IV–29. Strength and Ductility Tests for the Design of Reinforced Brickwork Shear Walls

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ABSTRACT

Experimental research showed that a suitable steel reinforcement can provide to brickwork sufficient strength and ductility for a satisfactory and a safe behaviour under seismic-type actions. The present study is particularly devoted to shear strength and to ductility and energy absorption under alternate shear actions, in order to provide criteria for the design of reinforced brickwork shear walls.

A particular "beam-test" has been developed in order to submit portions of walls to alternate diagonal actions. Tests have been performed on couples of small walls formed with vertically perforated ceramic units; the objective of the tests has been the comparison of the different behaviour of walls without reinforcement, walls with horizontal, vertical reinforcement and walls having both horizontal and vertical reinforcement.

The experimental behaviour showed the different roles of vertical and horizontal reinforcement, as well as the different possible types of failure relevant to the specific technique: bond slip in the joints, diagonal cracking of the units and yield of steel; diagonal crushing in compression.

A satisfactory hysteretic behaviour with ample ductility and large energy absorption was expected and has been observed only when the yield of steel was allowed; practical suggestions are consequently derived for the limitation of the vertical load and of the amount of reinforcement.

The tests called the attention on the importance of the strength to diagonal compression, in the case of highly perforated units. Information about the deformability to shear has been obtained. Within the limited range of the research, the conclusions are positive for the possible development of a seismic-resistant reinforced brickwork.

INTRODUCTION

Earthquake resistance of structures needs strength and ductility. As regards the single element in the structure, besides the strength, the initial stiffness, the damping characteristics the ductility and the capacity of energy absorption in the plastic range and the change in stiffness and strength due to cyclic loading are worth determining.

The aforesaid quantities are often found statically by testing under alternate actions and extending the results to dynamic situations.

In the case of brickwork shear walls, the behaviour depends on the resistant characteristics of the constitutive materials, and on the ratio between vertical and horizontal actions.

Plain brickwork walls subject to shear actions fail by slip along the mortar beds or by diagonal cracking of the units according to the ratio between strength of mortar and strength of units, and depending on the amount of the vertical action.

A fundamental way of improving the behaviour of walls under repeated and cyclic loading is the use of reinforcing steel.

The presence of reinforcement, its amount and the yield point have an effect on the beginning of cracking, on the density of the cracks and their opening, on the strength and, chiefly, on the ductility of the wall: a load transfer from units to reinforcing bars after the first cracking allows the bearing capacity to increase and plastic deformations to take place without the wall loosing its entirety.

TEST SETUP

The principal tests employed for getting information about the shear strength and the ductility of walls are the "racking test" and the "diagonal compression test."[5]

In the "racking test", the uncertain boundary conditions make difficult the interpretation of the experimental results.

The present test is a kind of "beam-test", as the figures 1 to 3 show. The two elements of each couple are joined at the top inner face by a steel hinge providing the transmission of the horizontal action, named H, between the two walls. At the bottom the elements are connected by a steel tie, by means of which the horizontal action is applied at the outer bottom corner of each wall. Vertical actions, named V, are supplied by the testing machine.

Only square elements were tested: this involves the equality of the vertical and horizontal forces.

The test setup allowed to load the walls with alternate horizontal actions by means of a rearrangement of the elements every half cycle. The diagonal subject to tension in the first phase (½ cycle) is therefore in compression during the second phase (Fig. 5).

The instrumentation used in the tests is shown in figures 1, 2, 4:it consists of dial gauges, transducers and electrical strain gauges.

Dial gauges and transducers are set along both diagonals in order to measure strains across the expected cracks.

The readings of the transducers referring to the opposite faces of the same wall are averaged and registered by a X-Y recorder. The curves so obtained and shown in Figs. 6, 7, 8 plot the load (H = V) versus the deformation on the gauge length of 400 mm.

ELEMENTS AND MATERIALS

The brickwork elements were tested in couples as the figure 3 shows. Four couples, named T1, T2, T3 and T4,

are examined: the only difference among the couples is the reinforcement, being:

- —T1 a couple of plain brickwork walls,
- —T2 a couple of horizontally reinforced walls,
- —T3 a couple of vertically reinforced walls,
- —T4 a couple with both horizontal and vertical reinforcement.

Each wall is made of four courses of hollow units in running bond executed between upper and lower R.C. beams. Local failures due to concentrated loads are avoided by the presence of the R.C. beams.

The mechanical properties of the constitutive materials (units, mortar, reinforcement) and of masonry itself are collected in Table I as well as the distribution of the horizontal and vertical reinforcement in the wall.

The horizontal reinforcement consisted in 2 wires ø 4 mm connected diagonally by a ø 3.75 mm wire; such trusses being disposed in every bed joint, the percentage of horizontal reinforcement is: $0.25/20 \times 17.5 = 0.07\%$.

The vertical reinforcement (ø 8 mm bars every 30 cm) is: $0.50/30 \times 17.5 = 0.096\%$.

Such values are below the minimum required[19] by the "Design guide" of B.C.R.A. (0.2%) and near to the minimum suggested by the current U.S.A. Building Code, only when both reinforcements are present (T4). Additional compressive tests orthogonally to the holes have been performed on the units, in order to provide information about the possible strength of the inclined struts in the failure mechanism of the walls. Three types of tests (see Fig. 9) gave the following mean strengths computed on the gross area:

—test a): $f_{ch} = 3.60$ N/mm^2

—test b): $f_{ch} = 3.20$ N/mm^2

—test c): $f_{ch} = 0.76$ N/mm^2

Test c) shows how dramatic can be the reduction of the strength to the horizontal component of the action when the vertical joints are not filled with mortar, and this latter is only surrounding the vertical bars; this was the unfavorable situation simulated in all the tested walls. Such results suggest a suitable study of the coring pattern in order to improve the diffusion of the horizontal actions, for instance by a truss-shaped cross section of the units.

EXPERIMENTAL RESULTS

It should be noticed that the shear stress referred to by different authors in the interpretation of test results is variously defined. Herein the conventional definition is used giving an apparent value for f_v as:

$$\tau = \frac{H}{A}$$

where A is the gross area of the wall (different values can be obtained if the "net mortar contact area" is used, according to N.B.S.[5]). Such a definition involves the hypothesis of a uniformly distributed shear stress.

Other definitions, more significant from the theoretical point of view, are given in[18].

The values given in Table II correspond to the first cracking and the maximum load reached for the examined walls.

The 4th and 7th column in Table II give the value of the principal tensile stress at first cracking[18]:

$$\sigma_t = \frac{2\sqrt{2}\,H_c}{\pi A}$$

and the horizontal compressive stress at the maximum load[18]:

$$\sigma_h = \frac{2\sqrt{2}\,H_u}{\pi A}$$

The presence of reinforcement increases the load of first cracking as well as the shear strength and modifies the behaviour under repeated and cyclic loading.

Cyclic Behaviour

For the interpretation of the cyclic histories of Figs 6 to 8 is essential to notice that the abscissas do not represent a displacement (as usually done for similar tests), but the opening of one of the cracks. In this way the study of the two orthogonal cracks is separated: each one shows in fact the plastic deformation when in tension, and the stiff behaviour in the subsequent half close-up by compression. The overall behaviour of the panel can be obtained by the combination of the two separate measurements.

Plain Brickwork Walls (T1)

Cracking occurs along the mortar bed joints, and, after cracking, the characteristics of a friction phenomenon can be noticed, as the Fig. 6 shows; the shear stress increases with increasing vertical load. Only the first 1/2 cycle has been performed, because of the loss of the masonry entirety. The obtained values of the maximum shear stress (Table II) are in reasonable agreement with the literature data for hollow blocks[5] (sensibly less than for solid bricks).

Horizontally Reinforced Walls (T2)

The shear stress values obtained in this case, and the recorded H-δ curves seem to show a behaviour similar to the plain brickwork walls. The mechanism was in fact slip along bed joints; the horizontal reinforcement had probably a function in redistributing shear in the joint.

The formation of a truss mechanism was not evident; in any case, in such a model the maximum shear strength that the reinforcement could provide is considerably less than the sustained shear (115 kN).

At the first reversal the strength fell down to 80 kN.

Vertically Reinforced Walls (T3)

Vertical reinforcement proved to be efficient and to contrast shear across the diagonal cracks formed partly along the joints and partly across the units. The theoretical strength provided by the steel bars is:

$$V_t = 4 \times 50.2 \times 0.570 = 114.5 \text{ kN}.$$

The actual values reached are:
—V = 125 kN at the first half cycle
—V = 145 kN at the second
—V = 135 kN at the third.

This shows that no load degradation was observed, even with an imposed deformation corresponding to a ductility factor higher than 16. Fig. 7 shows that the test was carried out up to an opening of 7 mm of the crack (in wall A1B2), and the sustained load was still 80 kN.

A different behaviour was shown by the similar wall A2B1: after having developed a ductility of 8 at the second half cycle, it appeared to begin to fail by compression of the inclined strut. According to the theory of elasticity, the horizontal stress under V = H = 135 kN was:

$$\sigma_h = 0.66 \text{ N/mm}^2$$

value very close to the limited strength ($\sigma_h = 0.76$ N/mm^2) found in the horizontal tests of the units connected by the partially filled vertical joint. This calls the attention on the risk of brittle compressive failure before yielding of the steel and therefore on the necessity of defining a maximum percentage of reinforcement.

Vertically and Horizontally Reinforced Walls (T4)

The results obtained with the vertical reinforcement are apparently in contrast with the conclusions of other authors[6] in favour of the horizontal reinforcement. The problem is very complex, and reasons of divergency can be found in many different details of the tested specimens. A clarification is certainly necessary, but the authors feel that is urgently needed to assess the conditions at which a wall can develop satisfactory behaviour under vertical and horizontal reinforcement. Vertical reinforcement is in fact necessary for the in-plane flexural strength of the wall subject to horizontal actions, and horizontal reinforcement helps to provide flexural strength for actions orthogonal to its plane.

This is the case of walls T4. In spite of the very low percentages of both reinforcements, no brittle failures of steel occurred, and the walls developed satisfactory strength and ductility. The maximum shear reached 175 kN (showing in this way the effect of the addition of the horizontal steel); it corresponds approximately to 1 N/mm^2, value mentioned by other authors.

In the successive 4 half-cycles the max loads are:

$$165 - 160 - 175 - 165 \text{ kN}$$

No load deterioration therefore appeared under large deformation, corresponding to a ductility factor of 14 for wall A1B2 and of 26 for wall A2B1 (see Fig. 8).

The horizontal compression reached the value: $\sigma_h = 0.86$ N/mm^2, with the consequent risk of compression failure; at the 4th reversal in A1B2 some symptoms of compression deterioration in fact appeared.

The stiffness deterioration due to close-up of cracks increases with the maximum width previously reached; Fig. 8 shows in fact the maximum in A2B1 after having reached a crack width of 4 mm; which is exceptionally large on the gauge length of 400 mm.

CONCLUSIONS

1. Shear walls constituted by vertically perforated clay blocks can withstand without reinforcement limited horizontal forces, depending on the amount of the vertical load; under moderate actions (and therefore without cracking) the stiffness can be satisfactory; under elevated actions leading to slip in the horizontal joints or to inclined cracks in the units, the walls can develop considerable inelastic deformation without loss of strength, by a friction-type behaviour; nevertheless, it is questionable if this behaviour can be considered sufficiently reliable under alternate repeated actions, because the wall loses its entirety.
2. Horizontal reinforcing steel bars included in the bed joints can increase the strength to shear by a redistribution of shear stresses in the joint and so delating the beginning of slip in the weakest point; after cracking such a reinforcement can only be effective across inclined cracks formed in the units (case of weak units), but cannot help in the case of slip along horizontal joints, case in which the behaviour of the wall can be similar to that of the unreinforced wall.
3. Vertical reinforcing bars can provide after cracking satisfactory strength, ductility and energy absorption capacity, provided some complementary conditions are satisfied:
   i) running bond pattern provides an efficient interlocking resistance to vertical shear;
   ii) the bars are efficiently bonded to the units by mortar applied in convenient cavities;
   iii) rupture by inclined compression is avoided.
4. The brittle failure of reinforcement must be avoided by adopting a minimum amount of reinforcement able to sustain the forces acting when cracking occurs.
5. The brittle failure by inclined compression must be avoided by a convenient limitation of the amount of reinforcement; such a maximum reinforcement is a function of the strength of the units to in-plane horizontal compression and has to be determined so as to ensure that the steel yields in the truss mechanism before the inclined compression failure of the units; due account has to be taken of the way in which the horizontal actions are transmitted between the units (partial filling of the vertical joints). Such risk of failure of compression struts is considered by the authors the main concern in developing the considered type of walls; therefore the void percentage of the blocks has to be limited, the coring pattern accurately studied for a sufficient horizontal strength of the unit, the vertical joints well defined and tested.
6. Vertical and horizontal reinforcements can cooperate and increase the ultimate strength to shear when inclined cracks are formed; the net of the two perpendicular reinforcements seems desirable also for a convenient preservation of the entirety of the wall after several cycles of alternate horizontal actions, therefore also for the preservation of its strength to vertical load and to actions perpendicular to its plane.
7. Under the above listed conditions, reinforced shear walls made by vertically perforated clay blocks seem to be in measure of behaving similarly to corresponding reinforced concrete walls under alternate actions. Further

research is needed for the definition of precise design rules.

ACKNOWLEDGEMENTS

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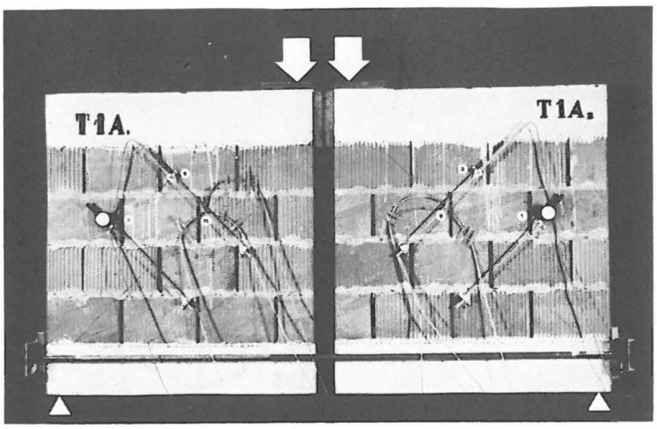


Figure 1

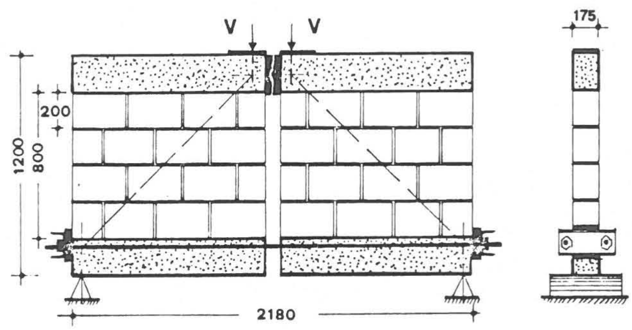


Figure 3. Test setup

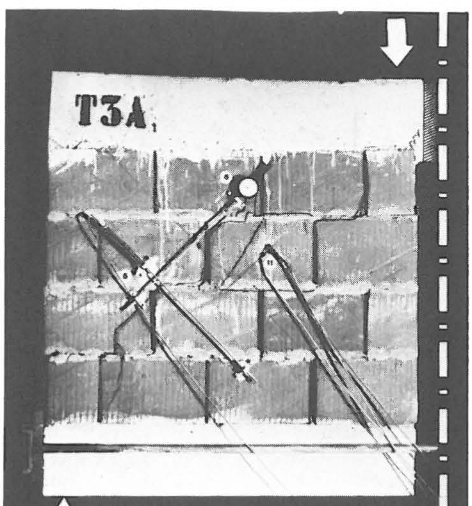


Figure 2

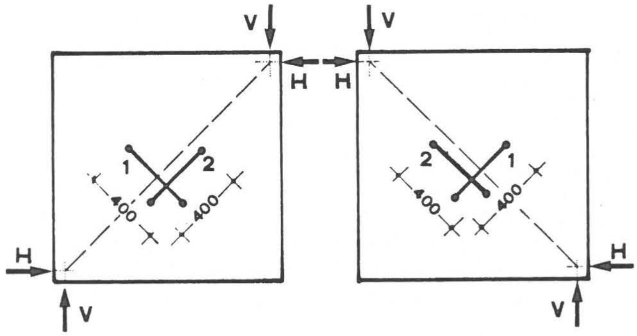


Figure 4. Instrumentation: 1 transducers; 2 dial gauges

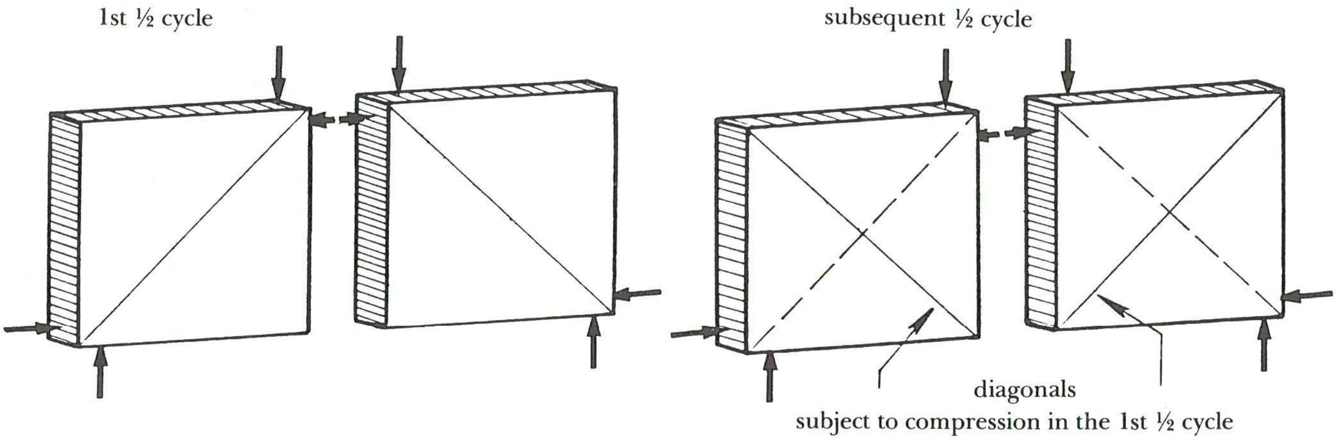
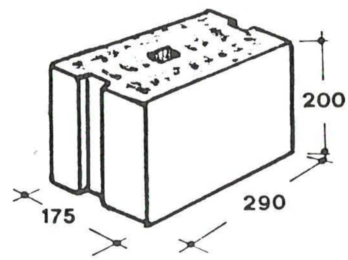
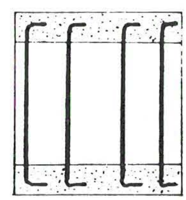


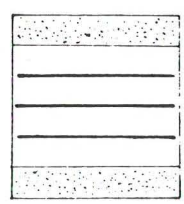
Figure 5. Load conditions in two subsequent phases of the test

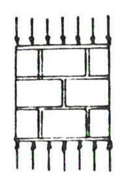
TABLE 1—Mechanical Properties

UNITS (hollow clay blocks)		
nominal dimensions 290×175×190 (mm) – 41% coring		
mean compressive strength parallel to holes	=	32.3 N/mm^2
characteristic compressive strength (// holes)	=	27.2 N/mm^2
mean compressive strength (⊥ hole)	=	3.6 N/mm^2
CEMENT MORTAR 1:0:3 (by volume)		
mean compressive strength	=	12.1 N/mm^2
mean modulus of rupture	=	2.6 N/mm^2
modulus of elasticity	=	13400 N/mm^2
REINFORCEMENT		
i) *vertical*—ø 8 mm deformed bars/300 mm		
yield point	=	380 N/mm^2
strength	=	570 N/mm^2
ii) *horizontal*—continuous wire truss Ø 4 mm		
strength	=	600 N/mm^2
MASONRY		
mean compressive strength (// holes)	f_{me} =	7.3 N/mm^2
characteristic compressive strength (// holes)	f_k =	5.7 N/mm^2
mean modulus of elasticity (// holes)	E =	7600 N/mm^2
shear modulus in the uncracked stage:		
plain masonry	G =	1300 N/mm^2
horizontally reinforced masonry	G =	2460 N/mm^2
vertically reinforced masonry	G =	3180 N/mm^2
hor. and vert. reinforced masonry	G =	3290 N/mm^2









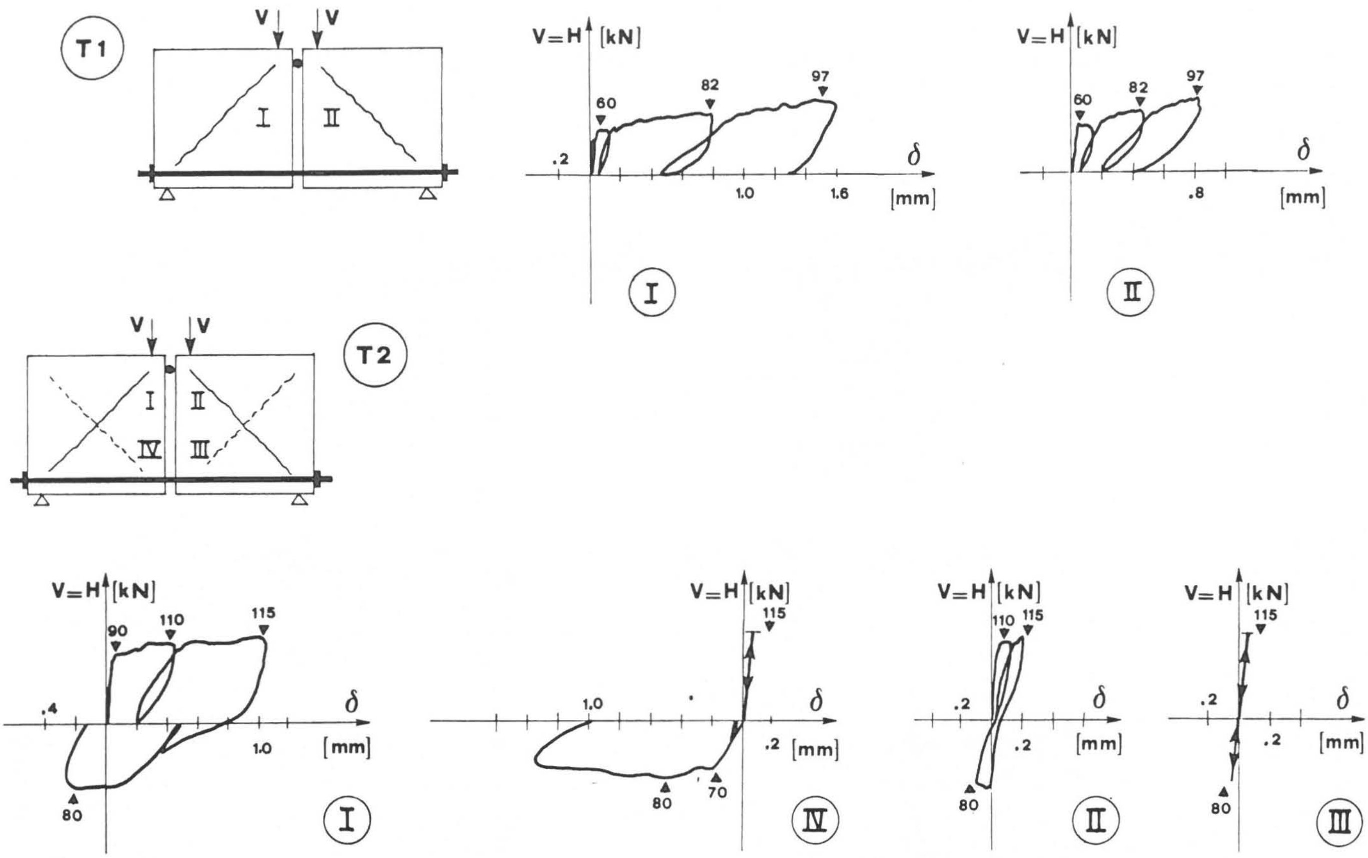


Figure 6. Loading cycles for the couples of walls T1 (plain masonry) and T2 (horizontally reinforced masonry)

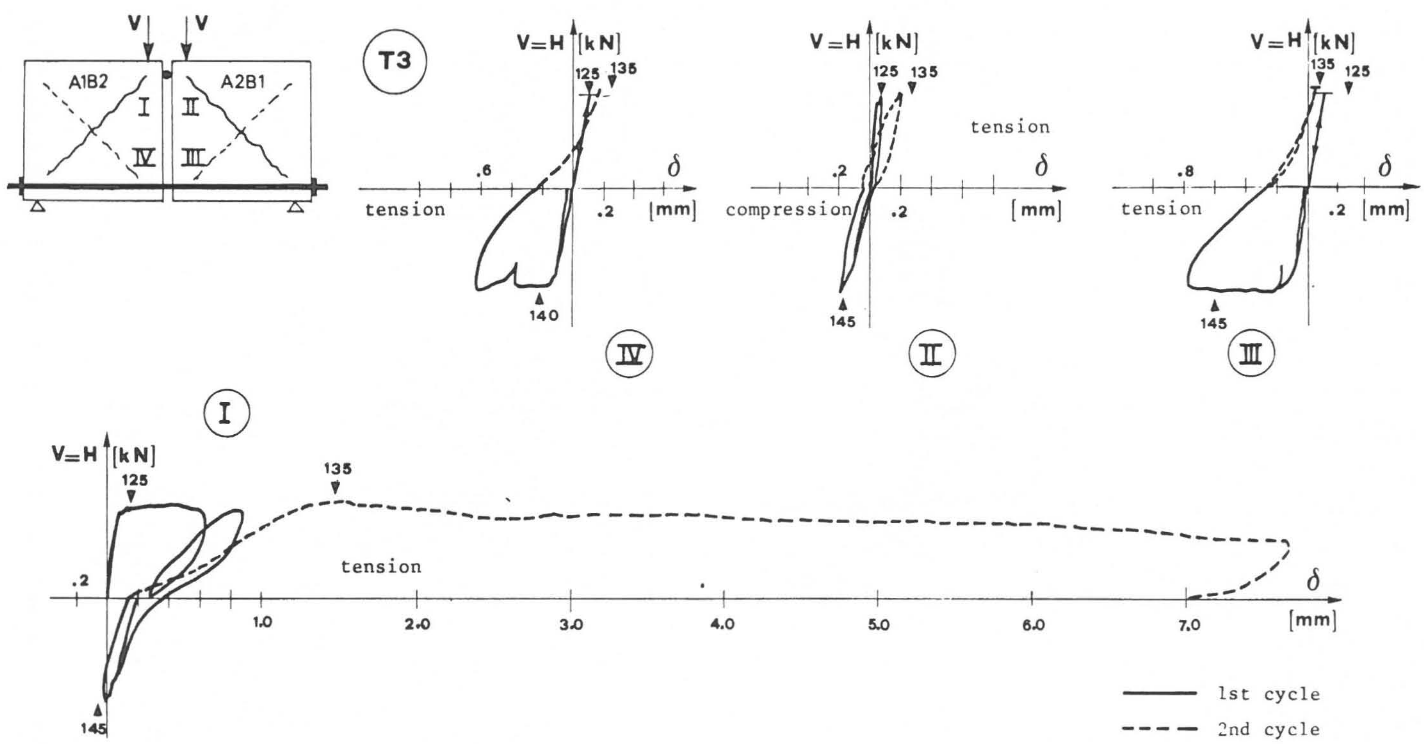


Figure 7. Loading cycles for the couple of walls T3 (vertically reinforced masonry)

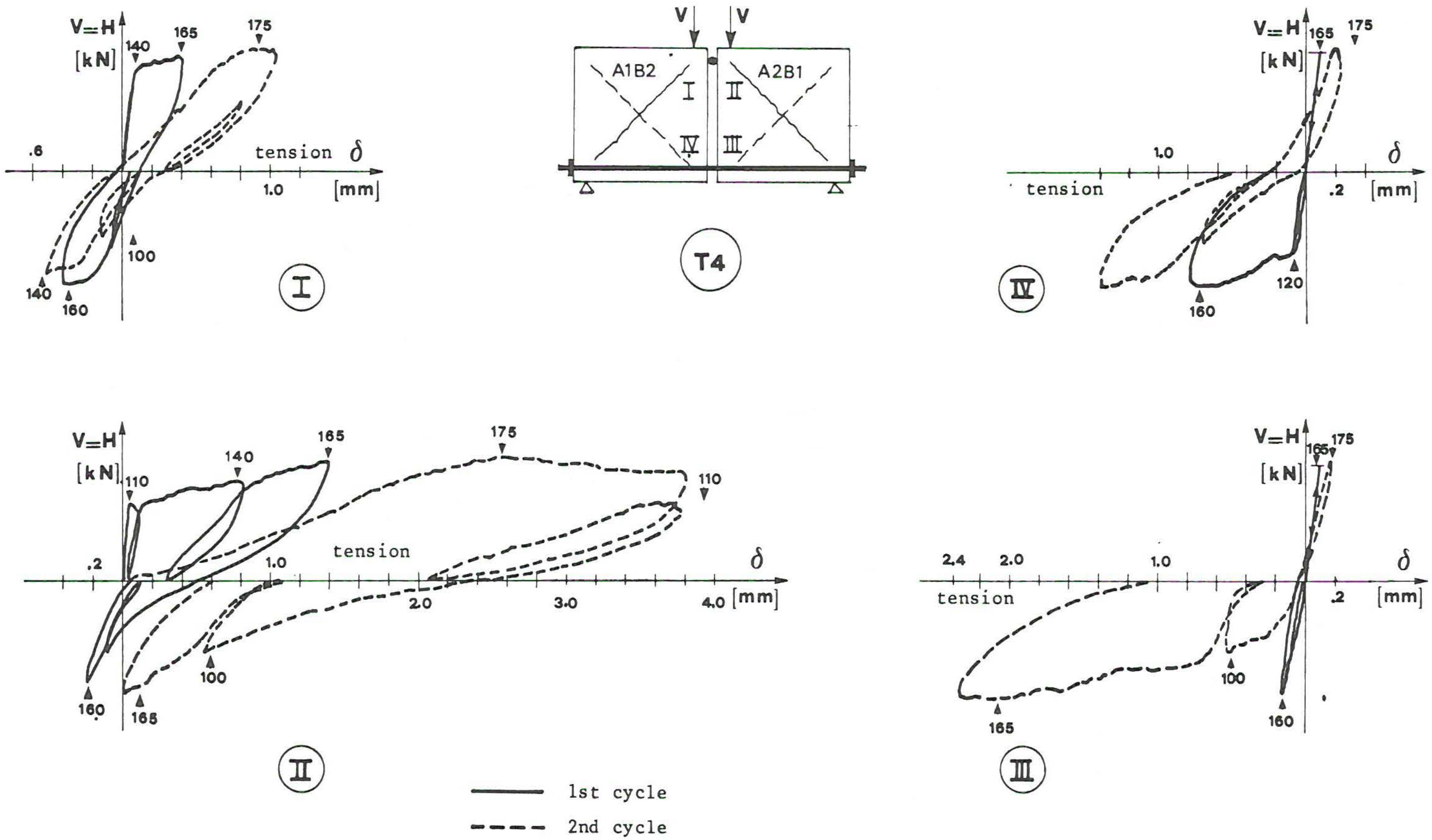
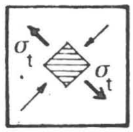
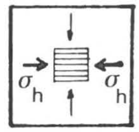


Figure 8. Loading cycles for the couple of walls T4 (both horizontally and vertically reinforced)

TABLE II—State of Stress





COUPLE OF ELEMENTS	FIRST CRACKING			MAXIMUM LOAD		
	H_c |kN|	$\tau = \frac{H_c}{A}$ |N/mm^2|	σ_t |N/mm^2|	H_u |kN|	$\tau = \frac{H_u}{A}$ |N/mm^2|	σ_h |N/mm^2|
T1 (plain)	60	0.33	0.30	97	0.53	0.48
T2 (hor.reinf.)	90÷110	0.49÷0.60	0.44÷0.54	115	0.63	0.56
T3(vert.reinf.)	110	0.60	0.54	145	0.79	0.71
T4(hor.+vert.)	110	0.60	0.54	175	0.95	0.86

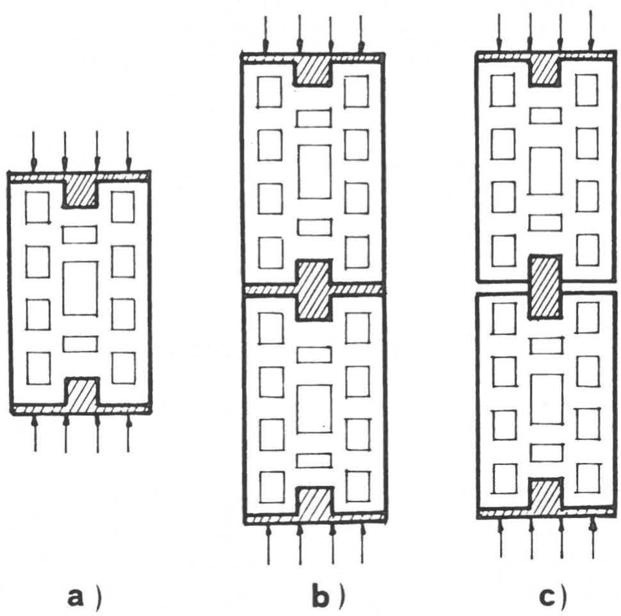


Figure 9.