A METHOD FOR THE ASSESSMENT OF THE SEISMIC PERFORMANCE OF UNREINFORCED MASONRY WALLS

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SUMMARY

The method was elaborated in the perspective of the revision of the Romanian masonry design code. It addresses unreinforced masonry walls, subjected to lateral in-plane forces. The method is suitable for both new and existing buildings, in seismic areas.

Three deformation stages considered are: cracking, yielding in compression , ultimate.

The shear strength is determined at the intersection of the limit curves: the curve corresponding to shear failure and the curve corresponding to failure in bending and compression stress.

For the basic shapes of sections (rectangular, T- or I-shaped) a simplified method, adequate to manual calculations, was derived. For complex shapes of sections, the basic method is applied, which requires the use of a computer program.

The method was applied in a numerical study, presented in the paper. For three shapes of wall, the relation between axial stress and shear stress was plotted for three different distributions of shear stresses across the flange in compression.

1. INTRODUCTION

A method for the design of unreinforced masonry walls, subjected to in-plane bending, shear and axial force (*Fig.1*), was elaborated in the perspective of the revision of the Romanian masonry design code. The method was implemented into a design manual [Popescu *et al.*, 1995], addressing either new and existing buildings.

This method was presented in a design handbook [Popescu *et al.*, 1995], which can be used both for designing new shear walls and analysing the shear walls in existing buildings.

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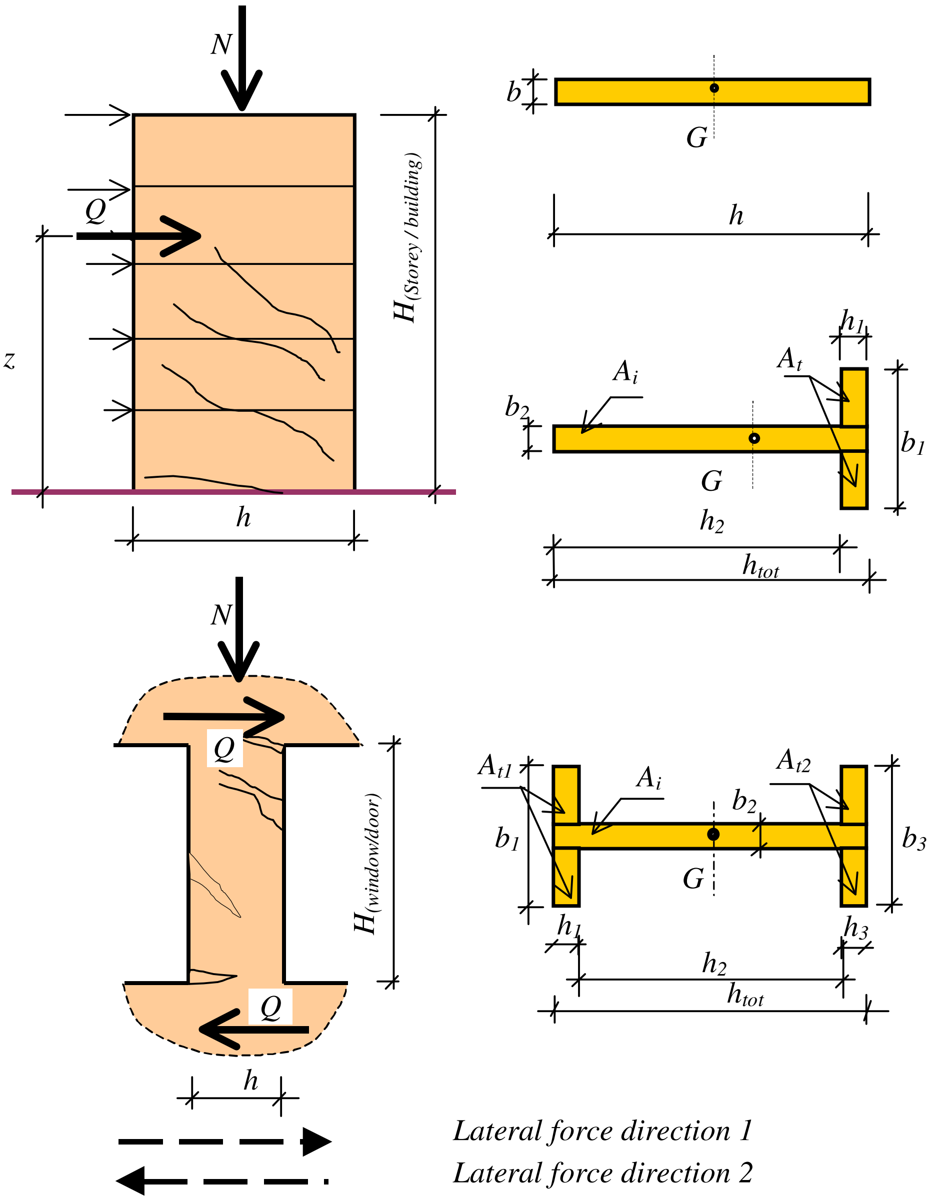


Fig.1. Vertical elements – elevations and sections

2. ASSUMPTIONS OF THE METHOD

The assumptions used in the method are presented below.

1. It is accepted that, for structural unreinforced masonry walls, the main failure criterion is diagonal failure, due to shear stresses.

2. The law of Bernoulli applies.
3. The mortar in the bed joints at the bottom part of the wall has null tension strength.
4. The normal compression stresses (σ) have a linear variation on the elastic zones ($\varepsilon<\varepsilon_c$) of the section.
5. On the plastic zones of the section ($\varepsilon>\varepsilon_c$), the normal compression stresses are constant and equal to the compression strength of masonry (σ).
6. The distribution of the shear stresses τ along the height of the section conforms to the average flexural shear stress formula (i.e. is parabolic); the shear stresses are distributed only over the compressed, elastic zone of the section ($\varepsilon<\varepsilon_c$).

3. DEFORMATION STAGES

The section at the basis of a structural masonry shear wall, subjected to simultaneous actions, consisting of compression and shear forces, can pass through different stages of deformation. There are three stages of reference, characterised by the stress and strain distributions as shown in *Fig. 2, 3*and *4*.

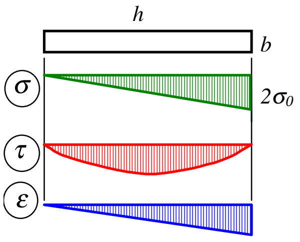


Fig. 2. Cracking stage(F)

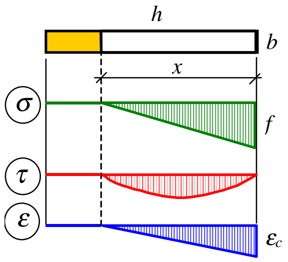


Fig.3. Yielding in compression stage (C)

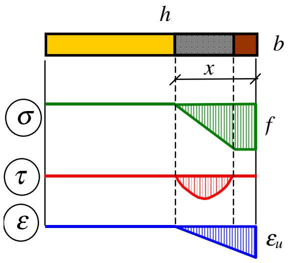


Fig.4. Ultimate stage (U)

The distribution of shear stresses over the flange width was taken according to the following three hypotheses:

a) only on a width equal to the web width, with a variation shown in *Fig.5 a*;
b) on the whole width of the flange, with a variation shown in *Fig.5 b*;
c) on the whole width of the flange, with a variation shown in *Fig5 c*.

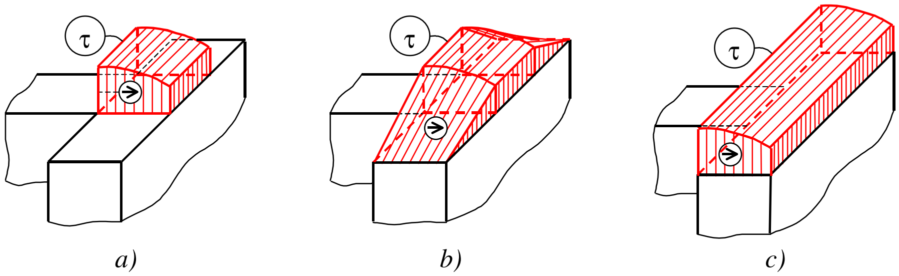


Fig. 5. Hypotheses used to model the shear stress distribution over the width of the flange

The constitutive law of the material (compression stress-strain relationship) for masonry is shown in the *Fig. 6*, while in *Fig. 7* the relationship M-θ is shown, for the section from the basis of an element subjected to compression and shear forces.

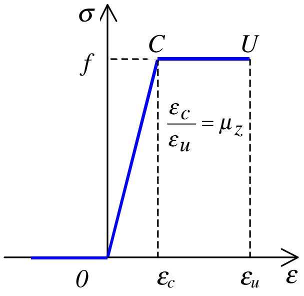


Fig. 6. The constitutive law of the material

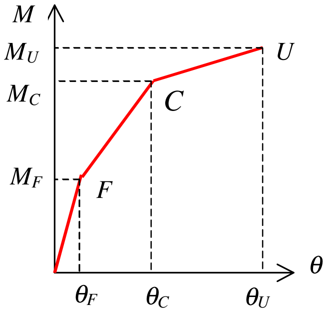


Fig. 7. The relationship M-θ

4. COMPUTATION OF SHEAR STRENGTH

The method consists in computation of the shear strength for the unreinforced masonry walls (*Fig.1*), admitting as a main failure criterion the failure due to main tensile stresses.

In order to determine the value of the shear strength in the shear masonry wall the next steps have to be followed:

1. for each of the deformation stages must be determined, admitting the distribution of stresses and strains presented in *Fig.2…4:*
   - the values of the shear strength, associated to the deformation stages F, C and U ($Q_{M,F}$, $Q_{M,C}$, $Q_{M,U}$);
   - the strength corresponding to the diagonal failure due to the principal stresses, for the stages F, C and U ($Q_{Q,F}$, $Q_{Q,C}$, $Q_{Q,U}$);
2. with the six values above, can be determined, analytically and graphically, the value of the shear strength Q_R, defined by the condition:

$$Q_R=Q_Q=Q_M \,. \tag{1}$$

These values [Popescu *et al.*, 1995] can be determinate manually or automatically.

5. ESTABLISHMENT OF THE FAILURE MODE

By comparing the values of the strength capacity *(Q_Q)* with the values associated to the bending strength *(Q_M)*, the failure mode can be determined, for each of the three deformation stages, as follows:

1. ductile type (***MMM***), if (*Fig. 8*)

$$Q_{Q,F} > Q_{M,F}\,,$$
$$Q_{Q,C} > Q_{M,C}\,,$$
$$Q_{Q,U} > Q_{M,U}\,;$$

2. limited ductility type (***MMQ***), if (*Fig. 9*)

$Q_{Q,F} > Q_{M,F}$,
$Q_{Q,C} > Q_{M,C}$,
$Q_{Q,U} < Q_{M,U}$;

3. brittle type (***MQQ***), if (*Fig.10*)
$Q_{Q,F} > Q_{M,F}$,
$Q_{Q,C} < Q_{M,C}$,
$Q_{Q,U} < Q_{M,U}$;

4. brittle type (***QQQ***), if (*Fig.11*)
$Q_{Q,F} < Q_{M,F}$,
$Q_{Q,C} < Q_{M,C}$,
$Q_{Q,U} < Q_{M,U}$.

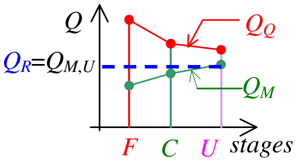


Fig. 8. Failure mode MMM

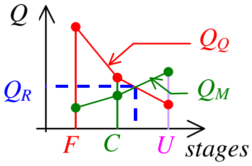


Fig. 9. Failure mode MMQ

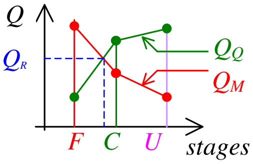


Fig. 10. Failure mode MQQ

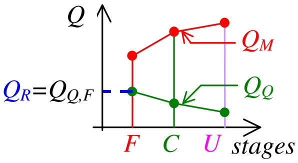


Fig. 11. Failure mode QQQ

6. APPLICATIONS

In order to provide an example, three sections with rectangular, *T*- and *I*- shapes were chosen (*Fig. 1*), having the following common properties: $H=5.75\ m$, $h_{tot}=5.75\ m$, $f_p=0.90$ [kgf/cm^2], $f=14.00$ [kgf/cm^2].

Table 1. Sectional properties

Rectangular section	*T*-shaped section	*I*-shaped section
$h=5.75\ m$	$h_1=1.50\ m$	$h_1=1.50\ m$
$b=0.25\ m$	$b_1=0.48\ m$	$b_1=0.48\ m$
	$h_2=5.27\ m$	$h_2=5.02\ m$
	$b_2=0.25\ m$	$b_2=0.25\ m$
		$h_3=1.75\ m$
		$b_3=0.25\ m$

The sections were analysed for a variation of the stress $\bullet_0$ from *0* to the value of the design compression strength, *f*. The curves of the shear strength $\bullet_R$ ($\bullet_R = Q_R/A_i$), for

• $_0$=*0.5f* were determined (*Fig. 12...14*), together with the diagrams • $_0$ – • $_R$ (*Fig. 15...17*). For the *T*- and *I*-shaped sections the diagrams were calculated by considering the three hypotheses from paragraph 2, points 7 a), b) and c).

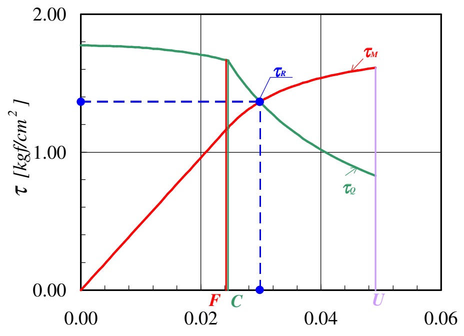
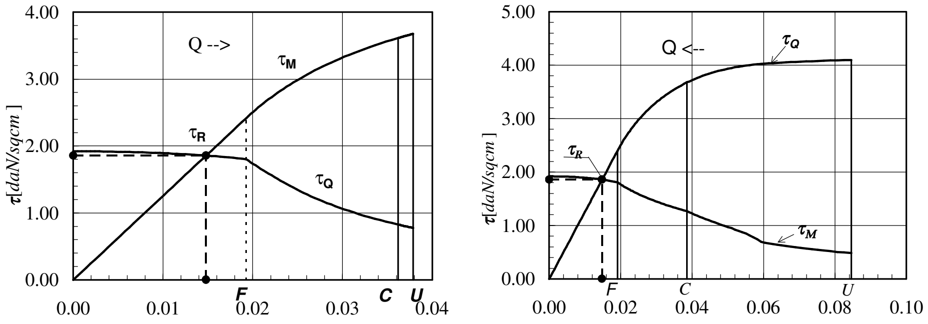


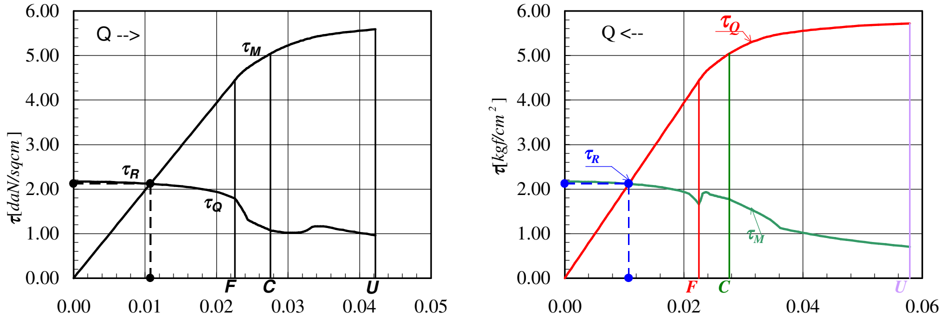
Fig. 12. Shear strength τ_R *– rectangular section*



a)

b)

Fig. 13. Shear strength τ_R *– T-shaped section*



a)

b)

Fig. 14. Shear strength τ_R *– I-Shaped Section*

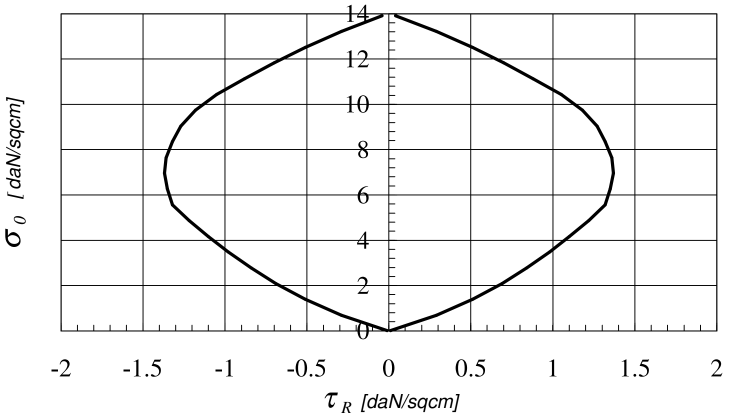


Fig. 15. σ_0–τ_R diagram – rectangular section

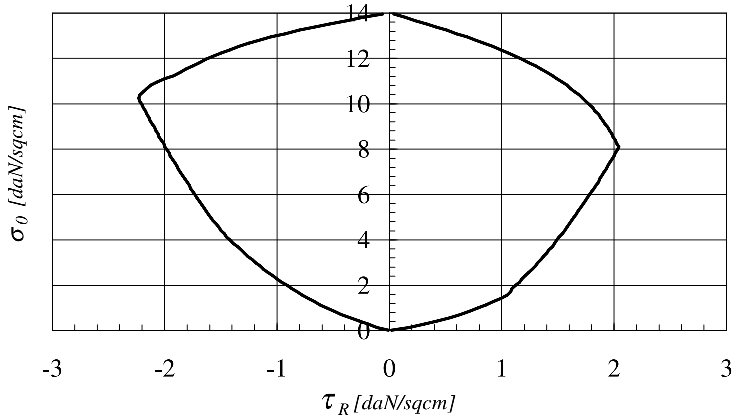


Fig. 16. σ_0–τ_R diagram – T-shaped section

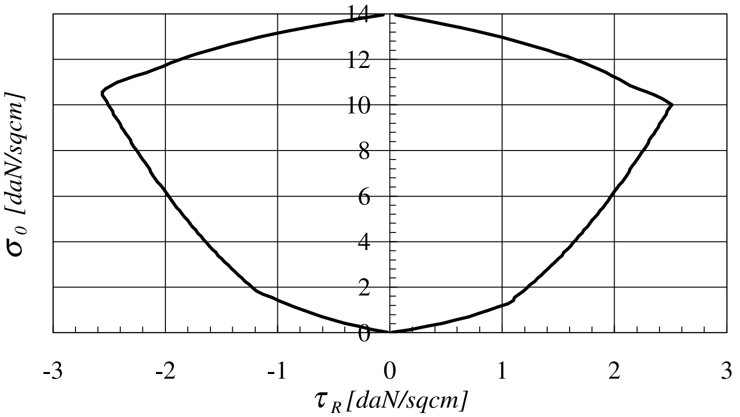


Fig. 17. σ_0–τ_R diagram – I-shaped section

Some conclusions drawn from the study are presented below.

1. For the rectangular sections, the maximum shear strength is obtained for $\tau_0=0.5f$.
2. For the *T*- and *I*-shaped sections, the maximum shear strength is obtained for $\tau_0=0.7f$.
3. For the *T*-shaped sections, there is an increase of the shear strength of about *40%*, compared to the shear strength of the rectangular section with the same cross-section height, when the flange is in tension.
4. For the *T*- and *I*-shaped sections, an abrupt decrease of the strength capacity is observed, when the compression stress increases above *0.7f*.

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