DESIGN OF THE UNREINFORCED MASONRY SHEAR WALL RESISTANCE IN THE HIGH SEISMIC AREAS. CASE STUDY.

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LISBOA 2012

SUMMARY:
Because of diagonal cracking failure (due to main tensile stresses as the main failure criterion) of the unreinforced masonry shear walls subject to lateral in-plane forces and eccentric compression, it is very important to evaluate the capacity of the shear resistance and to perform the strength request verification in the critical section of the wall.
In the Romanian Codes, capacity of shear resistance of the wall (V_{Rd}) is calculated at modified ULS, by limiting tensile length zone, at eccentric compression. Although the numbers of levels are sever reduced with this method is difficult to respect the strength request.
In Romania was realized another methodology (authors: G&R Popescu), with capacity of shear resistance evaluated taking into account the principal failure by diagonal cracking, according with the materials resistance theory. The values of (V_{Rd}) are close with those obtained from tests. Case Study contains walls calculated with the two methods.

Keywords: shear strength, unreinforced masonry wall

1. INTRODUCTION

According to Romanian masonry Codes, the buildings with unreinforced masonry structure in zones with high seismic intensity, are made with unreinforced masonry shear walls, reinforced concrete ties, spandrels and floors. The floors must be rigid diaphragms. In zone with $a_g \geq 0.24g$ (Case study of the article) only one level is accepted by requirement of compliance from Table 8.1,P100/1-2006.

2. DESIGN SHEAR RESISTANCE AND MOMENT OF RESISTANCE CAPACITIES OF THE MASONRY SHEAR WALL

The capacity of the shear resistance (V_{Rd}) for rectangular unreinforced masonry wall will be calculated in two alternatives, presented further.

2.1. Capacity of the shear resistance (V_{RD}) calculated in modified ULS at eccentric compression action in vertical plane of the wall, after Romanian Codes

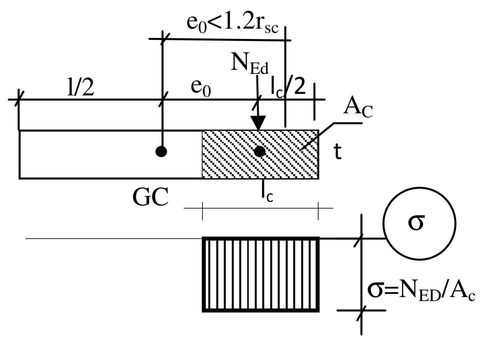


Figure 2.1. Ultimate Limit State (modified); stress distribution

Is assumed that, for shear walls in seismic zones:

- the tension zone is limited by reducing the eccentricity of axial force N_{Ed} (Figure 2.1):

$$e_0 < 1.2\ r_{sc} \tag{2.1}$$

- the compression stresses σ is uniformly distributed in compression zone:

$$\sigma = \frac{N_{Ed}}{A_C} \tag{2.2}$$

where:

e_0 is the eccentricity of axial force N_{Ed}
r_{sc} is limit of the central core in compression zone (l/6 for rectangular section)
l is length of the cross section
N_{Ed} is design axial force from the analyses for the seismic situation
A_C is compressed area on the section of the wall
σ is average of the compression stresses on the A_c

In the present, in Romanian Codes of Masonry, the equation of the capacity of the shear resistance at the base of the masonry wall is presented as follows:

$$V_{Rd} = 0.30 f_{vd} t l_c \quad \text{(CR6+P100/1-2006)} \tag{2.3}$$

where:

$$l_c = l - 2e_0 \qquad \text{(Annex II/2010, 8}^{\text{th}}\text{ Ex; Calculation examples for CR6/2006)} \tag{2.4}$$

$$e_0 = \frac{M_{Ed}}{N_{Ed}} \tag{2.5}$$

With condition: $e_0 < 1.2\ r_{sc}$

M_{Ed} is design value of the bending moment from the analyses for the seismic situation
t is thickness of the cross section of the wall
l_c is the length of the compressed part of the wall
0.30 is a reduction coefficient of the shear resistance capacity at the base of the wall - P100/1; chap. 8 (without scientific justification)
f_{vd} is the design shear strength of masonry

The equation for the moment of resistance is:

$$M_{Rd} = e_0 N_{Ed} = 0.2 l N_{Ed} \tag{2.6}$$

2.2. Capacity of the shear resistance of the wall (V_{Rd}), calculated with the Methodology of failure in diagonal cracking (G&R Popescu)

The capacity of shear resistance is calculated differently for cantilever or embrasure wall (Figure 2.2). It is assumed:

- it is accepted that, for structural unreinforced masonry walls, the main failure criterion is diagonal cracking due to principal tensile stresses;
- the law of Bernoulli applies;
- the mortar in the bed joints at the bottom of the wall has null tension strength;

- the normal compression stresses (σ) have a linear variation on the elastic zones ($\varepsilon \le \varepsilon_c$) of the section;
- on the plastic zones of the section ($\varepsilon > \varepsilon_c$), the normal compression stresses are constant and equal to the compression strength of masonry (f);
- the distribution of shear stresses, (τ over the length of the section conforms to the average flexural shear stress formula (i.e. it is parabolic); the shear stresses are distributed only over the compressed, elastic zone of the section (where $\varepsilon \le \varepsilon_c$).

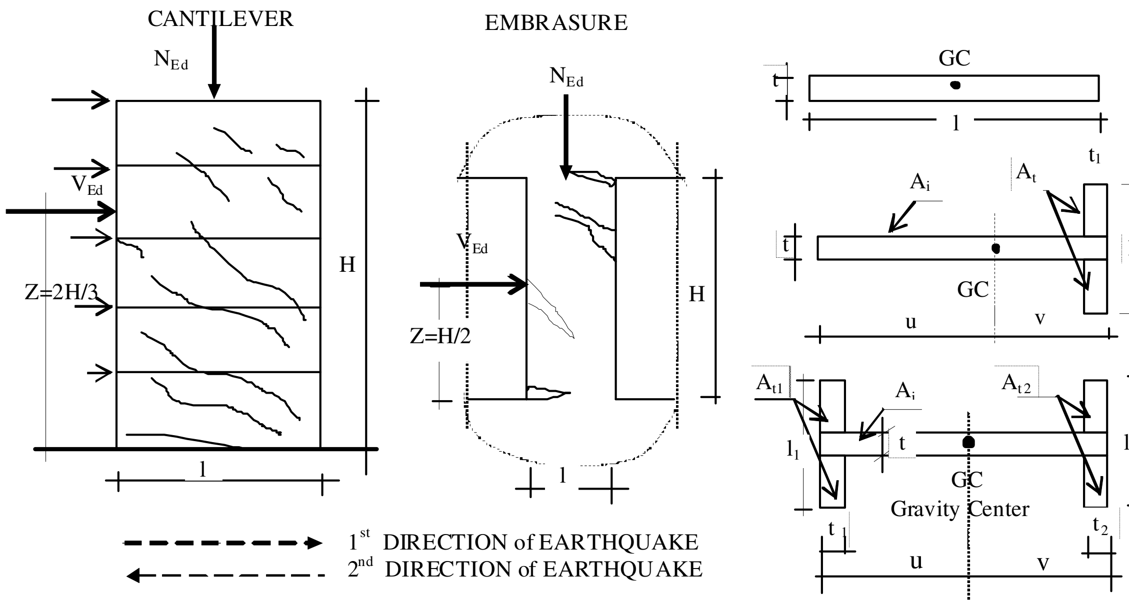


Figure 2.2 Wall subjected to lateral in plane forces and eccentric compression. Geometry of the horizontal section

- the stress-strain curve of masonry is assumed to be of the type shown in Figure 2.3, where ε_c is the yield strain in compression, ε_u is the ultimate strain in compression, f is the design compression strength of masonry, and the letters C and U are the yielding and the ultimate stage, respectively.

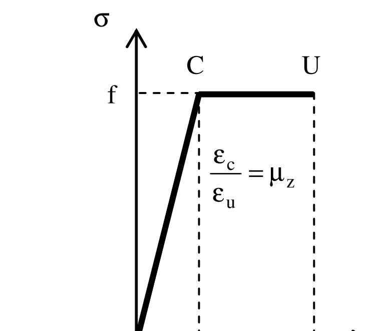


Figure 2.3 Assumed stress-strain relationship for masonry

The capacity of shear resistance at the base of the unreinforced masonry wall (V_{Rd}) is calculated by considering diagonal failure due to main tensile stresses as the main failure criterion. The gradually increasing force (V_{Ed}), determines the wall to pass (theoretically) through successively deformation stages, until diagonal cracking or until ultimate state in eccentric compression (rare situations if geometry of the wall, i.e. aspect ratio, properties of the materials and level of forces acting in plane are favourable).

The method considers three reference stages, characterised by the stresses and strain distribution shown in the Figure 2.4. It is considered actual distribution of the stresses and strain along section length.
The calculation involves the following steps for each deformation stage (F, C , U), and the following capacities of resistance are determined:

- the shear resistance associated to the bending moment of resistance ($V_{Rd,M}$) at each stage of stress and strain distributions: $V_{Rd,M,F}$, $V_{Rd,M,C}$ and $V_{Rd,M,U}$;
- the shear resistance, $V_{Rd,Q}$, corresponding to diagonal failure due to principal tensile stresses, at each stage of stress and strain distributions: $V_{RdQ,F}$, $V_{RdQ,C}$ and $V_{RdQ,U}$

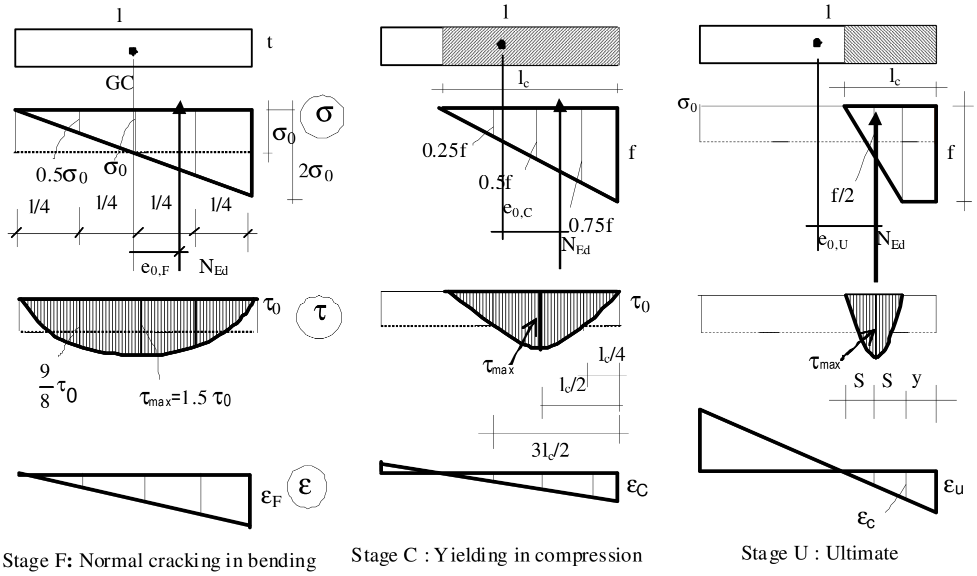


Figure 2.4 The three deformation stages of the wall; stress and strain distributions - cross rectangular section

With two pairs of three values $V_{Rd,M}$ and $V_{Rd,Q}$ must be traced two curves $V_{Rd,M}$-θ and $V_{Rd,Q}$-θ. At the intersection of two curves is found the value of shear resistance V_{Rd} (Figure 2.5).
The seven values :$V_{Rd,M}$,$V_{Rd,Q}$ and V_{Rd} resistances are calculated with a computer software [Cazinds].

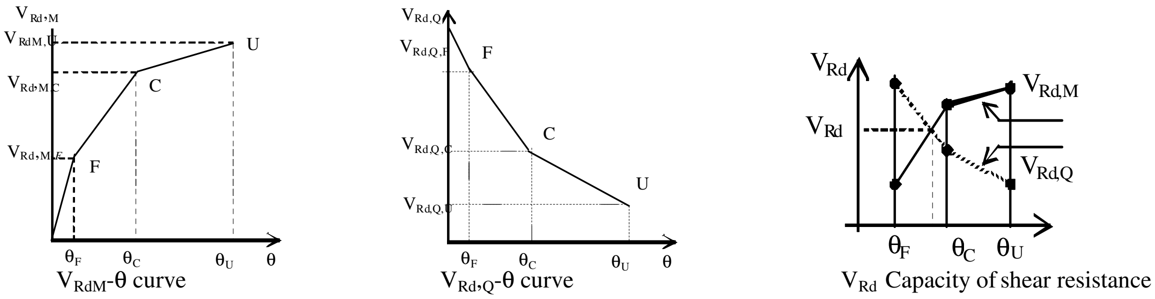


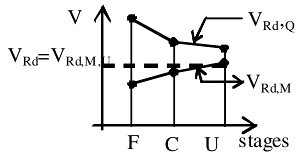
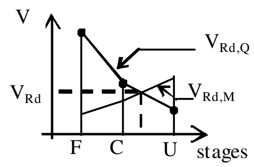
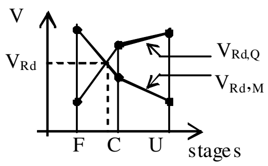
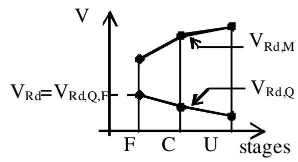
Figure 2.5 Graphical determination of shear resistance capacity

where:

$V_{Rd,M}$ is capacity of shear resistance associated to the bending moment of resistance
$V_{Rd,M,F}$ is capacity of shear resistance F stage
$V_{Rd,M,C}$ is capacity of shear resistance C stage
$V_{Rd,M,U}$ is capacity of shear resistance U stage
$V_{Rd,Q,}$ is capacity of shear resistance corresponding to diagonal failure due to principal tensile stresses
$V_{Rd,Q,F}$ is capacity of shear resistance F stage
$V_{Rd,Q,C}$ is capacity of shear resistance C stage
$V_{Rd,Q,U}$ is capacity of shear resistance U stage

By comparing the values of two pairs of three values $V_{Rd,M}$ and $V_{Rd,Q}$, the failure mode can be determined, as follows (see Table 2.1):
a. ductile failure: MMM (Table 2.1a) - no intersection of two curves and $V_{Rd,M} < V_{Rd,Q}$
b. limited ductility failure: MMQ (Table 2.1b) - intersection of two curves after stage C
c. brittle failure: MQQ: (Table 2.1c) - intersection of two curves before stage C
d. brittle failure: QQQ: (Table 2.1d) - no intersection of two curves and $V_{Rd,Q} < V_{Rd,M}$

Table 2.1 Failure mode

a. Failure mode MMM	b. Failure mode MMQ
	
c. Failure mode MQQ	**d. Failure mode QQQ**
	

3. THE STRENGTH REQUEST

3.1 In accordance with the Romanian Codes – modified ULS, and eccentric compression action in plane of the wall

The strength request on the cross section of the wall is accomplished if the next equations are accomplished (for modified ULS). Else, the cross section area or the materials resistance will be changed. All the shear walls in a building structure must respect all the strength request.

$$M_{Rd} \geq M_{Ed} \tag{3.1}$$

$$V_{Rd} \geq 1.25 V_{Edu} \tag{3.2}$$

$$V_{Rd} \leq q V_{Ed} \tag{3.3}$$

if $$V_{Rd} > q V_{Ed} \tag{3.4}$$

and $$M_{Rd} \geq q M_{Ed} \tag{3.5}$$

than $$V_{Rd} = q V_{Ed} \tag{3.6}$$

where:
V_{Edu} is design value of shear force associated of the bending resistance
1.25 is a coefficient without scientific justification
q is the behaviour factor

3.2 In accordance with the Methodology of failure in diagonal cracking (G&R Popescu)

The strength request on the cross section of the wall has the same limitation for the capacity of shear resistance (see eqn. 3.3÷3.6) and in addition the following equations:

$$M_{Rd} \geq M_{Ed} \tag{3.7}$$

$$V_{Rd} \geq V_{Ed} \tag{3.8}$$

Equations for the capacity of the moment of resistance are:

$$M_{Rd} = V_{Rd}\frac{2H}{3} \quad \text{- for cantilever walls} \tag{3.9a}$$

$$M_{Rd} = V_{Rd}\frac{H}{2} \quad \text{- for embrasure walls} \tag{3.9b}$$

The comparison (Eqn. 3.8) is made with the elastic shear force because the author's Methodology captures the right moment of diagonal cracking of the masonry wall.

4. CASE STUDY

Three buildings with unreinforced masonry structure, one to three levels, the same conformation in layout, situated in a seismic zone: ag=0.24g, T_C=1.6s (Figure 4.1) were calculated. The structure has regularity, both horizontally and vertically and the value of the behaviour factor is q=1.87 (single level) and q=2.20 (more levels).
Were selected three walls, on the two main directions of the earthquake action: ML1 (1-axis, between C and D-axis), MT1 (A-axis, between 1 and 7-axis) and MT2 (D-axis between 5 and 3 axis)-Figures 4.1÷4.3. The results are presented in the tables 4.1÷4.2 in comparing with the capacities calculated after P100/1-2006 and CR6/2006 and Annex II and Methodology of principal creaking in diagonal failure.
The main materials used are: mortar M10, masonry unit C10 in the walls, reinforced concrete C16/20 in the ties and floors. The characteristics of the materials are: E_c=270000 daN/cm^2, E_m =43000 daN/cm^2; f=f_d=14.7 daN/cm^2, f_p=f_{vd} =1.23 daN/cm^2.

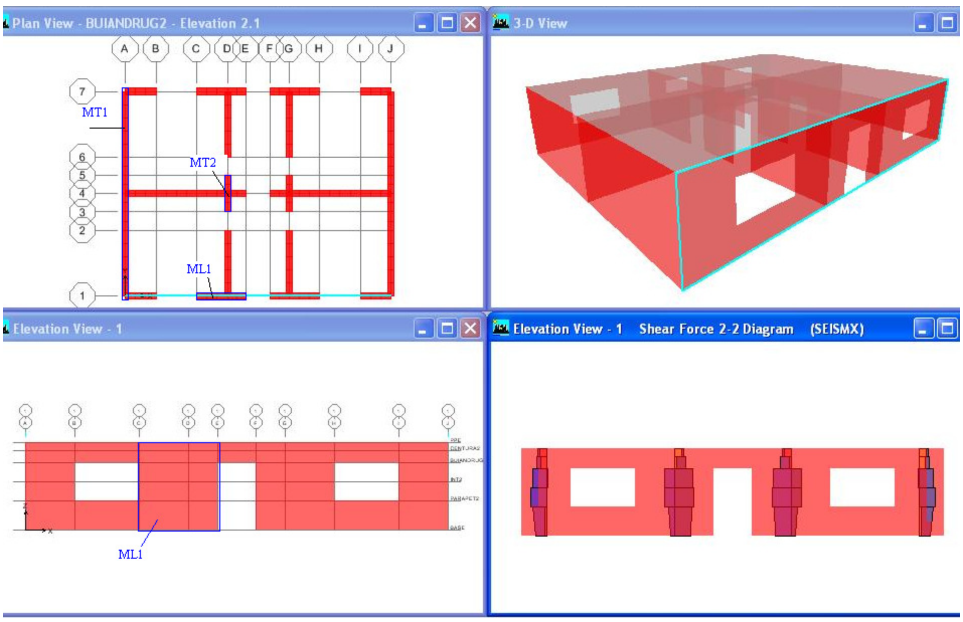


Figure 4.1 Unreinforced masonry building - single level [Etabs software]

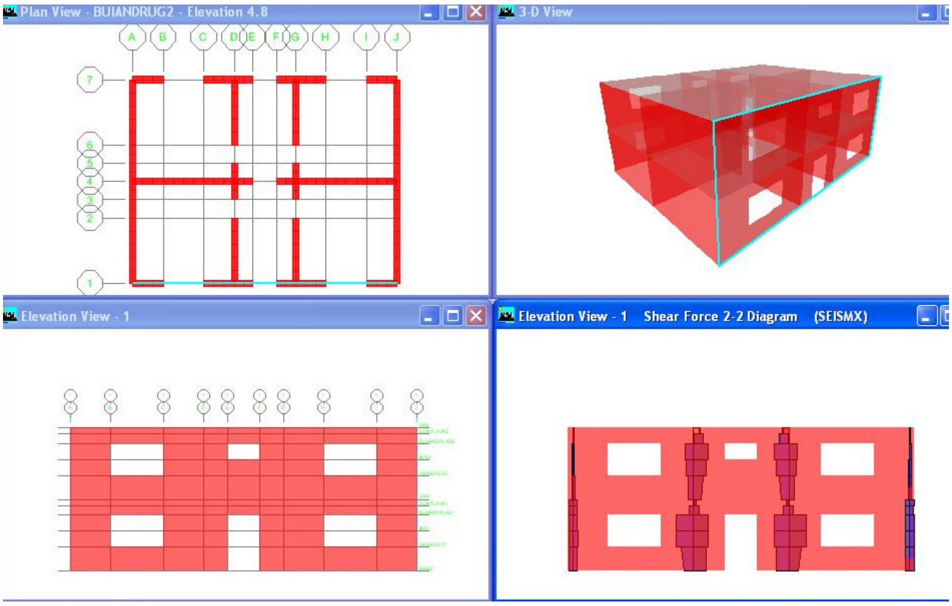


Figure 4.2 Unreinforced masonry building - two levels [Etabs software]

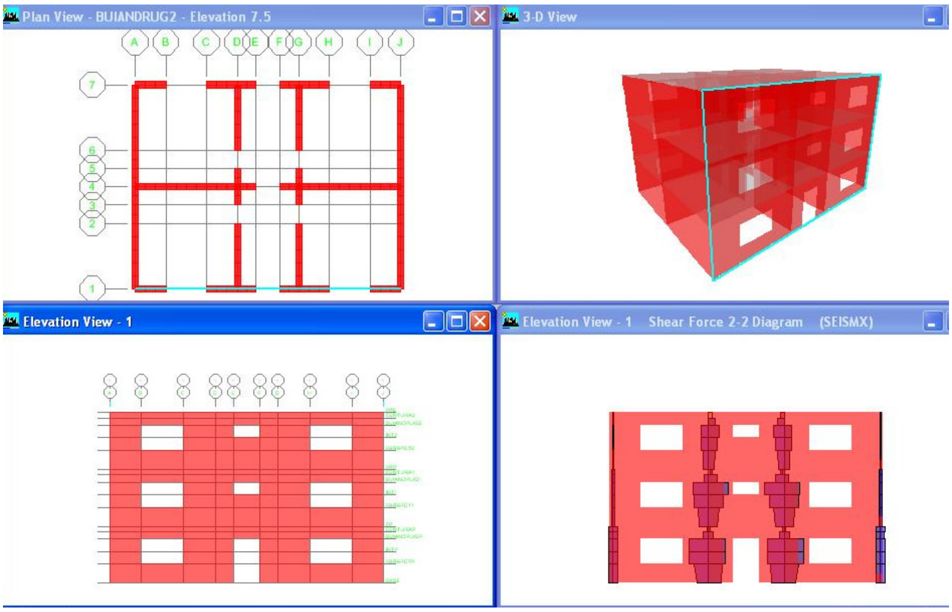


Figure 4.3 Unreinforced masonry building - three levels [Etabs software]

The main materials used are:
-mortar M10,
-masonry unit C10 in the walls,
-reinforced concrete C16/20 in the ties and floors.
The characteristics of the materials are: E_c=270000 daN/cm^2, E_m =43000 daN/cm^2; $f=f_d$=14.7 daN/cm^2, $f_p=f_{vd}$=1.23 daN/cm^2.
The requirement of structural regularity of the building is respected in plan and in elevation and all the compliance conditions of the Romanian Codes.
The seismic calculation of the building was done with a specialized software structure [Etabs].
In the following tables are the results (N_{Ed}), (V_{Ed}), (M_{Ed}), in comparison with the shear and the moment strength, evaluated with the two above mentioned methodologies.
Two tables with design forces and capacities which were calculated after both methodologies are presented in comparing in the further.

Table 4.1. Walls in a building with one to three levels – (CR6/2006 - Annex II/2010 Examples for)

Cross section wall name [m]	N_0 of level/H [m]	q	l_c (2.4) [m]	N_{Ed} [kN]	V_{Ed} [kN]	qV_{Ed} [kN]	V_{Rd} [kN]	0.3x V_{Rd} (2.3) [kN]	1.25x V_{Edu} [kN]	M_{Ed} [kNm]	M_{Rd} (2.6) [kNm]	qM_{Ed} [kNm]
0	1	2	3	4	5	6	7	8	9	10	11	12
ML1-rectangl 0.30x2.40	1/ 2.7	1.87	1.75	82	28	51	(64) 51	15.3	50.5	27	39	50
	2/ 5.4	2.20	1.71	170	45	99	68	20.4	79.5	58	82	128
	3/ 8.1	2.20	1.68	248	77	169	67	20.1	114.5	100	119	220
MT1-double T 0.3x10.3 2x0.3x1.65	1/ 2.7	1.87	8.94	404	90	166	(322) 166	49.8	601.7	215	1150	402
	2/ 5.4	2.20	8.11	792	192	422	315	94.5	610.5	747	1900	1643
	3/ 8.1	2.20	7.05	1194	334	735	293	87.9	804.2	1760	3390	3872
MT2-rectangl 0.30x1.80	1/ 2.7	1.87	1.45	79	12	22	(54) 22	6.6	27.8	14	28	26
	2/ 5.4	2.20	1.37	145	27	59	55	16.5	56.7	31	52	68
	3/ 8.1	2.20	1.33	203	47	103	53	16.5	75.2	57	73	103

Strength request for bending moment is accomplished for all walls in study - col (10) in comparing with col (11): $M_{Rd}>M_{Ed}$,.
Strength request for shear force is not accomplished for all walls in study - col (8) in comparing with col (9) – even for those in one level building: $0.3V_{Rd}<1.25\ V_{Ed,u}$
This will result, in consequence, an increase of the dimensions of the walls cross-sections.

Table 4.2. Walls in a building with one to three levels – (G&R Popescu Methodology)

Cross section wall name [m]	N_0 of level/H [m]	q	N_{Ed} [kN]	V_{Ed} [kN]	qV_{Ed} [kN]	V_{Rd} (CAZIN) [kN]	M_{Ed} [kNm]	qM_{Ed} [kNm]	M_{Rd} (3.9a) [kNm]	Failure Mode
0	1	2	3	4	5	6	7	8	9	10
ML1-rectangl 0.30x2.40	1/ 2.7	1.87	82	28	51	40	27	50	(72) 50	MQQ
	2/ 5.4	2.20	170	45	99	45	58	128	(162) 128	MMQ
	3/ 8.1	2.20	248	77	169	41	100	220	220	MMQ
MT1-double T 0.3x10.3 2x0.3x1.65	1/ 2.7	1.87	404	90	166	(400) 166	215	402	(720) 402	QQQ
	2/ 5.4	2.20	792	192	422	(450) 422	747	1643	1620	MQQ
	3/ 8.1	2.20	1194	334	735	520	1760	3872	2808	MQQ
MT2-rectangl 0.30x1.80	1/ 2.7	1.87	79	12	22	(31) 22	14	26	(56) 26	MQQ
	2/ 5.4	2.20	145	27	59	28	31	68	(103) 68	MMQ
	3/ 8.1	2.20	203	47	103	27	57	103	(132) 103	MMM

Strength request for bending moment is accomplished for all the walls in study - col (7) vs col (9): $M_{Rd}>M_{Ed,}$
Strength request for shear force is not accomplished only for those in three level building- col (4) vs col (6) : $V_{Rd}>V_{Ed,}$ in a building with one and two levels,
and
$V_{Rd}<V_{Ed,}$ in a building with three levels; this case, will result, in consequence, an increase of the dimensions of the walls cross-sections.
May be considered the building can have two levels and is not necessary to modify anything in the structure or in the materials from the masonry walls.

5. CONCLUSIONS

It can considers that the Methodology of failure in diagonal cracking is close of the true behaviour of the masonry walls because:

5.1 This methodology is build on the theory of materials resistance and detects exactly the moment of the cracking in diagonal failure of the unreinforced masonry walls, due to main tensile stresses. According to this methodology, with a regularity compliance on horizontal and vertical directions it is possible to build edifices with more than one level, in zones with a_g>0.24g.

5.2 Romanian codes CR6 and P100/1-2006 are based on assumptions that lead to undervalued shear resistance capacities. For this reason, the results cannot meet simultaneous the strength requests for the bending moment and for the shear resistance than for the limited height of the buildings. Even a single level building requires structural walls with unusable and uneconomical cross section for a common design of architecture.

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CAZINDS- Masonry structure design software.

ETABS – Integrated Building Design Software

CR6 /2006 – Romanian Masonry Design Code

P100/First part 2006 – Romanian Design Code of Structures for Earthquake Resistance –Chapter 8 Masonry

Eurocode 6 – Design of the Masonry Structures

Eurocode 8 –Design of structures for earthquake resistance

SR EN 1998-1:2004/NA: 2008 Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for building National Annex