidity with dimensions of the section 20x20 cm. The tower is made of four wooden columns with vertical wind bracings. Because of the vibrations introduced by the bell, important horizontal and vertical displacements can be observed at the tower.

Fig. 6.9 Breaks of joints between the wooden horizontal beams

Fig. 6.10 Rotting of wooden rafters

The settlements, winds and earthquakes weaken the joints which are not rigid and so, the structural elements rotate and move on any direction. There are joints reinforced with metal elements but the wood is rotten so the joint is weakened.
Can be concluded that due to all this actions as settlements, vertical and horizontal forces, and the fact that the elements don't ensure a good spatial behaviour, the structure shows important displacements and in the bearing elements the strain values exceed the maximum allowed values.

6.1.1.3 Strengthening solutions

Strengthening solutions aim to increase the bearing capacity of bearing elements, to increase capacity of seismic energy dissipation and to increase the level of seismic risk. The majority of the chosen consolidation solutions were tested experimentally and theoretically within the European Research Contract PROHITECH FP6 "Seismic Protection of Historical buildings by reversible mixed Technologies", and were published in international journals **[6.19]**, international conferences in the restoration field **[6.20][6.21][6.22]**. The studies were continued by the author within the contract COST FP 1101 "Assessment, Reinforcement, and Monitoring of Timber structures", where the author is the contract director for Romania **[6.24]**. To ensure the spatial behaviour of the structure and to keep the structure stiffness unchanged after realizing of consolidations was also aimed. For this purpose, the proposed interventions are reversible, using metal, epoxy resin and original wooden materials (fig. 6.11). The strengthening solutions will be executed at the exterior of the church to avoid damaging of paintings realized on the wooden interior walls and will be made at ground level and at roof framing.

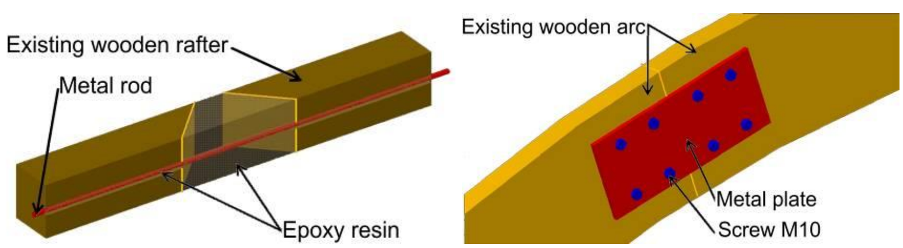


Fig. 6.11 The consolidation solution of the wooden arches and rafters

The consolidation solutions were established based on the efforts and displacements from the analysis using AxisVM software. For a better understanding of the structural behaviour under the seismic load, wind and snow, there was made a 3D simulation of the response of the church.
Both the consolidated and unconsolidated versions of the church under horizontal and vertical loads were subjected to 3D analysis in the elastic domain. Nonlinear analysis for the material

was not performed because wood is an anisotropic material due to the fibres and inhomogeneous due to the presence of defects in the wooden body. It is difficult to simulate with the aid of computer programs the real behaviour of degraded wood in such a construction **[6.4]**. The simulation is difficult to achieve due to particular way in which the connection between the elements was made, using different types of wooden nails and different positioning of the bearing elements in the nodes. In these cases, churches developed different failure mechanisms.
The bearing elements were outlined in the form of bars. The panels between the columns were considered as a series of horizontal bars with hinges at both ends in the area joining the columns. All the connections were considered to be pinned between the elements in the unconsolidated version. For the control of displacements due to damages, the dimensions of the sections were considered to be reduced. To check the efficiency of the consolidation solutions, static and dynamic calculations were performed even for the consolidated version.
The stone masonry foundation will be strengthened by replacing of deteriorated mortar between stones with higher resistant mortar. To avoid disturbing the ground under foundation, no casting of concrete under foundation was made. The foundations will be connected with metal ties. The bottom wood beams, on foundation, will be completely replaced with beams of the same size and same material, which will be installed in parts. If possible it will be reused after the necessary treatment. Continuity of the parts will be provided through metal rods inserted in the wood and protected with epoxy resins (fig. 6.12).

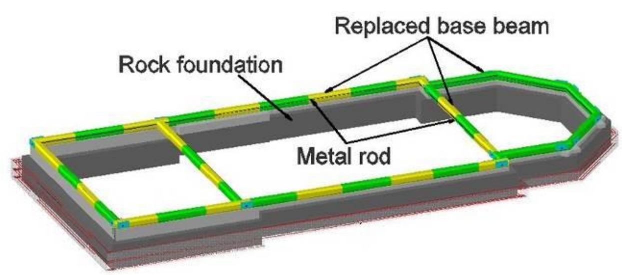


Fig. 6.12 The consolidation solution of the foundation and of the base beam of the church

At levels -0.28 m and +2.80 perimeter and transversal tie rods were mounted, in a cave made in wooden beam and filled with epoxy to ensure protection and bounding with wood beams.
Between the ground and floor will be left ventilation channels which eliminate accumulated rainfall moisture. Perimeter drains will be made.
On the wall height will be strengthen the joints with metal parts or restore with wood nails as historical technology.
The roof framing will be strengthened with wooden and metal bearing elements as shown in figure 6.13. Each wooden arch of the existing nave is consolidated by connecting to new arch made of oak wood (fig. 6.14c). The painted wooden ceiling is not connected to the new arches. The weight of the arches is taken by the rafters trough metal vertical rods. Perimeter wooden beams at the level +2.80 m are strengthened by suspending them to new metallic profiles placed over wooden beams. With rigid joints between steel beams is ensured the rigidity in horizontal plane (fig. 6.14a). All joints are made with bolts to prevent a fire during execution.
In the perimeter wooden beam, at level +2.80 are mounted two additional tie metal rods positioned under these two strengthened beams (fig. 6.14b). Roof framing was strengthened with wooden planks disposed only on one side of the rafter or by partial replacement of the rafter with

same kind of wood from other parts. Some areas will be restored with metal and epoxy resins. Damaged joist will be strengthened with wood and metal parts. To restore the initial historical solution, metal sheet roofing will be replaced with shingle roofing.

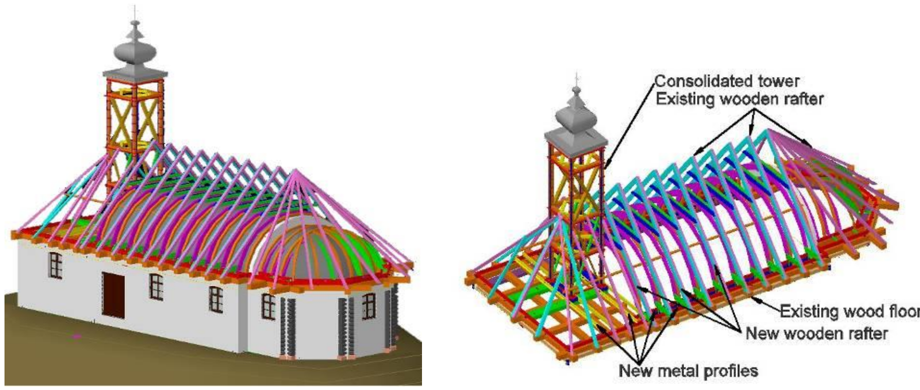


Fig. 6.13 Consolidation solutions of the arches, rafters and wooden beams of the church

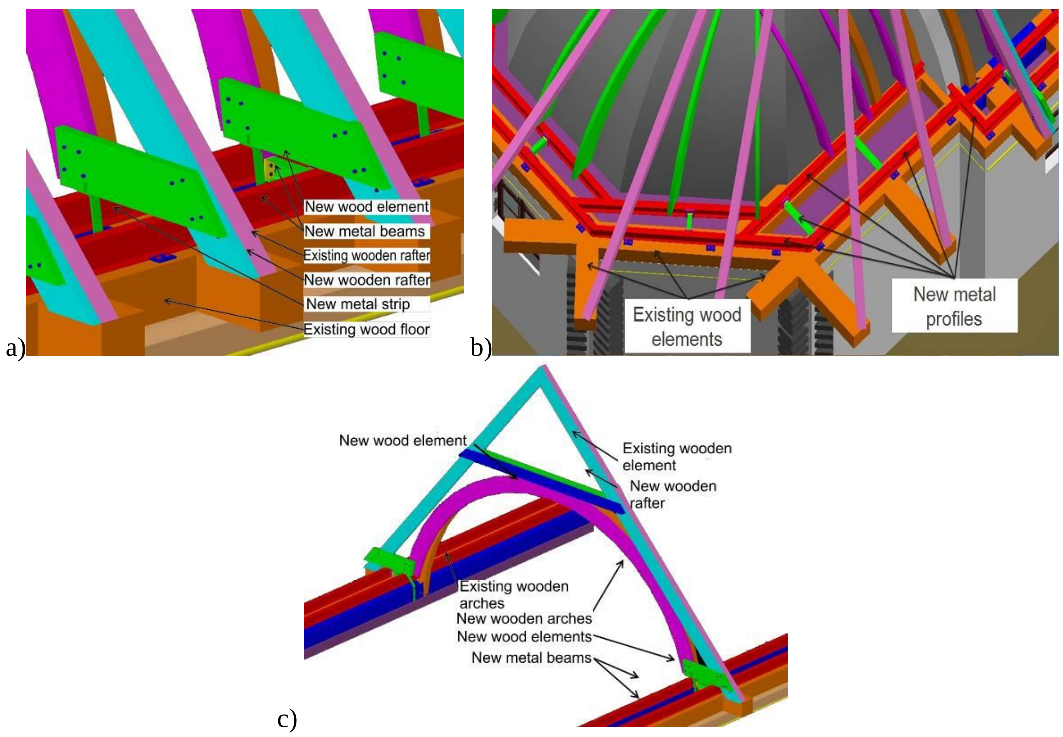
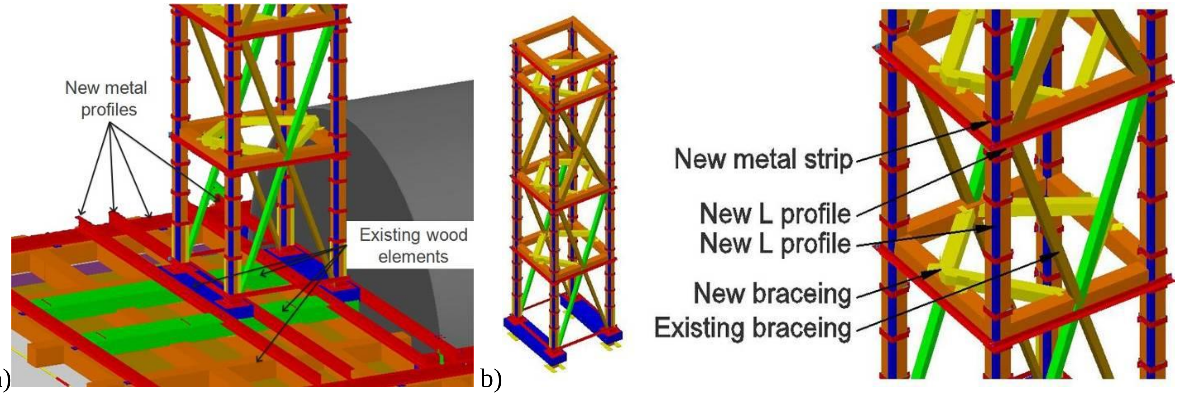


Fig. 6.14 a), b), c) Consolidation solutions for beams and arches from nave and altar

Bering capacity of the tower will increase by strengthening of wood beams and columns with steel profiles. The wooden beams which support the columns of the tower will be strengthened with steel profiles (fig. 6.15a). The stiffness will increase after mounting vertical wood or steel wind bracings between columns and strengthening of beam-column joints with metal pieces. Lateral horizontal displacements will decrease by realizing of horizontal bracings made by wood beams at levels +5.38, +7.28 and + 9.70m (fig. 6.15b). Vertical displacements of the tower will decrease by strengthening of wooden beams which support tower with steel profiles. The bell will be dismantled and will be mounted outside the Church in the bell tower to eliminate the vibrations introduced in the bearing structure.



a) b)

Fig. 6.15 a), b) Consolidation solutions for the wooden columns and beams of the tower of the church

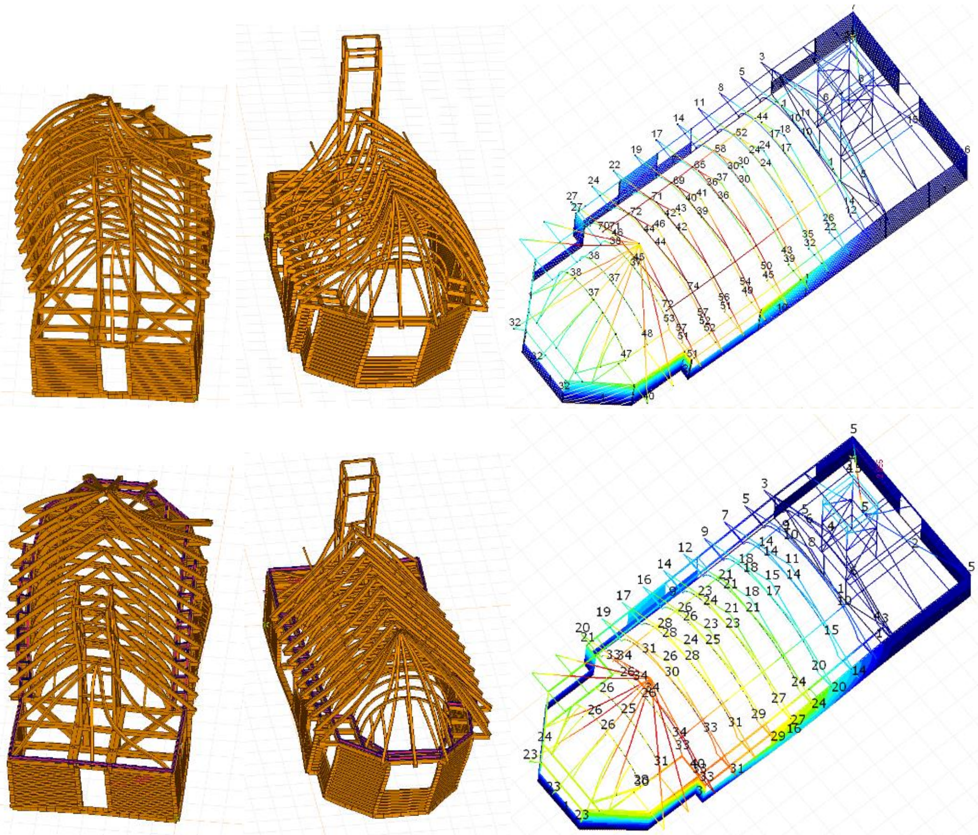
The consolidation solutions of the bearing structure were established based on the obtained values of efforts and deformations from 3D analyses with AxisVM software in linear elastic domain. The author has presented comparatively the values of the efforts and registered displacements of the bearing elements before and after consolidation.

The bearing structure was modelled with linear elements of frame type. For each element were defined the physical-mechanical characteristics of oak wood and the dimensions of the minimum cross-section measured. Combinations of loads including self-weight, wind, snow and seismic load were introduced. The maximum efforts and displacements were obtained from the combinations with self-weight, snow and wind. Due to the fact that the inertial mass of the church is not very large and the seismic area has the peak ground acceleration a=0.12g, the efforts resulted from the combinations with seismic load are reduced. The monks and the locals have confirmed this hypothesis, stating that the degradations appeared mostly in time of winters with snow and not after the earthquakes from 1991.

By strengthening of roof framing was ensured a good spatial connection between bearing elements. In figure 6.16 are shown comparatively the values of the horizontal maximum displacements before and after consolidation. It can be observed that while at the bell tower the displacements are slightly reduced, in the nave area the maximum displacements are reduced about 54% and at the sanctuary about 32% because of the presence of steel profiles and strengthened rafters. The maximum values of horizontal displacements are shown in Table 6.1.

Table 6.1 – Maximum horizontal displacements

Areas	**Unconsolidated [mm]**	**Consolidated [mm]**	**%**
Narthex	6	5	-17
Nave	74	34	-54
Sanctuary	38	26	-32



a) b)

Fig. 6.16 Deformed shape of the church: a) before and b) after consolidation

The same tendency of reducing of efforts values after consolidation appears at the columns. The axial compression efforts are reduced about 10-13% and the bending moment efforts are reduced about 13-20% as shown in Table 6.2. The axial efforts N at the columns are shown in figure 6.17 and the bending moments efforts M are shown in figure 6.18. It can be observed increased values of the bending moments at the columns from the separation area between sanctuary and nave, in the joint area.

Table 6.2 – Maximum values of axial forces and bending moment in columns

Areas	Axial forces N_{max} [kN]			Bending moments M_{max} [kNm]		
	Uncon-solidated	Conso-lidated	%	Uncon-solidated	Conso-lidated	%
Narthex	55	48	-13	8	7	-13
Nave	48	44	-9	5	4	-20
Sanctuary	35	49	+13	5	4	-20

The stress values, bought compression (fig. 6.19) and tension (fig. 6.20) from the columns, also decrease after consolidation, excepting the columns from the nave. Nevertheless the compression stress values do not exceed the maximum allowed stresses as can be seen in Table 6.3.

Table 6.3 – Maximum values of compression and tension stress in columns

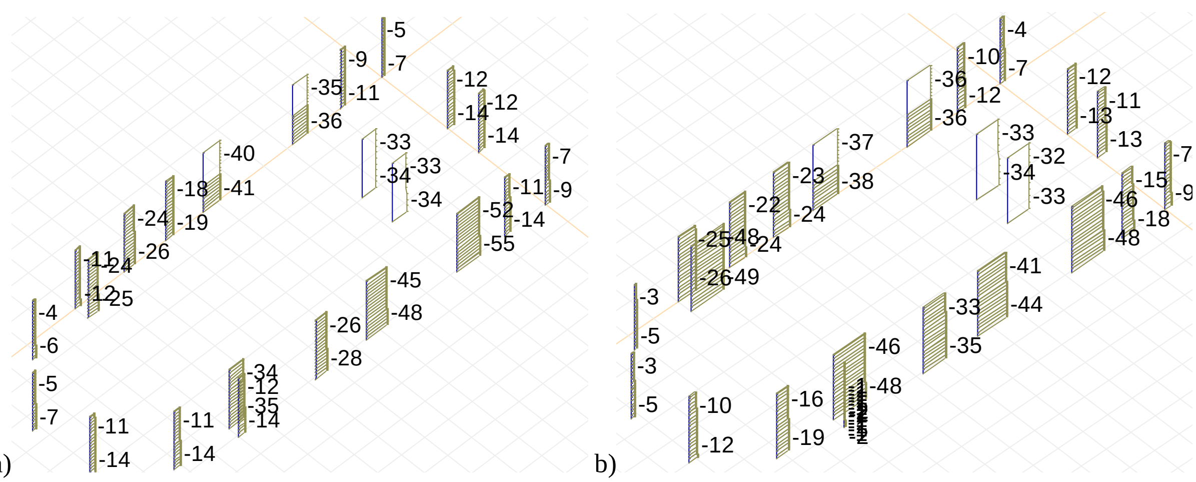
Areas	Compressions stress [dan/cm^2]			Tensions stress [daN/cm^2]		
	Uncon-solidated	Conso-lidated	Allowed	Uncon-solidated	Conso-lidated	Allowed
Narthex	79	67	130	68	47	90
Nave	48	65	130	40	53	90
Sanctuary	265	200	130	258	194	90

The only columns whose stress values exceed the maximum allowed stress value before and after consolidation are the columns from the sanctuary, at the extremity of the Church. These values are registered only at the extremities of the columns. In these areas, the joins between columns and horizontal beams, over the foundation and from the roof framing, must be strengthened. So can be explained the brakes of the joints.

By comparing the maximum stress values from bending, shown in Table 6.4 and figure 6.21, at the half span of the rafter, can be observed that the proposed strengthening solutions determine a very important reduction of the stress values in the nave and narthex rafters and a less important reduction of stress values at the sanctuary rafters. So it can be explained the presence of rafters with large deformations in this area. Nevertheless the maximum stress values from bending in rafters, before consolidation do not exceed the maximum allowed stress value.

Table 6.4 – Maximum values of stress in rafters

Areas	Stress [daN/cm^2]		
	Unconsolidated	Consolidated	Allowed
Narthex	168	33	130
Nave	105	4	130
Sanctuary	117	87	130



a) b)

Fig. 6.17 Maximum values of axial forces N in columns: a) before and b) after consolidation

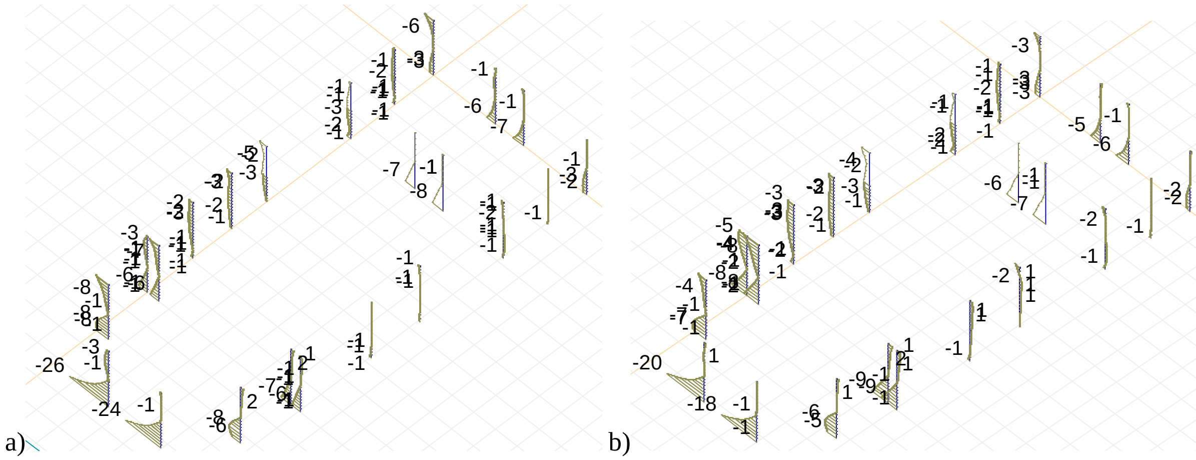


Fig. 6.18 Maximum values of bending moments M in columns: a) before and b) after consolidation

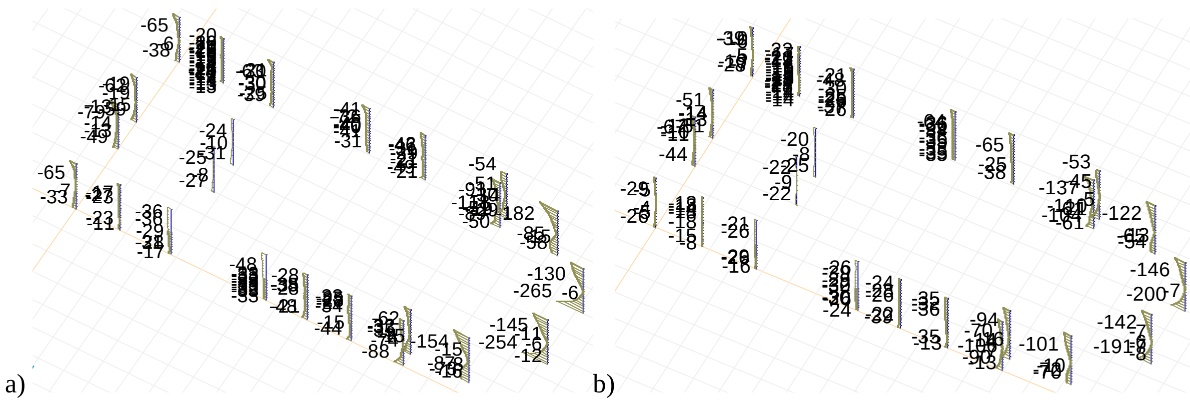


Fig. 6.19 Maximum values of compression stress in columns: a) before and b) after consolidation

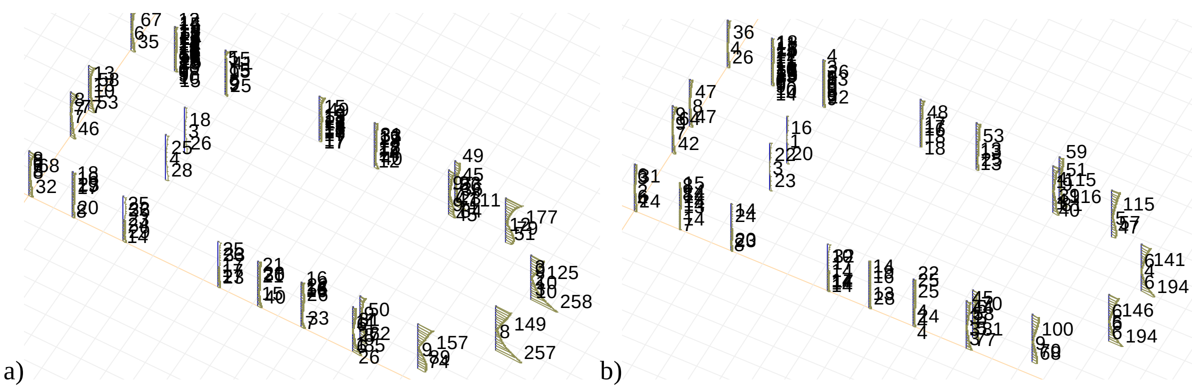
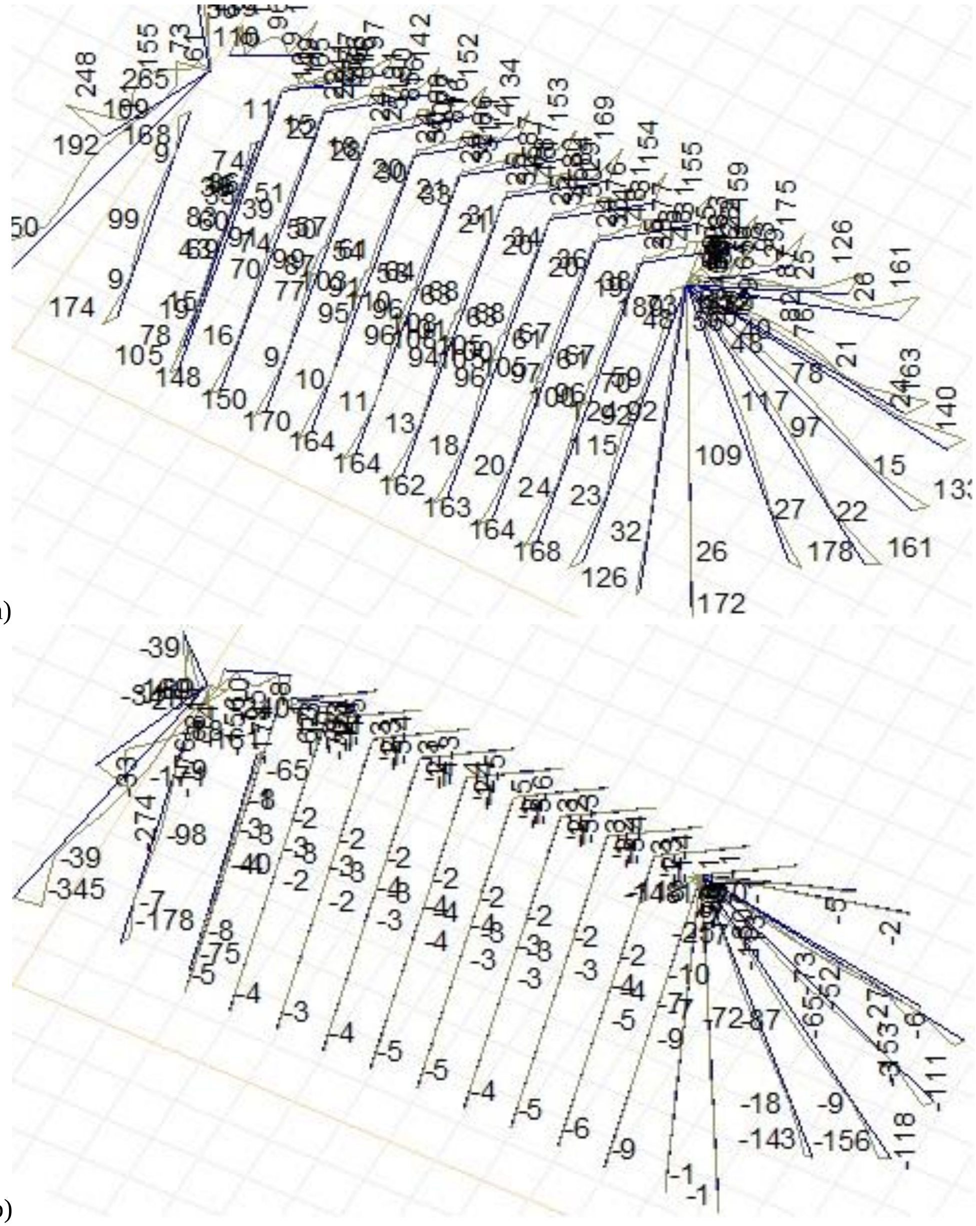


Fig. 6.20 Maximum values of tensile stress in columns: a) before and b) after consolidation



a)

b)

Fig. 6.21 Maximum values of stress in rafters: a) before and b) after consolidation

6.1.1.4 Conclusions

- Wooden churches can develop different failure mechanism due to the particular historical manufacturing technologies of the bearing structure and of the connection between bearing elements.
- The procedure of strengthening the bearing structure of wooden churches in Romania is very difficult because the painting is applied directly on walls and ceilings wood.
- In the future it is necessary to develop methods and techniques for the structural system and painted surfaces restoration of wooden churches located in seismic areas.
- The author has presented for the first time in Romania, based on static and dynamic analysis of these historic buildings, consolidation measures in case of emergency for churches of cultural heritage based on the reversibility principle, developed within a European research contract.

- The presented strengthening solution is original, reversible and solves the bearing capacity and stiffness of the structure requirements. The presented solution brings novelty at an international level due to the originality and uniqueness of the analysed constructions.

6.2 Roof framing systems

6.2.1 Historical wood framing systems from Banat region, Romania. The Great Synagogue from the Citadel

6.2.1.1 Architecture

The Great Synagogue in the Citadel is one of the most spectacular buildings of the city **[6.23][6.24]**. In 1863 the Jewish people were granted the full civic rights; in the same year the community started to build the Great Synagogue, the Citadel, on the former grounds of the Franciscan monastery garden. Built under the direction of the architect Carol Schumann from 1863 to 1864, it was inaugurated in 1864; another ceremony was held during the Emperor Franz Joseph I visit in 1872. It was fully functional until 1985; since 2001 the Philharmonic Society organised musical events and made several proposals with the help of WMF to restore the monument.

The ongoing project for the restoration of the Synagogue and its conversion into an auditorium revealed the complexities of the wooden structures – the lightweight and delicate balconies were able to sustain hundreds of worshippers for more than a century: the complex roofing system, stretched between the exterior tympani, the skylight of the dome and the exterior profile of the vaults, display elegance and minimalism. Both structures withstood the test of time, neglect and constant seismic activities.

The synagogue is a domed masonry cube, three stories height. An oculus on the top of the cupola brings the light inside. Four arched aisles “pack” the main cubic volume. Four towers flank the square plan, containing the stairs to the upper balconies. The two towers on the front façade are covered with cupolas and flank a stone Moorish horseshoe arch supported by Corinthian columns. The main façade features an eight pointed star rose window and religious symbols. The materials are brown brick and deep blue ceramic ornaments. The other three identical facades are partially plastered and crowned by triangular pinnacles. The roof – mostly invisible – was shaped with a skylight at the central crossing.

The Moorish deep-blue ornamentation continues into the majestic inner space crowned by the massive cupola. Three of the four aisles correspond to the three pinnacle facades; the entrance aisle is rendered as a three arched interior façade.

Inside the ornate masonry box, there is a hidden gem: the magnificent to tiers wooden balconies and, the most surprising of them all the roof structure. The balconies being surrounded on three sides by the massive brick box, there is no need for stiffening the delicate, graphic structure. The third balcony was added later, in the 1920’s, over the arched entrance façade and under the pipe organ platform. It was meant to reproduce the slenderness and elegance of the lateral balconies but without the encasement. The anchorage into the brick structure is more massive and metal brackets are connecting this balcony to the inner façade.

Forced between the outer shells of the cupola, the supporting arches and the geometrically inflexible cruciform shape of the roof, managing also the massive oculus and skylight – a marvel of a roofing structure appears.

6.2.1.2 Description of the bearing structure

Until 1716, Romanian worship places from this area had a wood bearing structure. After 1716, brick masonry bearing structures for the Catholic and Orthodox churches began to be built in this area, but the architecture was to be realised in the Baroque style. The load bearing structure of the Synagogue differs from the other churches in Timisoara built at that time through plan and elevation shape, and the opening dimensions. In plan, the initial load bearing structure is shaped like a cross. Two staircases in the shrine area were added later, resulting in a very stiff structure having in the staircases area four masonry tubes in extremities. These staircases work together with masonry, metallic ties, wooden floors from the balconies and the roof framing of the building. The masonry was made from bricks with lime mortar. In plan the maximum dimensions of the building are 25.80x25.24 m. The walls and the foundations are made from bricks and have dimensions between 100-130 cm. As it can be noticed from figure 6.22, the diameter of the central dome is 22.60 m, the balconies having the span of 11.30 m and the width of 5.60 m. The structure of the balconies is realised from wooden frames with the maximum height of 120 mm and 5.60 m span, and distributes the load to the cast iron columns (fig. 6.23). Wooden floors of the balconies are coffered, in order to provide a high stiffness in its plane and work together with metallic ties. The floor at level +14.25m which overtakes the loads from the central brick dome (with the maximum height of 4.85m), is made of brick vaults. At the bridge level, the perimeter brick masonry has the maximum height of 5.60 m and is fixed to a framework with metallic parts. The arrangement of the load-bearing elements of brick and wood masonry in the Synagogue building is shown in figure 6.24.

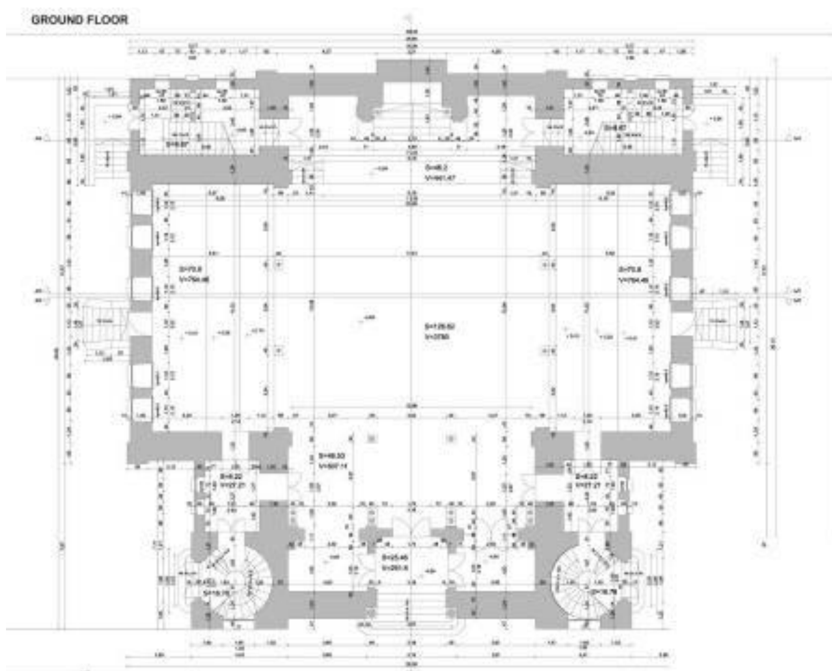


Fig. 6.22 Ground floor horizontal section of the building

Fig. 6.23 Wooden truss beam at the balconies

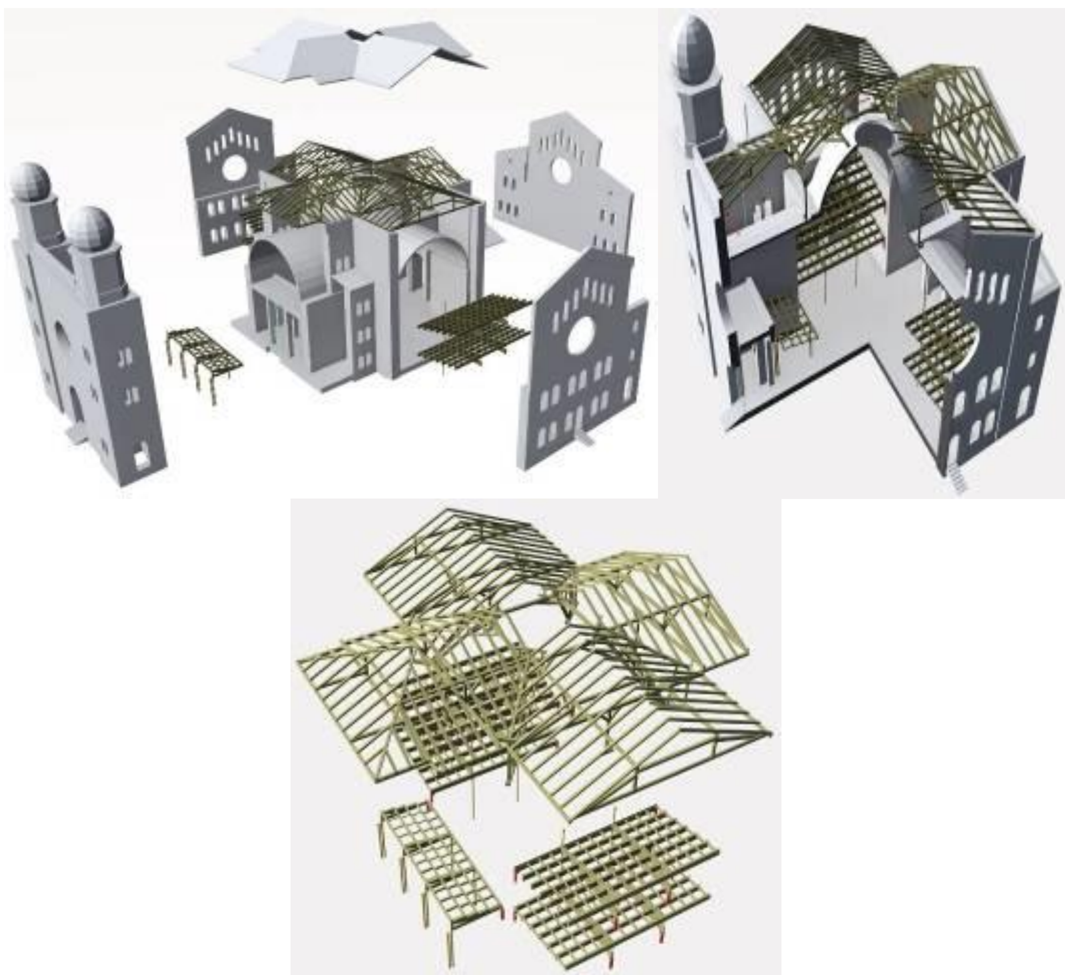
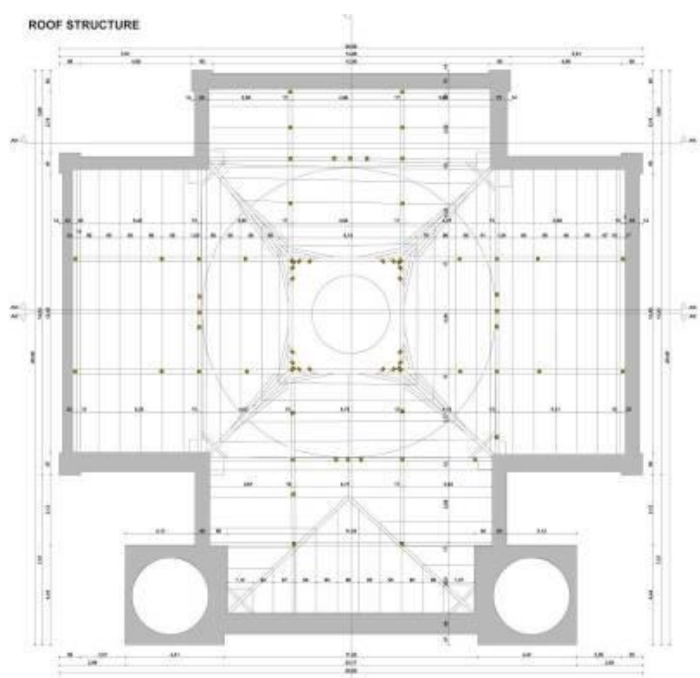


Fig. 6.24 Arrangement of the load-bearing masonry walls and of the wooden elements at Timisoara Synagogue

The plan shape and section of the roof framing are presented in figure 6.25. Framing is made of oak wood cut from the nearby forests of Timisoara. The maximum height of the roof framing is

6.50 m in the balcony and central dome area. Columns, beams, rafter have dimensions of 15x15 cm, 17x20cm respectively 15x17 (with a spacing of 75 cm between rafter). The framing in the balcony and shrine area transmit the vertical loads through columns to the horizontal beams, which are supported on the perimeter load-bearing masonry walls (fig. 6.26).
In the dome area, the roof structure consists of four inclined columns, which transmit the loads directly to the junction of perimeter walls, in order to avoid transmitting the load to the central brick dome of 22.60m diameter (fig. 6.27).
The roof framing is elegantly highlighted by the way in which the intersection of inclined columns with horizontal wooden beams was solved. They form a square in the central dome area. By introducing some supplementary beams to this intersection, it can be obtained not only an efficient transmission of the loads, but also a high stiffness of the roof framing in all directions. This way of transmission was possible by imposing optimal distances between all perimeter walls as a result of a right and effective compliance concept of the bearing structure from the foundations up to the roof framing structure. Thus, resulted a stiff roof structure with a high bearing capacity from both structural and architectural considerations.
Through a correct dimensioning of timber elements sections, imposing correct details of timber-timber and timber-metal joints, and also providing some rigid links between framing and brick attic (fig. 6.28), the result is a space efficient cooperation between supple and elegant framing and perimeter cantilever masonry, which for 148 years has not developed collapse mechanisms. Today is recorded local degradation of roof wood due to poor maintenance between 1980-2005, when through broken tiles, water infiltrated into the central dome of the building.



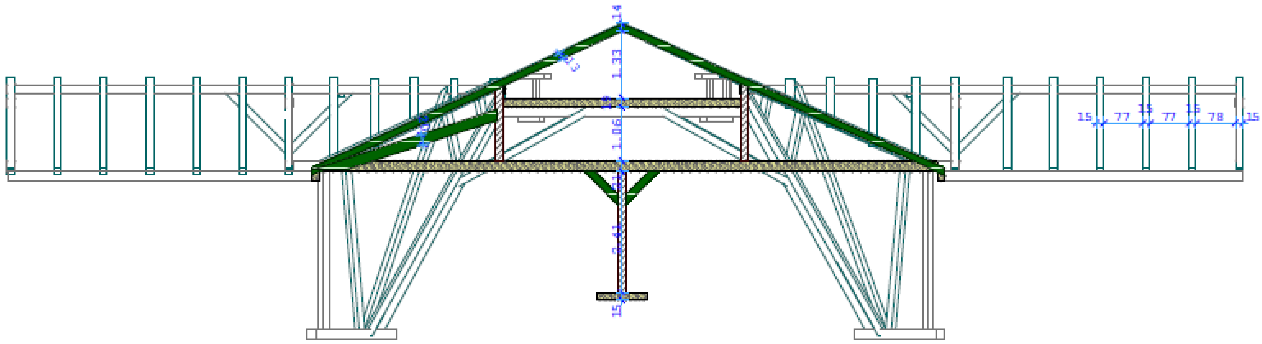


Fig. 6.25 a) Arrangement plan of the roof framing elements b) Transversal roof framing section

Fig. 6.26 Framework details and element joints in the balcony area

Fig. 6.27 Framework details and element joints in the central dome area

Fig. 6.28 Detail of building framework post in the perimeter load bearing masonry

6.2.1.3 Bearing wooden structures influence against the spatial behaviour of Synagogue

The analysis of the bearing structure was explained in the European Research contract COST FP1101, where the authors are members in this program **[6.23][6.24]**. In the 148 years of existence, after 40 earthquakes, the building did not register significant damage. This is due to a correct conformation in plan and elevation, use of brick structural walls with correct dimensions and metal ties placed in the masonry thickness. Besides these factors, an important role in the seismic response of the structure had the load-bearing timber structures from the balcony floors and roof structure. The balconies have ensured a smooth rigid floor transmission of seismic forces between the 4 brick masonry tubes, through metal parts and ties which ensured cooperation of masonry and wood-panelled floor. Currently, there are no observations of wooden beams failure or degradation of the joining area of metal parts with wooden beams. Through the dimensions of wooden beams and their arrangement, these floors together with metal parts ensured the working with the brick masonry. A very important role had the structural wood framing. Through the shape in plan, arrangement of wooden beams, their dimensions and the ways of combining them, a high stiffness element on upper size of the construction has been obtained. Thus, the upper part of structural brick walls in the roof structure is secured against failure out of plane by setting it with metal pieces on the Synagogue framing, avoiding thus brittle failure modes. This is demonstrated by the absence of cracks in masonry walls at the attic level and by making brick masonry around wooden roof structure elements a good cooperation roof framing-brick masonry is obtained.

After 2001, several attempts were made to start the process of restoration / reconversion of the Synagogue in the Citadel World Monument Fund supported a comprehensive survey of the entire structure which underlined the surprisingly good state of preservation of the entire structure, including the timber parts: the roof and balconies. Only local damage due to neglect appeared.

Two proposals for the reconversion of the synagogue were elaborated: one more complex, including wrapping of the synagogue in a steel and glass structure containing public spaces and another more basic, a landscaped and shaped reshaping of the courtyard including small pavilions for the ancillary spaces.

Minimal interventions were also to be considered for the roofing structure, whose excellent configuration with the V shaped rafters rising over the cupola and the cross connection to the corner towers, and excellent craftsmanship – prevented any major damage during several earthquakes.

6.2.1.4 Conclusions

- The good configuration and – above all – the excellent cooperation of the timber structure with consistent brick case ensured not only the elegance but also the reliability of those structures during 100 years of continuous use, several earthquakes, and over 40 years of neglect;
- In general, design codes neglected real contribution brought to stiffness and bearing capacity of historical buildings made of wood structures located in areas of low seismicity. In this study it is presented the case of a brick masonry building without reinforced concrete elements, subjected 40 times to seismic action, that in a correct conformation of wood floors from intermediate levels and roof structure elements, and efficient use of joints with metal parts between masonry and wooden structures, has shown that the occurrence of significant damage to masonry load-bearing walls can be prevented, although the span of arches and dome are important. Through great seismic response of Synagogue building in Timisoara, it is highlighted the importance over the years in historic bearing structures of floors and roof framing wooden structures together with ties and metal joining pieces. This compound effect: timber beam-metal tie-brick masonry is a very effective structural system in low seismicity areas, whose behaviour must be correctly understood by further studies in the future in this area;
- The scientific contribution of the study performed by the author together with a team of architects emphasizes that historic wooden bearing structures were considered as simple, supple and elegant solutions to increase the bearing capacity and stiffness of such buildings located in seismic areas;
- The study brings original information in the domain of vulnerability assessment of timber and masonry structures situated in seismic zones.

(b-ii) Scientific, professional and academic future development plans

As presented in (b-i), after the completion of my PhD thesis in 2004 until the present time, within the team of the Faculty of Architecture from Timisoara and the design office H. I. STRUCT SRL, I have performed research, design and academic activities in the field of design and consolidation of historic and modern bearing structures made of: reinforced concrete, steel, masonry and timber. The future development plan of my career is based on the principles and experience gained during 15 years of collaboration with my mentor, Prof. Victor Gioncu, the creator of the engineering design subjects for bearing structures within the Faculty of Architecture from Timisoara. The teaching activity for a professor is and will be a complex activity for an engineer, due to the fact that in the collaboration with students and fellow architects, one must rapidly identify a bearing solution for the proposed structures. The chosen solutions must satisfy aesthetical, economical and logistical requirements and must be easy to be implemented, indirectly forcing the engineer to be updated about latest technologies and research. For this, I will continue the multidisciplinary research with engineers, architects, restorers, painters, archaeologists, historians, chemists, software engineers. In the future, according to the development plan, I will intend to develop my career in the following academic, professional and research directions:

A. Design and consolidation of reinforced concrete structures

Reinforced concrete structures have a large coverage in Europe, especially for industrial, civil, agricultural buildings and bridges. Since the beginning of the XX^{th} century these structures were used due to the high resistance of concrete and due to the fact that elements having artistic shapes could have been casted, thus satisfying the aesthetical requirements.
From historic point of view, in Romania there are the following types of reinforced concrete bearing structures:

- historic r.c. structures, listed as monuments, erected between 1900 and 1940;
- r.c. structures with historic value, not listed as monuments, erected between 1945-1977, before modern provisions for seismic design;
- modern r.c. structures, erected after 1977 until the present time;

I. Future objectives

I will propose some new directions of international research based on theoretical and experimental research by:

- Elaboration of rapid evaluation methods for the vulnerability of r.c. historic buildings;
- Preparation of a database which should contain historic information on reinforcing, type of reinforcements, concrete recopies used for the construction of r.c. elements;
- Development of reversible consolidation methods according to the provisions of the Chart of Venice for historic buildings;
- Proposal of consolidation methods of RC frames having masonry infills, placed in seismic areas, erected between 1950 and 1977 with no seismic design. Due to the large number of these buildings, and because the classical consolidation methods require a long time and the evacuation of the people, new rapid exterior consolidation methodologies are required.

- Elaboration of a calculation methodology and innovative technologies for the construction of brick masonry in frame structures;
- Elaboration of methodologies for the calculus, reinforcement, and consolidation of r.c. shear walls with staggered and ordered openings, based on the behaviour recorded by these walls after the Chile earthquake from 2010;
- Study of the seismic behaviour of composite walls with staggered and ordered openings, in function of the position and dimension of the steel profiles placed centrically or eccentrically inside the wall, as well as the reinforcement layouts;
- Analysis with the help of computer software of the effect of the loading speed of the seismic waves and the large values of the vertical components of the seismic acceleration on the failure modes of the r.c. structural walls;

II. Impact of the study

The directions of research proposed by the author in the field of r.c. bearing structures will bring information in new research domains on national and international level and will facilitate the elaboration of design provisions for consolidation, construction for historic and modern bearing structures. Elaboration of a rapid evaluation methodology and consolidation of historic r.c. bearing structures constitutes a future research direction for specialized organizations such as ICOMOS. Research performed by the author are original, in the domain of r.c. frame structures will bring new information for the reduction of the seismic vulnerability of these structures. It will propose new construction technologies and calculation methods of infill walls capable to withstand out-of-plane actions, aiming to complete the seismic design codes at European level.

III. Methodologies for performing the research

(i) By creating a database which should contain data about the type of r.c. historic buildings, recorded structural damages, types of materials used, technologies of construction and by identifying the forces which acted on these building, a rapid evaluation of the vulnerability will be proposed. Only through an exact knowledge of these buildings from archives, in situ and laboratory testing on the concrete and reinforcement, statistics, and by tracking the evolution in time and state of stresses and deformations of the building, can important contributions in the domain of conservation of historic bearing structures be found;

(ii) The seismic behaviour of frame structures with infill walls will be studied in the laboratory by experimental testing and with the help of computer software for the calculus in the elastic and post-elastic domain. The out-of-plane failure mode of the infill walls will be tested on two categories of walls made from ceramic blocks with vertical holes: a simple wall as a reference, and a wall having a BAUMIT thermo-insulation system (Fig. 7.1). The efficiency of the consolidation solutions for these walls will be tested on the first two damaged walls, using mesh polypropylene bands on the exterior face of the walls and using a GeoSteel grid mesh developed by KERAKOLL **[7.1]**, placed also on the exterior. The testing procedure will be as follows:

- Establishing, based on current available international bibliographic references, the experimental models. In this stage, the type of experimental stand was defined, the dimensions of the walls and the resistance of all the materials were established, and also an agreement on the loading protocol (Fig. 7.2) was reached in accordance with ECCS **[7.2]**.

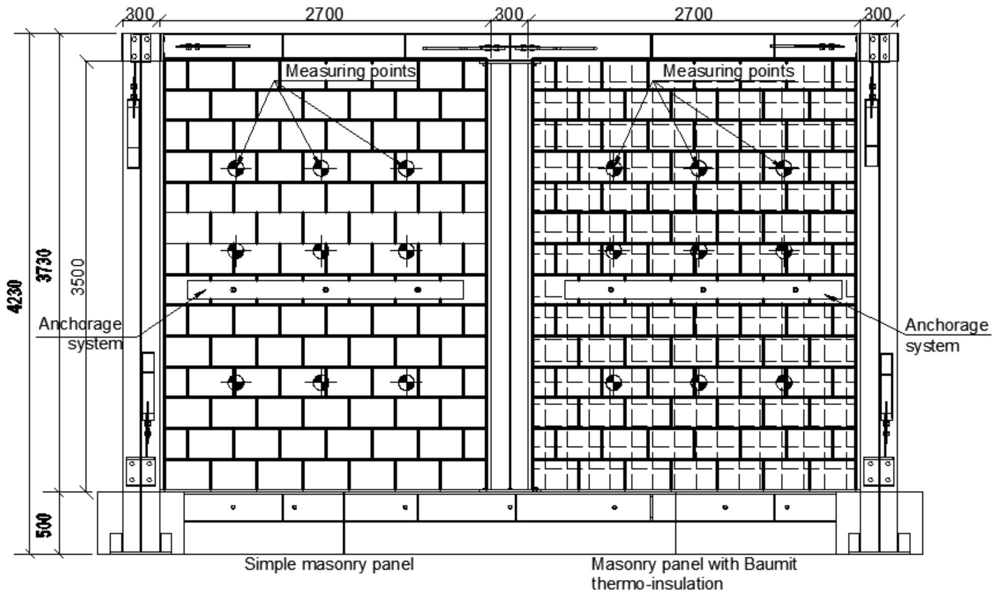


Fig. 7.1 Dimensions of the wall specimens

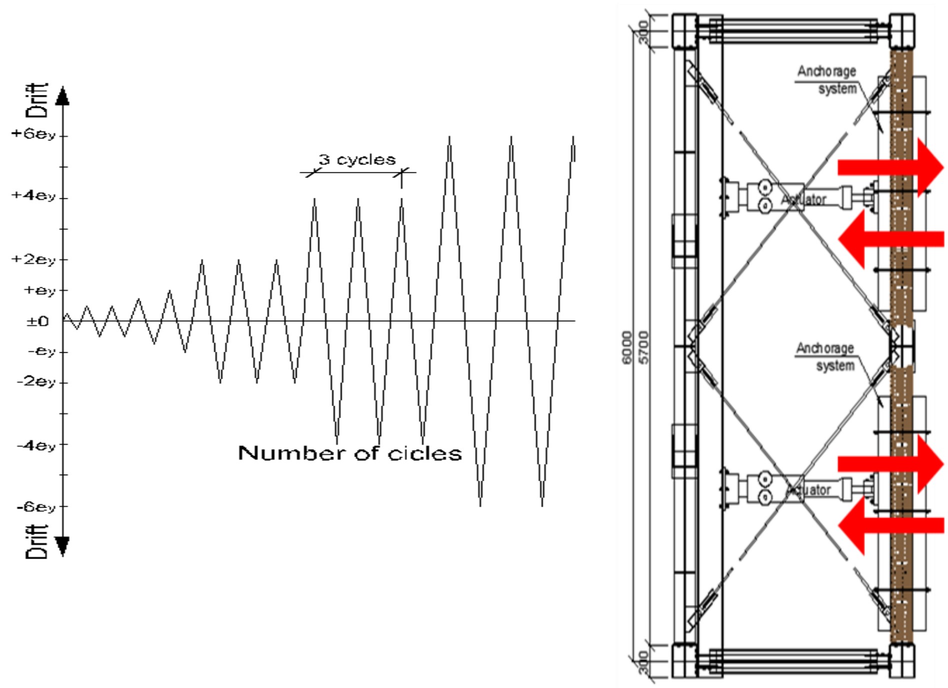


Fig. 7.2 Loading protocol and testing procedure

- Analysis of the seismic behaviour of the masonry walls loaded by a cyclic alternating force with the following computer software: SAP 2000 **[7.3]** in the elastic domain, and in the post-elastic domain with ATENA **[7.4]** by horizontal displacement control. There will be determined the maximum displacement for the elastic domain, the state of efforts and the deformed shape for each loading step, evolution of the cracking state and finally the failure mechanisms. There will be analyzed the simple reference wall and the one with the thermo-insulation system already applied in order to highlight the contribution of the glass-fibre mesh on the bearing capacity of the wall;

- Construction of the experimental models will be done after a clear definition of the elastic limit, loading protocol and experimental stand;
- A report presenting the results obtained from the experimental test will be prepared, and the first consolidation recommendations will be established;
- A theoretical analysis will be performed with the help of FEM software in order to establish the seismic out-of-plane behaviour of the two damaged walls consolidated with PPB (polypropylene bands), as it can be seen in figure 7.4, and with the GeoSteel grid, as it can be seen in figure 7.3. The failure modes will be identified; the state of efforts and the deformed shape for several loading steps will be recorded. The elastic limit and the value of the ultimate force and displacement will be identified. A new testing methodology will be proposed for the consolidated walls, and all the execution details will be finalized;

Fig. 7.3 GeoSteel grid developed by KERAKOLL can be used for RC elements and masonry walls

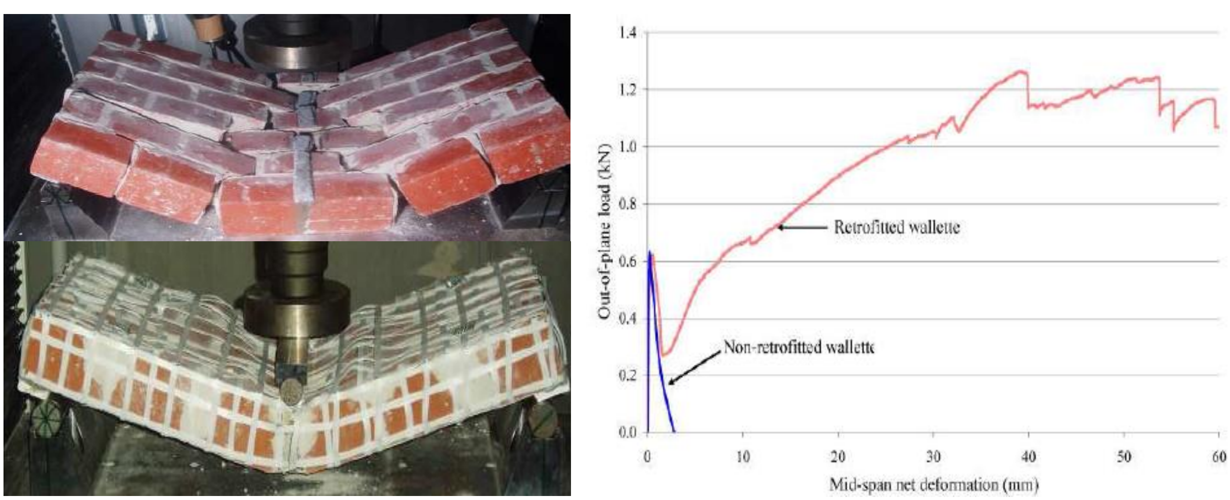


Fig. 7.4 Failure patterns of brick masonry wallets with and without retrofitting by PPB mesh and the out-of-plane load variation **[7.5]**

- A workshop will be organized in which there will be presented the constructive and execution methods for two experimental models: one with PPB and one having a GeoSteel grid system;
- The experimental testing will be performed and the state of efforts, deformations and cracking state of the elements will be recorded in each loading step and the failure mechanisms will be identified;
- A report based on a comparative presentation of the theoretical and experimental results will be drafted for all testing specimens;
- A methodology of calculus for the design and consolidation of these walls will be proposed ;
- The final research report will be prepared in which all the activities, conclusions on the seismic behaviour of these consolidated and unconsolidated walls subjected to a cyclic alternant motion and methods of calculation for design engineers and constructive recommendations will be shared with design firms and companies producing building materials;

(iii) The seismic behaviour of r.c. shear walls with staggered openings represents a complex research activity which will aim to establish calculus and design methods for these walls, having various constructive details. These studies will open new international directions of research because they will study particular types of walls that are not foreseen in the design codes and thus there is no information about their seismic behaviour. Reduced financial efforts, capability to overtake seismic forces and the ease of construction, represent the motivation of studying these types of walls. Based on the results obtained by the author and Assoc. Prof. G. Tamas Nagy, Prof. D. Dan, coordinated by Prof. V. Stoian, future studies will be performed on the following types of shear walls with staggered and regular openings:

- Composite shear walls;
- Composite eccentric bulbs with respect to the middle plane of the wall;
- Consolidated with FRP, in all the previous tested variants;

The research activity will be carried out in the following steps:

- Based on existing international bibliographic references there will be identified the types of buildings having such type of walls. The dimensions, types of buildings, resistances of materials, reinforcement layouts, and aspects regarding the loading protocol and the execution details of the experimental models will be established;
- With the help of FEM software, the analysis of the seismic behaviour of these walls subjected to a cyclic motion will be studied. There will be identified the elastic limit, rotation and displacement capacity, the sequence in which the reinforcement yields, rigidity degradation and the maximum value of the horizontal force and ultimate displacement etc.;
- An intermediate report will summarize the collected data from the numerical analysis;
- The elastic limit and the loading protocol for the cyclic test and the details for the experimental stand will be set. The construction of the experimental models will be performed;
- Following the experimental testing, a report will present a comparison between the numerical and experimental data, and the first recommendations of consolidation will be made;
- A numerical analysis of the walls, consolidated with FRP will be performed with the help of FEM software. The failure modes will be identified and also the state of efforts and displacements in several loading steps will be recorded by means of displacement control. A new loading protocol will be proposed and the constructive details will be finalized;
- A workshop which will present the construction and consolidation of the experimental models with FRP will be organized;

- The experimental testing will be performed and the efforts, deformations and cracking state of the models at each step will be recorded in order to identify the failure mechanisms;
- A comparative report will present the results of the theoretical and experimental results obtained for the experimental models consolidated with FRP;
- A new calculus methodology will be proposed for the design and consolidation of these walls based on the experimental results;

(iv) Seismic behaviour of RC structures being subjected to large values of the vertical component of the seismic action (a_g> 1.0g) and large strain rates is a recent direction of research of great importance on an international level, after the strong earthquakes recorded between 2009 and 2011. For this research, the author will continue the activity started with Dr. A. Anastasiadis under the coordination of Prof. V. Gioncu. Due to the fact that the large loading speeds of the vertical components of the seismic action are difficult to simulate in today's laboratories, the author will perform only theoretical research with the help of FEM software.
According to recent seismic recordings, the studies will be original and will be presented in prestigious international journals and will be carried out in the following steps:

- Based on international bibliographic references, the values of the accelerations and speeds of the seismic waves recorded in situ for different bearing elements from different storeys will be established;
- Identification of the types of r.c. structures which developed unknown failure mechanisms after these earthquakes;
- Definition of the physical-mechanical characteristics of the concrete and reinforcement subjected to large loading speeds of the horizontal and vertical seismic forces;
- Nonlinear dynamic analysis with the help of FEM software will determine the failure modes developed by these structures, the capacity of energy dissipation and degradation of rigidity;
- Preparation of a comparative report between the results obtained from FEM analysis and the real seismic behaviour of these buildings;
- Calibration of the theoretical results with those recorded in situ;
- Elaboration of recommendations in design codes regarding the application of these seismic forces in combination of loads and some particular reinforcement layout for r.c. bearing elements.

B. Structural Steelwork: design, repair and strengthening as well as architectural interaction

I. Future objectives

(i) Investigation of the influence of the cyclic loading type and of the loading rate on the local and global ductility of steel moment resisting and braced frames as well as on the steel members and connections. It was widely accepted the fact that as a function of some characteristics of the seismic action (near vs. far fault earthquakes/ strain rate vs. cyclic action, directivity, the influence of vertical component) the structures and the component members behave in a different way, therefore it is critical for the public safety to better analyze such a response and further on to create new codes and practical recommendations taking the aforementioned influences.
(ii) Development of new more efficient solutions to repair and/or strengthen the damaged steel members ;
(iii) Development of new details or structural systems, based on conventional design and traditional construction techniques, targeting to the controlled behaviour of the different

structural systems used in practice such as moment resisting, concentrically and eccentrically braced frames, taking into account the concepts and the principles of the ultimate and capacity design;
(iv) Development of a methodology to implement the ductility verification in a direct way in the practical design and the corresponding design codes in an equal manner as the strength and stiffness verifications. This is a topic that constitutes the continuing of Prof. Gioncu's studies, which developed the methodology of the local plastic mechanism;
(v) Development and investigation of the inelastic behaviour of the steel composite shear walls both locally and globally. Such types of systems could be used not only to undertake the seismic action but also the blast loading coming from terrorist attacks. Therefore, it is crucial to analyze their behaviour and contribution, when it is used in steel-shear wall systems, against progressive collapse.
(vi) Research and development of new ways of teaching steel design in the Architectural schools introducing the factor of the steel structural aesthetics, which attempts to implement the interaction of the architectural solution with the structural dimension.
(vii) Investigation and the impact of the architectural design waives on the structural design of steel constructions and particularly on the earthquake behaviour. The freeform design which creates long span, complex and asymmetric structures, until now was not analyzed under the seismic and blast actions. Therefore, it is a field for interdisciplinary collaboration between architects and structural engineers, in order to study the limits of the complex forms and furthermore allow the implementation of them in highly earthquake prone countries.

II. Impact of study

The directions of research proposed by the author in the field of steel bearing structures will bring information in new research national and international domains of reduction of seismic vulnerability of these buildings and will propose new technologies of construction and calculation which will complete the European seismic design codes by proposing simple, cheap and rapid consolidation technologies for these buildings.
The theoretical studies regarding the large vertical components of the seismic acceleration and the propagation speed of seismic waves will bring important information on the failure modes, details of joints and quality of materials used in the construction of steel elements, allowing the developing of a favourable failure mechanisms for buildings in seismic zones which can complete the seismic protection codes for buildings.
Studies related to the education of architects will bring contributions not only in the academic field but also will lead to a better development of the interactions between architects and design engineers for steel structures which will eventually lead to structures that conform to the basic principles of Vitruvius.

III. Methodology and research planning

Steps:
(i) The basic introductory phase, where the motivation, scope, as well as the objectives and the approaches were will be stabilized based on a thorough literature review and the existing knowledge, experience and skills of the head of the project and collaborators that constitutes the research group;
(ii) The main working phase, where the investigation process should be performed, developing and sub structuring the way of the work (numerical, experimental), developing the analytical models, calibrating them with other experimental and/or analytical works from the literature

review or by the own experimental work and further on carrying parametrical and sensitivity analysis in order to unveil the main influential parameters;
(iii) The conclusive phase, where proper organization and collection of the results will be performed as well as the suitable developing of design charts, illustrations and other characteristic and representative material of the corresponding work will be presented in the project report;

C. Assessment, strengthening methods and regeneration strategies for wood bearing structures

As responsible for the Romanian part of the COST FP1101 **[7.6]** research contract, the author will carry out the following research activities in the following domains related to timber structures:
(i) perfecting the in-situ evaluation methodology of the bearing capacity of timber elements, in function of their exact state of degradation. For this, non-destructive tests will be performed in order to assess the potential problems in the structures **[7.6];**
(ii) development of modern consolidation technologies for wooden bearing structures due to the change of the function of the building, deterioration due to lack of monitoring in time and poor maintenance. There will be studied consolidation measures which will not affect the historic value of the timber elements and the cultural value of the building, consolidation measures from mechanical fasteners (glued-in rods, self taping screws, adhesive systems, covering with steel plates) to the increasing use of FRPs, and more recently, nanotechnology **[7.6]**.
These research activities will be continued for wooden orthodox churches by following the next steps:

- Elaboration of complete surveys which will identify all dimensions of buildings and bearing elements, as well as the types of recorded damages;
- Non-destructive in-situ testing for the identification of the resistances of materials. From the areas not affecting the cultural value of the building, samples of wood will be collected and tested in the laboratory in order to calibrate the results obtained on site;
- Numeric simulations with the help of FEM software in order to obtain the overall behaviour of the wooden structures;
- Calibration of the theoretical results with the real situation on site;
- Proposal of innovative solutions which will be experimentally tested in laboratories;
- Preparation of a rapid evaluation methodology of the vulnerability of wooden churches;
- Consolidation recommendations for the structural wood on which painting is applied, with the help of fellow consulting chemists, medics and painters, etc.

(iii) urban regeneration of cities by the exploitation of historic roof framing systems
Looking at other cities like Graz, Stuttgart, Bamberg, together with fellow architects, the author will perform studies in order to regenerate historic areas from Timisoara, by applying a conversion of the attic areas historic buildings. In this direction there will be performed surveys with the help of drones, in-situ measurements for roof framing systems and there will be identified all types of degradations. Based on static and spatial dynamic calculations, methods of consolidation for the wood bearing structures will be proposed in order to exploit the attic space. Based on this analysis, there will be elaborated architecture plans and urbanism rules for these historic areas which will allow the regeneration of historic areas by introducing them in the touristic circuit and by exploitation of the attic areas without altering the historic character of the urban area.

There will be studied the effect of the spatial wooden structures which form the roof framing systems of historic buildings from Timisoara, on the seismic behaviour of brick masonry structures.

D. Vulnerability of buildings and urban centres:

In Romania there are a large number of urban historic areas and sites, having buildings which recorded damages in the bearing structure and which are vulnerable to the seismic activity. A great part of these historic buildings require urgent consolidation interventions in order to strengthen them. For this, the author will develop and adapt for Romania, studies for the rapid evaluation of the seismic vulnerability of individual and complexes of buildings, based on rapid evaluation methodologies developed by researchers from Italy and England, by following the next steps:

- In situ measurements with drones and other modern investigation technologies for buildings;
- Performing experimental in situ and laboratory tests on various construction materials particular for Romania;
- Performing exact calculations in the linear and non-linear domain for various building materials from Romania;
- Elaboration of rapid calculation methods for the vulnerability of historic and religious buildings from Romania;
- Calibration of the results and elaboration of final methodologies for the seismic evaluation;
- Development of the theory of rigid block failure of the orthodox churches from Romania;
- Implementation of vulnerability maps for various historic urban centres and archaeological sites;

Resources and dissemination

(i) Theoretical research will be performed in U.P. Timisoara, H.I. Struct design office, together with other teams of researchers from universities from Europe, or with private companies;

(ii) As until now, the financial aid will be obtained by the participation of the author in research contracts with national or European financing, or from private companies;

(iii) The experimental tests will be performed in the laboratories of "Politehnica" University, INCERC Timisoara and other laboratories capable to perform the given set of objectives. For the in situ behaviour of historic buildings, there will be established partnerships with national or international laboratories.

The obtained results will be disseminated by the completion of design of structures courses for the academic environment, publications within national and international conferences and international journals, recommendations for design codes, presentations within workshops and on websites, elaboration of PhD, MSc and BSc thesis.

Autonomy of the scientific activity

As recognition for the quality of the performed scientific activity:

(i) *Research autonomy*. The author has participated in 4 national contracts and 3 international contracts: 2 as coordinating director for Romania, and in 1 as a member:

- FP6 PROHITECH "Seismic Protection of Historical buildings by reversible mixed Technologies" – 2004-2008, member of the research team;
- FP7-SME-2013 INSYSME "INnovative SYStems for earthquake resistant Masonry Enclosures in RC buildings" – 2013-2016, coordinating director for Romania;

• COST FP1101 action: “Assessment, Reinforcement, and Monitoring of Timber structures” – 2011-2015, coordinating director for Romania;

The participation in 3 European research grants provided me with the skills and competences to manage other contracts. Currently I take part in research projects without financing together with other researchers from Europe with which I have and I will further publish articles, as sole representative of “Politehnica” University of Timisoara.

I have reviewed articles for prestigious publishing houses for international journals:

- Ed. Elsevier: *Engineering Structures* (2013), *Thin walled structures* (2013), *Journal of Constructional Steel Research* (2013), *Engineering Failure Analysis* (2012);
- Ed. Taylor and Francis: *International Journal of Architectural Heritage*(2014), *European journal of Environmental and Civil Engineering* (2014)
- Ed. Springer: *Bulletin of Earthquake Engineering* (2014)
- Ed. Ernst & Sohn Whiley: *Structural Concrete* (2013)

(ii) Academic autonomy. After the late departure of Prof. V. Gioncu, I have coordinated the technical subjects within the Faculty of Architecture and Urbanism from Timisoara. As a director of research contracts, I coordinate Erasmus exchanges for students and professors from universities from Italy and Turkey. I have made changes in the teaching plan for the students, in correlation with the subjects studied in other universities from Europe and I have organized national and international workshops.

(iii) Professional autonomy. In the next two years, I will coordinate the design of bearing structures for investments of over 20 million euro.

Visibility of the scientific activity

I have elaborated a total number of 103 papers, among which 12 ISI journal papers, 14 ISI proceedings papers. Since 2004, I have published a total of 40 papers in International Conferences, 4 books as author and one book as associate editor.

I have been a member in international scientific committees at the following conferences:

1. PROHITECH ’14, *“2nd International conference on protection of historical constructions*”, 2-7 May 2014, Antalya, Turkey;
2. VANEQS 2013, “*International Van earthquake symposium”,* October, 2013, Van, Turkey;
3. RICH 2014, *“2nd International Conference Robotics: Innovation for Cultural Heritage”,* 6th-7th October, 2014, Venice, Italy;
4. SHATIS’15, “*3rd International Conference on Structural Health Assessment of Timber Structures*”, September 9-11, Wroclaw, 2015, Poland;

As until the present time, as a member of the Faculty of Architecture and Urbanism from Timisoara, I will dedicate my entire research, professional and academic activity, based on the principles imposed by Prof. V. Gioncu, to participate, together with architects and other specialists, to the salvation of historic bearing structures, improvement and development of the design of bearing structures in the future, as well as to continuously improve the professional training of student architects and engineers.

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Chapter 7: b-ii - Scientific, professional and academic future development plans

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