HOLLOW BRICKS IN BEARING WALLS: AN EXPERIMENTAL AND THEORETICAL INVESTIGATION

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SUMMARY

Walls made of hollow bricks with vertical holes,1x1 m in dimension have been tested, loaded by forces in their own plane. A common feature was the incomplete filling by mortar between bricks of the same layer, while the brick dimensions and hole percentage ranged around the values commonly met in European practice.

The experimental limit shears exhibit a linear correlation with the normal stresses, as in a friction type failure. However no evidence of friction was experienced in the failure mechanisms. A theoretical model for different limit states was therefore developed, to explain experimental findings, in particular this linear correlation and, besides, crack pattern and amplitude, strength and stiffness degradation in repeated cycles. Such theoretical model is fairly convincing for h llow bricks but seems promising for brick masonry in general.

Large ductility and large hysteresis loops were also experienced. This suggests that the brittle behaviour of such walls during earthquake shaking be connected to out-of-plane forces.

PURPOSE

Since the early tests of Musschenbroek and of Coulomb about two centuries ago, a variety of experiments have been undertaken on unreiforced masonry. Great portion of these has been included in at least three critical reviews: one prepared at the University of Illinois [1] ,one at UCB [2] and one by Yarar at ITU, Istanbul [3]

When attention is focused on seismic behaviour, the number of significant experimental findings is sharply reduced: in fact, due to the peculiar anisotropy of masonry, only tests reproducing a seismic-type state of stress are representative. The loading apparatus of Fig.4 was selected for the present research with this intention, and, besides, with two peculiar aims:

1) to give a picture of the actual strength,during a severe earthquake,of a class of brick-work that has not received too much attention by research. Namely: a) walls made of bricks of large size (typically 20x20 cm) with vertical holes for a large portion of their plane, and b) clay-extruded bricks, that have shown quite a brittle behavior in recent earthquakes in Italy as well as in other mediterranean countries and in South America.

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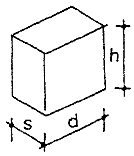
2) to interpret the failure mechanism and develop a mathematical model suited for more complex structures.

DESCRIPTION OF THE TESTS

A typical specimen is shown in Fig.1. All the other specimens have the same dimensions. Based on widespread evidence among rural houses in the mediterranean area, incomplete mortar filling was left between bricks on the same layer, and besides, the mortar was not selected to avoid horizontal shrinkage. The quality of the mortar used was kept as constant as possible, with a compression strength ranging around 140 kg/cm^2.

Different types of bricks were used, according to the following table:

Brick type	Dimensions (cm) dxsxh	Percentage of holes	Fig.
1	 12x20x20	50%	1
2	24x24x20	50%	2
3	24x12x6	12%	3
4	20x12x10	50%	

The loading apparatus is shown in fig.4: the reinforced concrete base of the specimen is fixed to the Lab.floor ,while two actuators apply forces (A and B) at 45° with respect the vertical axis in the plane of the specimen. Varying the intensity of A and B one can have any resultant force on the pier, passing through point C. The apparatus is particularly suited for identification, since the forces on the specimen are always known, i.e. no unforeseen increase in the vertical forces arise, as in the apparatuses that constrain the rotation of the top of the specimen.

The average stresses (normal σ and shear τ) on the horizontal cross section of the wall are:

$$\sigma = (A + B) / (S\sqrt{2}) \qquad (1)$$

$$\tau = (A - B) / (S\sqrt{2}) \qquad (2)$$

where S is the gross section, i.e. the total section surface without substracting the holes.

EXPERIMENTAL RESULTS

The first loading cycle for every specimen has been applied in two steps, 1) loads A and B were proportionally increased. and 2) load A only was increased, till a full mechanism was evident. An overview of the limit loads obtained in the first cycle for each specimen is given in fig. 5. More details on the results can be obtained in Ref. [4] .

The following conclusions were derived:

1) The correlation between limit shear τ_L and simultaneous normal stress σ has the format:

$$\tau_L = \tau_k + \mu \sigma \qquad (3)$$

well known in the literature. However no evidence of friction was observed in the experiments, since the mortar penetrates into the vertical holes and makes slip impossible.

2) Wall type 1 is weaker than type 2. Everything being the same, except brick size, the influence of the latter feature must be incorporated into the theoretical failure model.

3) Walls type 1 and 2 (with a large ratio h/d) exhibit cracks in the bricks starting at the corner (see fig.6) while walls type 3 and 4 (with low ratio h/d) exhibit cracks crossing amid the bricks (see fig.3). Two different failure modes therefore seem to be involved.

In the load cycles following the first, some common features were recognized, namely:

1) If the shear stress has the same direction both in the first and in the second cycle, the latter limit shear is substantially lower (see fig.7) than that in the first cycle. On the other hand, if the direction of the load is reversed, the second cycle is similar to the first, and only the third shows lower (see fig.8) limit shear.

2) Large ductility and large hysteresis loops are shown to develop (see fig.8). Therefore the brittle behaviour experienced in real earthquakes should be due to the simoultaneous presence of out-of-plane forces.

3) As far as the deformations for extreme loads, a picture of the measured amount of relative vertical displacements bridging the horizontal cracks on two mortar layers is given in fig.10.

THEORETICAL MODEL IDENTIFICATION

Three different stages of behaviour were identified during a monotonic increase of lateral force:

(I) An elastic range, during which the bricks are in full contact. Nonlinearity and anisotropy are already present in this stage.

(II) The brick rotation range, during which the single brick equilibrium is satisfied by the typical picture of stresses of Fig.9 a. Here it is shown that the horizontal-shear moment is unlikely to be equilibrated by the vertical shear-moment, thus producing brick rotation and reduction of the contact surface.

(III) A collapse range, during which bricks or mortar are expected to fail either by excessive compression or by excessive shear in a vertical section. Both types of failure may be present in the same specimen, see fig.9b.

A discussion on the feature of theoretical models for these stages is here given in the sequel.

Stage I: A few theoretical models have been implemented for analyzing the elastic range, mainly for service loading. In the model of Page [5] the two phase composite is replaced by a homogeneous,anisotropic, elastic medium. A different approach, originally proposed for layered and fiber reinforced composites, has been mentioned as applicable to masonry, [6] and, recently, Mengi and McNiven considered the material as a mixture consisting of two phases, and used the therory of mixture to obtain approximate equations governing the dynamic behaviour in the elastic realm, [7] .

With reference to the model of Page for this realm, the main characteristics of the masonry, as distinguished from those pertaining to the single brick or to mortar, are:

- The shear deformation versus shear stress diagram is a monotonic curve independent of the normal load;
- The Poisson modulus is approximately zero;
- the normal and tangential moduli are between those pertaining to bare bricks and those pertaining to mortar.

Whether these characteristics are to be incorporated into a linear or nonlinear elastic model has not been clarified yet. Even a rigid model could be appropriate,provided that it be able to describe the normal load distribution at the transition between the first and the second stage. Deformations during this stage are in fact meaningless in comparison with maximum deformations during an earthquake.

Stage II: A number of tests are reported for wall samples under vertical and horizontal loads in the second stage [1] , [2] , [3] . On the other hand, to the authors' knowledge, the mathematical models so far developed aim only at the description of the conditions under which the limit shear is reached in one horizontal section: i.e., no mathematical model aims to describe the evolutive behaviour in the second stage. Such a model is attempted here.

To this purpose the equilibrium equations for a single brick are introduced. Symbols are referred to fig.9 a.

$$M_t - M_b = T' h \tag{4}$$

$$N_t - N_b = - S' h \rho \cong 0 \tag{5}$$

where M_t and M_b are the moments around any point of the normal stresses acting on the top and bottom surfaces of the brick, T' is the shear force on a brick, ρ its density, h is the brick height, and N_t,N_b the total vertical normal forces acting at the top and bottom of the brick.

With reference to fig.9 one can express eq.(4) as:

$$N_b + N'b' + N''b'' = T'h \tag{6}$$

where $N_b = N_t = N$ because of eq.(5).

Dividing both sides of eq.(6) by the brick area and by h

$$\sigma \frac{b}{h} + \frac{N'b' + N''b''}{hS'} = \tau \tag{7}$$

where σ , τ , are the average normal and shear stresses on the brick.

The comparison of eq.(7) with eq.(3), widely reported in the literature, suggests that the quantity τ_k depends on the tension resistance of the mortar, and that $\mu = b/h$.

Stage III: Stress states $\tau < \tau_L$ pertain to stage I of behaviour and those $\tau = \tau_L$ to the brick rotation range (stage II). The collapse range, stage III, is reached after a given brick rotation and can be characterized mathematically by the attainment of a given deformation.

Cyclic loadings: two different situations occur depending on whether the previous loading has been kept withinthe elastic range or has substantially proceeded into the brick rotation range. In this second case, mainly if the load sign is reversed, the lateral resistance of the specimen can never again reach the value of the first cycle, even if this did not exploit all of the available ductility.

In spite of the great variety of tests reported,not a single precise model of masonry behaviour under progressive cyclic loads has ever been described. As to the experiments at the authors' Institute one valuable result was found, and physically explained [4] : the ultimate lateral load τ_L ,after cyclic loadings can be expressed as:

$$\tau_L = \mu \sigma \tag{8}$$

where again σ is the normal load on the specimen, and μ assumes the same values as in the monotonic tests. Eq.(8) can be easily related to the quantities of Fig.7: during cyclic tests the tensile resistance of the mortar is nullified, and therefore the terms N',N" of Eq.(7) become zero.

Fig.9 b explains also the two different types of brick failure:the first (diagonal) due to concentration of stresses in the corner, the second (vertical amid the brick) when the shear force on a vertical plane (equal to the force N for the brick) exceeds the limit strength in shear of the brick itself.

CONCLUDING REMARKS

The main conclusions that are pertinent to the experimentally observed behaviour have been already stated at the beginning (see Summary).

As far as the theoretical model, the local equilibrium conditions at various stages are here obtained. This will help in building a full mathematical model to follow the evolution during cyclic loads. This aspect of the research is now in progress. The basic conditions that have to be added are compatibility conditions on relative displacements. Based on the experimental results and on results in the literature [5] , the latter conditions for stage II and III are basically equality on vertical and horizontal displacements of selected corners and mid points of two adjacent bricks, that behave as quasi-rigid elements.

REFERENCES

1 SÅHLIN S., "Structural Masonry", Prentice-Hall Inc., Englewood Cliffs 1971.

2 MAYES .C., CLOUGH R.W., "State of the Art in Seismic Shear Strength of Masonry", Rep. No.EERC 75-21, Univ. of California, Berkeley, 1975.

3 YARAR R., "Behaviour of Masonry Brick Walls under Earthquake Loading", 5th Regional Seminar on Earthquake Eng.,Udine (Italy),1977.

4 CASTELLANI A., VITIELLO E., "Sperimentazioni su murature di mattoni a fori verticali soggette ad azioni di tipo sismico nel loro piano", Atti Convegno CNR-Geodinamica, E.S.A. Editrice, Roma, 1979.

5 PAGE A., "Finite Element Model for Masonry", ASCE, J. of Struct. Eng. Div., Vol.104, ST8, aug.1978.

6 HEGEMIER G.A., BACHE T.C.,"A General Continuum Theory with Microstructure for Wave Propagation in Elastic Laminated Composites" J. of Applied Mechanics, Vol.41, 1974.

7 MENGI Y., McNIVEN H., "A Mathematical Model of Masonry for Predicting its Linear Seismic Characteristics", Rep.n° EERC 79/04, Univ. of California, Berkeley, 1979.

FIG. 1 - SPECIMEN WITH BRICK TYPE 1

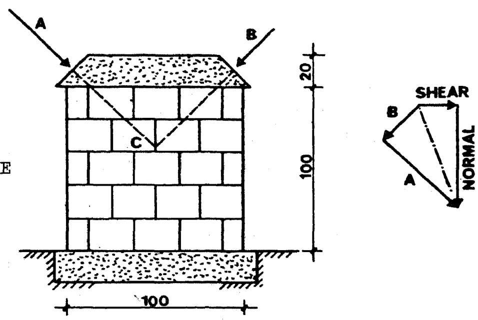


FIG. 4 - LOADING SCHEME

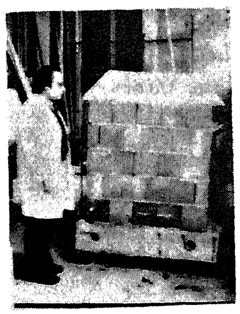


FIG. 2 - SPECIMEN WITH BRICK TYPE 2

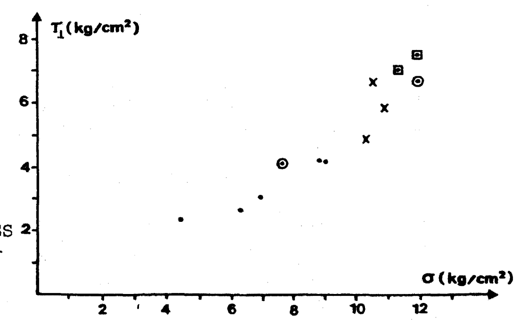


FIG. 5 - LIMIT SHEAR τ_L VERSUS NORMAL STRESS σ, AT THE FIRST LOADING CYCLE.

Brick type 1 •
" " 2 x
" " 3 ⊙
" " 4 ⊡

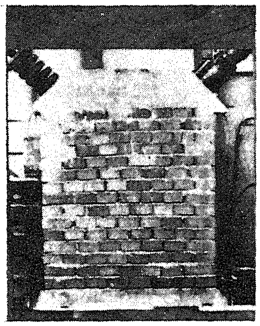


FIG. 3 -
SPECIMEN WITH
BRICK TYPE 3

FIG. 6 - (ABOVE)
Details of cracks in brick type 2.

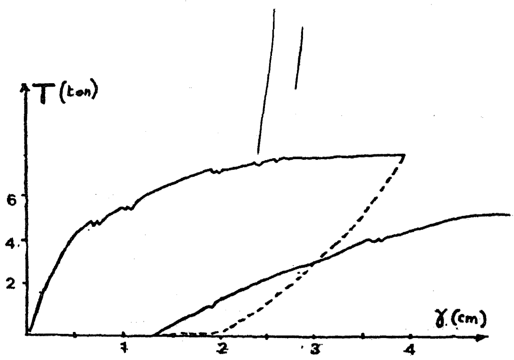


FIG. 7 - (LEFT)

Brick type 2. Shear - deformation diagram for two loading cycles having the same direction of shear force T. γ is the relative horizontal displacement between the concrete blocks at the top and at the bottom.

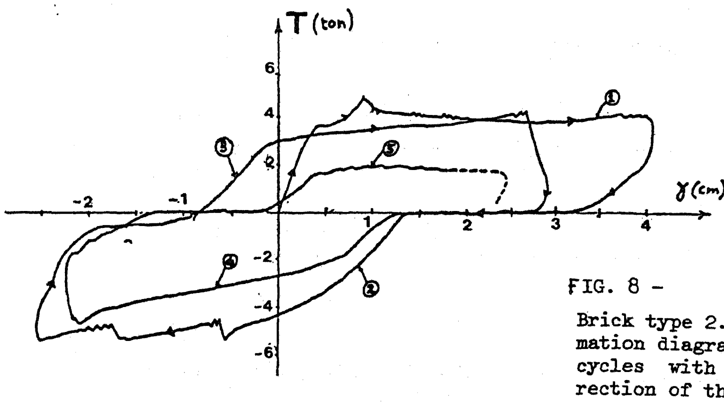


FIG. 8 -

Brick type 2. Shear deformation diagram for loading cycles with alternate direction of the shear force T.

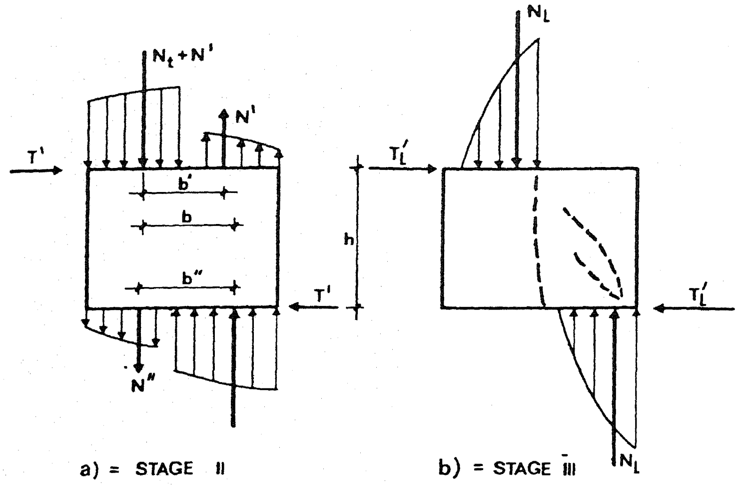


FIG. 9 - BOUNDARY FORCES ON A BRICK AT STAGES II AND III.

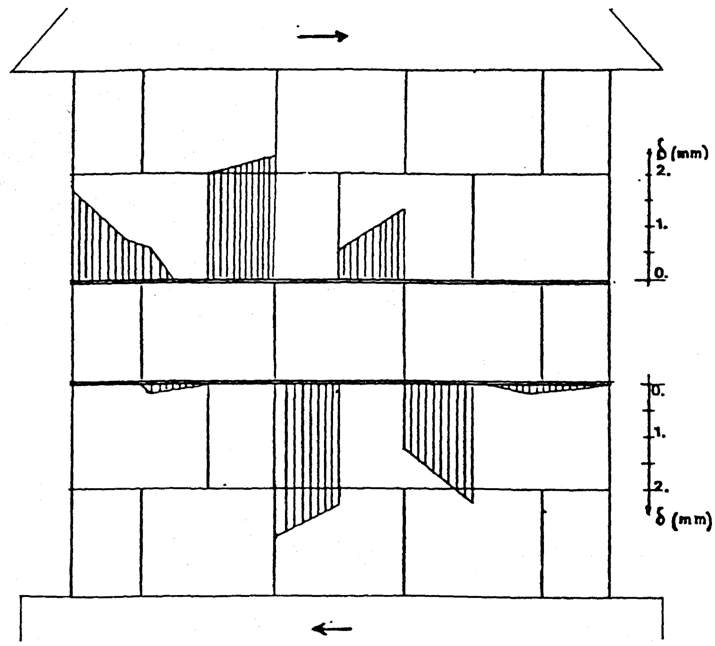


FIG. 10 - STAGE III - DIAGRAM OF MEASURED OPENING OF CRACKS IN TWO LAYERS OF MORTAR