Prestressed Masonry Diaphragm Walls

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SYNOPSIS

Two full sized prestressed brick diaphragm walls 7.62m high x 7.62m long were built in a laboratory at the University of Manchester Institute of Science and Technology, England. They were subjected to various levels of axial prestress and, at each level of prestress (including zero prestress), they were tested under uniform lateral load to a serviceability limit. This paper describes some of the results of these tests, proposes a theoretical method for the prediction of the behaviour of prestressed diaphragm walls and gives design guidance.

INTRODUCTION

The masonry diaphragm wall is simply a wide cavity wall with the two leaves bonded together by cross-ribs of masonry (see Figure 1). The leaves and cross-ribs act integrally to form a series of connected box sections. Such walls have been used successfully in tall single storey structures where they are subjected to relatively light axial loads from the roof of the building and to side loads from wind. The predominant design criterion is the tensile strength of the wall.

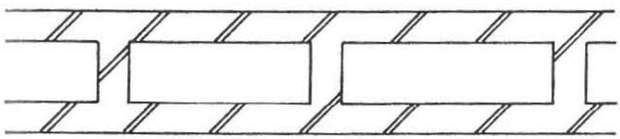

Fig. 1

Tensile stresses can be eliminated by compressive prestress and furthermore, if the prestress is increased sufficiently, much higher lateral loads than wind can be sustained without tensile fracture. Thus prestressing gives diaphragm walls the potential to act as retaining walls and the walls to bins and silos.

The section modulus, Z, of the diaphragm wall can be many times that of the cavity wall it replaces for only a small increase in cross-sectional area, A, (i.e. for only a small increase in number of bricks). The Z/A ratio of a diaphragm wall is therefore much higher than the cavity wall and it is this property which makes the diaphragm wall an efficient section for prestressing. Furthermore the open box section makes the application of prestressing by post-tensioning very simple - the tendons are simply placed in the box cavities at locations convenient from the point of view of stress.

Two full sized post-tensioned prestressed brick diaphragm walls 7.62m high x 7.62m long x 0.45m overall width were built in the laboratory for testing. They were subjected to various levels of axial prestress and, at each level of prestress (including zero

prestress), they were tested under uniform lateral load to a serviceability limit. The strength of this type of wall at collapse, or ultimate, has been studied by Williams and Phipps(3). The tests examined in detail the strength, deflection, strains and cracking behaviour of the walls at each load stage.

THE TESTS

Curtin and Sawko(2) tested six 3m high diaphragm walls built of half scale bricks. Three of the walls were tested under vertical loading and three under lateral loading. None of the walls were prestressed.

To eliminate any scaling effects the 7.62m x 7.62m walls in this test programme used full sized bricks. Each wall had five one brick wide diaphragms spaced at 14.5 times the leaf thickness (see Figure 2). Each box cavity formed by the diaphragms

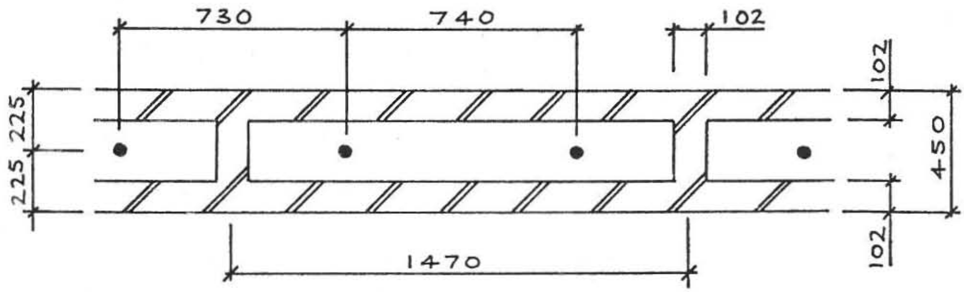

Fig. 2

contained two prestressing tendons at mid depth of the walls (see Figure 2). The walls were built side by side, on a common reinforced concrete foundation, with an air bag in between to apply lateral load. They were pinned together at the top so that they acted like propped cantilevers (see Figure 3).

One wall (Wall A) was built in quarter bond and the other (Wall B) was built in half bond. Approximately 8000 London Brick Company Class III fletton bricks were used in each wall. All the bricks were laid frog up, the frogs being filled with mortar. To distribute the prestress evenly over the walls the top 225mm of each wall was built in concrete blocks. The average brick strength was 11.0 N/mm^2 (frog unfilled) and 35.0 N/mm^2 (frog filled). The 28 day mortar strength was 7.3 N/mm^2 (nominal mix 1:1:5). The flexural tensile strength along the bed joints of standard brickwork test panels was 0.4 N/mm^2 average and 0.2 N/mm^2 characteristic. The compressive strength of 215mm x 215mm piers was 11.0 N/mm^2 and that of 215 x 102mm piers was 14.5 N/mm^2. All the control tests were carried out according to BS.5368.

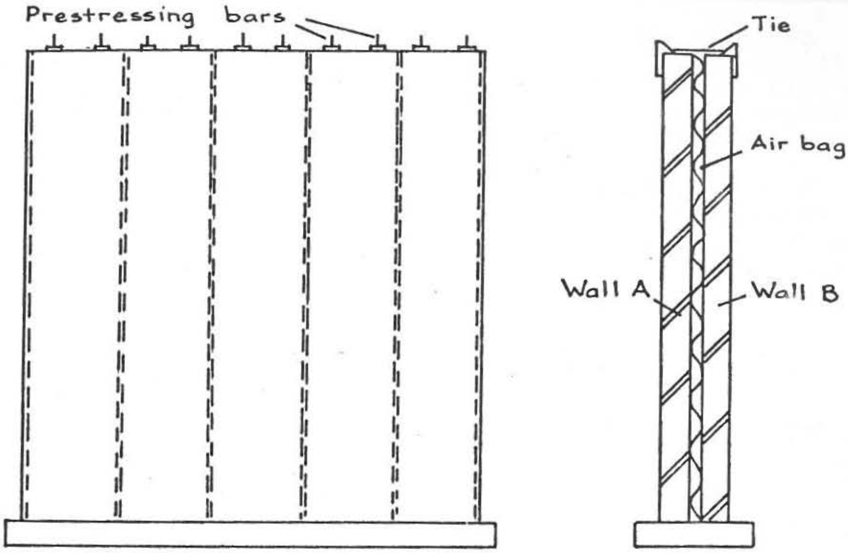

Fig. 3

There were 10-40mm diameter Macalloy prestressing bars per wall. For ease of construction a single bar was made up of five sections, each 1.5m long. This enabled the prestressing bars to be built up with each lift of brickwork. All the fixings - foundation anchorages, end plates at the top of the walls and couplers - were of the standard type supplied by the manufacturer. The last section of each bar was strain gauged and so allowed the load in the tendons to be determined throughout the tests. The stressing operation on the walls started on the centre pair of bars and worked outwards to the ends of the walls by using two jacks simultaneously.

Two rows of 14 strain gauges (gauge length 300mm) were placed on the exposed face of each wall, one row running vertically up the centre line of the wall and the other row, also vertical, running opposite one of the diaphragms closest to the centre line. Deflection gauges (38 on Wall A and 29 on Wall B) were used to measure lateral deflections at various positions across the walls' faces.

The test procedure was as follows. Wall B was heavily post-tensioned to act as the reaction frame for tests on Wall A under various states of vertical prestress (including zero prestress). Later Wall A was prestressed to act as the reaction for a second set of tests on the undamaged Wall B. Side loads were applied in increments until a critical load was reached. A critical load was defined as when either pumping an ever larger volume of air into the air bag did not produce an increase in side pressure (plastic critical load) or when, in the interests of safety, a predetermined or calculated, side load was reached (elastic critical load).

In total there were ten lateral load tests. Each test fell into one of three distinct groups: walls without prestress (three tests), Wall A under prestress (four tests) and Wall B under prestress (three tests).

Behaviour of walls without prestress

The first of the tests on an unprestressed wall was on Wall A in the as-built condition i.e. it was uncracked before testing. The wall reached a plastic critical load at a side pressure of $1.82kN/m^2$. At this load a tensile crack developed horizontally across the wall 2.7m from the top.

When Wall B was tested without prestress it had been cracked at its base in a previous test. Its strength had thus been impaired but it still withstood a plastic critical load of $1.10kN/m^2$ before forming a second horizontal tensile crack 2.9m from the top of the wall.

The final test on a wall without prestress was on Wall A after it had been cracked both along its base and in its face, 2.7m from the top. In this condition the wall's resistance to side pressure is due almost entirely to its own weight. Its plastic critical load was $0.57kN/m^2$ which shows that a diaphragm wall can have a considerable in-built fail-safe capacity after it has been subjected to previous failure causing stresses (cf a wall in service subjected to accidental overload).

Figure 4 shows the deflected shapes of the walls at maximum lateral load. Figure 5(a) shows plots of side pressure against strain for the three tests. The strain readings are from gauges over the tensile cracks that appeared in the faces of the walls at approximately 0.375 h from the top of the walls, where h is the height of the wall.

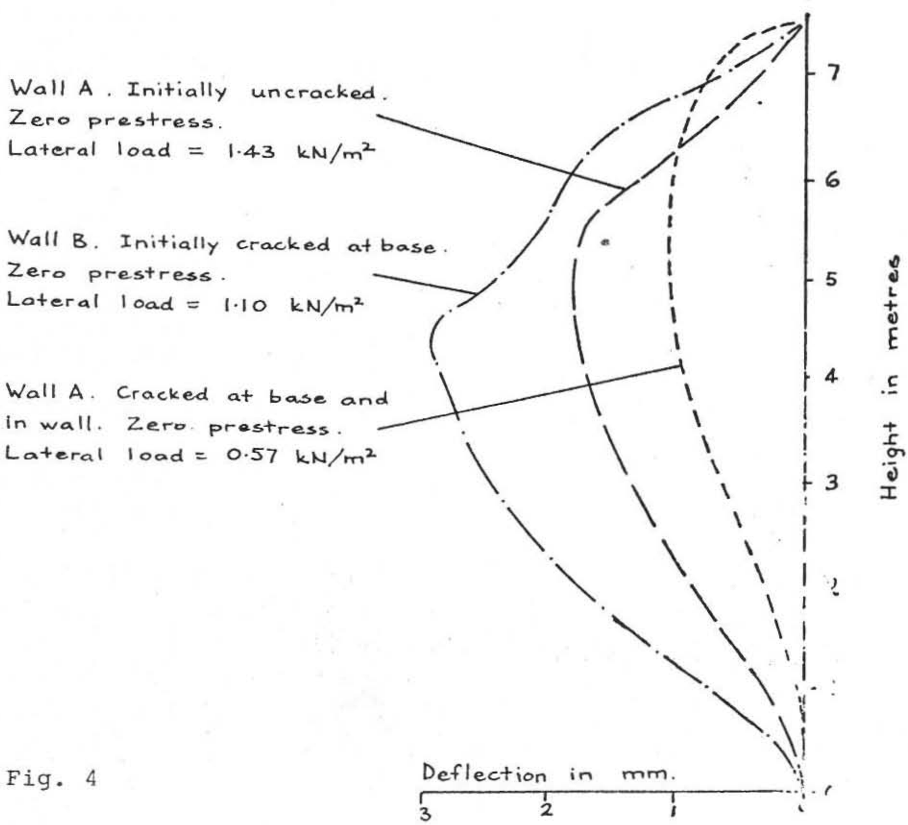

Fig. 4

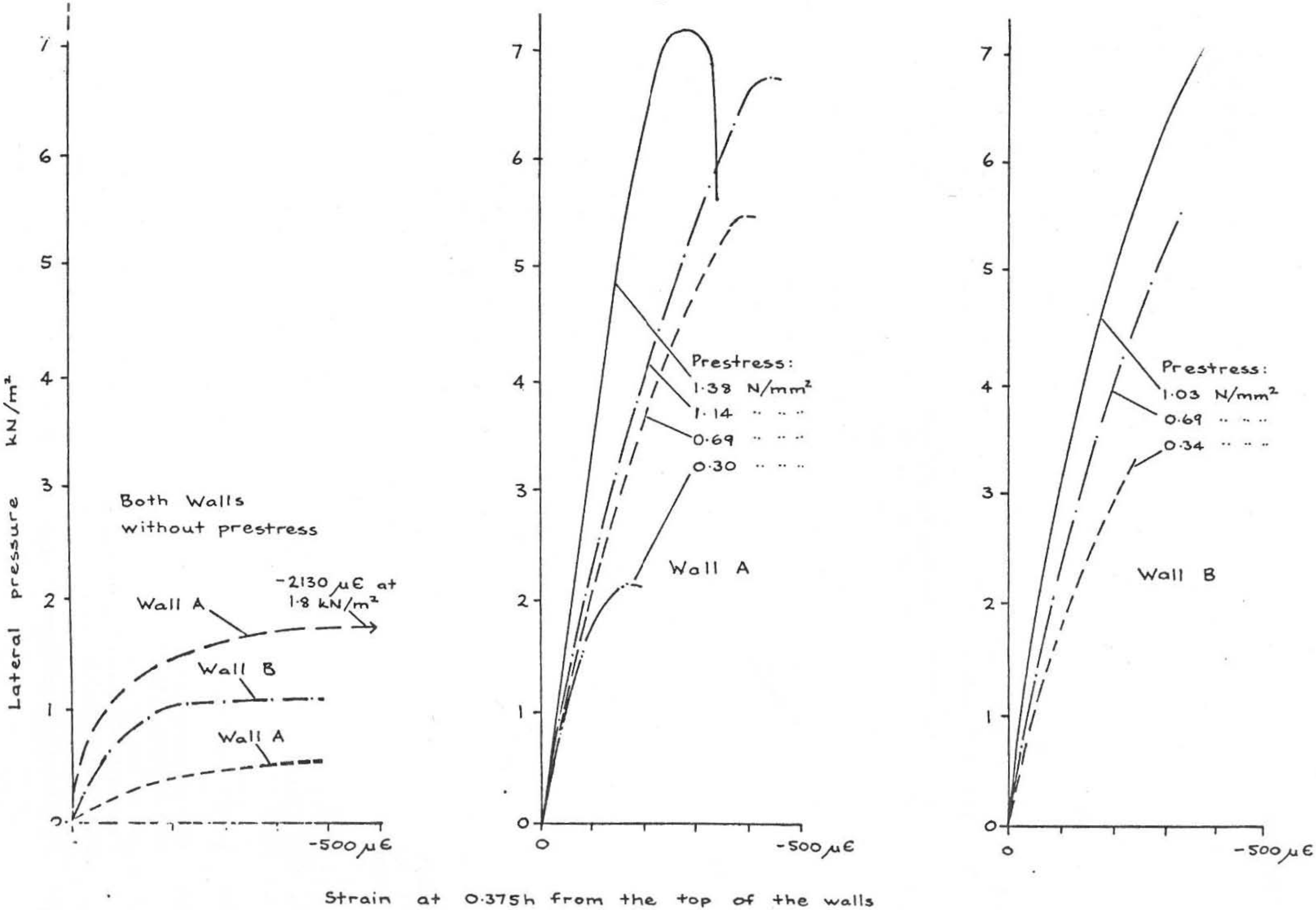
Lateral pressure kN/m²
Both Walls
without prestress
Wall A
-2130 µє at
1·8 kN/m²
Wall B
Wall A
-500 µє
Prestress:
1·38 N/mm²
1·14 .. .. ..
0·69 .. .. ..
0·30 .. .. ..
Wall A
Prestress:
1·03 N/mm²
0·69 .. .. ..
0·34 .. .. ..
Wall B
0
1
2
3
4
5
6
7
Strain at 0·375h from the top of the walls

Fig. 5 (a) (b) (c)

Behaviour of walls under prestress

After it was cracked in the first test without prestress Wall A was tested under four different levels of prestress, 0.30, 0.69, 1.14 and 1.38 N/mm^2. Before a crack at the base occurred in the last of the four tests the wall sustained the plastic critical side loads given in Table 1. The largest lateral deflection, 7.3mm, measured in all the tests was recorded at the load of 7.18kN/m^2 in the last of this group of tests: this is approximately span/1000.

The performance of Wall B under prestress was similar to that of Wall A except that it was cracked along its base prior to all the tests. The elastic critical loads are given in Table 1. No other cracks formed.

In all the tests the walls behaved in the manner of a vertical propped cantilever, the moments at the base being due to the self-weight of the wall, the tensile strength of the wall (if uncracked at the base) and any prestress acting. They behaved in a non-linear manner although (cracking excepted) they appeared to be elastic, always springing back to the vertical once the side pressure was released. The deflected shapes of the walls under prestress were similar to the deflected shapes shown in Fig.4. Plots of side pressure against strain are shown in Figure 5(b) and (c), the strain readings, once again, being from gauges over the tensile cracks at approx. 0.375 h from the top of the walls.

THEORY

The serviceability limit state can be defined as when a particular value of tensile or compressive stress has been reached, or a certain deflection has been attained or cracking has occurred. The tests showed that tensile cracking in the masonry appeared at low lateral wall deflections (both in absolute values and in terms of span/depth ratios) so deflection need not be considered a critical design problem and therefore no theory is put forward here for prediction purposes. Tensile stress is important in so far as it can be related to tensile cracking but compressive stress, in serviceability limit state terms, can only, at this stage, be arbitrarily catered for. Until more information is available it is better to consider compressive stress in ultimate limit state terms.

Thus in the following theory it is assumed that the serviceability limit state has been reached when flexural tensile cracks in the masonry have occurred either in the span of the walls or at the base of the walls or both. It is further assumed that:

(1) plane sections before bending remain plane after bending,

(2) the stress-strain relation for brickwork in both tension and compression is linear,

(3) cracking failure is determined by reaching a limiting tensile stress, and

(4) deflections are small and can be ignored.

In general at any section the extreme fibre stress, f, in the masonry due to an applied bending moment, M, and axial load, P, is therefore given by:

$$f = \frac{P}{A} \pm \frac{M}{Z}$$

For tensile failure this becomes:

$$f_t = \frac{P}{A} - \frac{M}{Z},$$

where f_t is tensile stress at the extreme fibre.

In the tests P is made up of the prestress force and the self weight of the wall so

$$\frac{P}{A} = f_p + f_w$$

where f_p = stress due to prestress
and f_w = stress due to welf weight of masonry.

$$\therefore M = Z(f_t + f_p + f_w).$$

f_p is a constant over the height of the wall and f_w varies linearly from zero at the top of the wall to a maximum at the bottom of the wall. f_t is a constant over the full height of the wall until a critical value, f_{kt}, is reached when locally, at a crack, f_t falls to zero. Thus the moment of resistance of the wall looks like the diagram of Figure 6(a).

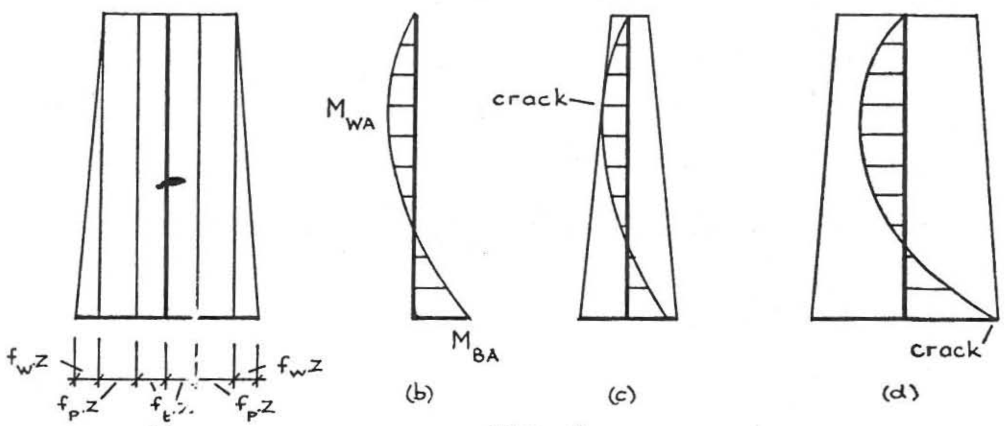

Fig. 6

The test walls were propped cantilevers, i.e. they had a pinned, or moment free, horizontal support at the top and a support at the base which provided both horizontal restraint and some resistance moment. The value of the base resistance moment is governed by the values of f_t, f_p and f_w. Consequently the applied moment diagram is as shown in Figure 6(b).

It is clear that when the applied moment is superimposed on the moment of resistance diagram failure can occur initially either in the span (Figure 6(c)) or at the base of the wall (Figure 6(d)). Once failure occurs at one or other of these positions the tensile strength, f_t, is lost at that position and the moments redistribute themselves. For instance assume that:

(a) initially the applied bending moments are those due to a linear elastic propped cantilever,

(b) the critical (or characteristic) flexural tensile strength of the brickwork, $f_{kt} = 0.21\ N/mm^2$,

(c) the density of brickwork, $w = 1725\ kg/m^3$ so that with a height of wall, $h = 7.62m$ $f_w = 0.13\ N/mm^2$ at the base of the wall and $f_w = 0.05\ N/mm^2$ at 0.375 h from the top of the wall, and

(d) the applied prestress, $f_p = 0.32\ N/mm^2$.

Then the moment of resistance at the base,

$$M_B = z\ (0.21 + 0.32 + 0.13) = 0.66z$$

and the moment of resistance at 0.375 h from the top of the wall,

$$M_W = z\ (0.21 + 0.32 + 0.05) = 0.58z.$$

The applied base moment, $M_{BA} = 0.125\ qh^2$ and the applied span moment, $M_{WA} = 0.07\ qh^2$ at 0.375 h from the top of the wall. If the lateral load, q, is increased until

$$0.125\ qh^2 = 0.66z$$

$$\text{i.e.} \qquad q = 5.28 \frac{z}{h^2}$$

$$M_{WA} \text{ is then } 0.07 \times 5.28 \frac{z}{h^2} \times h^2$$

$$\text{or } M_{WA} = 0.37\ z$$

which is less than the moment capacity in the span. However at this point the base cracks and the moment of resistance falls to:

$$M_B = z\ (0.32 + 0.13) = 0.45z.$$

Thus the applied base moment falls to 0.45Z and with q remaining constant at 5.28 $\frac{z}{h^2}$ the maximum applied span moment, M_{WA}, becomes 0.54z at 0.425 h from the top of the wall. If this value of M_{WA} had exceeded (approximately) 0.58z then the applied load would have to be reduced accordingly.

As another example if f_{kt} is assumed to be zero, as some designers might prefer, and if the prestress is zero then:

$$M_B = Z\ (0.13) = 0.13z$$

and $$M_W = Z\ (0.05) = 0.05z.$$

An applied base moment of $0.125\ qh^2$ gives:

$$q = 1.04\ \frac{z}{h^2}$$

while in the span, $0.07\ qh^2 = 0.05\ Z$ gives:

$$q = 0.71\ \frac{z}{h^2}.$$

Failure occurs in the wall before the base so the side load is limited by the strength of the wall in the span.

This analytical procedure has been used in analysing the test results applying throughout a value of $f_{kt} = 0.21\ N/mm^2$ where the wall is uncracked prior to testing and $f_{kt} = 0$ when the wall is cracked prior to testing. Table 1 compares the theoretical results with the test results.

Wall	Prestress N/mm^2	Calculated Side Load kN/m^2	Experimental Side Load kN/m^2	Experimental / Calculated
A	0*	1.3	1.8	1.35
	0.30	2.4	2.3	0.96
	0.69	4.8	5.5	1.15
	1.14	6.7	6.8	1.01
	1.38	7.2	7.2	1.00
	0**	0.4	0.6	1.50
B	0†	1.3	1.1	0.85
	0.34	3.1	3.3	1.07
	0.69	5.2	5.7	1.10
	1.03	6.3	7.1	1.13

* Wall in as built condition, uncracked before test.
** Wall cracked in span and at base before test.
† Wall cracked at base before test.

TABLE 1

CONCLUSIONS

1. Diaphragm walls can easily be prestressed to increase their working load capacity under lateral loads many times over that of diaphragm walls without prestress (in the experiments up to four times).

2. A simple theory is proposed which predicts the wall cracking loads, or serviceability loads, accurately at all the levels of prestress (including zero prestress) tested. The method is suitable for use in a design office.

3. The method of analysis also predicts the position of the tensile cracks either at the base of the wall or in the span of the wall accurately.

4. Deflection is not a critical serviceability load case.

5. Even after cracking diaphragm walls can have a considerable in-built fail-safe capacity.

REFERENCES

1. Curtin, W.G., "Brick diaphragm walls - development, application, design and future development", The Structural Engineer, Feb. 1980, Vol. 58A, No. 2.

2. Curtin, W.G., Sawko, F., "Brick diaphragm walls - research and testing", The Structural Engineer, March 1980, Vol. 58B, No. 1.

3. Williams, E.O.L., Phipps, M.E., "The bending behaviour of prestressed masonry box beams", 6th I.B.Ma.C., Rome, 1982.

ACKNOWLEDGEMENTS

Support for the work was given by The Brick Development Association; British Steel Corporation, Reinforced Steel Services; London Brick Company Limited; Fairclough Building Limited.