Confined Masonry Shear Walls in Seismic Areas

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ABSTRACT:
In seismic zones, the masonry buildings designed according to the new seismic codes, have reinforced or confined shear walls. They are evaluated by determining the cross-sectional forces using specialized software. Cross sectional verification relies on comparing the design shear load and design bending moment applied on the wall (V_{Ed}, M_{Ed}), with the representative design shear resistance and design bending resistance (V_{Rd}, M_{Rd}) : $V_{Ed} < V_{Rd}$ and $M_{Ed} < M_{Rd}$. However, the resistance values for shear walls determined in ultimate state seem to be conservative, because of extremely underestimate of shear resistance of masonry. Consequently, for a confined masonry structure, the strength requests can be satisfied only with an important consumption of materials, with a limited architectural plan and with high costs. Therefore, another detailed methodology is proposed by the authors, the resistance capacity calculated, seems to be well in line with experimental results obtained.

Keywords: confined masonry, design, resistance capacity

1. INTRODUCTION

In Romania, country with a high seismic risk, the most used confined masonry structural system is the one with concrete pillars and ties included into the masonry wall Figure 3.1. The Romanian Masonry Code rules establish the position of the concrete pillars and ties.
The rectangular section of masonry with pillars is transformed into equivalent unreinforced masonry section, "I" shaped (the pillar become a flange, Figures 3.1 and 3.2).
The design strength capacities (V_{Rd}, M_{Rd}) of the confined masonry shear wall are a sum of the unreinforced masonry capacities (V_{Rd1},M_{Rd1}) and of the additional capacities (V_{Rd2}, M_{Rd2}) of the reinforcement steel of the pillars.
In the following, two evaluation ways of the capacities of resistance (V_{RD}, M_{RD}) are presented: after Romanian Codes, and after the author's alternative methodology and the Case Studies with the compared results.

2. UNREINFORCED MASONRY: DESIGN CAPACITIES OF RESISTANCE (V_{RD}, M_{RD})

The shear resistance (V_{Rd}) and the moment of resistance (M_{Rd}) for a rectangular unreinforced masonry shear wall, subject to lateral in-plane forces and eccentric compression, will be calculated in two ways, presented in the following.

2.1. Shear Resistance and Moment of Resistance in according with Romanian Code (CR6/1-06)

The shear resistance formula is:

$$V_{Rd} = 0.30 f_{vd} t l_c \qquad (2.1)$$

where:
0.30 is a reduction coefficient (unspecified source)
f_{vd} is the shear strength of the masonry
t is the thickness of the cross section of the wall
l_c is the length of the compressed part of the wall

The design moment of resistance formula is:

$$M_{Rd} = 0.2lN_{Ed} \tag{2.2}$$

where:
0.2 is the coefficient witch limits the eccentricity of the axial compression force (N_{Ed})
l is the length of the cross section
N_{Ed} is the design value of the vertical load

2.2 Shear Resistance and Moment of Resistance in according with the authors' methodology

Unreinforced masonry elements are subjected to constant axial loads (N_{Ed}) and to gradually increasing lateral forces (V_{Ed}), as shown in Figure 2.1.

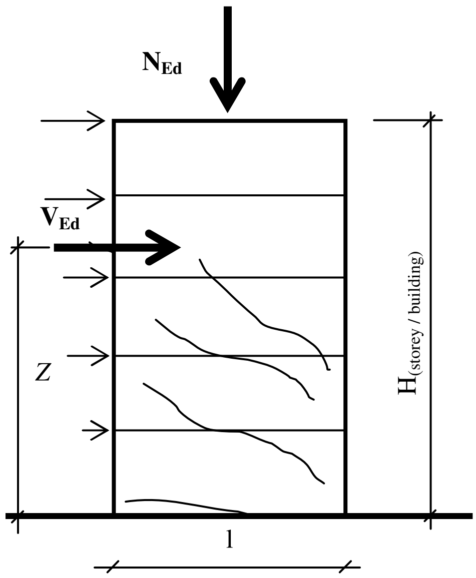

Figure 2.1. Masonry shear wall subjected to lateral in-plane forces and eccentric compression

The shear resistance of unreinforced masonry wall (V_{Rd}) is calculated by considering diagonal failure due to main tensile stresses as the main failure criterion.
The section at the base of a structural unreinforced masonry wall pass through successive deformation stages, as the lateral force gradually increases. The method considers three reference stages, characterised by the stresses distribution shown in Figures 2.2, to 2.4.
The calculation involves the following steps for each deformation stage (*F*, *C*, *U*) and the following expectable resistances are determined:
a) The values of the shear resistance associated to the bending moment of resistance (V_M) with the stress and strain distributions, is shown in Figures 2.2 to 2.4: $V_{M,F}$, $V_{M,C}$ and $V_{M,U}$;
b) The values of the shear resistance, V_Q, (corresponding to diagonal failure due to principal tensile stresses), is shown in Figures 2.2 to 2.4, $V_{Q,F}$, $V_{Q,C}$ and $V_{Q,U}$.
c) With V_M and V_Q values, the interaction curves V_M-θ (Figure 2.5) and V_Q-θ (Figure 2.6) can be design. The intersection of both curves gives the value of shear resistance V_R (Figure 2.7).

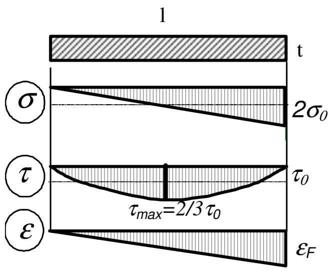

Figure 2.2 Stage F: normal cracking in bending

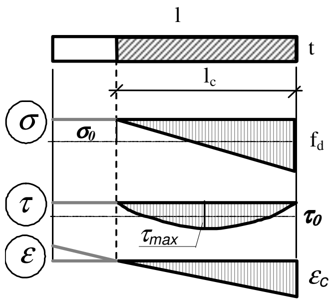

Figure 2.3. Stage C: yielding in compression

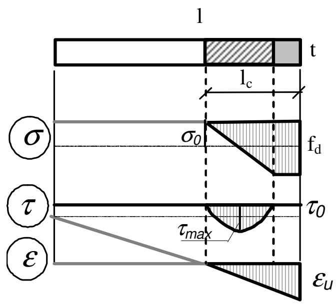

Figure 2.4.Stage U: ultimate

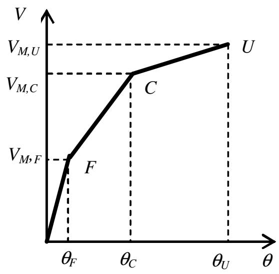

Figure 2.5. V_M-θ curve

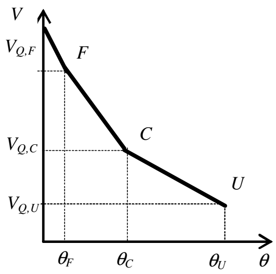

Figure 2.6. V_Q-θ curve

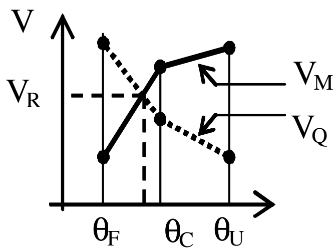

Figure 2.7.V_R Shear resistance

The moment of resistance is calculated with Eqn.2.3 similar with the Code formula:

$$M_{Rd} = y_{zc,i} N_{Ed} \tag{2.3}$$

where:

$y_{zc,i}$ is the distance between centre of gravity of the entire wall section and centre of gravity of the compressed part of the section

N_{Ed} is the design value of the vertical load

The values of V_M, V_Q, V_R are calculated with a computer program realized by the authors.

3.CONFINED MASONRY: DESIGN CAPACITIES OF RESISTANCE (V_{RD}, M_{RD})

The shear resistance (V_{Rd}) and the moment of resistance (M_{Rd}) for a confined masonry shear wall, subject to lateral in-plane forces and eccentric compression, will be calculate by two ways, presented in the following.

The rectangular masonry section with two pillars on the both ends (Figure 3.1) is transformed in an equivalent „I"-shaped section (Figure 3.2). V_{Rd1} is the design unreinforced masonry shear resistance and V_{Rd2} is the design shear resistance of the steel reinforcement in concrete compressed pillar. Both are calculated with the equivalent unreinforced masonry section.

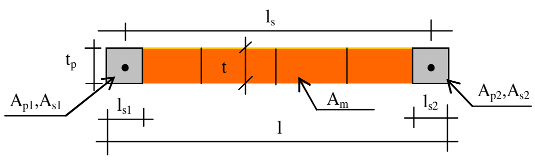

Figure 3.1 Confined masonry shear wall with concrete pillars - horizontal section

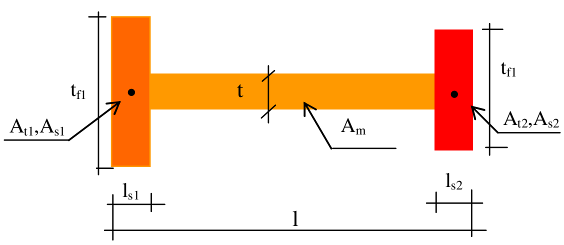

Figure 3.2 Equivalent I shape section of unreinforced masonry shear

The formula that transforms the concrete pillar in a masonry flange is:

$$A_t = 0.75 A_s \frac{f_c}{f} \tag{3.1}$$

where:

A_s is the cross sectional area of steel reinforcement in compressed pillar

A_t is the equivalent masonry area of the flange

f_c is the strength in compression of the concrete

f is the strength in compression of the masonry

3.1 Formulas for equivalent section of the confined masonry wall according to Romanian Code(CR6/1-06)

$$V_{Rd} = 0.30 V_{Rd1} + V_{Rd2} = 0.30 f_{Vd} * t * l_c + 0.2 A_{asc} * f_{yd} \tag{3.2}$$

$$M_{Rd} = M_{Rd1} + M_{Rd2} = y_{zc,i} N_{Ed} + l_s * A_s * f_{yd} \tag{3.3}$$

where:

N_{Ed} is the design value of the vertical load

V_{Rd1} is the design masonry shear resistance

V_{Rd2} is the design shear resistance of the reinforcement

M_{Rd1} is the design value of the masonry moment of resistance

M_{Rd2} is the design value of additional moment of the reinforcement

f_{vd} is the design shear strength of masonry

t is the thickness of the compressed equivalent cross section wall

l_c is the length of the compressed zone

A_{asc} is the cross sectional area of the vertical reinforcement in compressed pillar

f_{yd} is the design tensile strength of reinforcing steel

$y_{zc,i}$ is the distance between centre of gravity of the entire wall section and centre of gravity of the compressed part of the section

l_s is the distance between the reinforcement pillars' centres of gravity

3.2 Formulas for equivalent section of the confined masonry wall in according to the author's methodology

The design shear resistance of the confined section is the shear resistance of the "I"-shaped equivalent masonry wall section and of the additional shear resistance of the reinforcing steel (in concrete pillars). The shear resistance V_Q and bending resistance V_M for unreinforced equivalent section is determined according Eqn.2.2. The additional capacities of the reinforcing steel of the pillars $V_{S,Q}$ and $V_{S,M}$ are determined and added at (C) and (U) stages (Figures 3.3÷3.5):

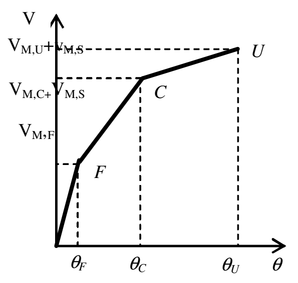

Figure 3.3. V_M-θ curve

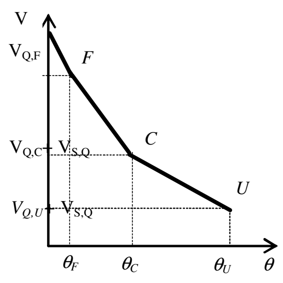

Figure 3.4. V_Q-θ curve

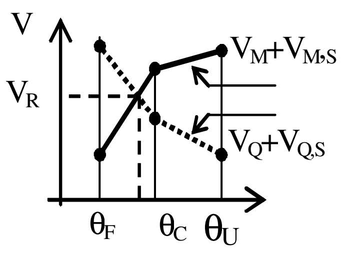

Figure 3.5. V_R Shear resistance

The moment of resistance is calculated with Eqn.3.3 similar with the Code formula.

3.3 Verification of the strength request

If the next equations are respected, the strength request on the cross section is satisfied. Else, the cross section area or the materials must be change.

$$V_{Rd} \geq 1.25 V_{Ed} \tag{3.5}$$

$$V_{Rd} \leq q V_{Ed} \tag{3.6}$$

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

Alternatively, if:

$$V_{Rd} > q V_{Ed} \tag{3.8}$$

And:

$$M_{Rd} \geq q M_{Ed} \tag{3.9}$$

Than:

$$V_{Rd} = q V_{Ed} \tag{3.10}$$

4. CASE STUDIES

4.1 Unreinforced Masonry Walls

In Table 4.1 is presented the comparison between the resistance capacities.

where:

N_{Ed} is the test value of the vertical load
H_{CR} is the values of the test lateral cracking force
V_{RK} is the shear resistance of the specimens, determined by method 2.1
V_R is the shear resistance of the specimens, determined by the method 2.2
f_k is the characteristic test compressive strength of masonry
l is the length of the specimen section
H is the high of the specimen section
t is the thickness of the specimen section.

$\tau_R = \dfrac{V_R}{l * t}$ is the shear stress on the section (determined with the computer program) (4.1)

The shear resistance value (V_R) that is determined by the authors' methodology (col 6) is nearest of the test lateral cracking force value (H_{cr}). The ratio H_{cr}/V_R (col.8) is between 1.01÷ 1.33.
The shear resistance value (V_{Rk}) that is determined by the Code (col 9) is lesser than the test lateral cracking force value (H_{cr}). The ratio H_{cr}/V_{Rk} (col.10) is between 6.33÷10.67.

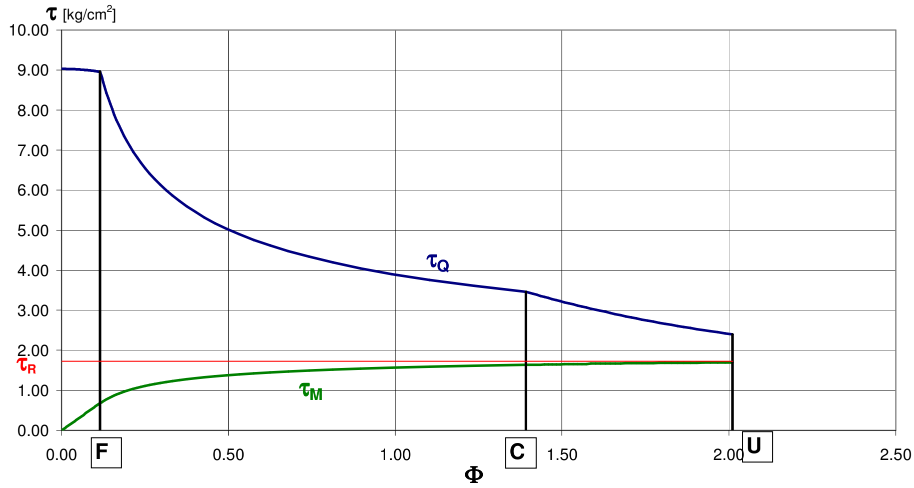

Figure 4.1. Diagrams with the value of shear stress on the section of the specimen BNL1

Table 4.1 Specimens with H/l=1.5

Wall series	Dimension: lxHxt	N test	σ_d test	fk test	H_{cr} test	V_R	τ_R	H_{cr}/V_R	V_{RK}	H_{cr}/V_{Rk}
	cm	kg	kg/cm^2	kg/cm^2	kg	kg	kg/cm^2	_	kg	_
0	1	2	3	4	5	6	7	8	9	10
BNL Fully joints	1- 102.8x151.0x30	18390	6.00	41.30	5453	5210	1.69	1.05	641.1	8.51
	2- 103.0x151.0x30	36770	11.90	41.30	9617	8350	2.70	1.15	1519.2	6.33
	3- 103.2x151.5x30	13890	6.00	41.30	5493	5220	1.69	1.05	641.1	8.57
	4- 102.5x151.4x30	36770	11.90	41.30	11200	8390	2.70	1.33	1519.2	7.37
	5- 102.7x151.1x30	36770	11.90	41.30	10936	8310	2.70	1.32	1519.2	7.20
	6- 102.6x150.8x30	18390	6.00	41.30	6313	5210	1.69	1.21	641.1	9.85
BGL Head filled at	1- 98.9x151.3x30	35340	11.90	43.10	10173	7850	2.65	1.30		
	2- 98.7x151.1x30	35340	11.90	43.10	10292	7840	2.65	1.31	1455.9	6.99
	3- 98.8x150.7x30	35340	11.90	43.10	9384	7870	2.65	1.19		
BPL - joints in the	1- 98.5x150.8x30	35340	11.90	62.80	10633	8990	3.04	1.18		
	2- 98.5x150.9x30	35340	11.90	62.80	10773	8990	3.04	1.20	996.3	10.67
	3- 98.6x150.7x30	35340	11.90	62.80	10450	9010	3.05	1.16		
BZL Dry groove	1- 98.8x151.0x30	35340	11.90	62.40	9640	9000	3.04	1.07		
	2- 98.7x151.2x30	35340	11.90	62.40	9670	8980	3.03	1.08	1005.6	9.59
	3- 98.6x150.8x30	35340	11.90	62.40	9100	8990	3.04	1.01		

4.2 Reinforced Masonry Walls

A masonry building with two levels situated in a hazard seismic zone, with ag=0.24g, T_C=1.6 s was calculated at lateral seismic action with a specialized soft. The structure is confined masonry with concrete pillars, ties and floor at each level. The value of the behavior factor is q=3.125. The characteristics of the materials are: mortar M10, masonry unit C10, concrete in the ties, pillars and floors C16/20, steel reinforcement PC52 (f_s=300N/mm^2).
The two types of capacities were compared with the design values, for a rectangular wall with pillars at both ends.

4.2.1. Verification of the strength request: (V_{Rd} , M_{Rd}) determined conform Romanian Code : (CR6/1-06)

$$M_{Ed} = 6.0tm \leq M_{Rd} = 68tm \quad (4.2)$$

$$1.25V_{Ed} = 6.19t > V_{Rd} = 5.36t \quad (4.3)$$

$$qM_{Ed} = 18.78tm \leq M_{Rd} = 68tm \quad (4.4)$$

$$qV_{Ed} = 15.47t > V_{Rd} = 5.36t \quad (4.5)$$

It can be seen that the strength request is not respected for the P100/1-06 Romanian Code's shear resistance relationship.

4.2.2. Verification of the strength request : (V_{Rd} , M_{Rd}) determined conform the author's methodology

$$M_{Ed} = 6.0tm \leq M_{Rd} = 68tm \quad (4.6)$$

$$1.25V_{Ed} = 6.19t > V_{Rd} = 16.20t \quad (4.7)$$

$$qM_{Ed} = 18.78tm \leq M_{Rd} = 68tm \quad (4.8)$$

$$qV_{Ed} = 15.47t < V_{Rd} = 16.20t \Rightarrow V_{Rd} = 15.47t \quad (4.9)$$

The resistance requirements of P100/1-06 Romanian Code are satisfied.

5. CONCLUSIONS

Concerning the unreinforced masonry

- The results of the tests confirm that the values of the shear resistance (V_{Rd}) according to Romanian Code (CR6/1-06) are under evaluated;
- The values of the shear resistance (V_{Rd}) calculated according to the author's methodology are very close to the test results.

Concerning the confined masonry

-In according to Romanian Code(CR6/1-06):

- The shear resistance (V_{Rd}) is much under evaluated. It is considered only 30% from the masonry shear resistance (V_{Rd1}) and 20% from the capacity of resistance of the reinforcement steel (V_{Rd2}). This determines an unjustified increase of the section area of the walls.
- The equivalent section is an unreinforced masonry "I" shaped, but the relationship of the (V_{Rd}) ignores the additional capacities of the flanges.
- The shear resistance (V_{Rd1}) is determined like the unreinforced masonry by the considering the plane of failure parallel to bed joints. However, the confined masonry has a diagonal failure due to main tensile stresses as the main failure criterion.

- In according to the authors' methodology:

- The shear resistance (V_{Rd}) is determined with the hypotheses that the diagonal failure due to main tensile stresses as the main failure criterion.
- This analyze allows to be established the failure mode of the vertical structural masonry elements, and one of the most important characteristic, to evaluate the ductile behavior of the overall structure.
- Although the European Code recommends ULS stage to design the structural masonry elements, an application of this recommendation in seismic risk areas without an attentive analysis, may create a disadvantage for the masonry structures, with economical and functional implications.

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