MODERN DESIGN METHOD FOR STRUCTURAL ANALYSIS OF MASONRY BUILDINGS IN THE SEISMIC AREAS

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ABSTRACT:
Romania is a country with a high seismic risk, two important seismic regions in Vrancea and Banat existing on the territory.
For seismic design of the buildings, the territory of the country was divided in 7 seismic hazard zones, with different intensity of design earthquake ag=0.8÷0.32g. Every hazard zone has seismic design rules to realize the structure of the buildings: antiseismic conformation, number of levels, characteristics of the materials, characteristics of design earthquake etc.
In zones with reduced seismic hazard, the masonry buildings can have an unreinforced masonry (URM) structure, but with a low height (1, 2 levels) or reinforced or confined masonry (RM) structure with multilevel height (3, 4 levels). In zones with high seismic risk, the masonry structures must have only reinforced masonry and reduced height.
In Romania are yet many old buildings over hundred years, which were not designed after seismic codes. These buildings, historical monuments, cannot be demolished and must be retrofitted by different methods to ensure the resistance and stiffness of the structure according with the actual design codes; one of this methods frequently used in our country is retrofitting of the shear masonry walls by jacketing with cement-mortar and ductile reinforcement steel. Sometimes, the building have wooden floor, who must be replaced with rigid concrete floor diaphragm.
KEYWORDS: masonry, reinforced masonry,antiseismic,design,capacity,ductility,retrofit, jacketing

1. INTRODUCTION

The new masonry buildings, reinforced or not, can be evaluated by determining the sectional stresses for every vertical structural element (walls) with a specialized soft. Design resistance of the wall can be determined after the Romanian Code of Masonry (CR6/2006). Finally comparison between design value of the shear load and of the design moment applied to the wall (V_{ED}, M_{ED}) and the shear resistance and the moment of resistance (V_{RD}, M_{RD}) must be: $V_{ED} \leq V_{RD}$ and $M_{ED} \leq M_{RD}$.
The old buildings can be evaluated by determining the nominal safety ratio (R) from Romanian Seismic Code (P100/92), who is the ratio between the actual capacity and the necessary of capacity of entire structure. In this analysis, an important step is to determine the mode of failure of the shear wall at the combined action of axial and lateral forces. This analysis shows the elements that must be retrofitted because have a brittle failure mode. The retrofit transforms the wall in an element with ductile failure mode.
Following is presented the methodology for the analysis of the ductile behaviour of a shear wall, in two situations: an old unreinforced masonry building retrofitted and a new reinforced masonry building. The evaluation algorithm respects the Romanian Codes. Both, the methodology and the algorithm of the analysis of the masonry structure are realized by the authors of the paper.

2.METHODOLOGY FOR ESTABLISH THE DUCTILE BEHAVIOUR OF THE STRUCTURAL UNREINFORCED MASONRY WALLS

2.1 Assumptions

- It is accepted that, for structural unreinforced masonry walls, the main failure criterion is diagonal failure, due to shear 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)has a linear variation on the elastic zones ($\varepsilon \leq \varepsilon_C$) 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 height 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 $\varepsilon \leq \varepsilon_c$).
- The stress-strain curve of masonry is accepted like in Figure 1, 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 denote the yielding and the ultimate stage. The ratio between ε_u and ε_c expresses the ductility of masonry, μ_z.
- The moment-rotation relationship is represented in Figure 2.

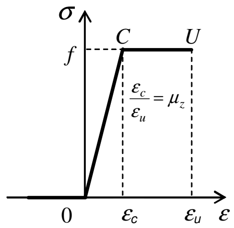


Figure 1. Accepted stress-strain relationship for masonry

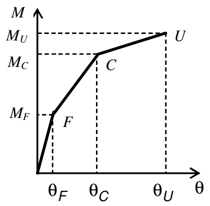


Figure 2. Moment-rotation relationship for unreinforced masonry cross-section

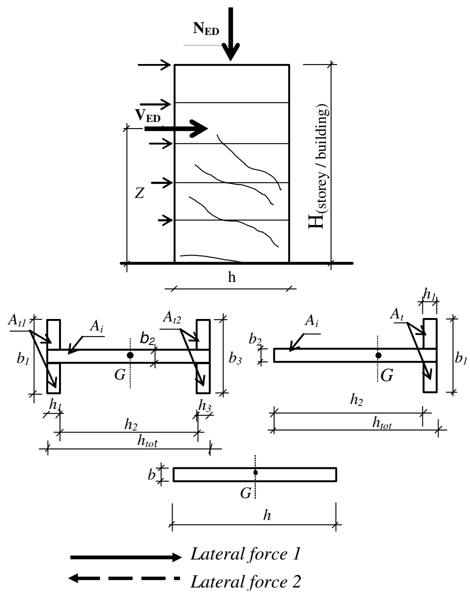


Figure 3. Masonry elements: elevation and sections

2.2 Deformation stages

Unreinforced masonry elements subjected to constant axial loads (N_{ED}) and to gradually increasing lateral forces (V_{ED}) are analysed, as shown in Figure 3.
The section at the base of a structural unreinforced masonry wall pass through successive deformation stages, as the lateral force gradually increases. The described method considers three reference stages, characterised by the stress and strain distributions shown in Figures 4, 5 and 6.

2.3 Calculation of the lateral shear resistance V_R

The lateral shear resistance of unreinforced masonry wall (Figure 3) is calculated by considering ***diagonal failure due to main tensile stresses as the main failure criterion.***
The calculation involves the following steps for each deformation stage (F, C and U) and the following expectable capacities are determined:

a) the values of the lateral force, V_M, associated to the bending moment of resistance with the stress and strain distributions, shown in Figures 4 to 6: $V_{M,F}$, $V_{M,C}$ and $V_{M,U}$;

b) the values of the lateral force, V_Q, corresponding to diagonal failure due to principal tensile stresses, shown in Figures 4 to 6: $V_{Q,F}$, $V_{Q,C}$ and $V_{Q,U}$.

c) with V_M and V_Q values can design the interaction curves V_M-θ (Figure 7) and V_Q-θ (Figure 8), (like M-θ curves) and the value of strength capacity (Figure 9) .

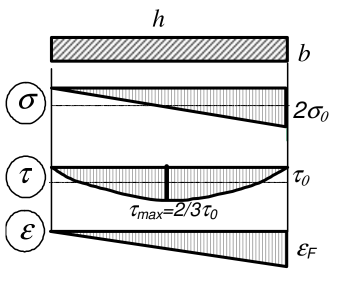


Figure 4. Stage F: normal cracking in bending

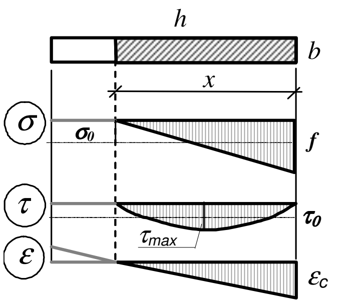


Figure 5. Stage C: yielding in compression

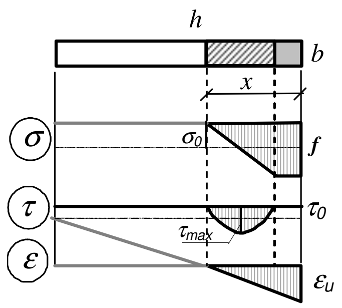


Figure 6. Stage U: ultimate

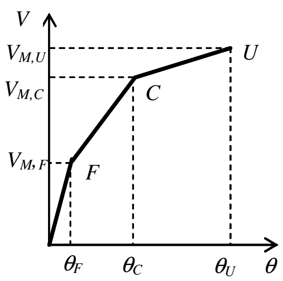


Figure 7. V_M-θ curve

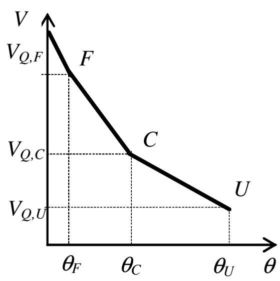


Figure 8. V_Q-θ curve

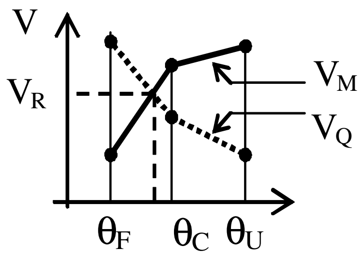


Figure 9. Strength capacity -V_R

2.4 *Determination of the failure mode and the value of V_R*

By comparing the values of two capacities (V_Q) and (V_M), the failure mode can be determined, analytically (Figure 10÷13) or graphically (Figure 14÷17):

$V_{Q,F} > V_{M,F}$	$V_{Q,F} > V_{M,F}$	$V_{Q,F} > V_{M,F}$	$V_{Q,F} > V_{M,F}$
$V_{Q,C} > V_{M,C}$	$V_{Q,C} > V_{M,C}$	$V_{Q,C} < V_{M,C}$	$V_{Q,C} > V_{M,C}$
$V_{Q,U} > V_{M,U}$	$V_{Q,U} < V_{M,U}$	$V_{Q,U} > V_{M,U}$	$V_{Q,U} > V_{M,U}$
$\mathbf{V_R = V_{M,U}}$	$\mathbf{V_R}$ –at intersection of the curves, between C and U stage	$\mathbf{V_R}$ = at intersection of the curves, between F and C stage	$\mathbf{V_R = V_{Q,F}}$
ductile failure – ***MMM*** (Figure 10)	low ductility failure – ***MMQ*** (Figure 11)	brittle failure – ***MQQ*** (Figure 12)	brittle failure – ***QQQ*** (Figure 13)

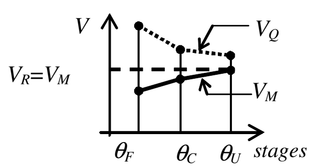


Figure 14. *MMM* (ductile) failure mode

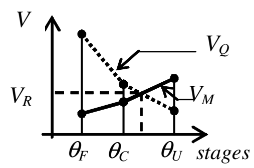


Figure 15. *MMQ* (low ductility) failure mode

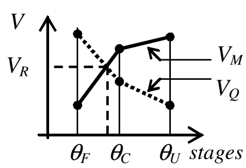


Figure 16. *MQQ* (brittle) failure mode

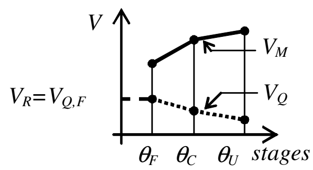


Figure 17. *QQQ* (brittle) failure mode

3. UNREINFORCED MASONRY OLD BUILDING EVALUATION AND RETROFIT

3.1 Dynamic analysis and failure mod evaluation of URM structure

The unreinforced masonry house, historical monument, was built in 1908 in a hazard seismic zone with ag=0.24g and Tc=1.6s (Fig.18; 19).

Figure 18 – Main front of the house

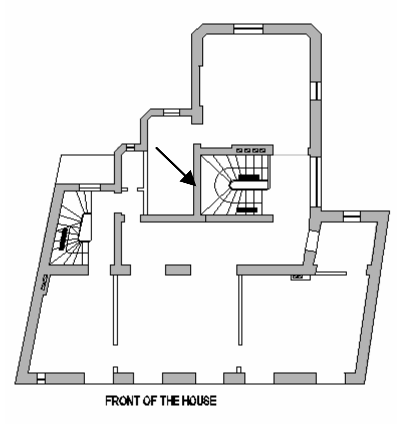


Figure 19 –First floor plan

The dynamic and failure mod analysis of the building gived a nominal safety ratio "R" of unreinforced structure for both principal directions of the earthquake x, y R=0.46, lower than R_{min}=0.50 recommended by the Romanian Cod P100/1992.

3.2 Retrofitting by jacketing solution

To improve the behaviour of the structure, was adopted retrofitting of the shear wall by jacketing with 6 cm mortar-cement M10 and steel Φ8/100-Φ8/100 (see Figure 20) on two side of the section. Because the house has a wooden floor, it must replace the last upper floor with a concrete floor and to realize a horizontal rigid diaphragm to ensure spatial work of the walls.

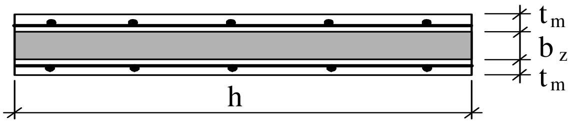


Figure 20. Horizontal active section of the wall (Ex.Figure 19 - h=257 cm; b_z= 28 cm; t_m=6 cm)

3.3 Analysis of the failure mod of retrofitted walls

The methodology presented in Chapter number 2 can be used to calculate the strength capacity of retrofitted wall by jacketing. It was based on the determination of two reference quantities: the equivalent compression strength (f^{eq}) and the equivalent strength to main tensile stresses (f_p^{eq}).
The equivalent strength capacity also includes two safety ratios related to:

- the interaction between the mortar jacket and the unreinforced masonry up to the ultimate stage,
- the materials quality of the old masonry.

The calculation of the equivalent design compression strength, f^{eq}, of the retrofitted masonry wall (Figure 20) is carried out based on the design compression strength of unreinforced masonry, f, and on the design compression strength of the jacketing mortar, f_m (see equations 3.1).

$$f^{eq}=c_c[nf + (1-n)f_m], \qquad (3.1)$$

where:
$n=b_z/b$

b: total thickness of retrofitted masonry wall (b_z+2t_m)
t_m: thickness of the jacket
b_z : thickness of unreinforced masonry wall
c_c : partial safety ratio for the interaction between mortar jacket and masonry, up to the ultimate stage
f : design compression strength of the masonry
f_m : design compression strength of the jacketing mortar

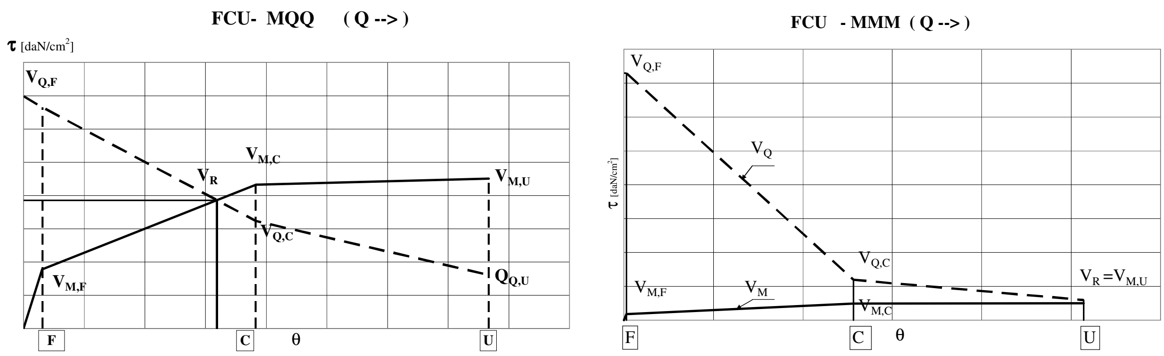
The calculation of the equivalent design strength to principal tensile stresses of the retrofitted masonry f_P^{eq} is carried out based on the design strength to main tensile stresses of the mortar of unreinforced masonry f_p, on the design strength to main tensile stresses of the mortar of the jacket f_{pm}, and on the design tensile strength of the reinforcement steel f_s (see Eqn 3.2).

$$f_p^{eq}=c_p[nf_p+(1-n)f_{pm}]+0.8n_af_s \tag{3.2}$$

where:
$n_a=A_{a0}/100b$
c_p partial safety ratio for the quality of the mortar in the joints
f_p design strength to main tensile stresses of the mortar in the bed joints of the masonry
f_{pm} design strength to main tensile stresses of the mortar of the jacketing
f_s design tensile strength of the reinforcement steel
A_{a0} reinforcement horizontal steel area
b total thickness after retrofitting: $b=b_z+2t_m$ for two side jacketing.

3.4. Comparison between a pre and post retrofitted wall

For the evaluation of the retrofitting solution efficiency, an unreinforced masonry wall with rectangular cross-section was considered (Figure 20) for example. The wall was calculated in two variants: pre-retrofitting (Figure 21a) and post-retrofitting (Figure 21b).



a) Wall pre-retrofitting (V=τh) b)Wall post-retrofitting (V=τh)

Figure 21- Analysis of the wall pre and post retrofitted

The characteristic conventional deformation stages of the two sections (F, C and U) were determined practically in the same way as for unreinforced masonry, only difference consisting in the calculation of the material design strengths.The diagrams of the axial compression stress (σ) and of the shear stress (τ) corresponding to the conventional stages of deformations (F, C and U), as well as the diagrams of the shear strength capacity, were determined for the original (not retrofitted) wall (Figure 21a) and for the retrofitted wall (Figure 21b). As shown in Figure 21a, the original wall section have a brittle type (MQQ) of failure. By retrofitting, a ductile type of failure (MMM) is obtained.

3.5 Conclusions

- By the methodology presented, it can be established the failure mode of the vertical structural masonry elements, one of the most important characteristic to evaluate the ductile behaviour of the overall structure.Thus, these information about the existent structure, can help the expert to decide the value of the behaviour factor *q*.

- With the same relationship is determined the failure mode of the masonry wall, retrofitted by jacketing and the unreinforced masonry walls.

- The diameter of the steel-bars reinforcing and the thickness of the jacket can be established so that the element shall have a ductile behaviour.

- The nominal safety ratio "R" for the retrofitted structure of the building increase from R=0.46 to R=0.82 (greatest than R_{min}=0.50).

4. DESIGN OF A NEW MASONRY CONFINED BUILDING

4.1 Seismic evaluation according Romanian Design codes: CR6/2006 si P100/2006

The building is situated in a hazard seismic zone, with ag=0.24g, T_C=1.6 s. The structure is with masonry, concrete pillars, and concrete ties and floor at each level. (Figures 22, 23). The value of the behavior factor after CR6, is: q=3.125. The characteristics of the materials are: mortar M10, masonry unit C10, concrete in the ties, pillars and slabs - C16/20, steel reinforcement PC52 (f_s=300N/mm^2).

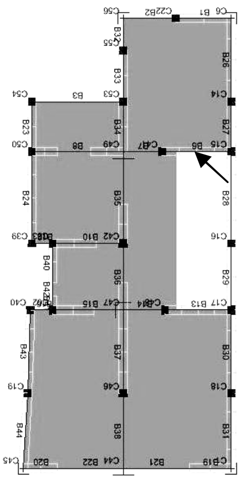


Figure 22. Current floor – plan view

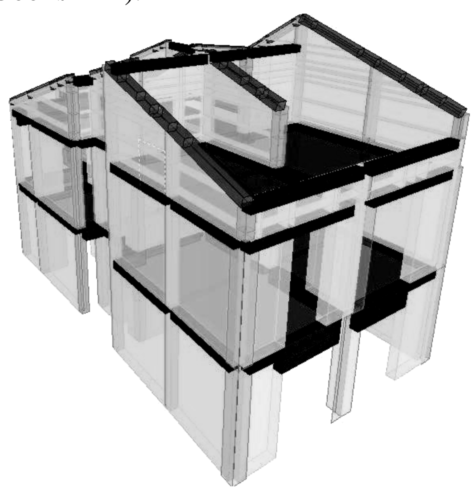


Figure 23. Spatial (3D) view

The structure was calculated at lateral seismic action, with a specialized soft. The design diagrams of the shear loads (V_{ED}) and of the moments applied (M_{ED}) for the masonry piers and concrete pillars are presented in the Figures 24 and 25 (for the element indicated in Figure 22 see arrow).

4.2 Design resistance of the shear wall with Romanian Masonry Code -CR6/2006

The rectangular masonry section with two pillars on the both ends is transformed into an equivalent I-shaped section (Figure 26b). For this section is calculated V_{RDm} (design masonry shear resistance) and V_{RDc} (design shear resistance of the the steel reinforcement in concrete pier).
The values of the shear resistance V_{RD} and the moment of resistance M_{RD} for composed section of the masonry wall and concrete pillar calculated after the Romanian Masonry Cod (CR6/2006), are:

$$V_{RD} = 0.30V_{RDm} + V_{RDC} = 0.30 f_{vd} * t * l_c + 0.2 * A_{sc} * f_{yd} \quad (4.1)$$

$$M_{RD} = M_{RDm} + M_{RDC} = y_{zc,i} N_{ED} + l_s * A_{si} * f_{yd} \quad (4.2)$$

where:
- V_{RDm} design masonry shear resistance
- V_{RDC} design shear resistance of steel reinforcement in concrete pier
- M_{RDm} design value of the masonry moment of resistance
- M_{RDC} design value of moment of steel reinforcement in concrete pier
- f_{vd} design shear strength of masonry

-t	thickness of the wall section
$-l_c$	length of the compressed part of the wall
$-A_{sc}$	cross sectional area of steel reinforcement in compressed pillar
$-f_{yd}$	design tensile strength of reinforcing steel
$-y_{zc,i}$	distance between centre of the wall section and centre of the compressed part of the wall

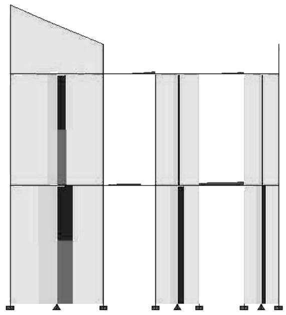


Figure 24. Shear load $-V_{ED}$ diagr

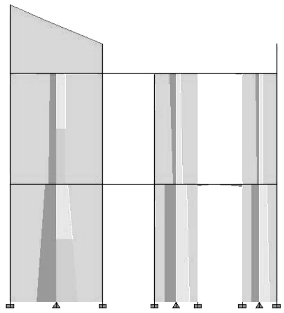


Figure 25. Moment-M_{ED} diagrams

The values of resistance must be verified thus:

$$qV_{ED}= \leq V_{RD} \text{ and } qM_{ED} \leq M_{RD} \tag{4.3}$$

If the relationships are not respected, the section of the wall or the characteristic of the materials must be changed.
An example of analyzed wall (Figure 22) provides the next results:

$$qV_{ED}= 15.47 \text{ t} > V_{RD}=5.36 \text{ t} \quad \text{and} \quad qM_{ED}=18.78 \text{ tm} < M_{RD}=68 \text{ tm}. \tag{4.4}$$

It can see that the requirement of Romanian Seismic Code (P100/2006, Chapter 8) is not respected and the materials and the dimension of the section must be modified.

4.3 Methodology for evaluation the ductile behaviour of confined masonry wall, and of the design capacity

The rectangular section of the wall with pillars at both ends (*Fig. 26*) is transformed into an equivalent I-shaped masonry section (Fig. 27). Formula of the equivalent section area of the flange is:

$$A_t = A_s \frac{f_b}{f} \tag{4.5}$$

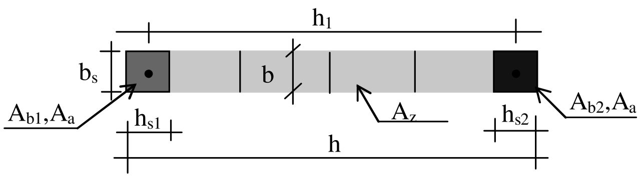


Figure 26) Structural masonry wall with concrete pillars (confined masonry)- horizontal section

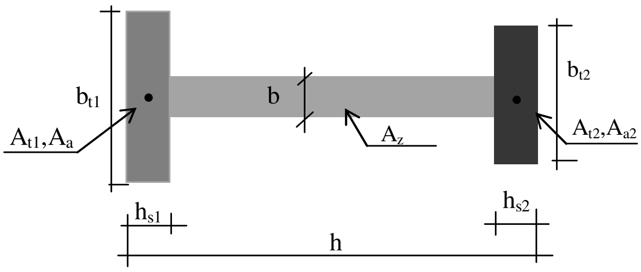


Figure 27) Equivalent masonry wall section

where:
A_s - the concrete pillar area;
A_t - the equivalent masonry area of the flange;
f_b - the strength in compression of the concrete;
f - the strength in compression of the masonry.
The design shear strength of the confined section is resultant of the shear strength of the I-shaped equivalent masonry wall section and of the shear strength of the steel from pillars of the actual section.

It is calculated the shear capacity V_Q and bending capacity V_M conform chapter 2 and ad the capacity of the steel-reinforcing of the pillars $V_{S,Q}$ (A_s,f_s,h_{s1}) is $V_{S,M}$ (μ_R,A_s,f_s) in (C) and (U) stages (Figure 27÷30):

$V_{Q,F} > V_{M,F}$
$V_{Q,C}+V_{S,Q} > V_{M,C}+ V_{S,M}$
$V_{Q,U}+V_{S,Q} > V_{M,U}+ V_{S,M}$
$\mathbf{V_R = V_{M,U}+ V_{S,M}}$

Figure 27 - MMM

$V_{Q,F} > V_{M,F}$
$V_{Q,C}+V_{S,Q} > V_{M,C}+ V_{S,M}$
$V_{Q,U}+V_{S,Q} < V_{M,U}+ V_{S,M}$
$\mathbf{V_R}$ –at intersection of the curves, between C and U

Figure 28 - MMQ

$V_{Q,F} > V_{M,F}$
$V_{Q,C}+V_{S,Q} < V_{M,C}+ V_{S,M}$
$V_{Q,U}+V_{S,Q} > V_{M,U}+ V_{S,M}$
$\mathbf{V_R}$ **=** at intersection of the curves, between F and C

Figure 29 - MQQ

$V_{Q,F} > V_{M,F}$
$V_{Q,C}+V_{S,Q} > V_{M,C}+ V_{S,M}$
$V_{Q,U}+V_{S,Q} > V_{M,U}+ V_{S,M}$
$\mathbf{V_R = V_{Q,F}+ V_{S,M}}$

Figure 30- QQQ

Then is determined the value of V_R and M_R . In our example the requirement of P100/2006 is satisfied:

$$V_R = V_{M,U}+ V_{S,M}=16.20 \text{ t} < qV_{ED}= 15.47 \text{ t}$$
$$M_R = V_R{*}z=69.6 \text{ tm } (z=2H/3 - \text{ Figure 3})$$

4.4Conclusions

The design shear resistance of masonry,V_{Rdm}, after Romanian Masonry Code (CR6/2006) is conservative because:

- the value of V_{Rdm} is decreased with 70%
- both the design value of V_{ED} and M_{ED} is calculated by multiplying the reference values from specialised soft with q value (behaviour factor).

The results obtained with the methodology described in chapter 2 and 4.3, was compared with the experimental tests performed in European laboratories. Though the results of the test was better than the calculated shear resistance V_R and V_M. (max. 20% plus).

Though, European Code recommend ULS stage to design the structural masonry elements, an application of this indications without of an attentive analyses may be create a disadvantage for the masonry structures with economical and dysfunctional implications.

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