IOSUD - Universitatea Politehnica Timişoara
Şcoala Doctorală de Studii Inginereşti

STUDIES CONCERNING THE BEHAVIOUR OF MASONRY WALLS UNDER SEISMIC ACTIONS. STRENGTHENING OF MASONRY WALLS USING COMPOSITE MATERIALS

PhD Thesis – Summary
for obtaining the scientific title of doctor at
Politehnica University Timișoara
in the Civil Engineering PhD field

author eng. Eleonora Eva PARTENE

scientific coordinator Prof. PhD eng. Valeriu STOIAN

November 2018

Abstract

The PhD thesis aims to study the behaviour of masonry walls built-up using ceramic blocks with vertical hollows, under seismic action. The thesis also studies the strengthening of masonry walls using composite materials.

The experimental program focused on testing unreinforced masonry walls and reinforced masonry walls, under horizontal cyclic loading and observing the difference between these types of walls in initial state. The second part consisted on strengthening the tested walls using fiber reinforced polymers and testing the walls again, in order to determine if the walls with applied strengthening materials are able to regain their initial capacity or even increase it. The efficiency of the strengthening method is assessed in the final chapter, at the end of the experimental program.

At the end of the PhD thesis there is presented a case study for a real building, made with load-initialing masonry walls, using the same ceramic blocks from our experimental program. A theoretical evaluation is made according to the romanian standard P100-1/2013 and then a evalution using a software, in order to observe the spatial behaviour of the entire building. The results for the case-study are compared with the results of the experimental program, in order to observe the maximum capacity of the masonry walls from the building and the maximum shear capacity from the experimental program.

1. INTRODUCTION

1.1. Overview

Masonry represents one of the oldest building material and is still used often nowadays. However due to the lack of knowledge, many types of masonry structures have been built without taking into account the horizontal loads that this buildings are going to be subjected to over the years. Thus, there are a large number of buildings vulnerable to seismic actions, built without any reference to seismic design rules, designed only for gravitational vertical loads.

However, in order to be able to successfully build masonry structures in seismic areas such as our country, a series of measures are being taken to strengthen it, namely the use of reinforced masonry, which by the presence of tie-columns and tie-beams, or the reinforcement of the horizontal bed mortar joints, allows a better energy dissipation of seismic energy.

It is also recommended that in seismic areas to realize concrete rigid slabs for a better behaviour under seismic loads.

1.2. Motivation

The main objective of the PhD thesis is the study of the behaviour of masonry walls built up with ceramic block with vertical hollows, unreinforced or confined, under seismic actions. The second part is the study of the behaviour of the strengthening of damaged masonry walls, using composite materials, also under seismic action.

1.3. General framework

The PhD thesis was carried out in the Civil Engineering and Building Services Department, Civil Engineering Faculty, Politehnica University Timișoara.

2. SUMMARY OF THE DESIGN CODE RULES FOR MASONRY BUILDINGS

2.1. Design rules – according to romanian code: CR6-2013

The CR6 code makes a classification of masonry structures and contains design rules for structures with load initialing walls, but also other types of walls.

The most important aspect in determinining the type of masonry to be used in a structure is the knowledge of their classification:

Unreinforced masonry (URM) is the masonry that does not contain enough reinforcement to fit into the reinforced category. [1]

Confined masonry (RC) is the masonry with concrete tie-columns and tie-beams on all the sides of the wall. [1]

Confined masonry and reinforced masonry is the confined masonry that also has reinforcement (materials with good tensile strength) in horizontal bed mortar joints, in order to increase the shear capacity and the ductility of the masonry walls. Fig. 2.1.a [1]

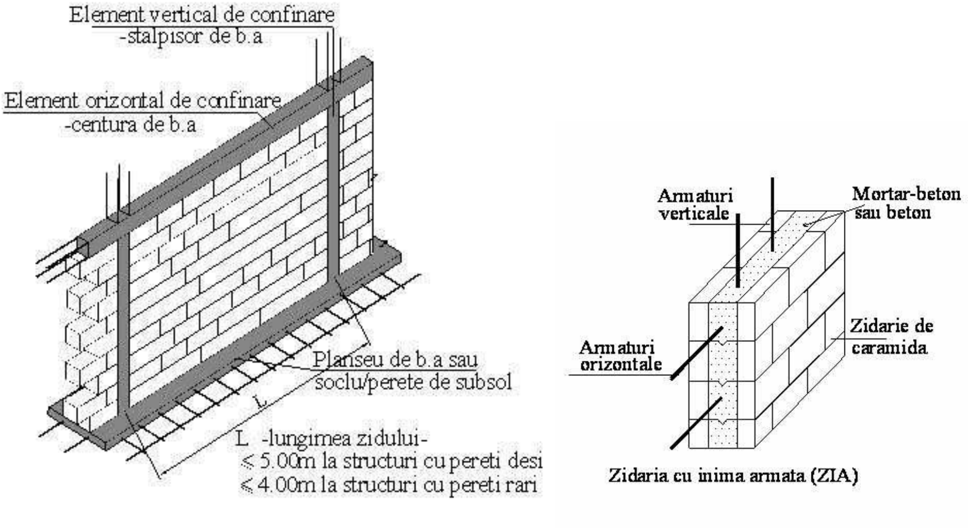
Reinforced masonry is the masonry that has between two layers of masonry, a layer of reinforced concrete/mortar with vertical reinforcement, with or without mechanical connections between layers and in which all components have a contribution for gravitational and horizontal loads. Fig. 2.1.b [1]

Infill masonry is the masonry used in concrete or steel structures with no load initialing part, but can contribute in some cases to the lateral stiffness and to the energy dissipation of the building. [1]

Another classification of the masonry can be: structural walls, structural stiffening walls and nonstructural or framed walls. [1]

The present paper only refer to structural, load-bearing walls designed to withstand vertical and horizontal in-plane loads. [1]

Structural load-initialing masonry walls are used for: buildings with maximum 5 storey height, depending on the seismic area, used for housing or similar functions, social-cultural buildings where large free spaces are not required, or hall buildings with moderate openings. [1]



a) b)

Fig. 2.1. Masonry wall design:
a – confined masonry; b – reinforced masonry [1]

2.2. Design rules – according to romanian code: P100-1/2013

Romania's territory is divided intro several seismic hazard areas (Fig. 2.2.), which are considered constant in each area. The seismic hazard is represented as the peak value of seismic ground acceleration a_g, which is determined for an average recurrence period (IMR). [2]

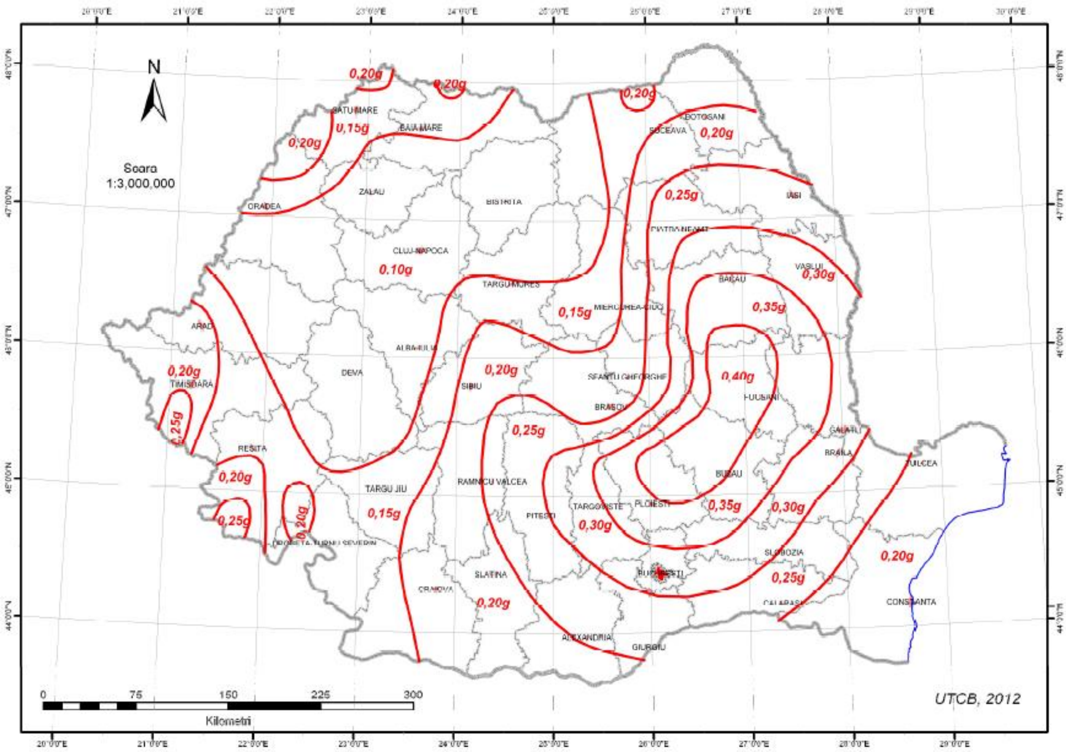


Fig.2.2. Zoning map of a_g values in Romania for IMR=225 years and 20% probability of overtaking in 50 years

The design code makes a clear distinction between structural load initialing walls and stiffening encreasing structural walls, so the structural load initialing walls are able to support vertical and horizontal in-plane loads and the stiffening encreasing walls are the walls that help the spatial conformation and cooperation of a building walls, and also helps the stability of the wall linked to it. [3]

At the moment, several types of masonry elements can be used for structural load initialing walls: clay blocks with or without vertical hollows (SR EN 771-1) or autoclaved cellular concrete blocks (BCA) (SR EN 771-4). These elements are divided into group 1 or 2 of materials, with properties according to table 8.1. of P100-1. (table 2.1) [3]

Table 2.1. Geometrical properties of masonry blocks [3]

<table>
<tr><td>Characteristics</td><td>Group 1 - clay și BCA</td><td colspan="4">Group 2 – Clay blocks with vertical hollows</td></tr>
<tr><td rowspan="2">Total hollow volume (% from gross volume)</td><td rowspan="2">≤25%</td><td colspan="2">a_g≤0,15g</td><td colspan="2">a_g≥0,20g</td></tr>
<tr><td colspan="2">>25%; ≤55%</td><td colspan="2">>25%; ≤45%</td></tr>
<tr><td>Each hollow volume (% from gross volume)</td><td>≤12,5%</td><td colspan="4">*for each of the multiple hollows ≤2%
*total handling hollows ≤12,5%</td></tr>
<tr><td rowspan="3">Declared value of the thickness of the interior and exterior walls (mm)</td><td rowspan="3">No demands</td><td colspan="2">interior wall</td><td colspan="2">exterior wall</td></tr>
<tr><td>a_g≤0,15g</td><td>a_g≥0,20g</td><td>a_g≤0,15g</td><td>a_g≥0,20g</td></tr>
<tr><td>≥5</td><td>≥10</td><td>≥8</td><td>≥12</td></tr>
</table>

Unreinforced masonry (URM) has a reduced capacity to dissipate energy from horizontal loads, so its use in seismic areas is restricted.

In the design code, in table 8.8 (table 2.2.) we have the maximum storey level allowed, depending on seismic area a_g and wall density (p%). [3]

Table 2.2. Number of storey levels above ground allowed for URM [3]

<table>
<tr><td rowspan="3">n_{storey}</td><td colspan="6">a_g</td></tr>
<tr><td colspan="2">0,10g și 0,15g</td><td colspan="2">0,20g și 0,25g</td><td colspan="2">0,30g÷0,40g</td></tr>
<tr><td>Clay gr.1 și 2</td><td>Clay gr.2S și BCA</td><td>Clay gr.1 și 2</td><td>Clay gr.2S și BCA</td><td>Clay gr.1 și 2</td><td>Clay gr.2S și BCA</td></tr>
<tr><td>1</td><td>≥4,0%</td><td>≥4,5%</td><td>≥5,0%</td><td>≥5,5%</td><td rowspan="3">NA</td><td rowspan="3">NA</td></tr>
<tr><td>2</td><td>≥4,5%</td><td>≥5,0%</td><td>≥5,5%</td><td>≥6,0%</td></tr>
<tr><td>3</td><td>≥5,0%</td><td>≥5,5%</td><td>NA</td><td>NA</td></tr>
<tr><td colspan="7">NA – not allowed</td></tr>
</table>

Reinforced and confined masonry (RC) can be used in seismic areas taking intro account the conditions in table 8.9. (table 2.3.) from the design code, depending on seismic area a_g and wall density (p%). Wall density (p%) from the table is for the ground floor and it can be reduced with maximum 1%/storey for the above ground storeys, maintaing the elevation regularity.

If the regularity condition is not satisfied it is necessary to perform a modal analysis in order to determine the base shear force of the building. [3]

Table 2.3. Number of storey levels above ground allowed for RC [3]

n_{storey}	a_g					
	0,10g și 0,15g		0,20g și 0,25g		0,30g÷0,40g	
	Clay gr.1 și 2	Clay gr.2S și BCA	Clay gr.1 și 2	Clay gr.2S și BCA	Clay gr.1 și 2	Clay gr.2S și BCA
1	≥3,0%	≥3,0%	≥4,0%	≥4,0%	≥5,0%	≥5,5%
2		≥3,5%		≥4,5%	≥5,5%	≥6,5%
3	≥4,0%	≥4,0%	≥5,0%	≥5,5%*	≥6,0%*	≥6,0%*
4		≥5,0%	≥6,0%*	≥6,0%*	≥6,5%*	**
5	≥5,0%*	≥5,5%*	**	**		NA

*confined masonry with horizontal reinforcement or reinforced masonry is required
**a nonlinear structural analysis is required
NA – not allowed

2.3. Design codes evolution in seismic areas

The first design code for masonry walls buildings:

"Instrucțiuni tehnice privind măsurile constructive la clădirile cu zidărie portantă, situate în zone seismice. Indicativ P32".

The second one: *"Normativ privind alcătuirea și calculul structurilor din zidărie. Indicativ P2-75"*, followed by *"Normativ privind alcătuirea, calculul și executarea structurilor din zidărie. Indicativ P2-85"*.

This design codes include a series of measures that underline the design and execution of load initialing masonry structures located in seismic areas.

The design codes introduce general measures for spatial conformation, and lateral rigidity ensurance and also how to create favorable dissipation mechanism under seismic actions. [4]

The latest design codes are CR6-2006 and its improved version CR6-2013 *"Cod de proiectare pentru structuri din zidărie"- Design code for masonry walls.*

In the latest years we can observe a considerable improvement of the design codes for seismic areas, due to the evolution of technology as well as the numerous research programs carried out in the field. [5]

3. GENERAL ASPECTS FOR THE BEHAVIOUR OF MASONRY WALLS. STRENGTHENING OF MASONRY WALLS USING COMPOSITE MATERIALS. BIBLIOGRAPHIC STUDY

3.1. Overview

Masonry is considered one of the oldest types of building structures, having a slow evolution from the structural concept point of view, but also of the technological process of production of the elements used for masonry. [6].

In the design of masonry structures, the design model uses the following simplifications:

- the material is considered homogeneous, with elastic response until failure stage;
- the sectional characteristics are determined witout taking into account the cracks of the walls.

For the determination of design loads and resistance of the structural walls, using a numerical or non-linear model, has to adequately represent the strength of the entire structural system.

3.2. Failure modes for masonry walls

Masonry walls have different failure modes – for in-plane loads Fig. 3.1.:
- sliding failure (a);
- shear failure (b) ;
- flexural failure (c).

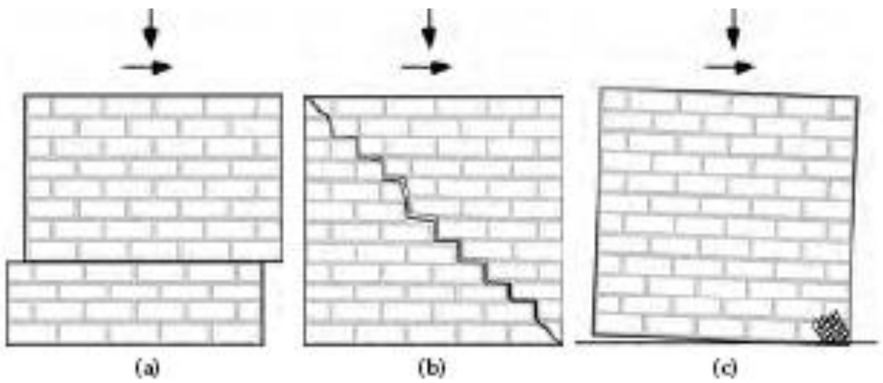


Fig. 3.1. Failure modes for masonry walls:
a – sliding failure; b – shear failure; c – flexural failure. [7]

Sliding failure is defined as the horizontal movement of entire parts of the wall on a single brick layer or mortar bed. It usually occurs in one of the lowest mortar beds of the wall in cases of walls with aspect ratio h/b lower than 1 and small level of vertical load. It is a non-ductile failure mode in the event of earthquake. [8]

Flexural failure occurs when the wall behaves as a vertical cantilever under lateral bending and either cracking in the masonry tension zone (opening of bed joints) or crushing at the wall toe. It occurs in slender walls with aspect ratio h/b bigger than 2. This failure mode usually involves large inelastic deformations without reduction of initialing capacity. [8]

Shear failure is characterized by a critical combination of principal tensile and compressive stresses as a result of applying combined shear and compression, and leads to typical diagonal cracks. In practice, two types of shear cracking can be observed, joint cracking by local sliding along the bed joint and diagonal cracking associated with cracks running through the ceramic blocks as well as the joints. It is the most common failure mode for masonry walls. [8]

3.3. Strengthening using composite materials

The advantages of the polymeric materials are as follows:
- reduced weight, 80% less than steel, thus reducing transport and installation costs, but also an advantage for buildings where permanent loads cannot be modified by the consolidation;
- high ultimate resistance, 3 time higher than steel;
- high strength-weight ratio, having less than 10% of the steel weight at the same strength;
- the posibility to choose orientation, position, volume of the fibres, in order to direct the maximum capacity in a certain way;

- high durability and posibility to use in agressive enviroments;
- dimensional stability, low thermal conductivity and low therman expansion coefficient;
- mangnetic and radar transparency;
- does not require maintenance;
- posibility of precompresion;
- possibility of production at any lengths/dimensions;
- low-time execution, minimizing production and traffic costs;
- can be used in places with limited acces, havind reduced thickness;
- increased impact/explosion resistance. [9]

However, composite materials also have a number of disadvantages:

- low fire resistance;
- easy mechanical damage, with cutting objects;
- degradation caused by ultraviolet radiation;
- elongation at tearing less than steel, resulting in fragile breaks;
- linear behaviour;
- high costs of materials. [9], [10]

3.4. Bibliographic study

This paper contains a bibliographic study of the specialized literature in order to be able to know the current stage in the studied field, but also to help in establishing the experimental program details, such as strengthening methods or applying loads method in order to obtain the shear failure. In the studied articles there are mentioned masonry walls in initial state using diffenrent ceramic block and also articles or papers with strengthening methods and the results obtained by strengtheing masonry walls with composite materials. [11], [12]

The paper presents summarized the conclusions of 32 important publications in the form of articles or Phd thesis, in the field of interest for the present paper.

The main conclusions of the studied publications were: it is proved that the confined and reinforced masonry is more efficient for seismic areas than the unreinforced masonry, for every type of ceramic block.

For the strengtheing of masonry with composite materials, the stiffness of the elements has decreased in most cases, but the inital capacity for the maximum loads is regained in most cases. The failure mode for the strengthened elements is fragile in most cases and occurs with the detachement of the composite material together with parts of the ceramic blocks.

4. EXPERIMENTAL PROGRAM

4.1. Introduction

The experimental program consisted in the testing of scale made masonry elements, with typology chosen to cover most of the situations encountered in practice. Three type of elements were tested, of which there were made 3 pieces for each, in order to validate the obtained results. The experimental program was conducted in the Civil Engineering and Building Services Departament, of the Civil Engineering Faculty, Politehnica University Timișoara. The nine masonry elements were tested in two stages: the first stage was the testing of the elements in initial state and the second stage was testing after strengthening the walls with composite materials.

4.2. Testing strategy. Experimental stand

The load applied to the masonry element were: a constant vertical load (V) and a cyclic horizontal load (H) – according to the theoretical model from Fig. 4.1. [9]

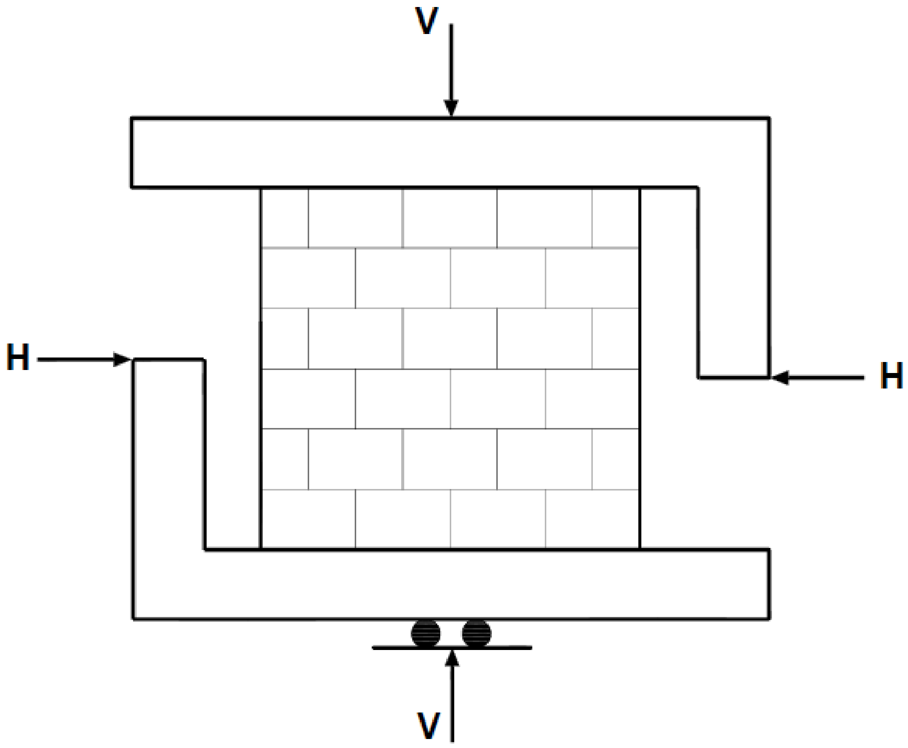


Fig. 4.1. Simplified model of the applying loads system [9]

In order to achieve the theoretical model it was necessary to build an experimental stand, seen in Fig. 4.2.

The experimental stand consisted of three main components: a reaction frame for the horizontal loads, a reaction frame for the vertical loads and two special sliding frames. The third component was the most important for obtaining the shear failure of the walls, with the correct mode of applying the vertical and horizontal loads. Fig. 4.3.

The sliding frames have bottom and top support beams which are designed to work with the experimental elements and to transmit the loads for the experimental stand. This beams are very rigid and cannot influence the load transmission and have also the role of simulating the real connection between walls and foundations and the walls and tie-beams, like we find in current practice in a building.

Fig. 4.2. Final experimental stand

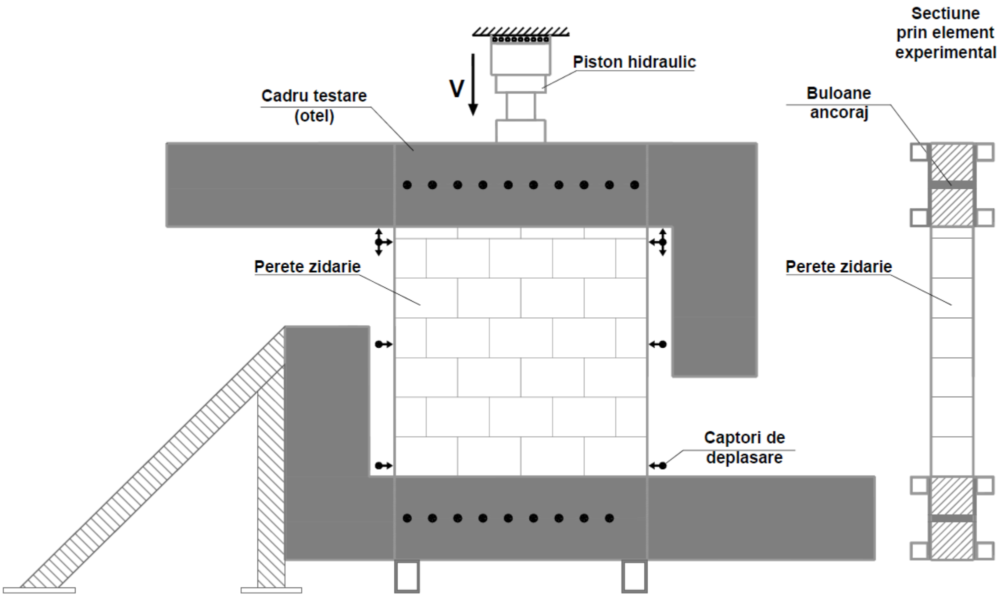


Fig. 4.3. Sliding frames

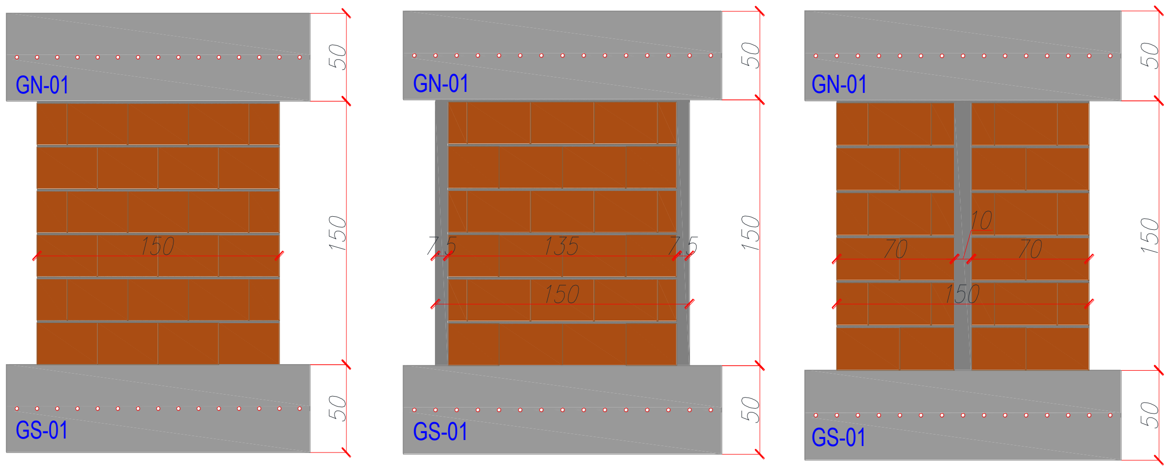
4.3. Experimental elements

The experimental elements tested for the current paper consist of masonry walls, made with ceramic blocks with vertical hollows and they have the dimensions 150x150x25 cm. Their label name is seen in table 4.1.

Table 4.1. Label of experimental elements

Experimental element type	Label
Unreinforced masonry	URM1
	URM2
	URM3
Confined masonry with lateral tie-columns	RM1
	RM1'
	RM1"
Confined masonry with central tie-column	RM2
	RM2'
	RM2"

The masonry walls were made using ceramic block with vertical hollows with dimensions 375x250x238 mm, having the following typologies: three elements of unreinforced masonry (URM), three elements of confined masonry with lateral tie-columns (RM1) and confined masonry with central tie-column (RM2). Fig. 4.4.



a) b) c)

Fig. 4.4. Experimental elements: a) URM; b) RM1; c) RM2

Material properties

The material properties for the ceramic blocks are in table 4.2.

Tabelul 4.2. Ceramic blocks material properties

Ceramic block	δ	$f_{b,med}$	f_b	$f_{b,min}$	$f_{b,max}$
		[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]
Porotherm 25	1.138	10	11.38	14.79	21.62

For the mortar used to make the experimental elements, bending and compression tests were performed on samples taken from the mortar, obtaining the results from table 4.3. – for the M5 mortar class, according to the standard rules for mortars used for load initialing walls. The results reflected the mortar we wanted to obtain in the experimental specimens.

Table 4.3. Results for mortar prisms tests

Prism label	Bending		Compression	
	Maximum force [N]	f_i [N/mm^2]	Maximum force [N]	f_c [N/mm^2]
PIa	480	18x10^{-3}	9200	5.75
PIb			9160	5.725
PIIa	600	22.5x10^{-3}	8300	5.187
PIIb			8470	5.293
PIIIa	590	22.1x10^{-3}	9650	6.03
PIIIb			9620	6.01

The concrete used to make the tie-columns was class C16/20, accordind to lab results seen in table 4.4.

Table 4.4. Result test on cubes

Element	**Test cube**	**Maximum force [N]**	**Compressive cube strength [N/mm^2]**	**$f_{cm,cub}$ [N/mm^2]**	**f_{ck} [N/mm^2]**	**Concrete class**
Masonry with lateral tie-columns	Cub 1	654.6x10^3	37.04	29.06	17.93	**C16/20**
	Cub 2	662.8x10^3	37.50			
	Cub 3	644.2x10^3	36.45			
Masonry with central tie-column	Cub 1	626.6x10^3	35.46	28.03	17.29	**C16/20**
	Cub 2	628.9x10^3	35.59			
	Cub 3	636.3x10^3	36.01			

For the strengthening of the masonry walls were used composite materials: carbon fiber plate and carbon fiber mesh, both applied using epoxy resins. The material properties can be seen in table 4.5.

Table 4.5. Material properties

Material	**Thickness [mm]**	**Density [g/cm^3]**	**Tensile strength [N/mm^2]**	**Viscosity [MPas]**	**Shear strength [N/mm^2]**	**Compression strength [N/mm^2]**
Primer	-	1.1	-	300	-	-
Thixotropic resin MapeWrap 12	-	1.70	-	800000	-	-
Medium viscosity resin MapeWrap 31	-	1.06	40	7000	-	70
Carbon plate E170/100/1.4	1.4	0.00161	3100	-	77	-
Carbon mesh C UNI-AX 300/40	0.166	1.8	4830	-	-	-

4.4. Strengthening of the experimental elements

The second part of the experimental program consisted in the consolidation of damaged masonry walls using polymeric composite materials. The strengthening solution consisted in applying the composite materials to the main diagonals of the elements, on both sides, where the important damages were produced. The first element of unreinforced masonry was strengthened using carbon plate and the other eight elements were strengthened using carbon mesh. This decision was made because of the fragile failure of the first element.

The carbon mesh MapeWrap C UNI-AX is a uni-directional mesh with high elasticity modulus and high tensile strength.

The application of the carbon mesh was made following these steps:

- cleaning the wall surface by removing mortar traces and masonry pieces, from the damages of the walls in the first tests;
- application of MapeWrap Primer for a better adhesion of the resin layer – application is made with a brush, only in the areas where the resin is to be applied;
- application fo MapeWrap 12 epoxy resin to smooth unevenness and seal the porous surfaces – application is made in a 2 mm layer for a better leveling;
- application of the first layer of MapeWrap 31 with a brush in a 0.5 mm thickness layer, followed immediatly by the carbon mesh and leveling with a rubber roll. In the end for the impregnation of the carbon mesh, we applied a second layer of MapeWrap 31 resin with a brush.

Between the application of different layers was expected at least 24 hours. After the manual application of resin MapeWrap 12, a mechanical grinding was necessary in order to smooth the surface, in order to apply to safely apply the carbon mesh.

This procedure was used for all the experimental elements on both sides of the walls, after they suffered damage from the inital tests.

4.5. Loading protocol

The cyclic horizontal loads were applied using an actuator that was able to perform cycles movements and the cycles were performed according to Fig. 4.5.

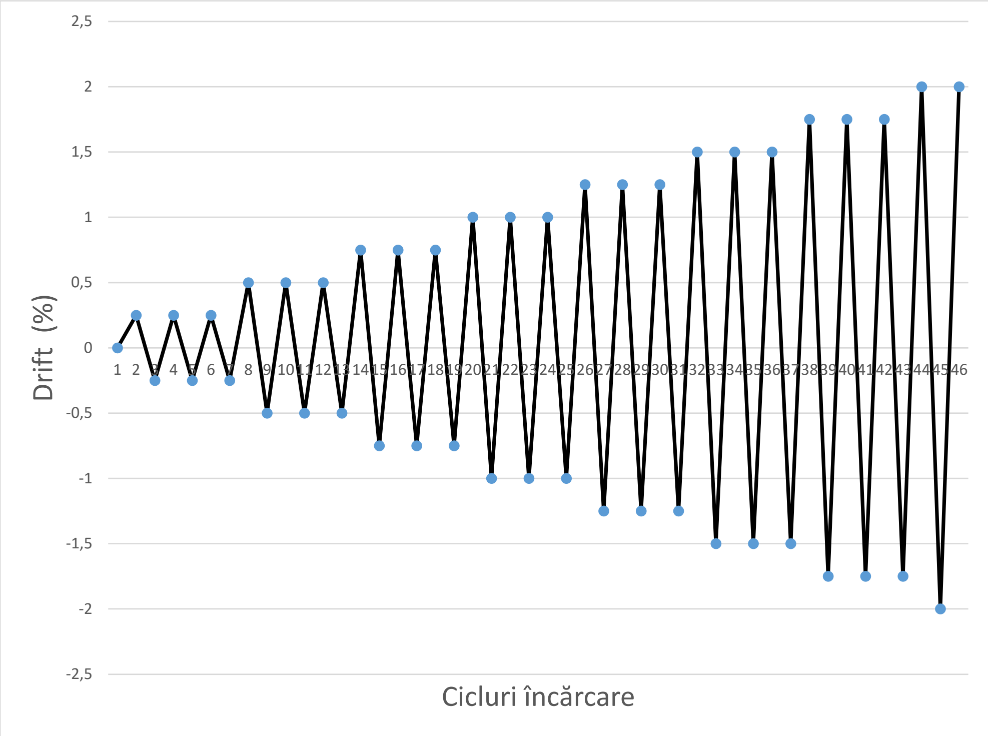


Fig. 4.5. Loading protocol

4.6. Instrumentation

During the experimental tests a series a measurements were made in order to determine the performance of the masonry walls, namely the displacements and the forces.

The measurements were made using displacement traducers and pressure traducers.

The pressure traducers registered the vertical load, which was maintained constant during the test and the horizontal forces obtained for each cycle.

The displacement traducers were mounted in several positions:

- on left/right sides of the wall, top-middle-bottom;
- on the sliding steel frame at the top, for measuring vertical displacements;
- on the sliding steel frame at the top, in order to observe if the testing frame is moving out of plane – which is undesirable.

The instrumentation of the experimental tests with the position of the displacement traducers can be seen in Fig. 4.6.

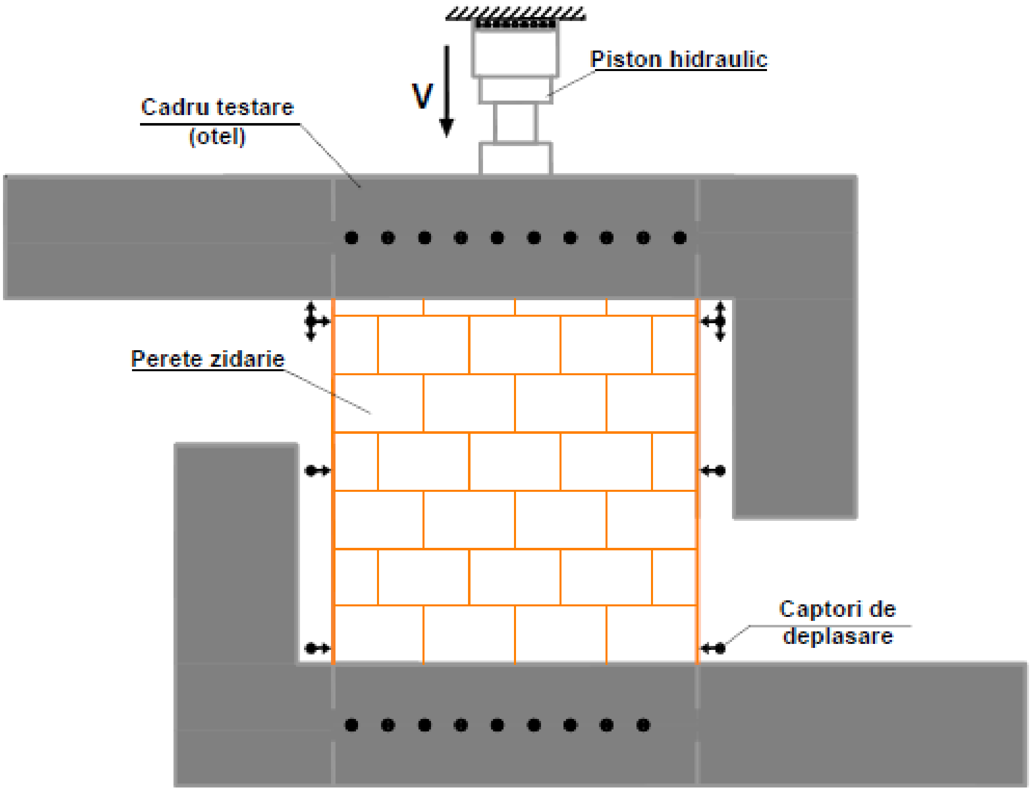


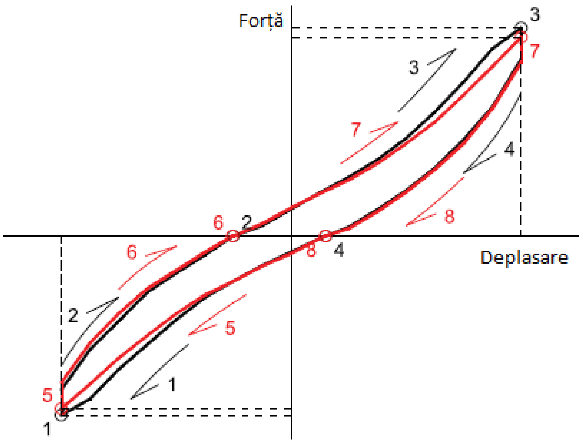
Fig. 4.6. Instrumentation of the experimental tests

5. RESULTS

5.1. Overview

The chapter presents the results obtained in the 18 experimental tests, for the walls in initial state and the strengthened walls.

The obtained results were processed as hysteresis loops, with all the loading-unloading cycles and also as envelope curves with the maximum values from the tests. Fig. 5.1. This way we could compare the results from the experimental program between different types of walls, and also the results from the numerical analysis with the experimental program.



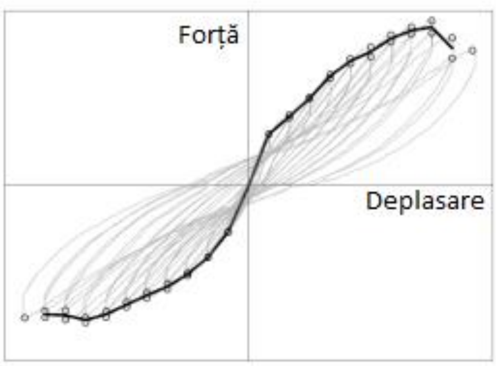


Fig. 5.1. Theoretical hysteresis loop curves and envelope curves

5.2. Experimental tests for walls in initial state

The chapter presents the results from the 9 elements in initial state: failure mode, the hysteresis loop curves, the envelope curves, in order to help us compare results.

The below image represents the envelope curves from all 9 elements in initial state, unreinforced masonry (URM), confined masonry with lateral tie-columns (RM1) and confined masonry central tie-column (RM2). From these envelopes we observe that the RM2 specimens had the best behaviour under horizontal loads. Fig. 5.2.

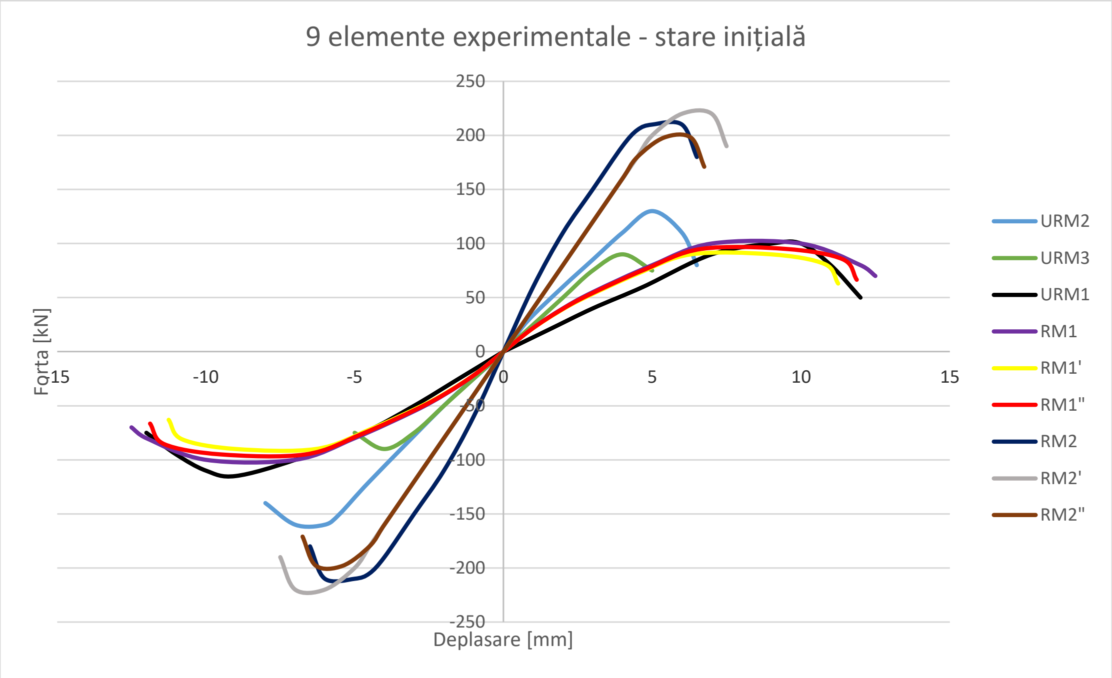


Fig. 5.2. Envelope curves force-displacement for the walls in initial state

The failure mode for all the walls in initial state was the shear failure, with diagonal cracking of the walls. For the unreinforced masonry and the confined masonry with lateral tie-columns the cracks appeared mainly in bed mortar joints, and for the confined masonry with central tie-column the cracks appeared mainly in the ceramic blocks, both following the diagonals of the walls.

5.3. Experimental tests for strengthened walls

After testing the walls they were strengthened using composite materials. The first element – unreinforced masonry – was strengthened with carbon-plate following the diagonals on both sides of the wall. The other eight elements were strengthened using a carbon fiber mesh, also following the two diagonals on both sides of the wall.

The results from the nine experimental walls can be seen in Fig. 5.3.

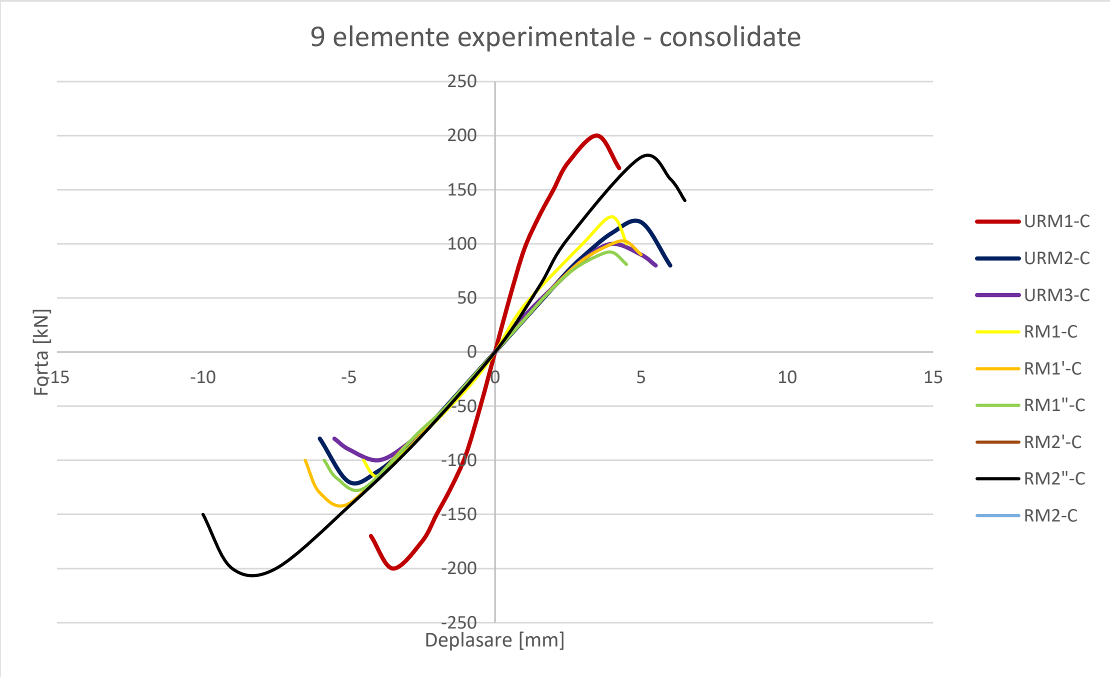


Fig. 5.3. Envelope curves force-displacement for the strengthened walls

The failure of the elements was also a shear failure, with the opening of the inital cracks until the composite materials starded splitting from the walls with peeling of a part of the ceramic blocks. For the carbon plate the failure was more fragile and produced the peeling of a big part of the brick ceramic blocks.

5.4. Conclusions of the experimental program

The results of the experimental program are centralized in Table 5.1.

Table 5.1. Results experimental program

Wall specimen	**URM1**	**URM2**	**URM3**	**URM1-C**	**URM2-C**	**URM3-C**
Vertical force [kN]	150	150	150	150	150	150
Maximum horizontal force [kN]	115	140	105	210	130	100
σ_0 [N/mm^2]	0,3	0,4	0,3	0,6	0,3	0,3
Maximum drift [mm]	11,0	6,0	5,4	4,0	6,0	5,8
Failure mode	shear	shear	shear	shear	shear	shear
Wall specimen	**RM1**	**RM1’**	**RM1”**	**RM1-C**	**RM1’-C**	**RM1”-C**
Vertical force [kN]	150	150	150	150	150	150
Maximum horizontal force [kN]	135	115	120	125	130	135
σ_0	0,4	0,3	0,3	0,3	0,3	0,4

[N/mm^2]						
Maximum drift [mm]	10,0	10,0	10,0	4,8	5,0	5,0
Failure mode	shear	shear	shear	shear	shear	shear
Wall specimen	**RM2**	**RM2'**	**RM2"**	**RM2-C**	**RM2'-C**	**RM2"-C**
Vertical force [kN]	150	150	150	150	150	150
Maximum horizontal force [kN]	230	220	200	210	175	160
σ_0 [N/mm^2]	0,6	0,6	0,5	0,6	0,5	0,4
Maximum drift [mm]	6,0	6,0	6,0	6,0	5,8	5,8
Failure mode	shear	shear	shear	shear	shear	shear

Analyzing these results, we can observe the following:

- for the unreinforced masonry, the strengthened elements manage to obtain 95% of the maximum horizontal force, and 83% from the initial drifts;
- for the confined masonry with lateral tie-columns, the strengthened elements manage to obtain 95-115% of the maximum horizontal force and 50% from initial drifts;
- for the confined masonry with central tie-column, the strengthened elements manage to obtain 80-92% of the maximum horizontal force and 80% from initial drifts.

Energy dissipation analysis

The cumulative energy dissipated for each experimental test was calculated and the results are centralized in în Fig. 5.4.

If we analize the results from the 18 experimental tests, regarding the cumulative dissipated energy, we cannot see a pattern of increase or decrease for the strengthened masonry walls. This cannot be a factor in evaluation the efficiency of masonry walls strengthened using polimeric materials.

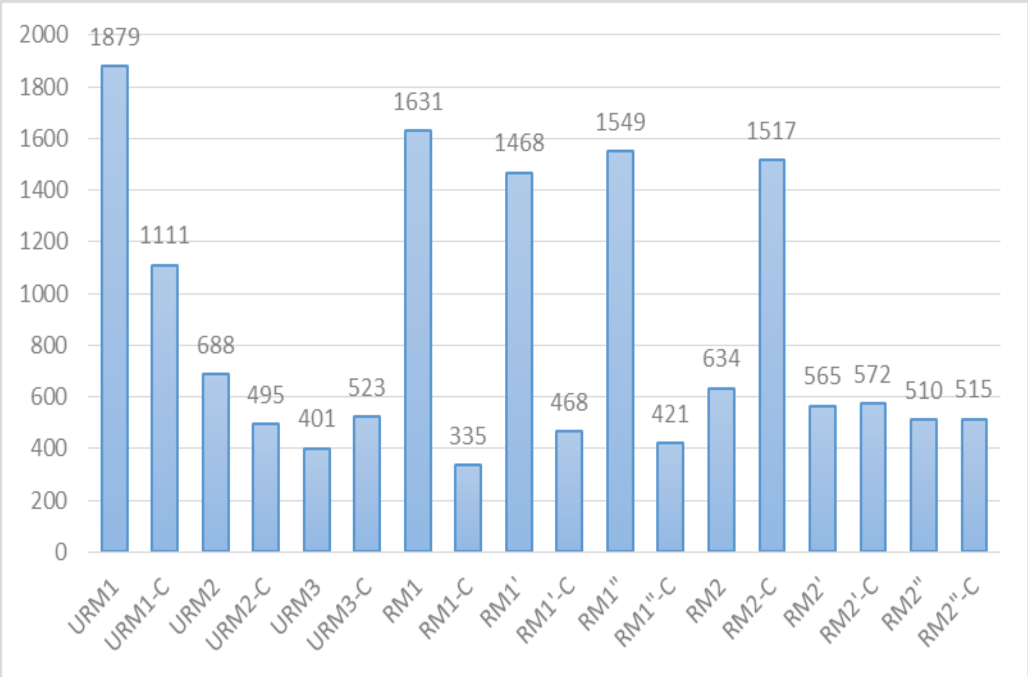


Fig. 5.4. Cumulative energy dissipation [kNmm]

Stiffness analysis

This paragraph evaluates the initial stiffness of the 18 masonry wall tests. We can observe that for the strengthened walls there is a small decrease of initial stiffness in each case, as seen in Fig. 5.5.

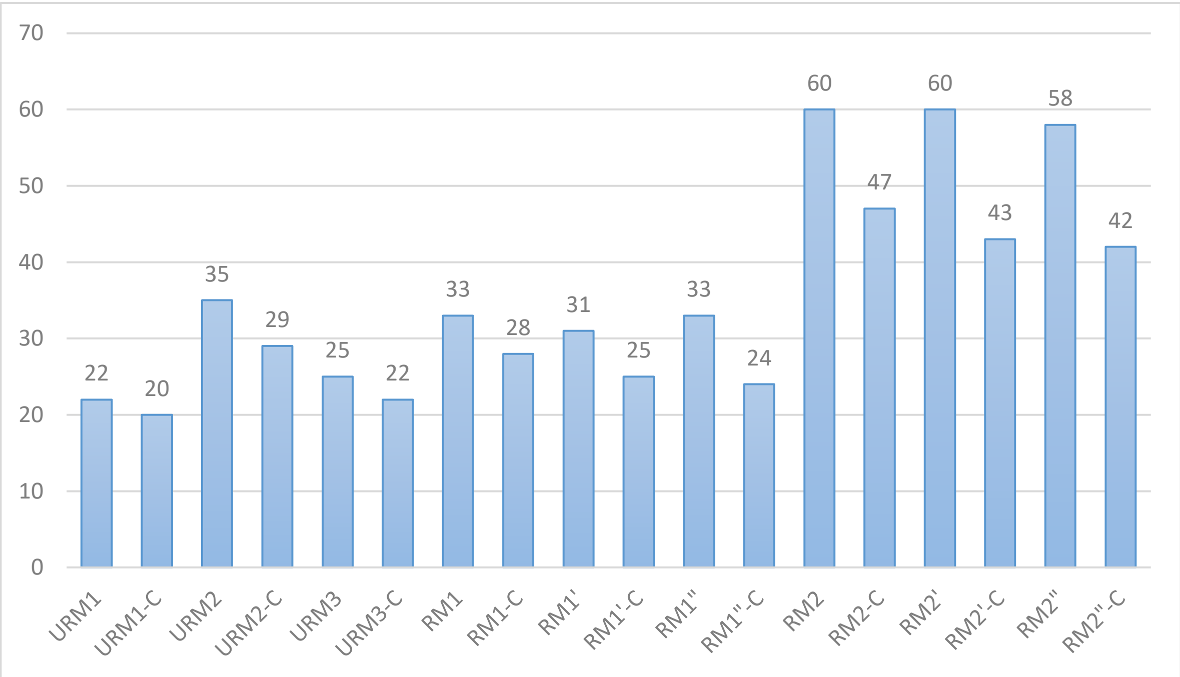


Fig. 5.5. Initial stiffness [kN/mm]

5.5. Numerical analysis and theoretical study

Theoretical study according to design code CR6-2013

Making a calculation according to the CR6-2013 design code, the shear design resistance is:

- unreinforced masonry: $V_{Rd,i}$=41,24 kN
- confined masonry: $V_{Rd,i}$=62,84 kN

If we compare the theoretical results with the experimental program (the maximum horizontal force), we can see the difference in Fig. 5.6.

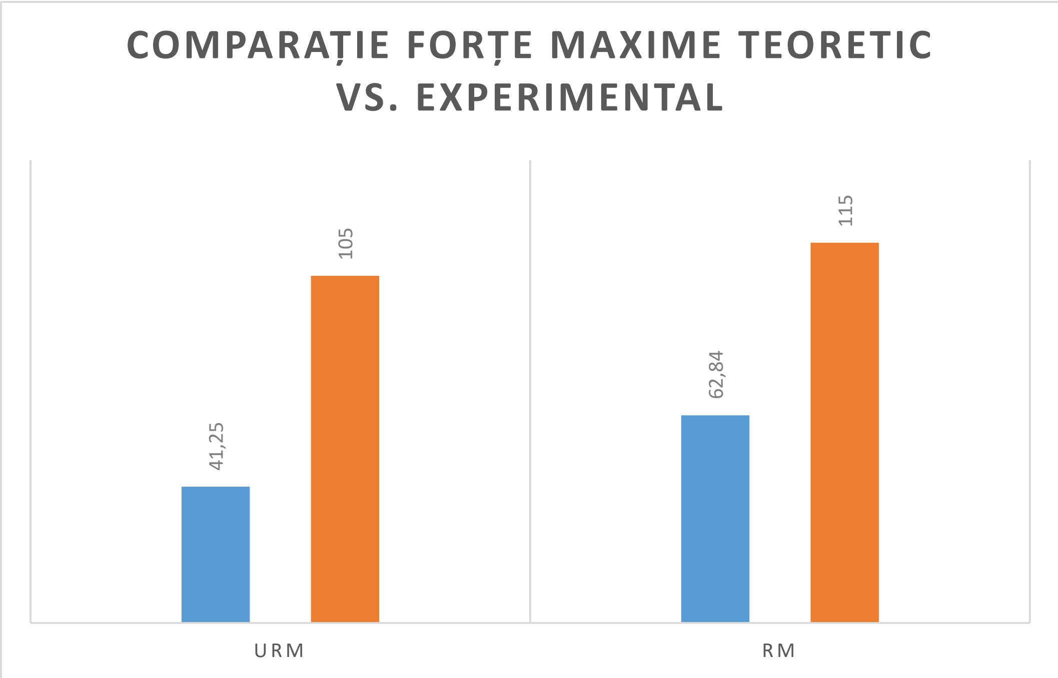


Fig. 5.6. Theoretical study vs. Experimental program – maximum horizontal force [kN]

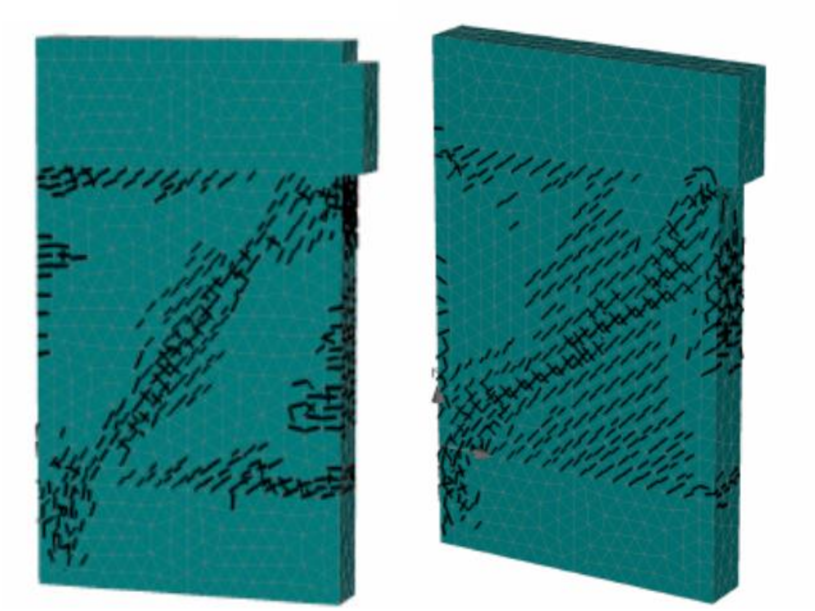
It can be concluded that the current design codes are very conservative in determining the shear resistance of masonry walls, due to the fact that we obtained in our experimental program maximum forces twice as high as the design code provides.

Numerical analysis with ATENA 3D

Numerical analysis were made for the walls in initial state for the unreinforced masonry and for the confined masonry with lateral tie-columns.

The analysis was made by assimilating the masonry with a homogeneous material, similar with concrete, using the material properties calculated according to CR6-2013.

The failure mode obtained was the same with the one obtained in the experimental program, namely the shear failure with cracking along the diagonal of the wall. In this case, the cracking appeared on a single diagonal, due to the fact that to loads were introduced form a single direction. Fig. 5.7.



a) b)

Fig. 5.7. Numerical analysis results a) unreinforced masonry; b) confined masonry

If we compare the results from the numerical analysis with the experimental results, we can see that we are very close to the maximum forces and also to initial stiffness, as seen in Fig. 5.8. and Fig. 5.9.

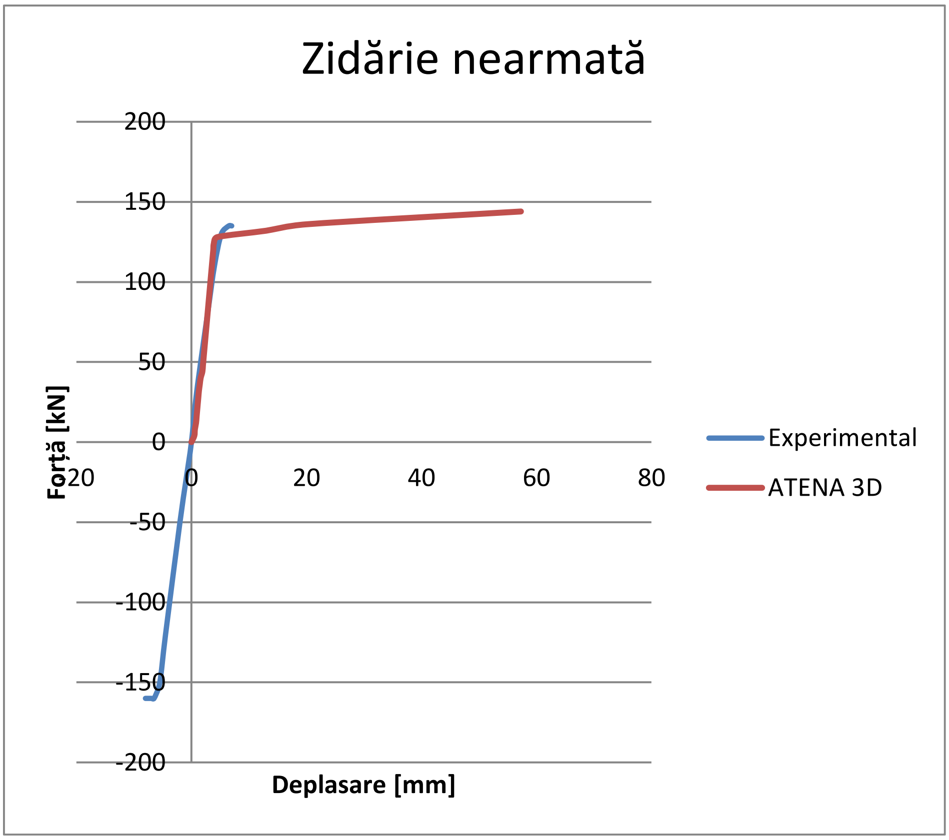


Fig. 5.7. Unreinforced masonry: Atena 3D vs. Experimental

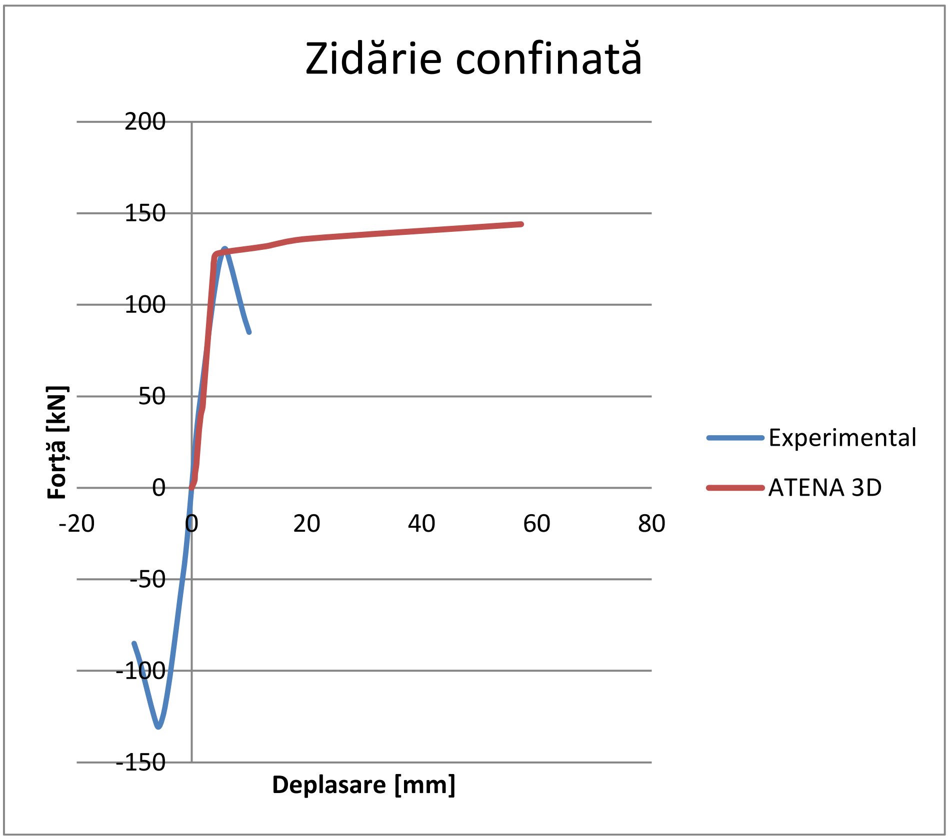


Fig. 5.8. Confined masonry: Atena 3D vs. Experimental

6. CONCLUSIONS. PERSONAL CONTRIBUTIONS

6.1. Conclusions

The paper addresses a topic of great interest at the present time because of the large scale using of the masonry walls built up with ceramic block with vertical hollows especially for residential buildings, but also due to the increased interest for the strengthening of this type of masonry wall using composite materials.

The experimental program was a complex one with different typologies of masonry walls, which were tested in initial state and then strengthened and tested again.

The originality of the paper consists on the experimental research, the interpretation of the results and the formulation of the conclusions of the results obtained. [20], [15]

After analysing the experimental, theoretical and numerical results we can observe the following:

- the failure mode for all the walls tested was the shear failure, with cracking along the diagonals of the walls;
- there were significant differences between the unreinforced masonry walls and the confined wall, with encreases from 20% up to 100%, in maximum horizontal loads;
- at the strengthening of the masonry walls we observed the regaining of the horizontal maximum force was between 80% and 115%, and for the maximum drifts between 50% and 80%;
- the strengthened walls have a decrease of the initial stiffness;
- the correct applying of the composite materials has a great importance in the final results;

- the theoretical study shows that the design codes are very conservative, due to the difference between our experimental program and the theoretical values obtained;
- the numerical analysis can be very useful in assesing the maximum capacity of the walls in initial state.

6.2. Personal contributions

Personal contributions are the following:

- extended bibliographic study on the behaviour of masonry walls subjected to seismic actions, and also masonry walls strengthened using composite materials;

- conceiving and participation in the realization of the experimental stand in the Civil Engineering Laboratory;

- designing an experimental test program that presents innovative features;

- conducting experimental test on 9 masonry walls;

- designing strengthening methods for the experimental elements and testing the 9 walls after their strengthening;

- analysis of the results obtained, compared with the results from numerical and theoretical studies made by the author.

7. CASE STUDY. PRACTICAL APPLICATION

Overview

- Rezidential building P+2E
- Storey level: h_{et}=3,00 m
- Maximum building dimensions: 12,40 x 12,45 m
- Built area: 134,70 mp
- Confined masonry structural system and horizontal reinforcements if necessary
- Interior and exterior walls made with Porotherm 30 and Porotherm 25 ceramic blocks
- Location: Timișoara, a_g=0,20g, T_c=0,7s.
- Geometry of the structure can be observed in Fig. 7.1.

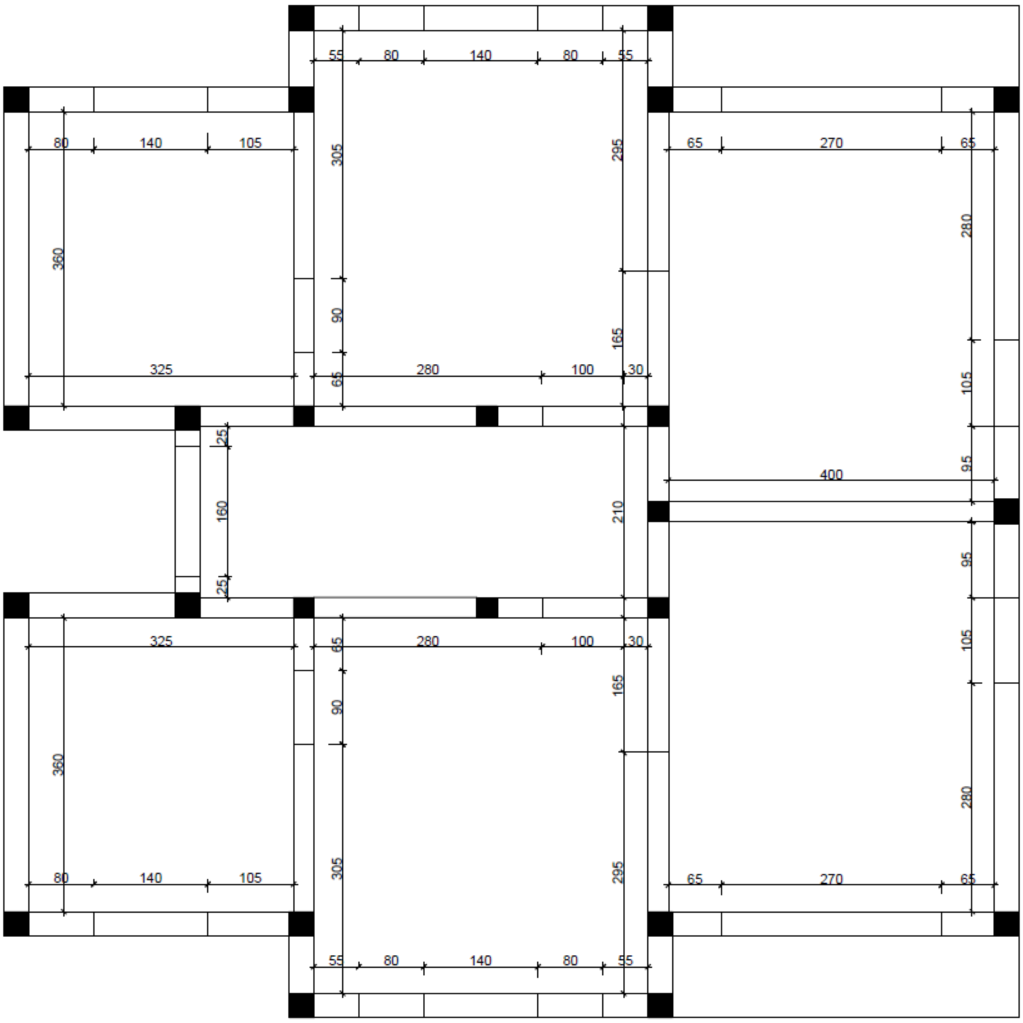


Fig. 7.1. Current level of the building

Shear resistance

According to CR6-2013, the shear resistance is verified with:

$$V_{Rd} \geq 1,25 V_{Edu}$$

The results obtained can be seen în tables 7.1-7.6.

Tabelul 7.1. Shear resistance- Transversal direction – Ground floor

Elem	$\mathbf{V_{Rd}}$**(ZC+AR)**	V_{Edu}	**1,25x** $\mathbf{V_{Edu}}$
	[kN]	[kN]	[kN]
T1	**130,49**	37,61	**47,01**
T2	**141,54**	45,59	**56,99**
T3	**117,67**	27,33	**34,16**
T4	**71,32**	2,95	**3,69**
T5	**121,26**	28,73	**35,91**
T6	**122,64**	31,48	**39,35**
T7	**197,14**	87,89	**109,86**
T8	**278,74**	199,54	**249,43**

T9	**91,03**	13,03	**16,29**
T10	**299,27**	204,29	**255,36**

Tabelul 7.2. Shear resistance- Transversal direction – First floor

Elem	**V_{Rd}(ZC+AR)**	V_{Edu}	**1,25x V_{Edu}**
	[kN]	[kN]	[kN]
T1	**128,92**	37,70	**47,13**
T2	**139,48**	46,06	**57,58**
T3	**116,44**	27,53	**34,41**
T4	**68,53**	3,19	**3,99**
T5	**119,75**	29,51	**36,89**
T6	**121,33**	31,57	**39,46**
T7	**193,15**	88,31	**110,39**
T8	**272,43**	193,25	**241,56**
T9	**90,32**	12,97	**16,21**
T10	**291,32**	200,06	**250,08**

Tabelul 7.3. Shear resistance- Transversal direction – Second floor

Elem	**V_{Rd}(ZC+AR)**	V_{Edu}	**1,25x V_{Edu}**
	[kN]	[kN]	[kN]
T1	**127,13**	37,08	**46,35**
T2	**137,09**	45,29	**56,61**
T3	**115,02**	27,20	**34,00**
T4	**64,98**	2,19	**2,74**
T5	**117,91**	29,34	**36,68**
T6	**119,79**	31,10	**38,88**
T7	**188,51**	85,39	**106,74**
T8	**265,18**	181,43	**226,79**
T9	**89,53**	12,76	**15,95**
T10	**281,90**	185,77	**232,21**

Tabelul 7.4. Shear resistance- Longitudinal direction – Ground floor

Elem	**V_{Rd}(ZC+AR)**	V_{Edu}	**1,25x V_{Edu}**
	[kN]	[kN]	[kN]
L1	**226,31**	196,14	**245,18**
L2	**141,42**	44,74	**55,93**
L3	**163,68**	96,69	**120,86**
L4	**116,48**	24,93	**31,16**
L5	**162,48**	94,18	**117,73**
L6	**176,67**	91,98	**114,98**
L7	**226,24**	148,00	**185,00**
L8	**203,23**	99,29	**124,11**
L9	**98,18**	15,25	**19,06**

Tabelul 7.5. Shear resistance- Longitudinal direction – First floor

Elem	**V_{Rd}(ZC+AR)**	V_{Edu}	**1,25x V_{Edu}**
	[kN]	[kN]	[kN]
L1	**222,08**	180,67	**225,84**
L2	**138,96**	42,53	**53,16**

L3	**161,33**	92,58	**115,73**
L4	**114,97**	24,20	**30,25**
L5	**159,77**	89,95	**112,44**
L6	**173,29**	84,96	**106,20**
L7	**220,82**	136,55	**170,69**
L8	**197,81**	93,06	**116,33**
L9	**97,52**	14,95	**18,69**

Tabelul 7.6. Shear resistance- Longitudinal direction – Second floor

Elem	**V_{Rd}(ZC+AR)**	V_{Edu}	**1,25x V_{Edu}**
	[kN]	[kN]	[kN]
L1	**217,24**	164,39	**205,49**
L2	**136,26**	40,19	**50,24**
L3	**158,74**	87,99	**109,99**
L4	**113,36**	23,39	**29,24**
L5	**156,64**	84,85	**106,06**
L6	**169,37**	76,95	**96,19**
L7	**214,54**	124,09	**155,11**
L8	**191,85**	86,37	**107,96**
L9	**96,84**	14,64	**18,30**

Concluzii:
The shear resistance safety is satisfied in the transversal direction, but on the longitudinal we can observe on the L1 element, on the first floor, that the shear resistance is exceeded.
In order to encrease the shear resistance of the masonry wall we will use reinforcement in horizontal bed mortar joints and this way the entire building has the shear resistance satisfied.
This simplified theoretical study does not take into account the spatial link between the elements of the building, so we considered necessary to make a spatial study using AmQuake software.

Modelling of the building with AmQuake

AmQuake is a software used for assessing the seismic performance of masonry buildings. The software is developed by one of the manufactures of ceramic blocks with vertical hollows used for load-bearing walls.

Seismic performance is assessed using a pushover static-nonlinear analysis and equivalent frame method according to Eurocode 6 and Eurocode 8. The software also make a static linear analysis under gravity loads according to Eurocode 6.

The program offers the possibility of checking the structures according to our national design codes: P100-1/2013 and CR6-2013.

Results:

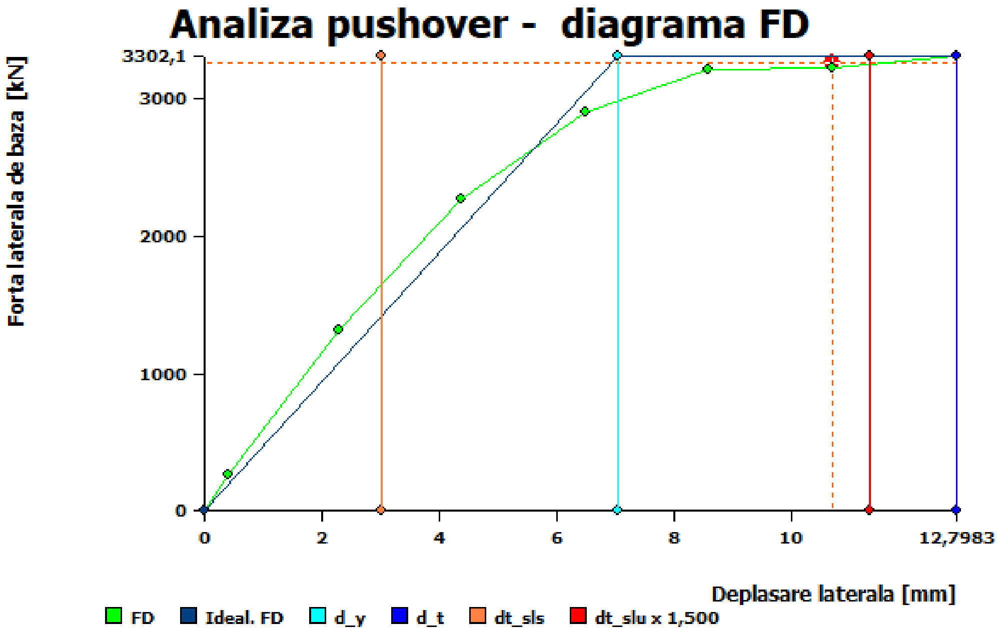


Fig. 7.1. Pushover X+

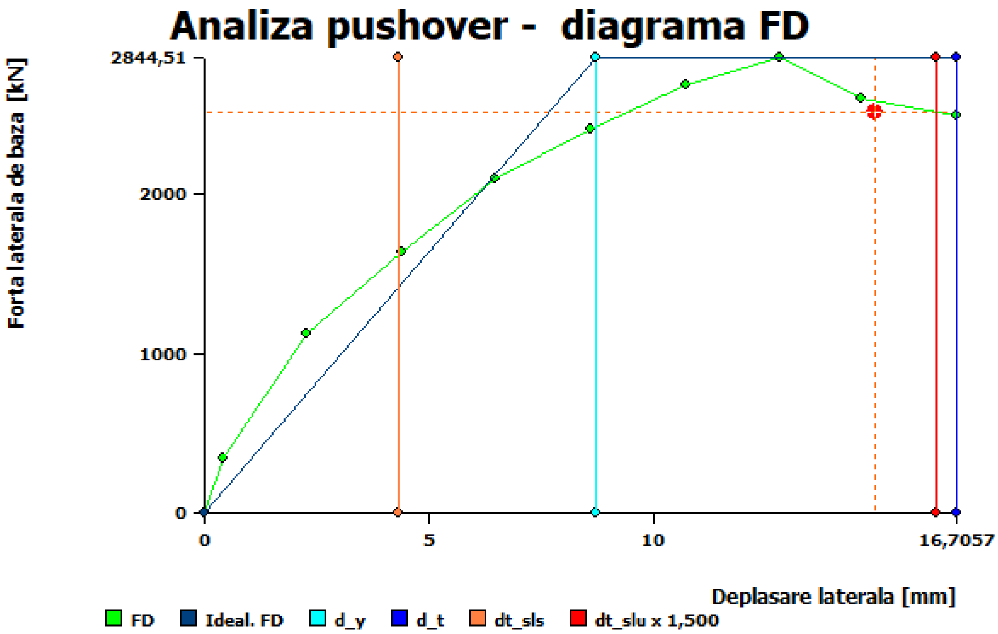


Fig. 7.2. Pushover Y+

AmQuake offers the results on the same building in form of force-displacements diagrams. The force refers to the base shear force of the building and the displacements are at the top of the building.

The criteria that need to be satisfied is based on displacements for the Life Safety state of the building and also the Immediat Occupancy state of the building.

The efficiency of the program, when in need to perform a evaluation on a building is also the reduced time of the modelling and analysis, and the program helped us identify rapidly the element that needed supplimentary horizontal reinforcement.

Also, there is taked into account the spatial behaviour of the building, compared to the theoretical study.

In some cases, the design code has made mandatory this type of analysis, due to the complexity of some buildings geometry.

BIBLIOGRAPHY

[1] CR6-2013 – Cod de proiectare pentru structuri din zidărie.

[2] P100-1/2013 – Cod de proiectare seismică, Partea I – Prevederi de proiectare pentru clădiri. Capitolul 3 – Acțiunea seismică.

[3] P100-1/2013 – Cod de proiectare seismică, Partea I – Prevederi de proiectare pentru clădiri. Capitolul 8 – Prevederi specifice construcțiilor din zidărie.

[4] Cretu D., Demetriu S. – Metode pentru calculul răspunsului seismic în codurile românești de proiectare. Comparații și comentarii. Revista AICPS, pg. 1-9, 2006.

[5] Anastasecu D. – Aspecte ale evoluției reglementărilor tehnice privind protecția antiseismică a construcțiilor din Municipiul Timișoara. Buletinul AGIR nr. 3/2012.

[6] Mărgărit R. – Contribuții privind concepția, calculul și execuția sistemelor structurale cu pereți realizați din blocuri ceramice cu goluri verticale, în zone seismice. Teză de doctorat. UTCB, București, 2011.

[7] Petersen R. – In-plane shear behavior of unreinforced masonry panels strengthened with fibre reinforced polymer strips, The University of Newcastle, Australia, 2009.

[8] Dogariu A. – Seismic retrofitting techniques based on metallic materials of RC and/or masonry buildings. PhD Thesis. Editura Universitatea "Politehnica" Timișoara, 2009.

[9] Nagy-Gyorgy T. – Materiale compozite polimerice pentru consolidarea elementelor din zidărie și beton. Editura Politehnica. Timișoara 2007.

[10] Babatunde S.A. – Review of strengthening techniques for masonry using fibre reinforced polymers. Composite Structures 161, pg. 246-255, 2017.

[11] Dăescu C. – Reabilitarea elementelor de construcție utilizând materiale compozite polimerice. Teză doctorat. Editura Universitatea "Politehnica" Timișoara, 2011.

[12] Niste M.S. – Contribuții privind tehnologia de realizare, reparare și consolidare a construcțiilor civile cu structura din zidărie amplasate în zone seismice. Teză doctorat. 2015.

[13] Fabian A.A. – Study on the performances of composite steel concrete structural shear walls under lateral loads. Teză doctorat. Universitatea Politehnica Timișoara 2012.

[14] Fofiu M. – Retrofitting the precast RC walls panels using externally bonded CFRP

laminates. Teză doctorat. Universitatea Politehnica Timisoara 2017.

[15] Demeter I. – Seismic retrofit of precast RC walls by externally bonded CFRP composites. Teză doctorat. Universitatea Politehnica Timisoara 2011.

[16] Matei C.L. – Contribuții asupra definirii caracteristicilor de rezistență și deformabilitate ale zidăriei utilizate în zone seismice. Teză doctorat. UTCB. 2013.

[17] Laurenco P.B. - Experimental and numerical issues in the modelling of the mechanical behaviour of masonry. Structural Analysis of Historical Constructions II, University of Minho, Barcelona 1998.

[18] Petersen R. – In-plane shear behavior of unreinforced masonry panels strengthened with fibre reinforced polymer strips, The University of Newcastle, Australia, 2009.

[19] Moșoarcă M. – Contribuții la calculul și alcătuirea pereților structurali din beton armat. Teză doctorat. Universitatea Politehnica Timișoara 2003.

[20] da Porto F., Mosele F., Modena C. – In-plane cyclic behaviour of a new reinforced masonry system: Experimental results. Engineering Structures 33, pg. 2584-2596, 2011.