LATERAL RESISTANCE OF CONFINED BRICK WALL UNDER CYCLIC QUASI-STATIC LATERAL LOADING

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ABSTRACT:

Usually in masonry building systems, the piers between openings are the most vulnerable in case of earthquake. The failure of such walls is due in the majority of cases to shear. Accordingly, this study is elaborated to compare three different analytical approaches regarding the prediction of lateral shear resistance of confined masonry walls. The principle is to calculate the total lateral resistance of the wall by considering the participation of both structural members, i.e. the brick panel and the RC tie-columns. In this study, the tie-columns participation is evaluated from the dowel action of confined columns' reinforcement and the brick panel participation is evaluated using the theory of elasticity with different assumptions. A full scale experimental study on confined clay brick wall subjected to a constant vertical and quasi-static cyclic lateral loads is performed to estimate the efficiency degree of each method.
Depending on the method, the comparison of calculated values to test results has shown that the real lateral resistance is overestimated by 17.54%, 42% and 54.1%.
On the light of these results, the appropriate approach to predict the shear capacity of the masonry structural member needs to be selected carefully to be consistent to a certain extent with the results of the experimental testing.

KEYWORDS: confined masonry, brick panel, tie-columns, cyclic load, dowel action, shear capacity.

1. INTRODUCTION

The development of practical methods for evaluating the shear strength of bearing confined masonry walls is a difficult task since experimental and analytical data of its mechanical behavior are less numerous and their interpretations are less precise. Indeed, the diversity of the available material and the imperfection of its production techniques make it complex to characterize and normalize this type of construction material.

Most of the research works and the existing regulations applied to the construction industry, simplify the behavior of the masonry with the aim to provide practical and easier criteria for the analysis of the structural behavior. In general, these regulations recommend the use of a linear model by considering the masonry such as a homogeneous material[1)]. Thus, despite of the high costs, the difficulties of carrying out experimental work in the laboratory and the complexity of the masonry characteristics; experiments remain incontestably imperative for identifying the mechanical parameters of this structural system.

As known, flexural failure is favored in seismic resistant design because it is accompanied by large plastic deformation and energy absorption and dissipation capacities[2]. Shear failure, on the other hand, is more brittle, with limited ductility, from where; confining the plain masonry can remedy in a certain extent this weakness. In this respect, Tomaževič and Klemenc[3] proposed a method to predict the shear capacity of confined masonry walls, where the seismic behavior of brick panel is modeled by considering the effect of interaction forces between bond-beam, tie-columns and masonry panel. According to Sucuoglu *et al.*[2] the ultimate shear strength of the masonry wall is reached when an inclined cracking initiates at the middle of the wall due to the principal stress components at their critical values. Another expression proposed by Turnsek and Cacovic[4] predicts shear failure of masonry wall according to tension failure criterion, which occurs when the principal tensile stress in the wall attain the tensile strength of the masonry. The tie-columns participation in any case is modeled by the dowel action of columns' reinforcement as developed by Priestley and Bridgeman[5]. In this study the three approaches mentioned here will be investigated to determine the appropriate method conduct to the best assessment of the shear capacity of such structural system.

2. TEST SPECIMEN, MATERIAL CHARACTERISTICS AND LOADING SYSTEM

The test specimen is the full scale of a common window pier with *h/l* ratio equal to 1.5 surrounded by RC ties, noted JCM.

The panel of the wall is composite of Japanese solid clay brick units with 210x100x60 mm of nominal dimensions. The bricks are laid with plain cement mortar, with a joint thickness of 10 mm.

The surrounding frame which serves to confine the brick panel is made by reinforced concrete. Mechanical characteristics of used materials, details of dimension, arrangement of reinforcement and member sections are summarized in Table 1 and Fig.1.

The wall is erected on an RC rigid beam foundation. A similar beam is cast in-place on the top of the wall after constructing the brick panel and the RC tie columns. This beam permits a sufficient anchorage of the vertical reinforcing bars in both columns and provides also an adequate transfer of the applied lateral load to the wall. The RC beams are bolted to the reaction frame (fixed-ended).

The compressive strength of masonry is obtained by testing 03 stack bonded prisms of five bricks each and jointed between them on the building face by 10 mm of mortar according to the specifications of LUMB1[6].

The masonry tensile strength is obtained by testing 03 masonry square panel specimens (35x35x10 cm) under diagonal compression load as specified by ASTM 519 (ASTM C1391). The value of the tensile strength is evaluated as the principal tensile stress in the center of the panel[1), 7)] which is equal to $0.519\ P_d/A$ where P_d and A are the maximum diagonal compressive load and the panel section, respectively.

Modulus of elasticity E_m,Ec and shear modulus G_m,G_c of masonry and concrete are determined at 1/3 of compression strength, their values are reported in Table 1.

The loading system used for performing the experiments of the confined masonry wall JCM is shown in the Fig. 2. The vertical and horizontal actuators exert forces self-controlled by computer. Two Transducers of LVDT type are used to measure the lateral displacement.

During the test process, the vertical pressure generated by the vertical actuator, the steel loading beam and the top RC beam is set equal to 4 Kg/cm^2. The cyclic loading test applied to the specimen JCM was considered as a quasi-static test.

Cyclic horizontal displacements of increasing amplitude have been imposed to the wall and expressed in drift angles, the first value is of 1/3200 and the latest one corresponds to the rupture of the wall. Two loading cycles have been performed at each amplitude.

The hysteresis loops, showing the relationship between the measured lateral load and the average horizontal displacements for the tested wall are shown in Fig. 3.

Table1 Material properties

Property		Experimental value (MPa)
Brick compression strength	f_b	30
Mortar compression strength	f_{mor}	27
Concrete compression strength	f_c	20
Yielding stress of vertical reinforcements	f_y	415
Masonry compression strength	f'_m	9
Masonry tensile strength	f'_t	1.1
Modulus of elasticity of masonry	E_m	8241
Shear modulus of masonry	G_m	1723
Modulus of elasticity of concrete	E_c	18320
Shear modulus of concrete	G_c	7633

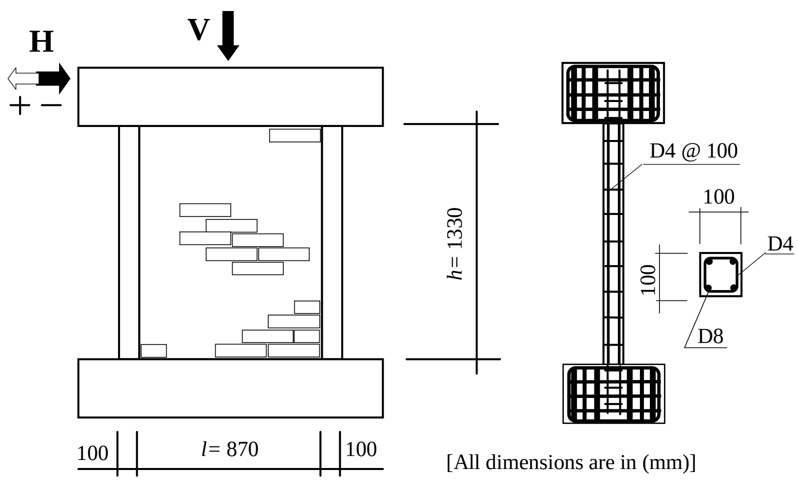

Figure 1 Test specimen (JCM)

3. TEST RESULTS AND CRACK PATTERN

During the third cycle of loading which corresponds to a drift angle of 1/1600, and at lateral load V= -66.25 kN and horizontal displacement δ= -0.49 mm, a small bending cracks appeared horizontally at the basis of the left RC tie. At about the same loading level during the fourth cycle (V= 70.25 kN, δ= 0.60 mm), other small cracks appeared at the interface of the brick panel and the left tie-column, as well as a diagonally oriented cracks were observed on the strut of the wall as mentioned in the Fig.4. Symmetric cracks were created quickly by a negative loading within the same level of drift angle. By definition this step marks the crack limit (elastic limit) of the wall. The followed cracks propagated and spread similarly to the crack pattern which initiated at the elastic limit.

The maximum lateral load was recorded in the ninth cycle which corresponds to the story drift angle of 1/500. The average values of the maximum shear load and its corresponding horizontal deflection obtained at loading in positive and negative direction were 81.25 kN and 3.26 mm, respectively.

The ductile behavior of the wall was confirmed by the large horizontal displacements and the decreasing of the lateral load in the last loading cycles as shown in the Fig. 3. The openings of the diagonal cracks became important and the cracks passed through the tie-columns to form the corner-to-corner diagonal shear cracks.

4. ANALYTICAL METHODS

Few attempts have been made to analytically predict the seismic behavior of confined masonry walls[8)]. In this respect, this study has been elaborated to compare three different approaches which lead to predict the maximum shear capacity of confined masonry walls expressed as the sum of the brick panel shear resistance and the shear supported by the R.C tie-columns.

4.1. Brick Panel Contribution

By considering the masonry wall as an elastic, homogeneous and isotropic structural element, the basic equation for the evaluation of the shear resistance of plain masonry walls can be derived by taking into account the assumptions of the elementary theory of elasticity. Under the combination of a vertical load V and a lateral load H, the principal stresses of compression σ_c and tension σ_t are developed in the middle section of the wall.

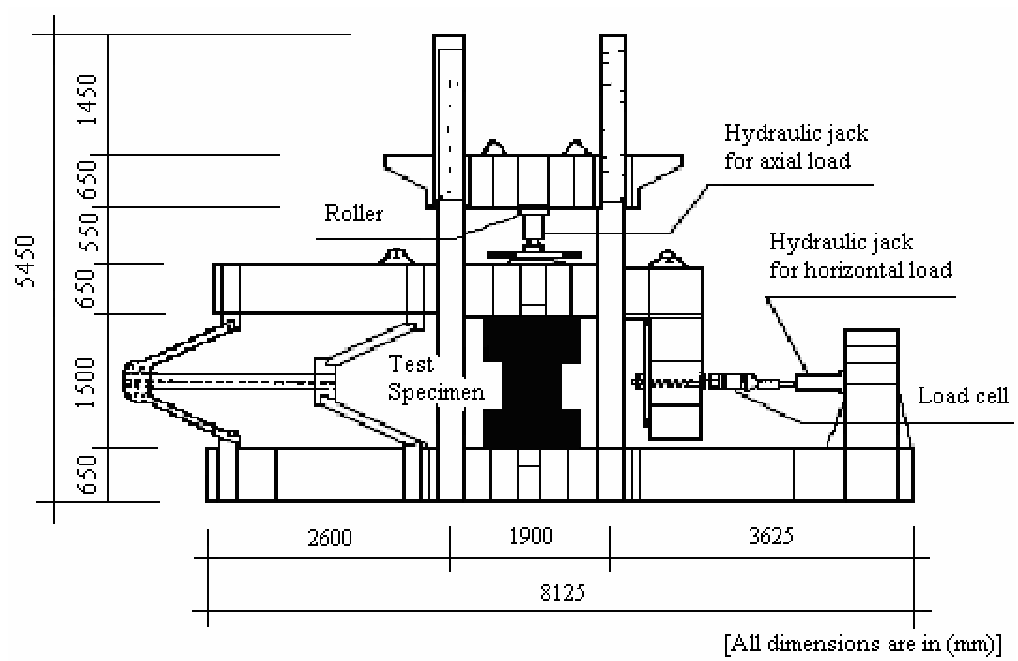

Figure 2 Loading system

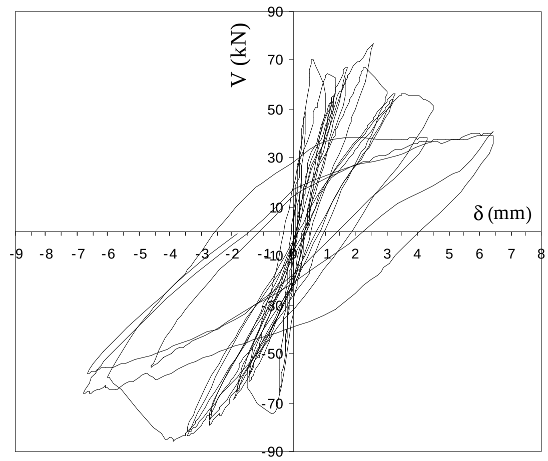

Figure 3 Lateral load-displacement hysteresis loops of JCM

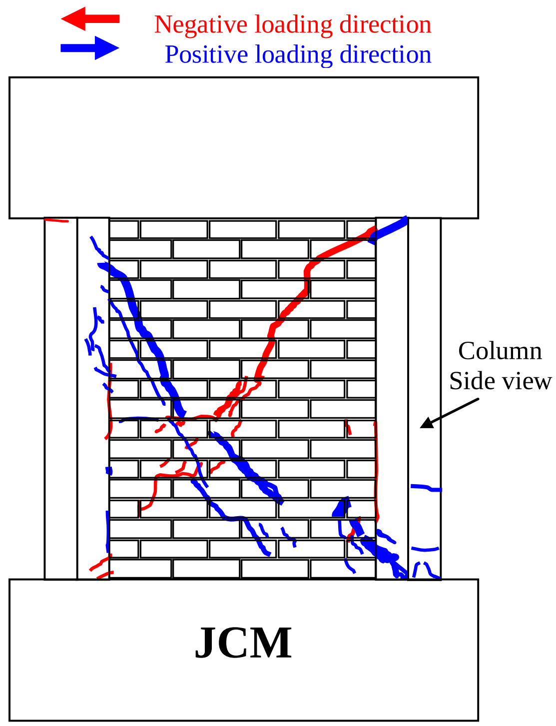

Figure 4 Final observed cracks

In this study, three failure hypotheses for splitting are considered; failure at a critical in-plane tensile strength as proposed by Turnsek and Cacovic[4)], failure at a critical tensile strength by considering the interaction effect of confinement elements which is developed by Tomaževič and Klemenc[3)] and failure by a critical biaxial combination of normal principal stresses according to Sucuoglu and McNiven[2)]. The three approaches are conveyed by the following Eqns. (1), (2) and (3):

$$H_m = A_m \frac{f_t^{'}}{b}\sqrt{\frac{\sigma_V}{f_t^{'}}+1} \qquad \textit{After Turnsek Cacovic} \qquad (1)$$

Where:
H_m : the lateral resistance force of masonry brick panel.
b: the shear stress distribution factor depends of the height h and the width l of the brick panel,
b= 1 for $h/l \le 1$, b= h/l for $1< h/l <1.5$ and b= 1.5 for $h/l \ge 1.5$.
σ_V = the average compression stress on the brick panel due to vertical load V.
A_m = the horizontal cross-section area of the brick panel only (tie-columns are not included).
f'_t = the masonry tensile strength, obtained by diagonal compression test as specified by ASTM C1391 as defined previously.

$$H_m = \frac{f_t^{'} A_m}{b C_i}\left[1+\sqrt{C_i^{\,2}\left(1+\frac{\sigma_V}{f_t^{'}}\right)+1}\;\right] \qquad \textit{After Tomaževič and Klemenc} \qquad (2)$$

Where:

$C_i = 2\alpha b \dfrac{l}{h}$ the interaction coefficient (α = 5/4 is a parameter of shape and distribution of interaction forces, *h* and *l* are the height and the width of the brick panel, respectively).

$$H_m = \frac{A_m}{b(1+\beta)}\sqrt{\left(\beta\, f_m^{'}\right)^2 + \beta\, f_m^{'}\sigma_V\left(1-\beta\right) - \beta\,\sigma_V^2} \qquad \textit{After Sucuoğlu et al.} \qquad (3)$$

Where:

$\beta = f_t^{'}/f_m^{'}$

f_m = the masonry compressive strength obtained by testing stack bonded prisms of five bricks according to the specifications of LUMB1(1994).

$f_t^{'} = 0.5187\, f_m^{'} \dfrac{P_d}{\left(f_m^{'} A - 1.683 P_d\right)}$ *obtained from diagonal compression test After Yokel and Fattal* (1976).

(P_d and A are the maximum diagonal compressive load and the specimen cross section, respectively).

4.2. Tie Columns Effect

In the confined masonry system the RC confining elements prevent the collapse of the masonry and cause additional compression stress in the vertical and horizontal directions; hence a certain amount of additional shear can be transmitted by dowel action of the vertical bars. This mechanism occurs mainly at wall corners.

The dowel strength of vertical reinforcements crossing the crack can be estimated by assuming the reaction of concrete on these bars as described in the Fig. 5.

In confined masonry system, the additive shear force which can be caused by stirrups is usually neglected because of the large span between two successive stirrups. However, the amount of shear generated by the tie-columns is evaluated by the Eqn. (4).

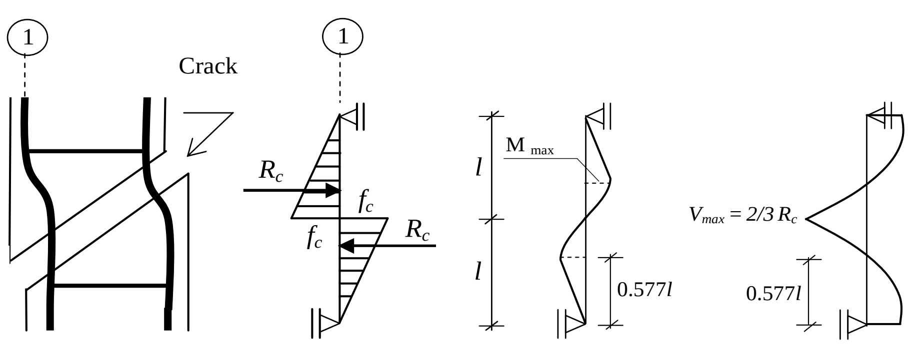

Figure 5 Dowel action mechanism of vertical reinforcements

$$H_{dowel} = n\frac{2}{3}R_c = n\frac{2}{3}\lambda d^2\sqrt{f_y f_c} \tag{4}$$

Where:

R_c = the reaction of the concrete on the main bar.
n = the total number of vertical bars in the RC tie-columns.
λ = 0.619 in case of elastic behavior and 0.8056 in case of a full plastic cross section of the main bar.
d = the main bar diameter.
f_y = the yield stress of vertical bar.
f_c = the compression strength of concrete.

As a result, the total shear resistance of a confined masonry wall H_w is then:

$$H_w = H_m + H_{dowel} = H_m + n\frac{2}{3}\lambda d^2\sqrt{f_y f_c} \tag{5}$$

Table 2 Analytical and experimental lateral shear resistance of the wall

	H_m (kN)	H_{dowel} (kN)	$H_w = H_m + H_{dowel}$ (kN)	$H_{w\,Exp.}$ (kN)	$(H_w - H_{w\,Exp.})/H_{w\,Exp.}$%
After Turnsek	74.5	21	95.5	81.25	17.54
After Sucuoglu	94.37		115.37		42
After Tomaževič	104.2		125.2		54.1

5. CONCLUSION

By applying Eqns. (1), (2), (3) and (4), one observes that the contribution of the masonry brick panel to the total shear capacity represents an average value of 81%, while the remaining 19% is attributed to the tie-columns which are in agreement with Umek (1971).

Results obtained from the three proposed approaches and test results summarized in Table 2 reveal however that the calculated value of the lateral shear resistance overestimates the real shear capacity by 17.54%, 42% and 54.1%, depending on the method. This can be explained in a certain way by the fact that while assessing the shear capacity of the masonry structural members, many factors influence their resistance; such as the heterogeneity caused by the mortar joints the variability of the compressive strength of the brick units, the intensity of the gravity load… etc.

On the light of these results, the appropriate approach to predict the shear capacity of the masonry structural member needs to be selected carefully to be consistent to a certain extent with the results of the experimental testing. However, for more precision on the general applicability of any approach, more experimental data are required.

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