Test on Single Leaf Brickwork Panels and Walls with Bed Joint Reinforcement

C. Southcombe BSc.(Eng)., MSc.(Eng).,C.Eng,M.I.C.E.,
Senior Lecturer, Civil Engineering Department,
Plymouth Polytechnic

G.D. Johnson B.Eng(Tech)., M.I.Struct.E., C.Eng.,
Research Engineer, Structural Clay Products Ltd., Hertford.

(England)

1.0 SYNOPSIS

The flexural resistance of unreinforced masonry panels is can be shown to be a function of the water absorption characteristics of the units[1]. Bricks of higher water absorption generally result in lower strength panels. Bed joint reinforcement in the form of high tensile steel wires can be used to enhance the strength but in this case serviceability criteria may be of significance. It is also noted[2] that the higher the water absorption of the bricks in a reinforced panel the greater the deflection at service load conditions. Lateral load tests were carried out on panels made of bricks of high water absorptions. Small panel tests and large wall tests compared flexural behaviour of unreinforced and reinforced specimens under lateral loads. It is found that reinforced and unreinforced panels exhibit first cracks at similar loads. This load is the limit state of strength for the unreinforced panels in the walls but for the reinforced walls and panels this is the serviceability condition. Thus given that deflections are not excessive in the reinforced case it is suggested that reinforced panels with small areas of bed joint reinforcement may be designed as unreinforced panels but with a reduced materials partial safety factor covering characteristic flexural strengths at service loads. This would then allow greater spans to be achieved in practice.

2.0 INTRODUCTION

The lateral load tests were carried out on small unreinforced panels over a 1m span and reinforced panels over a 2m span. In both cases they were 290mm in width (4 bricks). Two walls were also tested, one reinforced one unreinforced, with high tensile wires in every bed joint. The walls were 4.4m long by 2.6m high. The wire reinforcement provided increased moment of resistance and prevented the typical brittle failure of unreinforced panels. Steel strains were monitored in both the small panels and the test wall but considerable differences in behaviour were apparent. Steel strains in the small panels approached the plastic limit of the steel (3000 microstrain) whereas the strains in the walls at the maximum load tested was only of the order of 500 microstrain. Calculations show that moments, stresses and strains in the reinforced wall should give steel strains of the order of 2000 microstrain. The discrepancy between this figure and the 500 microstrain recorded may be due to the positioning of the strain gauges relative to the crack pattern in the wall. Tests to measure the stress profile in this type

of reinforcement either side of a crack will be carried out. A feature of the small reinforced panel tests was that even at "failure" the panels remained intact due to the wire near the compression zone not breaking. In the reinforced wall none of the reinforcement failed so that even at quite large deflections and when the wall would have been deemed to be unserviceable integrity was maintained. Calculation of EI values from tests on small reinforced panels suggest that these figures may be of use in determining the deflection of large walls under lateral loads.

3.0 SMALL PANEL TESTS

3.1 Materials

3.1.1 Bricks

The bricks used were three hole perforated of medium compressive strength, complying with BS 3921[3]. The volume of holes was less than 25%, hence brick is classed as solid. The mean compressive strength was 33.8 N/mm^2 and the mean water absorption was 18.2%. Suction rate was 1.49 $Kg/m^2/min$.

3.1.2 Mortar

The mortar mix was a 1:¼:3 cement:lime:sand mix (by volume) using Ordinary Portland cement to BS12[4] and a zone 2 crushed limestone sand to BS1200[5]. A water/cement ratio was not specified, the bricklayer being left to produce a workable mix. The average 14 day strength was 24.9 N/mm^2 using 100mm cubes, cured in water.

3.1.3 Reinforcement

The bed joint reinforcement consists of two parallel wires each of 9.48m in cross-sectional area. The wires are a flattened section approximately 5mm x 2mm. The steel conforms to BS4482[6] for hard drawn steel wire.

The parallel wires are at 55mm c/c, tied by 2.5mm diameter wires at approximately 300mm c/c, to form a ladder type reinforcement. Minimum breaking stress in the manufacturers' literature is 570 N/mm^2. Three lengths of wire were subjected to a tensile test. The average breaking stress, ignoring any reduction in cross-sectional area was 660 N/mm^2. There is no plasticity in this wire. It has a "brittle" type failure.

3.1.4 Test Specimen

The panels were built as stretcher bonded walls. The unreinforced panels were of 1m span and the reinforced panels of 2m span. Four point loading was used for both types of panel; loads were applied at third points.

3.2 Test Details

3.2.1 Apparatus

The loads were applied by a hydraulic test machine. Loads were transmitted to the brick through steel plates embedded in sand:cement:mortar.

3.2.2 Measurements

Deflection of the panels at the mid-point was measured by a dial guage. Steel strains were measured using electrical resistance strain guages positioned as shown in Fig.(1).

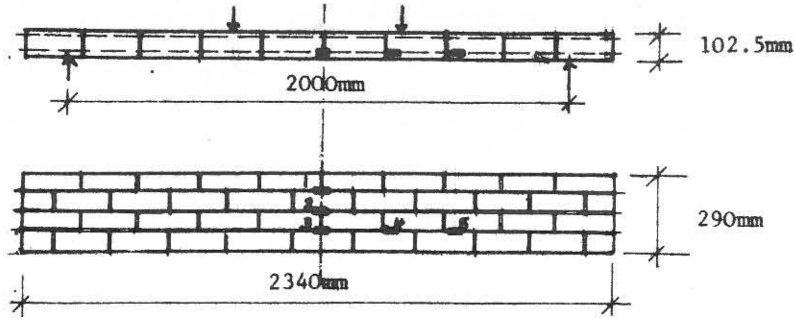

Fig. (1) Position of strain guages

3.3 Test Results

3.3.1 Unreinforced Panels

These were tested to destruction, loads being applied in increments of 0.25kN. Cracks were not visible before failure which was very sudden, the panels breaking across the central area of the beam. Recorded deflections are shown in Fig.(2).

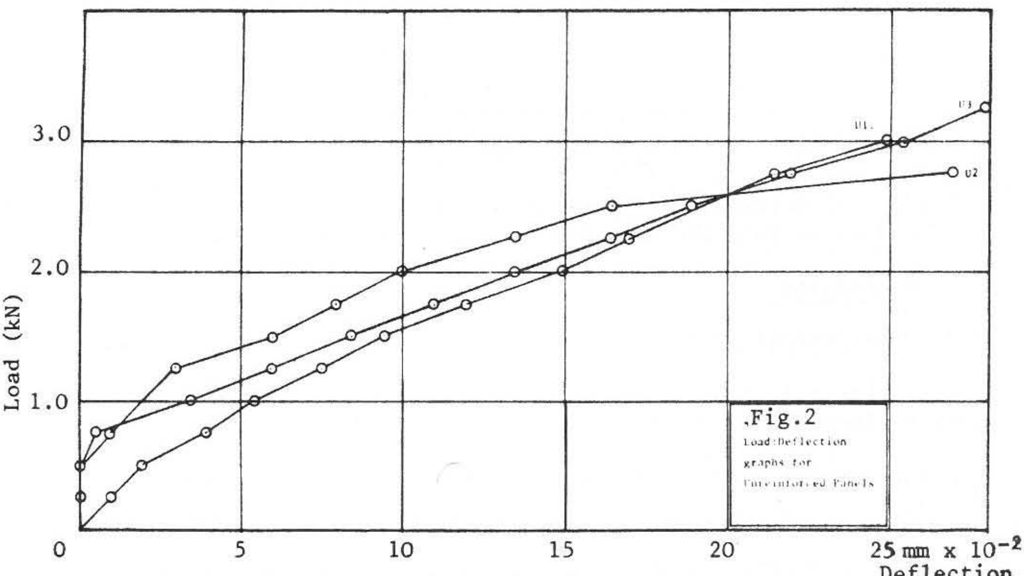

Fig.2 Load Deflection Graph for Unreinforced Panels.

Table 1 Unreinforced Panels

Unreinforced Panel No.	Characteristic Moment of Resistance to BS 5628 Part 1(kN.m*) (1)	Applied B.M. at failure (incl. self weight)(kN.m) (2)	$^2/_1$
1	0.560	0.57	1.02
2	0.560	0.57	1.02
3	0.560	0.65	1.16

* i.e. γ mm = 1.0

3.3.2 Reinforced panels

The reinforced panels were tested to destruction, loads being applied in 0.25 kN increments up to the load at which the initial mortar/brick bond failure was observed. Beam deflections and steel strains are shown in Figs. (3a,b,c). These graphs clearly show the loads at which the initial bond failure occurred.

Table 2 Reinforced Panels

Reinforced Panel No.	Design M.O.R. to BS 5628 Part 2† kNm	Simplified Design M.O.R.* kNm	Applied BM at Failure (incl. self weight) kNm
1	1.06	1.49	1.93
2	1.06	1.49	1.86
3	1.06	1.49	1.86
			mean = 1.88 kNm

* f_y = 660 N/mm^2 † f_s = 485 N/mm^2

The bottom reinforcement broke in all cases but the top steel remained intact and continued to hold the sections of the panel together.

3.4 Discussion

3.4.1 Unreinforced panels

The characteristic moment of resistance of the unreinforced panel was evaluated from:

$M = f_{kx} \cdot Z$ (Cl 36.4.3, BS5628[7] (omitting partial safety factor))

f_{kx} was taken as 1.1 N/mm^2, and $Z = 0.51 \times 10^6$mm^3

Table 1 compares this with experimental results.

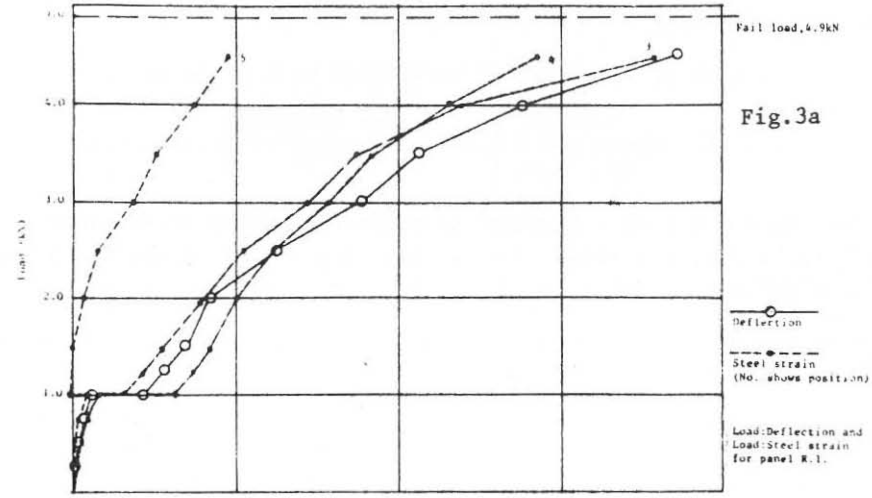

Fig.3a

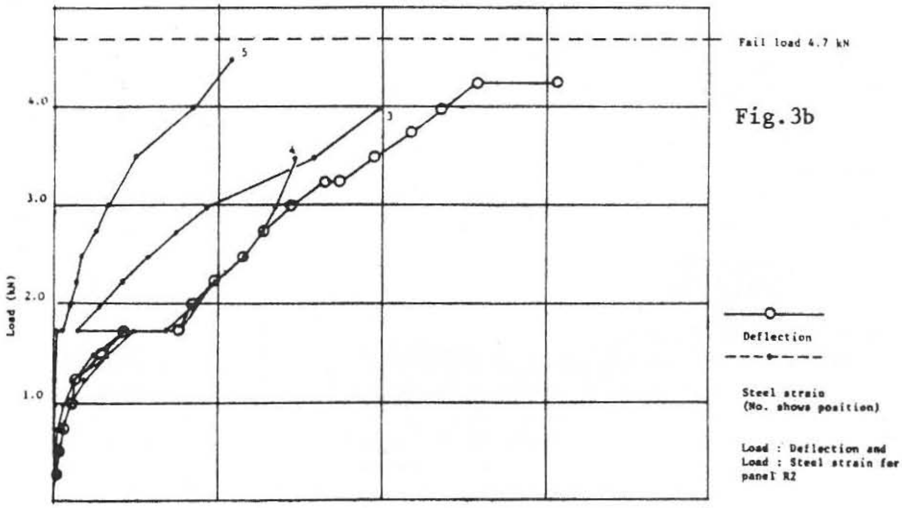

Fig.3b

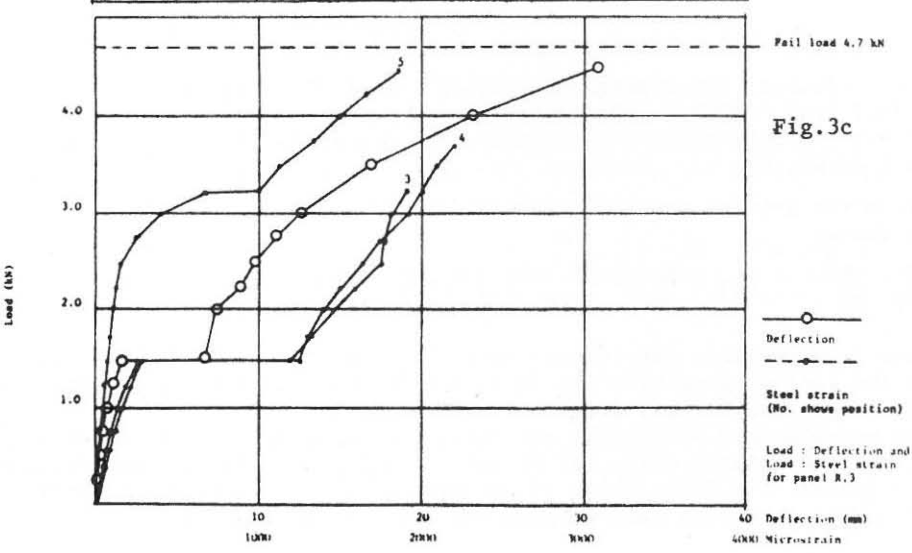

Fig.3c

3.4.2 Reinforced panels

The flexural strength of the panel in accordance with Draft BS5628[8] is:-

$$M_d = \frac{A_s f_y z}{\gamma ms} \quad .... (1) \quad \text{where } z = d\left[1 - \left(\frac{0.5\, A_s f_y \gamma mm}{b d f_v \gamma ms}\right)\right] \quad .... (2)$$

In this case $A_s = 3 \times 9.48 = 28.44\text{mm}^2$ (considering bottom strand only), $f_y = 485\ \text{N/mm}^2$, $\gamma_{mm} = 2.8$, $b = 290\text{mm}$, $d = 78.75\text{mm}$, $\gamma_{ms} = 1.15$. Hence $M_d = 0.896$ kN.m If $f_y = 660\ \text{N/mm}^2$ and $\gamma_{ms} = 1.0$ then $M_u = 1.37$ kN.m. The above analysis ignores the top steel.

A simplified analysis of the reinforced section, based on a rectangular stress block [2] but which considers top and bottom steel gives:-

$$M_d = \frac{A_s\, f_y}{\gamma_{ms}}\left(\frac{d_1^{\,2}}{d_2} + d_2\right) \quad ....(3)$$

Taking $A_s = 28.44\text{mm}^2$, $f_y = 485\ \text{N/mm}^2$ (3 bed joints), $\gamma_{ms} = 1.15$, $d_1 \doteq 24\text{mm}$, $d_2 = 79\text{mm}$,

then $M_d = 1.03$ kN.m $\equiv$ 4.58 kN.m/m. Using $f_y = 660\ \text{N/mm}^2$ and $\gamma_{ms} = 1.0$ then $M_u = 1.62$ kN.m $\equiv$ 7.20 kN.m/m.

Table 3 Comparison of results (mean of three each)

	Unreinforced (1m span)	Reinforced (2m span)
Experimental BM at failure	0.60 kN·m	1.88 kN·m (Equivalend to 6.48 kNm/m)
F.O.S. $= \frac{\text{Expt.BM}}{\text{Design BM}}$	3.33	2.11
Deflection at cracking	0.29mm	1.82mm
Max. deflection	0.29mm	30mm
Moment at first crack	0.60 kN·m (inc.own weight)	0.75 kN·m (inc. own weight)

3.4.3 Deflections of reinforced panels

From Figs. (3a,b,c) the average deflection (Δ) at the service load. (Service load = fail load $\div$ (3.1 x 1.2) where 3.1 = a materials partial safety factor and 1.2 = the load partial safety factor) is 3.2 mm and the average service load is 1.28 kN.(W).

Thus, from the general expression for deflection of a beam under four point loading where:

$$\Delta = \frac{23}{1296} \cdot \frac{WL^3}{EI} \quad ....(4)$$

Hence $EI = 56.8 \times 10^6\ \text{kN}\cdot\text{mm}^2$
and $E = 2182\ \text{N/mm}^2$

Test data from previous experiments (which did not monitor steel strains) yielded the following results for 2m span reinforced panels tested under four point loading. (see Table 5 over)

The best correlations between EI and the brick parameters shown in Table 5 are $EI = (112.17 - 20.02(\ln c)) \times 10^6\ \text{kN}\cdot\text{mm}^2$ ....(5) (correlation coefficient=0.93) and $EI = (67.91 - 11.34(\ln S)) \times 10^6\ \text{kN}\cdot\text{mm}^2$ ....(6) (correlation coefficent = 0.85) where c is cold water absorption and S is suction rate

Table 5 EI Values at Service Loads

Brick	Service Load (kN)	Deflection at Service Load (mm)	EI ($kN \cdot mm^2$)	S ($kg/m^2/s$)	c (%)
A1	1.56	2.8	79.1×10^6	0.829	7.3
B1	1.28	3.0	60.6×10^6	2.307	16.6
B2	1.13	2.0	80.2×10^6	0.218	4.1
C1	1.22	2.9	59.7×10^6	1.609	14.1
C2	1.22	2.0	86.6×10^6	0.410	4.4
D1	1.31	2.0	93.0×10^6	0.098	2.8
B3	1.35	3.0	63.9×10^6	2.103	7.0
E1	1.35	3.9	49.1×10^6	1.378	16.4
*F1	1.28	3.2	56.8×10^6	1.49	18.2

(* Brick described in 3.3)

Alternatively the brick types could be grouped into three convenient categories i.e. <7% , 7%-12% and >12% water absorptions so that the mean EI value for each group becomes:

Table 6 Mean EI values for water absorption ranges

Water Abs. Range (%)	Less than 7	7 - 12	Greater than 12
Mean EI($kN \cdot mm^2$)	86.60×10^6	71.50×10^6	64.05×10^6

4.0 CONCLUSIONS (Small Panels)

The provision of bed joint reinforcement increases the load carrying capacity but causes comparatively larger deflections before the onset of cracking. Reinforcement in every bed joint provides a system which may be over designed but this can be remedied by providing reinforcement in alternate joints if required. EI values calculated from the observed deflections can be correlated with some brick properties. Strains in the steel near failure are of the order of 3000 microstrain.

5.0 FULL SCALE TESTS ON WALL PANELS

5.1 Introduction

In order to compare the behaviour of larger walls with the results from the small scale tests two brickwork panels were tested for SCP Limited by BCRA* One wall was unreinforced and the other wall was reinforced with one layer of reinforcement in each bed joint but otherwise the panels were identical.

5.2 Experimental Walls

Details of the walls and test frame are shown. Fig. (4a-4d). The mortar was 1:¼:3 mix as for the small scale tests and the bricks, being of a high water absorption, were wetted before laying.

5.3 Loading and Instrumentation

The bottom of the wall contained a bitumenous dpc two courses above ground level to simulate walls in practice. The wall would thus be considered as being simply supported on three edges with the top edge free at the ultimate load condition but, at service loads, it may be valid to consider the wall as fixed at the base.

* *British Ceramic Research Association, Stoke-on-Trent*

The load was applied by means of air bags and deflections measured by transducers connected directly to data-logger and computer. Each transducer has a unique reference letter and the computer print out displays a graph in terms of continuous plot of each letter. In addition some of the reinforcement was strain guaged using electrical resistance guages at the locations indicated. The deflection patterns for the transducers of the unreinforced and reinforced walls are shown. Figs. (5) and (6).

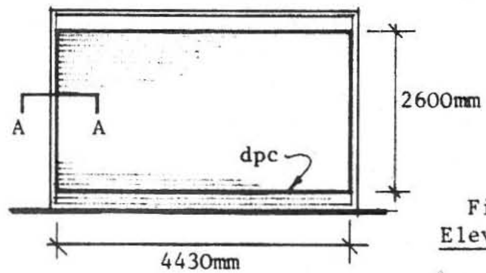

Fig.4a.
Elevation

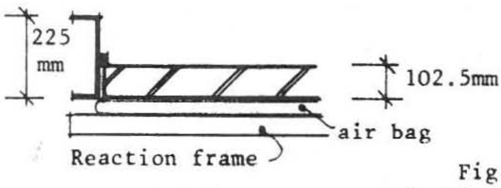

Fig.4b.
Section A-A

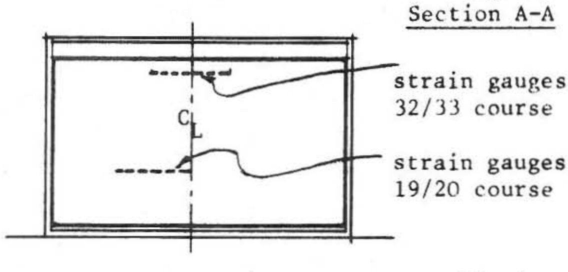

Fig.4c.
Position of strain gauges in wall.

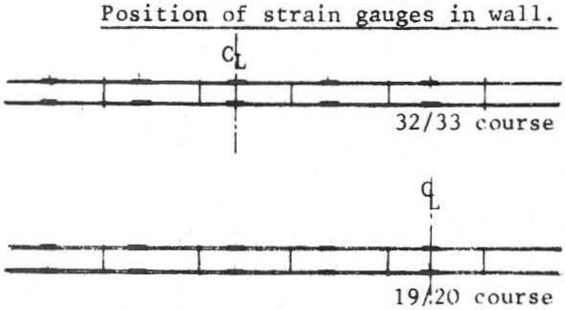

Fig.4d.

Location of strain gauges.

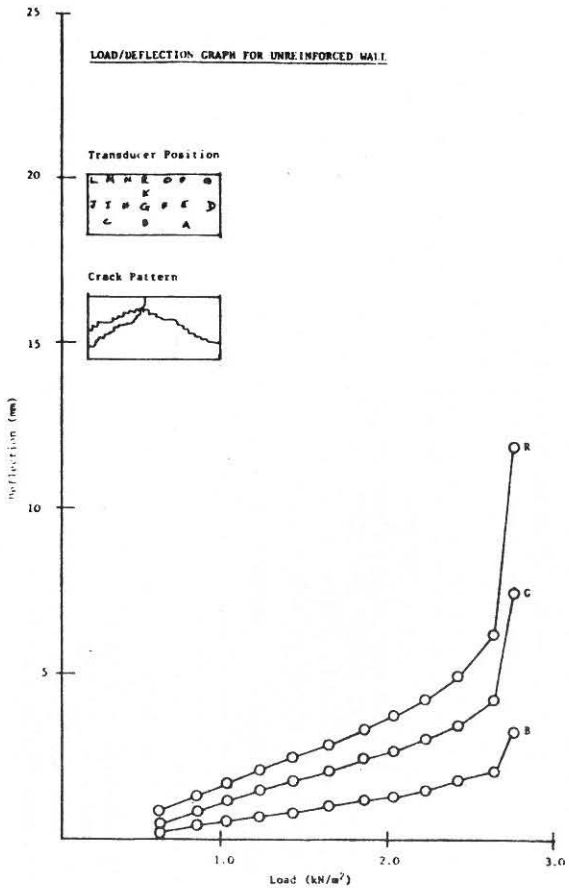
LOAD/DEFLECTION GRAPH FOR UNREINFORCED WALL
Transducer Position
Crack Pattern
Deflection (mm)
Load (kN/m²)
25
20
15
10
5
1.0
2.0
3.0
R
G
B

Fig.5

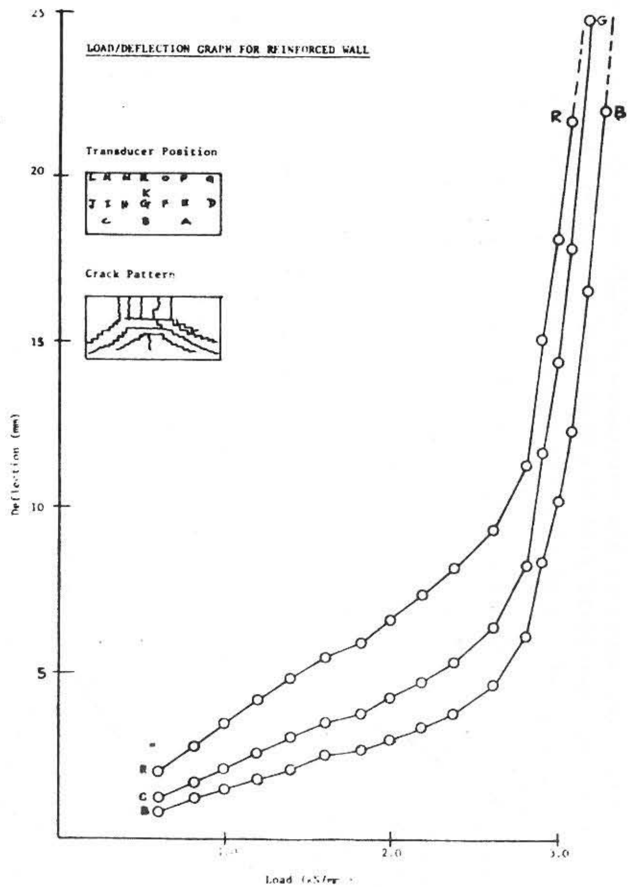
LOAD/DEFLECTION GRAPH FOR REINFORCED WALL
Transducer Position
Crack Pattern
Deflection (mm)
25
20
15
10
5
2.0
3.0
R
G
B

Fig.6

5.4 Discussion

As expected, reinforced walls exhibit greater lateral loadbearing capacity than unreinforced walls. However, results from the small panel tests indicated that deflections could be large near ultimate load and this was so in the large panel tests. Near the centre of the walls deflections in the reinforced wall were larger than those in the unreinforced walls at a given load, although up to the cracking load these differences were not significant.

At the ultimate load of 4.4 kN/m^2 in the reinforced wall the moment at the top is given by:

$M = 0.060\ qL^2$ ....(7) (assuming it is an elastic plate simply supported on three edges)

$\therefore M = 0.060 \times 4.4 \times 4.43^2 = 5.18$ kN·m /m width

But from eq(3)

$$f_s = \frac{M}{A_s\ (d_1^2/d_2 + d_2)}$$

where d_1 = 24mm, d_2 = 79mm, and A_s = Area of steel /m width

$= 10 \times \frac{1000}{75}\ mm^2 = 133 mm^2$/m width

$f_s = 451\ N/mm^2$

$$\therefore \text{steel strains} = \frac{f_s}{f_y} \times \varepsilon_{ult} = \frac{451}{660} \times 3000 = 2052 \text{ microstrain}$$

However, steel strains at the locations instrumented indicate only modest deformations, of the order of 500 microstrain. The reason for this is thought to be due to the position of the steel strain gauges relative to the cracks induced in the wall. Unless a gauge was positioned at or very near to a crack large strains will not be recorded because the full bond length of the wires will be small. Therefore, remote from the crack (remote > 50mm) steel strains will be small. At a crack the steel strains must be large ($\simeq 2000\mu$ strain) to sustain the moment. Between cracks, which usually occur at a line of perpend joints, the brick unit resists the bending moment.

5.5 Deflections

Figs. (5) and (6) show graphs of deflections/load for both walls at critical positions at the centre of the panel. The water absorption of the bricks used was 22.3% and the suction rate was 2.86 kg/m^2/min. Accordingly, using eq(5) and (6) the predicted EI values are 50.0×10^6 kN·mm^2 and 56.0×10^6kN·mm^2 respectively. Also, from Table 6 the mean EI value for the 12% + range of water absorptions is 64.05×10^6kN·mm^2. "I" of the specimens from which these results were obtained was $26 \times 10^6 mm^4$ so that the predicted elastic modulii are 1921 N/mm^2 , 2152 N/mm^2 and 2461 N/mm^2 respectively. The deflection (Δ) at the top of an elastic plate (free top edge, sides simply supported and bottom fixed*) is given by the standard formulae

$$\Delta = \frac{\alpha \cdot q\ h^4}{E \cdot t^3} \quad ....(9)$$

where α = coefficient depending on ht/length ratio
q = lateral pressure
h = height of panel
t = thickness of panel
and E = elastic modulus

height of panel(h) = 2.6m and length = 4.43m $\therefore \alpha = 0.38$. q (service load)= 1.2 kN/m^2 and t = 102.5mm. Consequently the three values of E predict Δ to be 10.0mm , 9.0mm and 7.9mm respectively. Results from the reinforced wall test give a maximum deflection at the top of the wall of approximately 6mm.

* *the assumed support conditions at service load*

5.6 Conclusions and Design Suggestions

Bed joint reinforcement of the type tested will improve the lateral load-bearing resistance of single leaf brickwork walls. The wall tests described above were on brickwork which had a low flexural strength due to the high water absorption of the bricks. The ratio of unreinforced to reinforced loads was 2.8:4.4 = 1:1.57. Tests on small panels indicate that the ratio for lower water absorption bricks will be less - approaching unity. The advantage of using reinforcement for walls subject to lateral loads - wind loads usually - is that material factors of safety may be reduced thus allowing greater design loads. For example, the minimum material partial factor of safety for brickwork is 2.5 (unreinforced flexural strength). For a reinforced section the appropriate material factor of safety is 1.15 for steel. It has been shown that deflections at critical points in the reinforced wall are of the same order at the cracking load of the unreinforced wall. Thus the service load could be calculated on the basis of the unreinforced flexural strength but with a lower materials partial safety factor. It is therefore suggested that a reinforced wall of this type can be designed in the following manner:

(i) Establish ultimate moment of resistance of unreinforced wall based on the flexural strength of the masonry

(ii) (i) becomes the basis of the service load condition for the reinforced wall but is factored by a smaller materials partial safety factor because steel reinforcement prevents collapse failure. It is suggested that the partial safety factor for masonry may be halved.

(iii) Check full design load (service load x factor) against moment of resistance of reinforced section based on steel strength

(iv) Check deflection at service load using EI obtained from small scale tests.

Prediction of deflection using elastic plate theory yields results somewhat higher than that observed in the wall tests. The deviation depends on the predictive method used. However, given that only one wall test was done, results are sufficiently encouraging to stimulate further tests to confirm the method.

6.0 SUMMARY

Small panels of unreinforced brickwork and reinforced brickwork with steel wires in each bedjoint were tested under lateral loads. Some tests included examination of steel strains. The results from these tests were compared with large wall tests. It was found that, in small (2m span panels with 4 point loading) that steel strains at and near mid-span were of the order of 3000μ strain - approaching the breaking strain of the steel. In the large walls with a uniformly distributed load the strains observed were only ≈ 500μ strain It is thought that this is due to the strain guages being remote (50mm?) from cracks in the wall. Further tests will be carried out to test this hypothesis The inclusion of bed joint reinforcement improves the flexural strength of a wall but reduces flexural stiffness. This reduction in stiffness is not serious up to cracking load (this load is similar for reinforced and unreinforced panels). It is suggested that calculation of EI values from small scale tests will allow prediction of deflection in larger panels. A method of design of large reinforced panels has been proposed whereby a reinforced panel is designed under service load conditions as an unreinforced panel but with reduced masonry material partial safety factors. This allows larger spans or pressure to be accommodated in design. Collapse of the wall under exceptional loads (greater than the cracking load) is prevented by the bed joint reinforcement. Results suggest that deflections of walls at service loads may be predicted from EI values derived from small scale panel tests.

7.0 NOTATION

b	Breadth of section
c	Cold water absorption
d	Effective depth
d_1	Effective depth to top steel
d_2	Effective depth to bottom steel
f_s	Stress in steel
f_y	Yield stress of steel
f_{kx}	Characteristic flexural strength of masonry
h	Height of wall
q	Pressure
t	Thickness of wall
z	Lever arm
α	Coefficient
γ_{mm}	Masonry partial safety factor
γ_{ms}	Steel partial safety factor
ε_{ult}	Strain in steel at failure
A_s	Area of steel
E	Elastic modulus
I	2nd Moment of area
L	Length of wall
M	Moment
M_d	Design moment
M_u	Ultimate moment
S	Suction rate
W	Load
Z	Section modulus
Δ	Deflection

8.0 ACKNOWLEDGEMENT

This work was sponsored by Structural Clay Products Limited and is published by permission of the Directors of Structural Clay Products Limited.

9.0 REFERENCES

(1) HASLETINE, B.A., WEST,H.W.H., "Design of laterally loaded wall panels: Part 2", B.Ceram.R.A., Technical Note No. 248, 1976

(2) JOHNSON,G.D, "Lateral load tests on single leaf brickwork panels with light reinforcement", 2nd Canadian Masonry Symposium proceedings, 1980

(3) British Standard No. 3921 "Clay Bricks and Blocks"

(4) British Standard No. 12 (Part 2) "Portland Cement (ordinary and rapid hardening)"

(5) British Standard No. 1200 "Building sands from Natural sources"

(6) British Standard No. 4482 "Hard-drawn mild-steel wire for the reinforcement of concrete"

(7) British Standard No. 5628 "Code of Practice for the structural use of masonry Part 1, Unreinforced Masonry"

(8) British Standard No. 5628 "Code of Practice for the structural use of masonry, Part 2, Reinforced Masonry" (in draft)