DIAGONAL TENSION TESTS ON REINFORCED AND NON-REINFORCED BRICK PANELS

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SUMMARY

This paper contains the results of diagonal tension test carried out on 29 brickwork panel specimens of three different dimensions, namely 1.10x1.10 m, 0.90x0.90 m and 0.50x0.50 m approximately, made with hollow or solid clay bricks; some panels were reinforced with steel bars.

The results indicate that the effective width of equivalent diagonal strut is about an eighteenth of its length, for square panels. The diagonal stiffness of the square panels results, on the average,of 600 N/mm^2 (related to one millimeter of shortening of diagonal strut and one millimeter of panel thickness) and do not depend on the dimension of the walls.

Curves are plotted to show the relation between the diagonal loading and the masonry deformations.

For reinforced panels, the width of equivalent diagonal was similar to that one of non-reinforced masonry.

INTRODUCTION

In modern buildings, it is frequent the case where a brick wall is built in a reinforced concrete frame and the infill is not integral nor bonding with the frame.

The design of such buildings to resist earthquake forces can be based on the assumption that an infilled frame subjected to lateral loads may be approximately represented by an equivalent frame in which the infills are replaced by diagonal struts (Fig. 1).

In this case the knowledge of the stiffness and strength of the infill becomes imperative.

The problem of the infilled frames has received much attention in recent researches.

Smith (Ref. 1), describes tests on unframed mortar panels loaded along a diagonal between steel blocks. The Author concludes that, assuming the diagonal load to be entirely applied near the corner of the panel, the effective width of the equivalent strut, w, varies from d/4 for a square infill to d/11 for an infill having a side ratio of 5 to 1 (d being the diagonal lenght).

Later, for infilled frames, Smith (Ref. 2) finds that the width of the equivalent strut varies as function of a parameter:

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$$\lambda = l\sqrt[4]{\frac{E_c t}{4EJl'}} \qquad (I)$$

in which E_c, t and l are the Joung's modulus, tickness, and length of side respectively, of the infill, and E, J and l', are the Young's modulus, second moment of area, and length of the side of the frame.

Mainstone in an experimental research on infilled frames (Ref. 3) obtained the following formula:

$$w = 0.175\ d\ (\lambda)^{-0.4} \qquad (2)$$

About the behaviour of brickwork panels filled within structure frames there are other researches besides the ones mentioned above.

Neither the quoted papers, nor the many others recently published, give, in the Authors' opinion, a satisfactory and complete answer on the behaviour of brickwork panels infilling structural frames.

To increase the knowledge on the mentioned behaviour, an experimental research was performed concerning the brickwork panels only (and therefore with orthotropic material organization) of square shape.

We assumed as variable parameters: the length of the side, the thickness of the panel, the brick type and, for hollow bricks, the ratio A_H/A_G between the area of the holes and the whole area of the cross section of a brick. The influence of a reinforcement put within the holes is also considered.

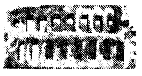
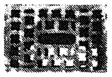
EXPERIMENTAL PROGRAM

Test Specimens

Tests were carried out on brickwork panels; overall dimensions of the panels and the bricks are described in table I.

The mortar used was 1:3 (cement: sand, by volume); the yield stress of the reinforcement bars was 517 N/mm^2.

TABLE I - SPECIMEN CHARACTERISTICS

Specimen name	Panel dimensions (m)	Reinforcement	Brick dimensions (cm)	Brick type	A_H/A_G
1PA,2PA,3PA,4PA,	1.11x1.11x0.12	non-reinforced	24x12x6	solid	0.00
5PA,6PA,7PA,8PA,9PA	0.51x0.52x0.12				
10PA,11PA,12PA,13PA	0.86x0.88x0.12				
1FB,2FB,3FB,4FB	1.11x1.11x0,12	non-reinforced	24x12x6		0.25
5FB,6FB,7FB,8FB,	0.51x0.53x0.12				0.21
9FB,10FB,11FB,12FB	0.86x0.88x0.12				0.21
1FC 8.6, 2FC 8.6	1.13x1.13x0.165	8Ø16	24x16.5x12		0.42
1FD 8.6, 2FD 8.6	1.12x1.12x0.165		24x16.5x18		

Test setup

The walls were tested by a diagonal compressive force. As it can be seen in figure 2, the panels were set diagonally in a MTS Load Frame 211.21, and were loaded between two steel V-support of two different dimensions (I and II) (Fig. 3). The V-support set on a cylindrical hinge specially designed (Fig. 4).

Tests were carried out controlling the diagonal relative displacement of the two hinges. Most panels were tested with a costant relative velocity of 3.75×10^{-3} mm/sec (Fig. 5 c); for three specimens the displacement history shown in Fig. 5, a or b was used.

Measurements

The load cell set in the MTS apparatus, was used to measure the load.

The relative displacement between top and bottom hinges was measured by a linear voltage displacement transducer that was integral with the piston of the hydraulic actuator. The horizontal relative displacement between lateral corners was measured by a W 50 Höttinger transducer. Moreover principal strains in several points of the two diagonals of the panel was measured by inductive extensometers, Höttinger D 1 (Fig. 6). All measured values were recorded on a graphic data recorder.

The diagonal load and the vertical displacement recorded continuosly by an xy recorder were used to check the progression of the tests.

TEST RESULTS AND DISCUSSION

A summary of the results of the experimental investigation is given in table II.

This table shows the ultimate (maximun) value of the load P_u, the maximun vertical displacement Δ_{yu} (Fig. 7), the maximun vertical strain at the center of the panel ε_{yu} (measured between two points, 200 mm spaced) the maximun horizontal extension Δx (measured as upon), the particular load story (Fig. 5), the V-support type (Fig. 3), the failure mode. Moreover in the table are shown the ideal maximun main tensile stress $\sigma_{tu} = 0,52\ P_u/lt$ calculated according to Frocht (Ref. 4) for elastic isotropic material (in which t is the thickness of the panels), the ultimate average shear stress $\tau_{au} = P\ /\sqrt{2}lt$ and the ultimate average compression stress under the V-support $\sigma_{cu} = P\ /at$ (for a, see Fig. 3). Finally in the last four columns the diagonal stiffness k and the effective width w of the equivalent strut, the w/d ratio and its mean for every specimen set are shown.

Failure

The tests have shown that for diagonal compression, four modes of failure are possible:

a) crushing of the loaded corners (that usually happened when employing type I of V-supports and hollow bricks (Fig. 8);
b) tension crack along the loaded diagonal (Fig. 9);
c) sliding along a mortar joint (Fig. 10);
d) combined sliding and diagonal crack (Fig. 11).

TABLE II - SUMMARY OF EXPERIMENTAL RESULTS

Specimen	Maximum load P_u (kN)	Loaded diagonal		Horizontal diagonal Δ_{xu} (mm)	Load story	V-support type	Failure mode	Stresses			$k=\frac{P_u}{\Delta_{yu}t}$ (N/mm^2)	$w=\frac{P_u d}{E_m \Delta_{yu} t}$ (mm)	$\frac{w}{d}$	$\left(\frac{w}{d}\right)_a$
		Δ_{yu} (mm)	ε_{yu} ($\times 10^{-3}$)					σ_{tu} (N/mm^2)	τ_{au} (N/mm^2)	σ_{cu} (N/mm^2)				
1PA	180	-	0.29	0.07	b	I	combined	0.70	0.95	15.00	-	-	-	0.053
2PA	152	-	0.23	0.05	a	I	sliding	0.59	0.81	12.67	-	-	-	
3PA	246	-	0.50	0.19	c	I	crushing	0.96	1.30	20.50	-	-	-	
4PA	279	4.02	0.49	0.18	a	II	diagonal	1.09	1.48	9.69	578	83.30	0.053	
5PA	116	1.29	0.86	0.14	a	II	diagonal	0.97	1.31	4.03	749	50.52	0.069	0.057
6PA	116	1.70	0.51	0.09	a	II	sliding	0.97	1.31	4.03	569	38.34	0.052	
7PA	120	2.14	0.97	0.08	a	II	sliding	1.00	1.36	4.17	467	31.51	0.043	
8PA	74	1.11	0.96	0.20	a	II	sliding	0.61	0.84	2.57	556	37.46	0.051	
9PA	182	1.99	0.67	0.10	a	II	diagonal	1.52	2.06	6.32	762	51.39	0.070	
10PA	145	1.90	-	0.08	a	II	combined	0.72	0.98	5.03	636	71.76	0.058	0.063
11PA	249	3.10	-	0.14	a	II	combined	1.24	1.69	8.65	669	75.53	0.061	
12PA	181	1.77	-	0.09	a	II	diagonal	0.90	1.23	6.28	852	96.16	0.078	
13PA	191	2.65	-	0.10	a	II	diagonal	0.95	1.29	6.63	601	67.78	0.055	
1FB	158	-	0.28	0.09	c	I	crushing	0.62	0.84	13.17	-	-	-	0.062
2FB	350	3.22	0.64	0.17	a	II	diagonal	1.37	1.86	12.15	906	125.80	0.080	
3FB	317	4.57	0.91	0.13	a	II	diagonal	1.24	1.68	11.00	578	80.30	0.051	
4FB	376	5.06	-	-	a	II	diagonal	1.47	2.00	13.06	619	86.00	0.055	
5FB	139	2.02	0.67	0.10	a	II	diagonal	1.16	1.58	4.83	573	37.30	0.051	0.050
6FB	186	4.16	0.82	0.12	a	II	diagonal	1.55	2.11	6.46	373	24.23	0.033	
7FB	166	3.25	0.61	0.07	a	II	diagonal	1.38	1.88	5.76	426	27.68	0.038	
8FB	119	1.12	0.81	0.14	a	II	combined	0.99	1.35	4.13	885	57.59	0.078	
9FB	211	2.61	0.46	0.11	a	II	diagonal	1.05	1.43	7.33	674	73.33	0.060	0.055
10FB	253	3.09	0.58	0.15	a	II	diagonal	1.26	1.71	8.78	682	74.26	0.060	
11FB	214	3.73	0.36	0.08	a	II	combined	1.06	1.45	7.43	478	52.04	0.042	
12FB	208	2.66	0.49	0.12	a	II	diagonal	1.04	1.41	7.22	652	70.92	0.058	
1FC 8.6	314	3.80	0.56	0.11	a	II	crushing	0.87	1.19	7.93	501	86.60	0.055	0.057
2FC 8.6	460	5.09	0.76	0.21	a	II	diagonal	1.28	1.74	11.61	548	94.60	0.059	
1FD 8.6	374	6.24	0.42	0.13	a	II	crushing	1.04	1.43	9.44	363	62.50	0.039	0.041
2FD 8.6	308	4.59	-	-	a	II	crushing	0.87	1.18	7.78	407	70.00	0.044	

Typical behaviour

The behaviour of the panels is quasi-linear until the failure; figure 11 shows the relation between the applied load and the displacement of the loaded corner.

The collapse is always brittle-type, i.e. the load has a sudden decrease (remember that the tests were displacement-controlled).

Walls with steel reinforcement, show a behaviour similar to that of non-reinforced panels;in particular, reinforcement does not change the cracking load.

After first cracking, load abruptly descreases as in non-reinforced panels, but the reinforcement prevents the separation of the various pieces.

Figures 12 and 13 show the relationship between diagonal load and internal strains.

Deformations

In figure 13 strains measured from three different specimens, at load level of 82.00 kN , are shown. In the AO and BC sections, the diagrams of ε_y measured at the centre and at 1/4 of the loaded diagonal are reported. The diagrams of ε_x and ε_y strains for the OD section are also reported.

The dashed parts of diagram indicate sections in which experimental measurements were not sufficient to allow a sure interpretation.

The trend of these diagrams, obtained from very reliable experimental measuraments agrees qualitatively with the results, in terms of strains and stresses, of theoretic analysis carried out by other Authors (Ref. 5) on panels with isotropic material structure.

The maximun value of strains at the centre of the panel, varies between 0.50×10^{-3} and 0.90×10^{-3} approx; smaller values were found when the failure happened with crushing of the loaded corners.

Ultimate load

Tests, carried out for three different size specimens , pointed out that the failure load P_u is proportional to the length of the side of the panel; in figure 14 is reported the trend of P_u for two different type of brick (solid and hollow).

Stiffness and equivalent strut

In every test the diagonal compression stiffness in the panel (at level of P_u), was directly determined; because of the linearity between P and Δ_y, the stiffness does not depend significantly on the load. The value of the specific stiffness k does not seem to depend significantly either on the dimensions of the panel or on small differences in the hole-ratio.

To calculate infilled frames, it seems possible to assume k = 600 N/mm approximately.

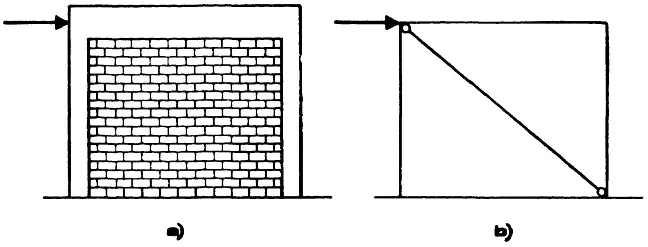
The width of the equivalent strut was calculated with the formula:

$$w = \frac{P_u d}{E_m \Delta_{yu} t} \qquad (3)$$

in which E_m is the Joung's modulus along the holes, determined by tests of uniaxial compression on three brick small pillars. The mean values $(w/d)_a$ pertinent to every group of specimens , vary from 0,041 and 0,063; therefore in square panels, the width of the equivalent strut may be considered, on the average, equal to one eighteenth of the diagonal length; this result may seem in contrast with that one obtained by other Authors (some of which tested with homogeneous and isotropic materials) who give, for $(w/d)_a$, values equal to 1/10 of the length of the diagonal.

This difference is due for the most part, to the fact that the diagonal load on the panels, is applied along a direction at $\pi/4$ with that one along which E_m has been calculated; along the load direction the modulus E_L is considerably smaller.

Because of the difficulty to determine experimentally E_L , while E_m can be determined by easy standard test, we preferred to give formulas in which the latter modulus appears.



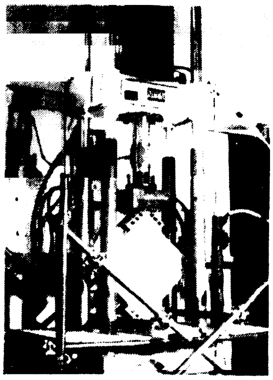


Fig.1: Infilled frame (a) and equivalent structure (b). Fig.2: Model test. Panel in MTS load frame.

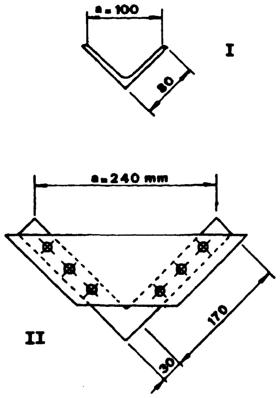


Fig.3 : Steel V-support on the corner of the panel.

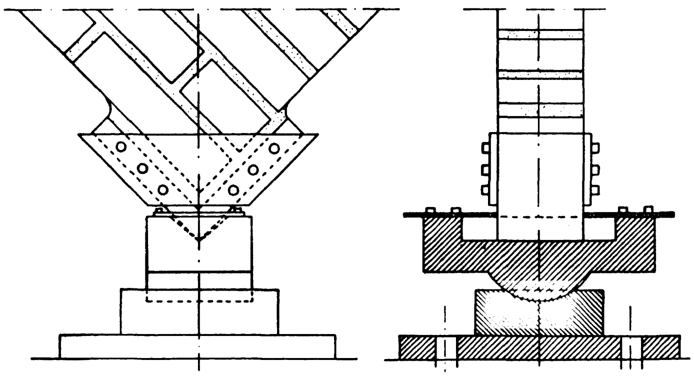
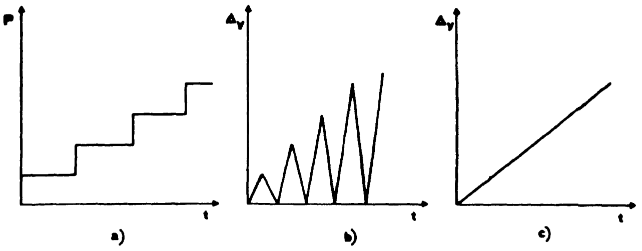


Fig.4: Cilindrical hinge to support the walls.



a) 1PA panel

b) 3PA and 1PA panels

c) All other panels

Fig.5: Load and displacement histories.

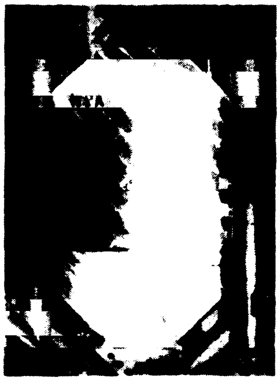


Fig. 6 : Inductive extensometers.

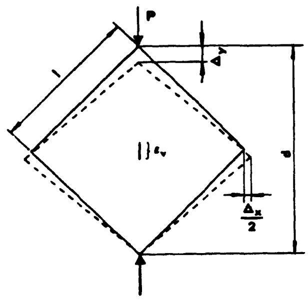


Fig.7: Indication of the displacements.

Fig.8: Crushing failure.

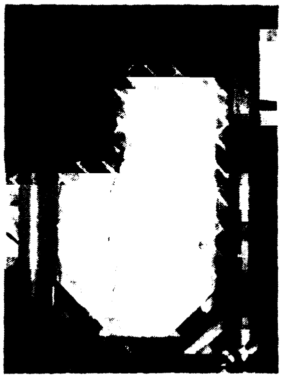


Fig.9: Crack along the loaded diagonal.

Fig.10: Sliding failure.

Fig.11: Combined failure.

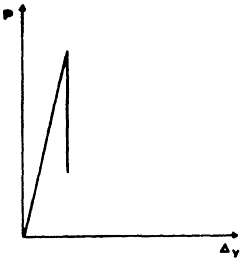


Fig.12: Load P versus displacement Δ_y(typical plot).

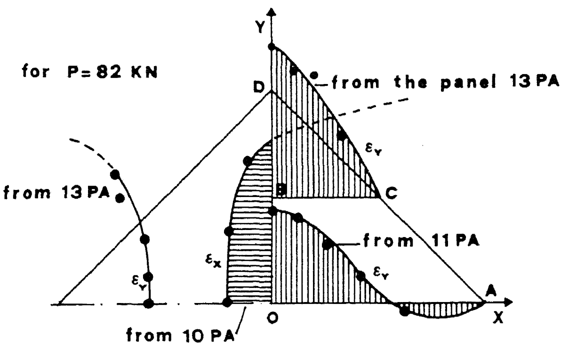


Fig.13: Internal strains along diagonals.

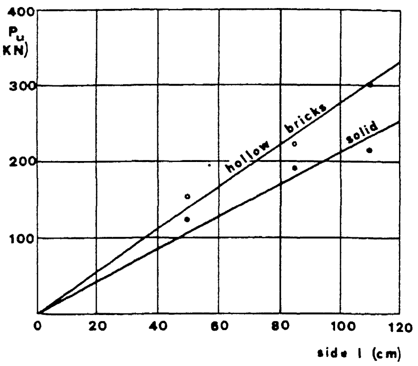


Fig.14: Relationship between the ultimate load and the side length of the panel.

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