IV–39. Research Into the Structural Behaviour of Model Brick Diaphragm Walls

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ABSTRACT

The structural use of brick diaphragm walls has been developed by the pratice of W.G. Curtin & Partners for single storey buildings of substantial height. Several buildings have been constructed using this technique during the last 15 years. Some 2 years ago a programme of research was started at the University of Liverpool to substantiate the design assumptions used and to explore the further potential of this type of construction. This paper summarises the results of this research and confirms the suitability of diaphragm walling for single storey construction.

INTRODUCTION

Diaphragm walls are essentially wide cavity walls of brickwork braced by transverse or cross ribs bonded to the inner and outer leaves thus creating a cellular section in plan. Thus the leaves and cross ribs, acting integrally, form a series of connected box sections having a much higher section modulus and radius of gyration than the normal cavity wall. This type of construction has been developed by the practice of W.G. Curtin & Partners and employed successfully in situations in which high frameless walls were required as in sports centres, theatres and various municipal buildings. During the last 15 years some 16 projects using this concept have been constructed mostly in the North-West and have survived extreme weather conditions and hard usage without visible signs of distress or need for maintenance.

In 1977 it was decided to commence an extensive series of research projects and tests to verify design assumptions employed by the practice (Ref 1) and further explore the full potential of this concept. This research was felt to be timely and desirable since the relevant Codes of Practice (CP111 . . . Ref 2, and the recently published BS5628 . . . Ref 3) give little or no guidance on the expected behaviour of these walls. The work commenced in the Department of Civil Engineering at Liverpool University, was continued and expanded with the involvement of Redland Brick Company who constructed and tested two walls for the authors in their testing laboratories The London Brick Company who investigated the resistance of such walls to rain penetration, Building Research Establishment who investigated the presence (or rather absence!) of cold bridging and possibility of pattern staining, Tyson (Contractors) Ltd who examined the organisation on site and speed and simplicity of construction and Tweed, Atkinson and Lewis, Chartered Quantity Surveyors, who examined the relative cost of diaphragm wall construction against the traditional framed structure soluton.

The various research findings are summarised under the main headings of behaviour of the test walls under vertical and lateral loading with a brief indication of their rain penetration resistance and thermal behaviour.

BEHAVIOUR UNDER VERTICAL LOADING

The load-bearing capacity of brick walls under vertical loading is a function of the strength of brickwork and the slenderness ratio of the walls. Both Codes of Practice (Ref 2 & 3) provided adequate information as to the expected strengh of masonry and it was not necessary to pursue this aspect of the problem. The slenderness ratio however, was subject to considerable uncertainty. The Codes employ the concept of "effective thickness" of a wall and define slenderness ratio as the ratio of effective height of the wall to its effective thickness. For a solid wall the effective thickness is taken as the actual thickness and information is provided on effective thickness of cavity walls and of walls stiffened by piers (Code CP111 also provided guidance on effective thickness of Chevron walls but this is not included in BS5628).

There is *no* information on the effective thickness of diaphragm walls. Because of lack of guidance various figures for the effective thickness have been suggested including centres of the inner and outer leaves, the depth of the void, the overall thickness of the wall and 0.7 times the overall thickness of walls. As part of the research programme it was decided to investigate this problem both experimentally and analytically. The theoretical explanation of their behaviour was developed first and this will be discussed briefly.

EFFECTIVE THICKNESS OF DIAPHRAGM WALLS

There are clearly similarities between the cellular nature of diaphragm walls and cellular construction used in concrete, steel-work and timber. The slenderness of struts in these other materials is defined by the radius of gyration of the section under compression. The slenderness ratio is defined as the effective height of the member under compression divided by the radius of gyration, l/r. It appeared to the authors that this concept was more meaningful than the use of effective thickness since it provided a better description of the *stiffness* of the section, that is to say its resistance to axial loading. It has been possible to develop a theory (Ref 4) relating the radius of gyration to

the effective thickness of walls and it was easy to demonstrate that the effective thickness, for the purpose of using CP111 and BS5628, was 3.46 times the radius of gyration for a solid wall. (It is interesting to note that Cl. 310 in CP111 for zig-zag or Chevron walls uses the same method for evaluating the effective thickness from radius of gyration).

The determination of the effective thickness of diaphragm walls using this procedure suggested that for the purpose of evaluating the slenderness ratio in Table 4 of CP111 and Table 5 and Fig. 2 in BS5628 the *effective thickness was greater than the actual overall thickness* of the wall. This result at first sight might appear confusing to some but it must be borne in mind that the effective thickness is used solely for the purpose of calculating the capacity reduction factor which determines permissible *stress* (or ultimate load in BS5628) which the wall can carry. The diaphragm wall can thus be subjected to a higher stress (*not load*) than a solid wall of the same overall thicknes.This appears to be sensible since the removal of material from the central core of a solid section creating a hollow box section improves its load carrying efficiency. It is for this reason that engineers use R.H.S. and I sections in steelwork and plywood box beams in timber.

It has been possible to demonstrate that the effective thickness (t_{ef}) of walls lies within the theoretical range $t < t_{ef} < 1.73t$, where t is the overall thickness of the wall (although the practical range is probably $t < t_{ef} < 1.4t$ .. Ref 5). It is felt that this result is significant since it demonstrates theoretically the superior structural efficiency of diaphragm walls when compared with solid or normal cavity wall construction.

BEHAVIOUR OF MODEL WALLS UNDER VERTICAL LOADING

To date three walls have been tested under vertical loading. The "Liverpool" wall (Fig. 1) tested in the Department of Civil Engineering at Liverpool University was loaded to the maximum capacity of the load-rig and jacks of 100 tons (about 30 times the working load of supporting lightweight roofs and in excess of the original estimate of ultimate load). Under this load the wall showed no visible distress, there was no evidence or sound of cracking and it fully recovered on unloading. The other two walls (Fig. 2) were constructed by Redland Brick Company Ltd for the authors and tested to the authors' specification to destruction in their 2,000 ton testing machine. They failed respectively at 485 tons and 675 tons. (It is felt that the lower figure for the first wall was caused by the absence of mortar bedding between machine platten and base course—an omission rectified in the second test).

During testing of the three walls opportunity was taken to measure stress and lateral deflections. The Liverpool wall built with a relatively soft Fletton brick, produced by the London Brick Company, exhibited linear behaviour over the whole range (100 tons) Fig. 3, but showed evidence of some leaf bulging as illustrated in Fig. 4, and a tendency to 'crumple' like a box section and not bowing like a solid wall. The two Redlands walls were built with very hard bricks. The stress strain graphs illustrate almost linear behaviour up to 50% of the ultimate load, both in terms of strain (Fig. 5-8) and deflections (Fig. 9-10). Bearing in mind that in practice they are extremely unlikely to be stressed to 20% of the ultimate limit load the authors feel justified in recommending the use of linear elastic theory to describe their behaviour. There was less evidence of crumpling in the Redland walls and the authors would like to repeat the Redland test using Fletton bricks since it is possible that the lateral deflections are a function of the wall's material as well as its shape.

An examination of the strain distribution on the two faces of the wall reveal little variation in strain confirming the fact that all individual components of the wall (the two leaves and ribs) act integrally as one unit.

On the basis of the results for the three walls tested under axial loading it was possible to conclude that:—

1. All individual components of the wall act together structurally as an integral box section. Had the walls acted as two separate leaves (as in a cavity wall) or as piered walls, an unacceptably high slenderness ratio of about twice the maximum permitted by the Code, would have led to a zero axial strength. There is no way in which the Code can be interpreted to account for the very high ultimate load obtained in the tests.

2. All the walls tested were relatively "stocky" in terms of slenderness ratio. It was not possible to confirm the theory about effective width being greater than actual, but it is possible to say, that it is *at least* equal to the overall depth. The authors have little doubt that testing of 'slender' walls should confirm their theory.

An estimate of ultimate load for the Redland walls is given in Appendix 1. It will be observed, that the estimate of 580 tons (using the radius of gyration concept) is close to the mean of 485 and 675 tons obtained in the two test. The dramatic view of the collapsing wall is shown in Fig. 11.

As a result of the practical experience and this research the BSI have set up an addendum committee of BS5628 to codify the design of such walls.

BEHAVIOUR OF WALLS UNDER LATERAL LOADING

Three walls were tested to destruction under simulated wind pressure. The test set-up is shown in Fig. 12. Polythene bags connected to a compressor were used to produce a uniformally distributed load on the vertical faces of the walls. The main object of the tests was to assess whether the various components of the wall act integrally as one section (i.e. that the section modulus of the wall is that for a box section—not merely the sum of the moduli of the two leaves.

The typical presure/strain graphs are shown in Figs. 13. It is evident that the behaviour of the walls is almost linear up to a pressure of 1.7 kN/M^2 (36lb/sq ft) when the gauges were removed. During the increase of pressure up to 2.2 kN/m^2 (45.5 lb/sq ft) the walls might have exhibited some plasticity, although pronounced plasticity in flexural tension in a brittle material such as brickwork is not expected. (It should be noted that the failing pressure of 45.5 lb/sq ft is equivalent to 50 lb/sq ft on a full size wall—this rep-

resents a massive factor of safety for such walls under wind pressure).

The failure of the wall occurred, as expected, by tensile cracking along a bed course just below the horizontal centre line of the wall, followed by an increase in lateral deflection as the pressure was maintained and opening of crack at the base and top of the wall (Fig. 14-16). Calculations summarised in Appendix 2 reveal, that the wall failed at a tensile stress of 299 kN/m² (43.39 lb/in²) compared with the characteristic stress for this type of brickwork of 290 kN/m² (42 lb/in²). It will be noted, that the whole section was assumed to act and these calculations and test results appear to justify this assumption.

The load deflection curves of the Liverpool wall (Fig. 17) under the lateral loading test show that the wall is tending (as expected) to deflect as a simply supported beam subject to a UDL. The change of curvature at the base is probably due to the partial restraint at its base, due to the action of the stability moment. In a full size wall, due to scaling up, the stability moment would be 16 times that of the test wall and there would, of course, be a larger restraining moment.

One of the walls was constructed with a very wide spacing of ribs of 24t in order to assess the effect of rib speacing on structural action. It is interesting to note that the failure in the wall with wide rib spacing occurred at approximately 50% of the pressure of that occurring in the other two walls indicating that the whole width of the leaves were not assisting in the load carrying.

An analytical investigation using a finite element method has been carried out to assess the magnitude (if any) of the shear lag effect in such walls. The graphs showing the variation of strain (Fig. 18) indicate a substantial variation with increased cross rib spacing. Since mortar has a limited tensile strain capacity, failure would commence at the rib positions, and quickly spread to other zones.

On the basis of the various tests and analytical investigations it is possible to conclude the following:—

1. All the constituent elements of the wall act as one integral cellular section. The walls acting as two separate leaves with a series of independent ribs, could not have withstood the lateral loads that they did. The characteristic flexural stress of brickwork multiplied by the section modulus of box section gives the ultimate bending moment the wall can develop.

2. Flange width equal to rib width plus twelve times flange thickness, should be adopted as the maximum permissible, until such time as further theoretical and experimental evidence provides further explanation.

RAIN PENETRATION

Considerable anxiety has been expressed by some architects and clients about the possibility of rain penetration through the brick ribs. None of the walls constructed up to date have shown any indication of such rain penetration (even though they have been constructed in an area with higher wind velocity and rainfall than average), it was nevertheless desirable to allay fears by carrying out a programme of rain penetration testing on a typical section of diaphragm wall. This work was carried out by Mr. R. Beard of the London Brick Company and funded by the Company. The test was carried out on a full sized diaphragm wall two bricks thick, in accordance with BS4315, Part 2, 1970. It was observed that under "normal" expected conditions of driving rain there would be no possibility of rain penetration. The eventual penetration of moisture to the inner leaf was achieved by applying gross overloading of moisture corresponding to a significant overloading in load carrying tests. It is possible to conclude that the walls would be expected to behave adequately under normal working conditions and that moisture would not penetrate to the inner leaf.

THERMAL BEHAVIOUR

The Building Research Establishment have carried out a theoretical analysis of heat flow through diaphragm walls. To meet the U value of 0.6, required by Part FF of the new building regulations they have found that fixing 75-100 mm insulating batts would be adequate. They also found that the dew point will always occur in the cross ribs and that condensation and pattern staining will not occur on the inner face—this agrees with the observations of the actual performance of buildings.

CONCLUSIONS

The main conclusions from the various investigations carried out to date are as follows:—

1. It is possible to consider the structural action of the wall under vertical loading as defined by the slenderness ratio using the radius of gyration of the cross-section.

2. In assessing the strength of wall under lateral loading it is possible to assume that all elements act as an integral section provided that the spacing of the ribs is not greater than 12 times their thickness.

3. It is possible to conclude that this type of construction is resistant to rain penetration.

4. It appears that brick diaphragm walls are a structurally viable, economic and attractive alternative to traditional framed structures.

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2. CP111—Structural Recommendations for Loadbearing Walls. British Standards Institution, London, 1970.
3. S5628: Part 1: 1978. Structural Use of Masonry. Part 1. Unreinforced Masonry, British Standards Institution, 1978.
4. Effective Thickness and Structural Efficiency of Cellular Walls and Piers by F. Sawko and W.G. Curtin. Proc.Inst. Civil Engineers, Part 2, 1978, 65, Dec. p893–898.
5. Discussion to 4—to be published.

ACKNOWLEDGEMENTS

Experimental work on Diaphragm Walls funded by the Brick Development Association and the Building Research Establishment, was carried out in the main Structures Laboratory of the Department of Civil Engineering at Liverpool University, using half scale bricks manufactured and donated by the London Brick Company. Tysons (Con-

tractors) Limited provided free services of a brick-layer for the first wall. Redlands Limited built and tested two walls to destruction in their testing facility at Horsham. London Brick carried out rain penetration tests in their testing laboratories and Building Research Establishment carried out an investigation in thermal efficiency of these walls. The authors are pleased to acknowledge all the support received from the various organisations.

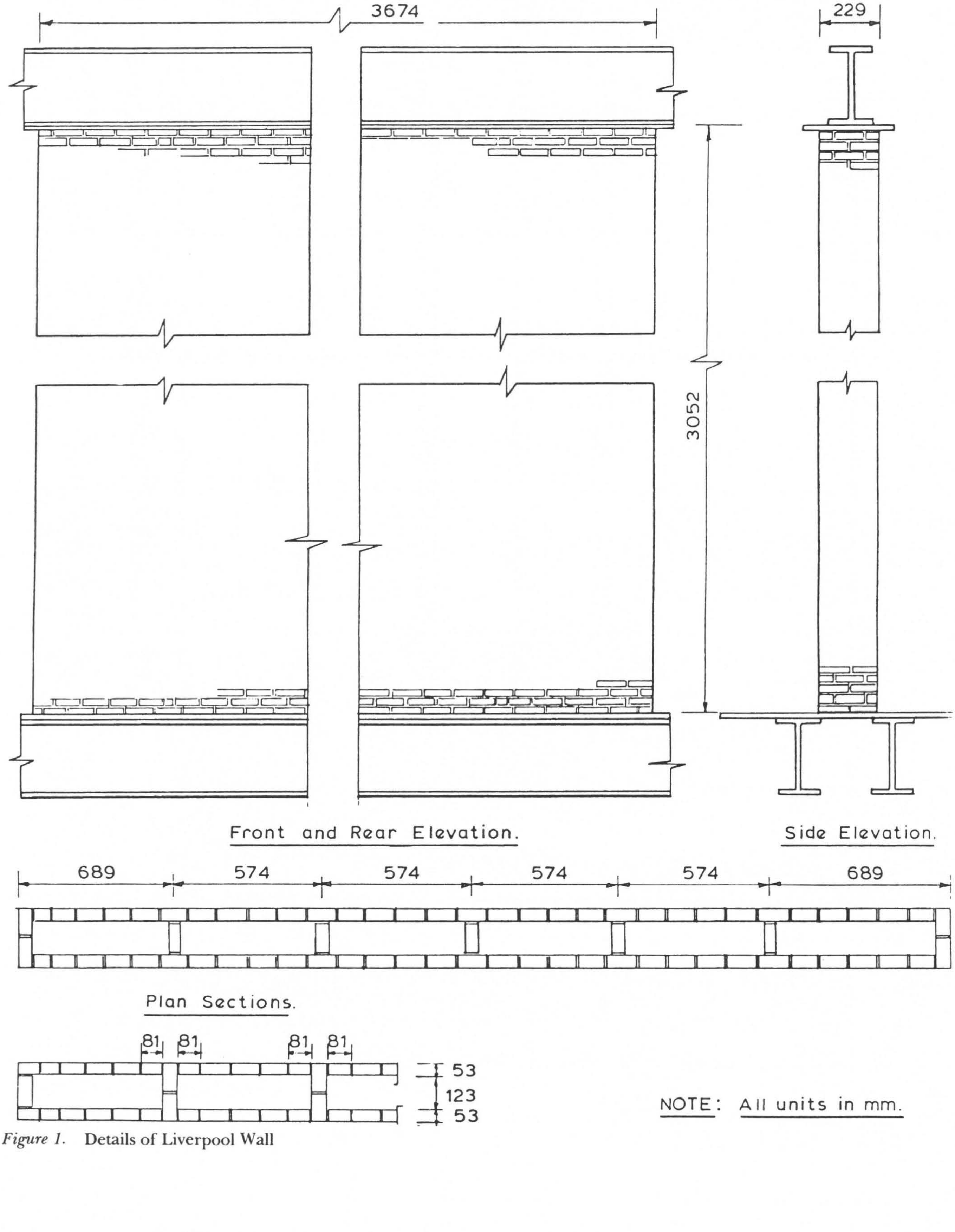

Figure 1. Details of Liverpool Wall

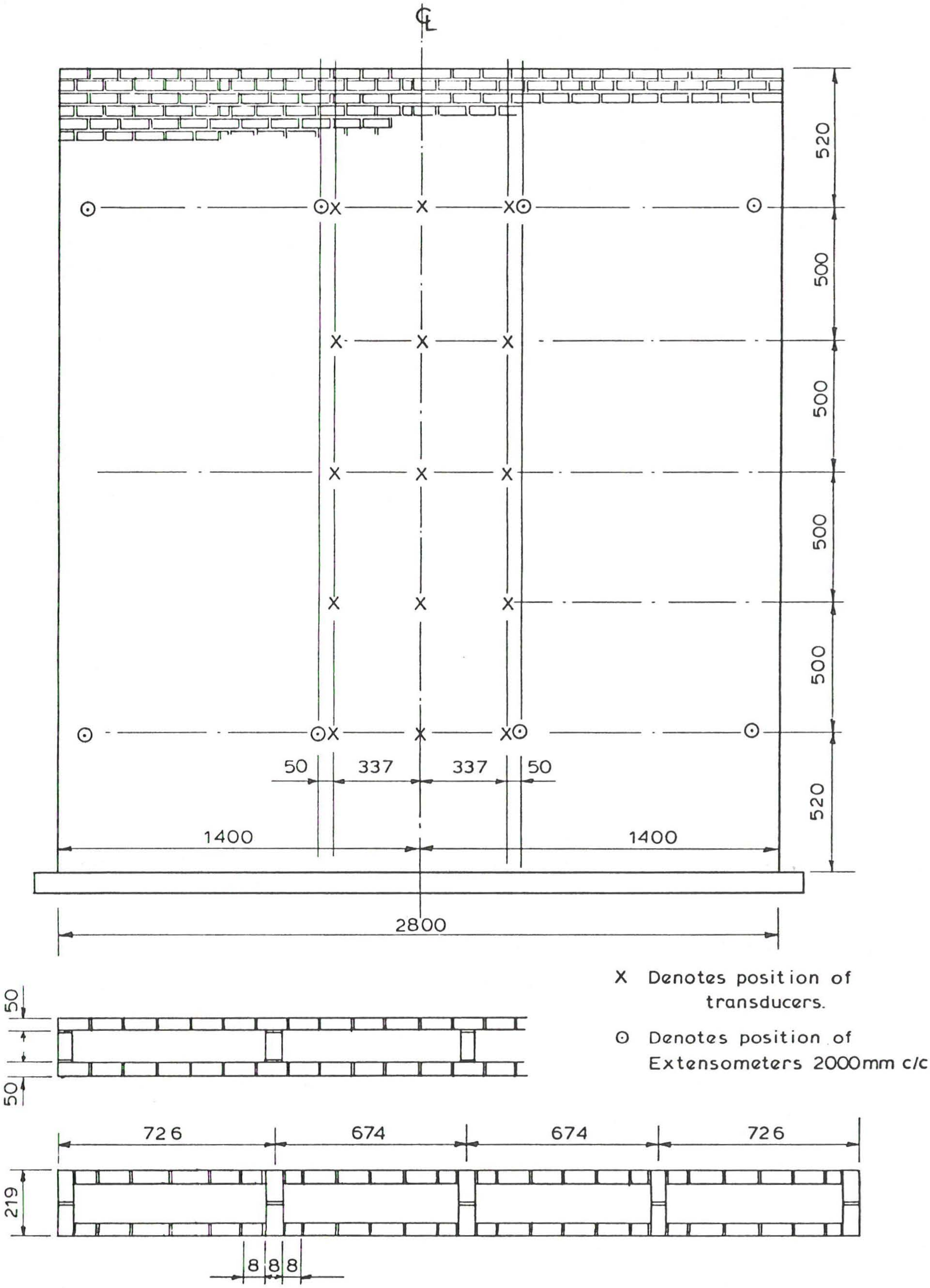

Figure 2. Details of Redland Walls

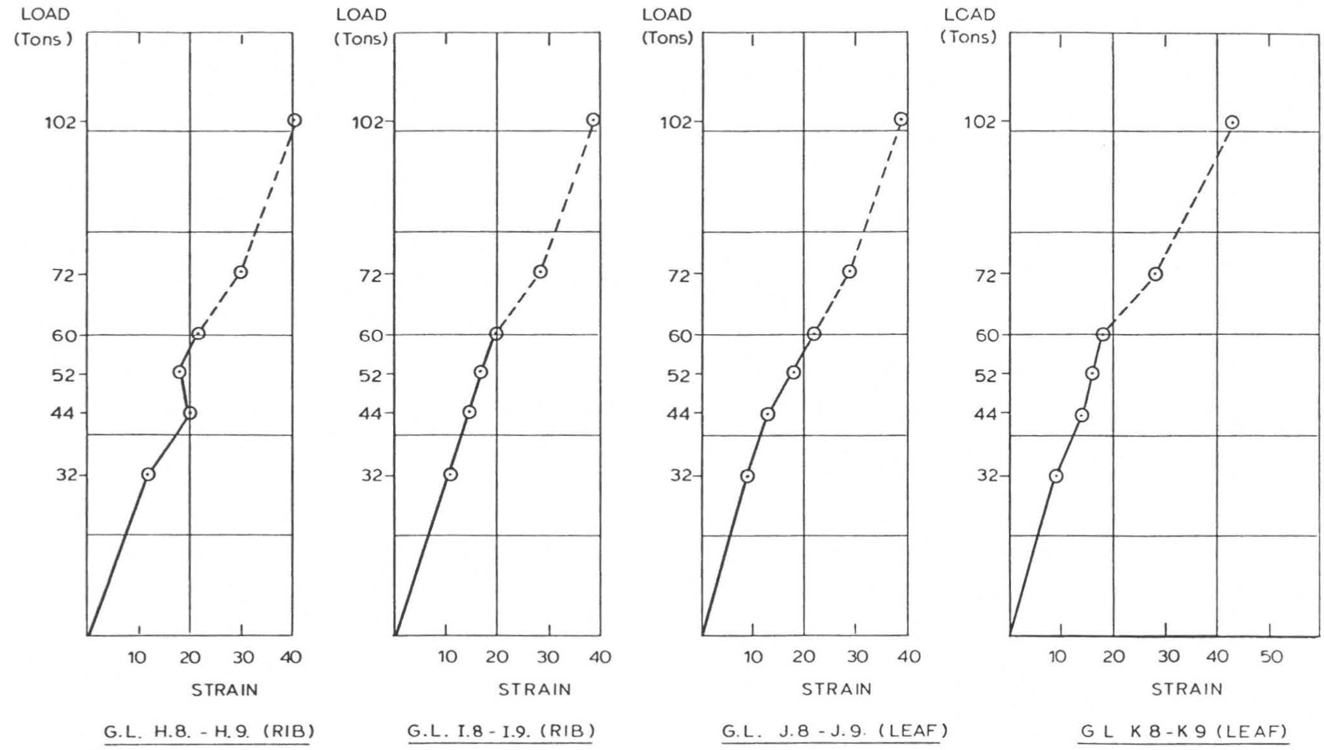

Figure 3. Liverpool Wall—Load Strain Characteristics

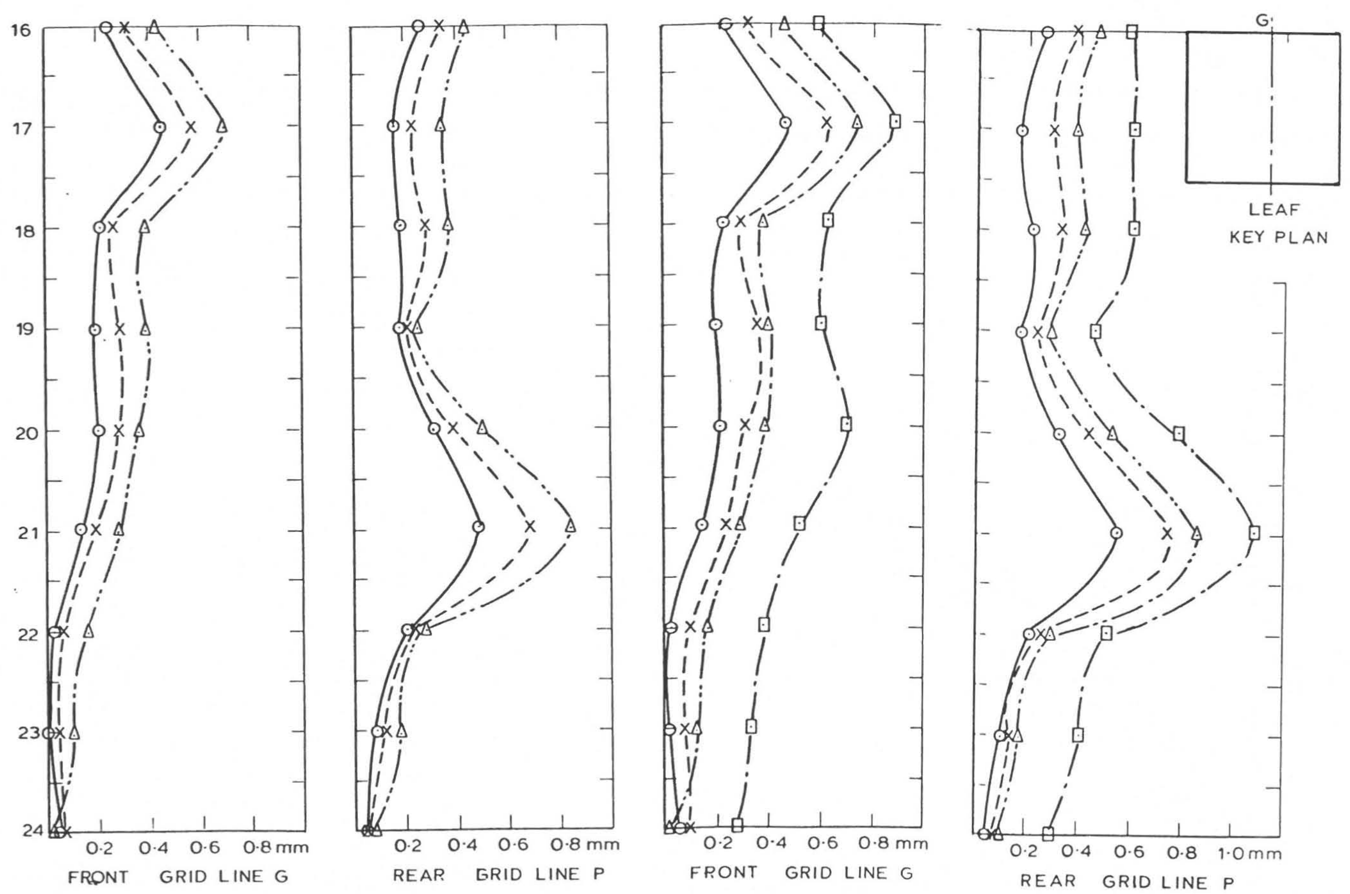

Figure 4. Liverpool Wall—Bulging of Leaves

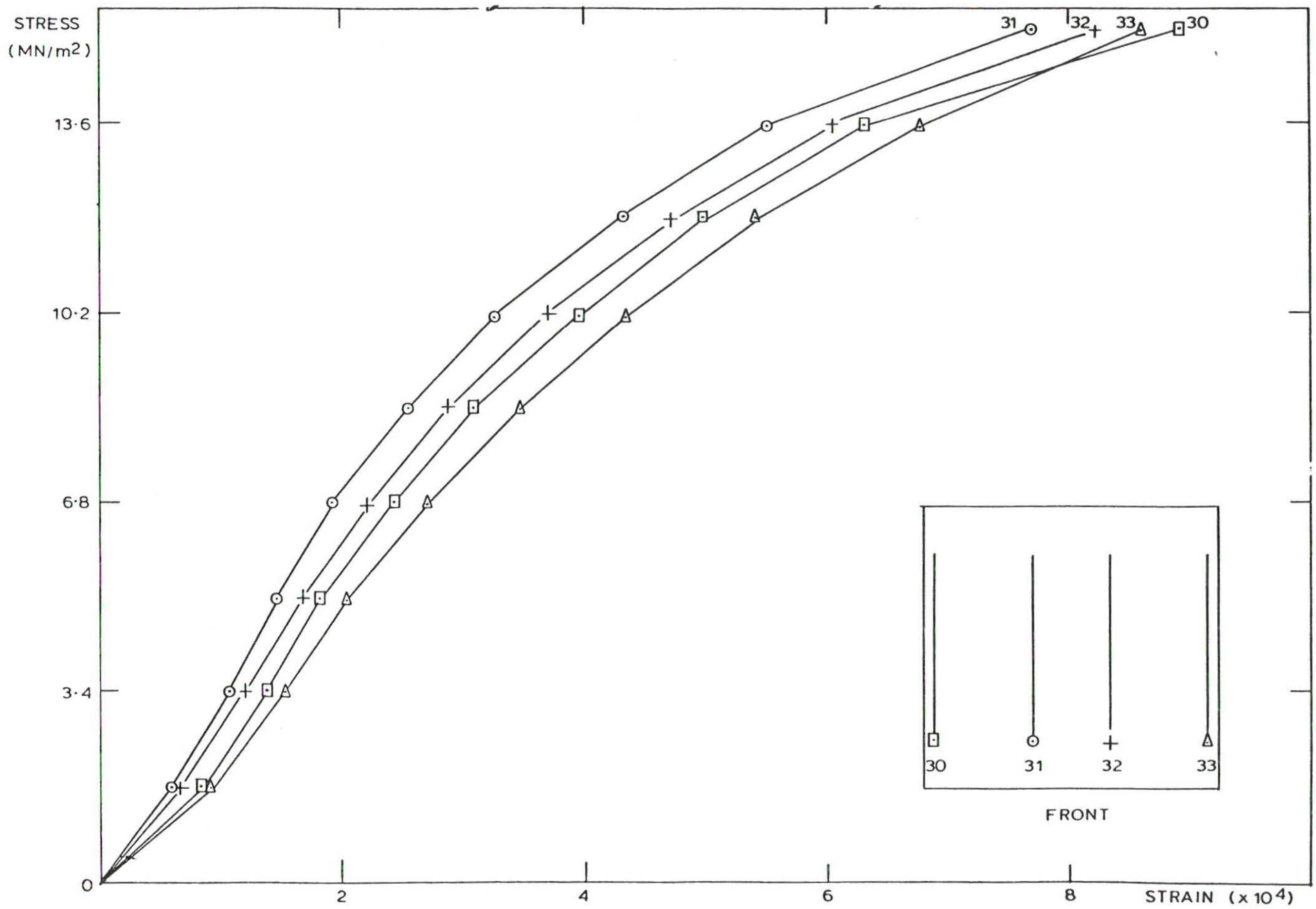

Figure 5. First Redland Wall—Stress Strain—Front Leaf

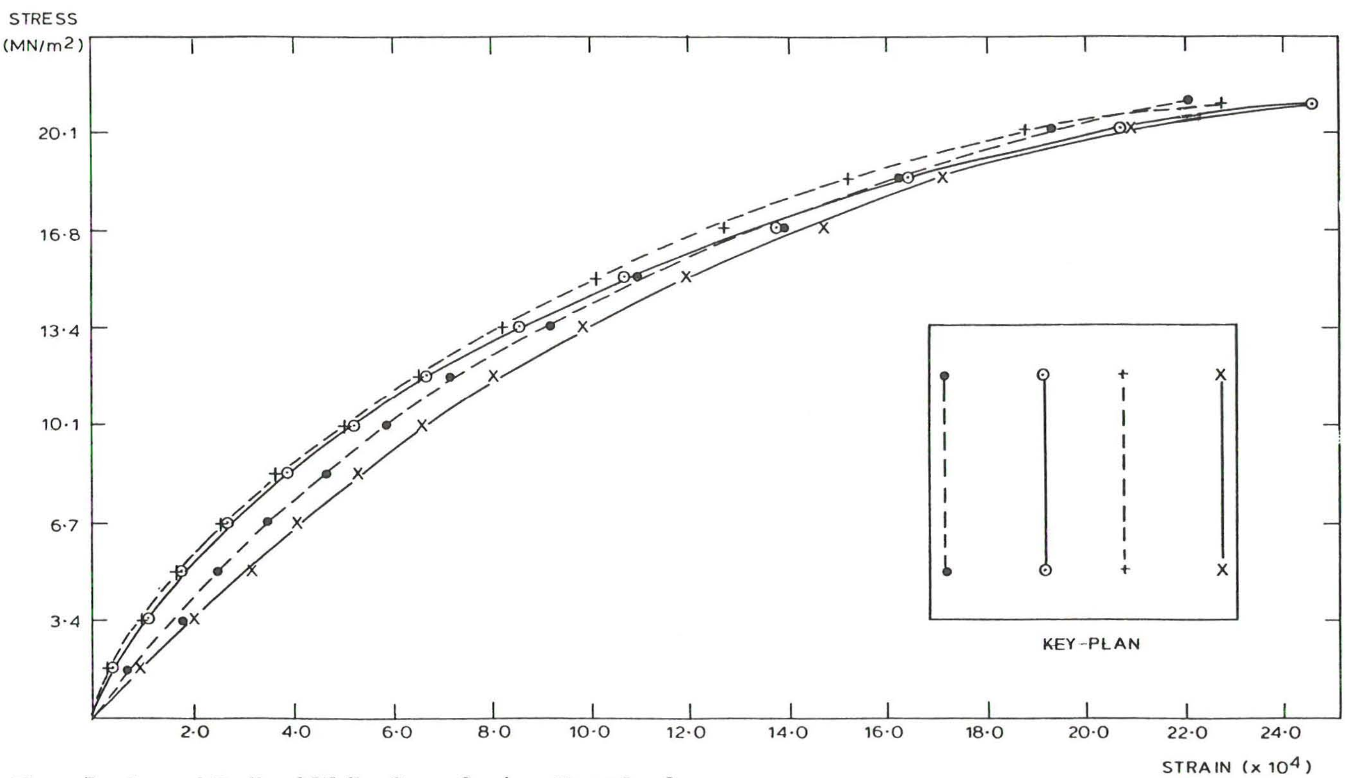

Figure 7. Second Redland Wall—Stress Strain—Front Leaf

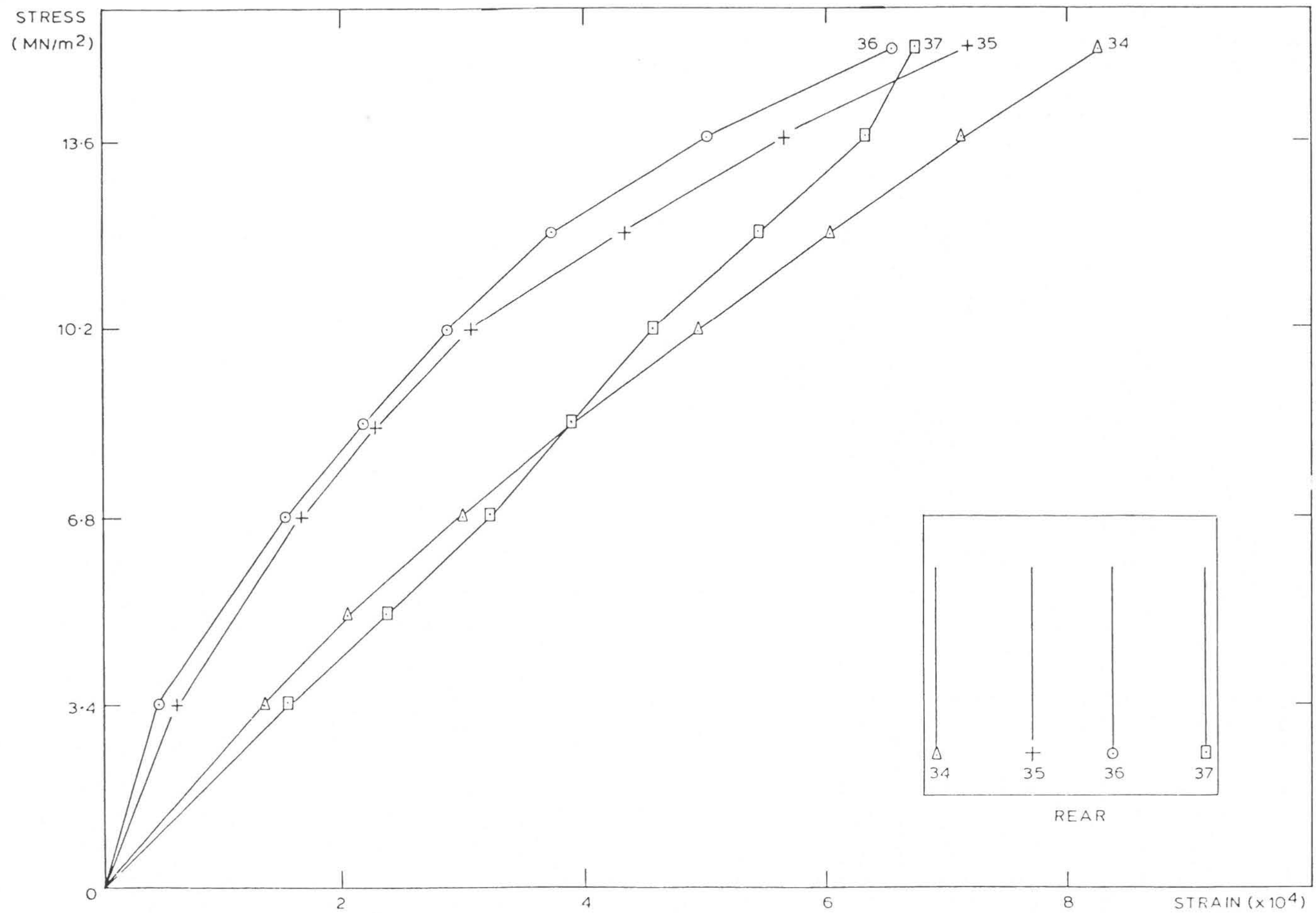

Figure 6. First Redland Wall—Stress Strain—Rear Leaf

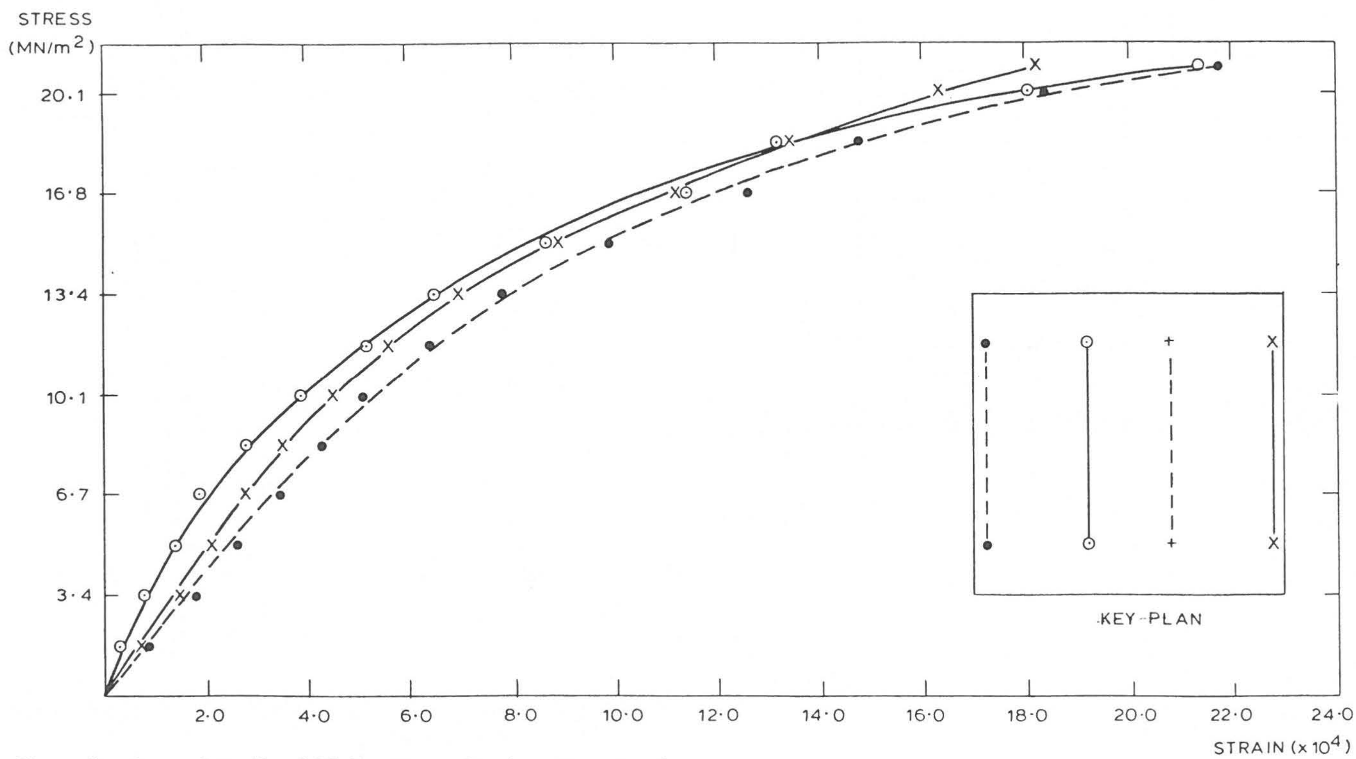

Figure 8. Second Redland Wall—Stress Strain—Rear Leaf

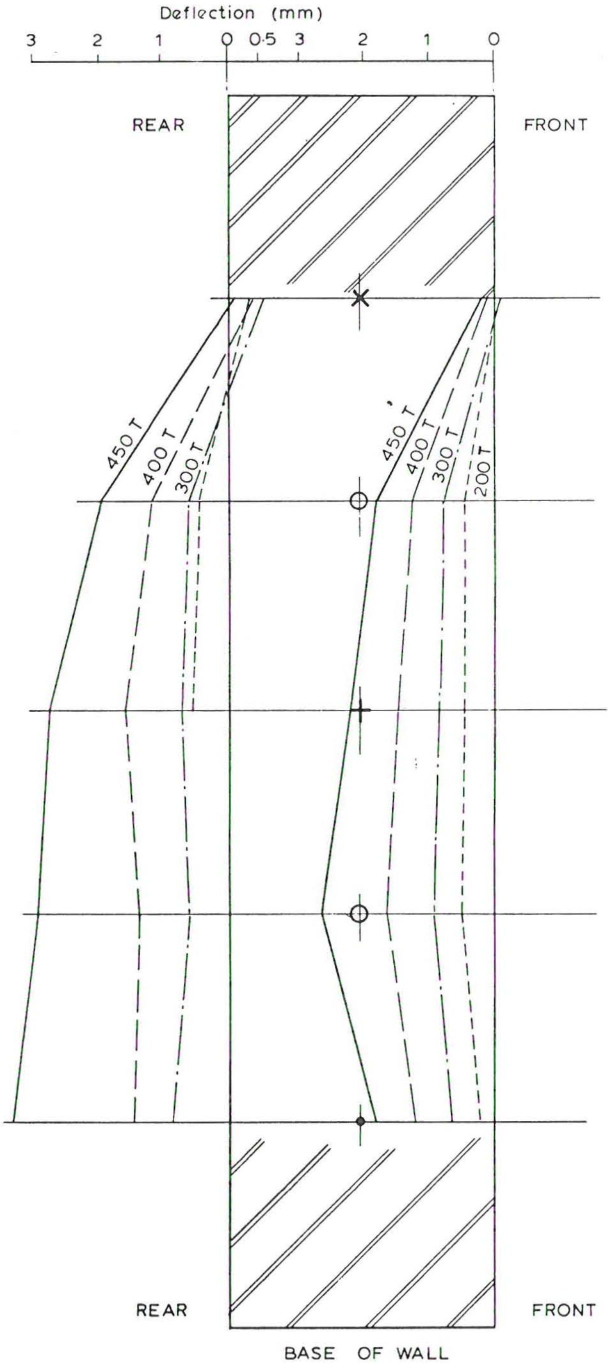

Figure 9. First Redland Wall—Load Deflection diagram

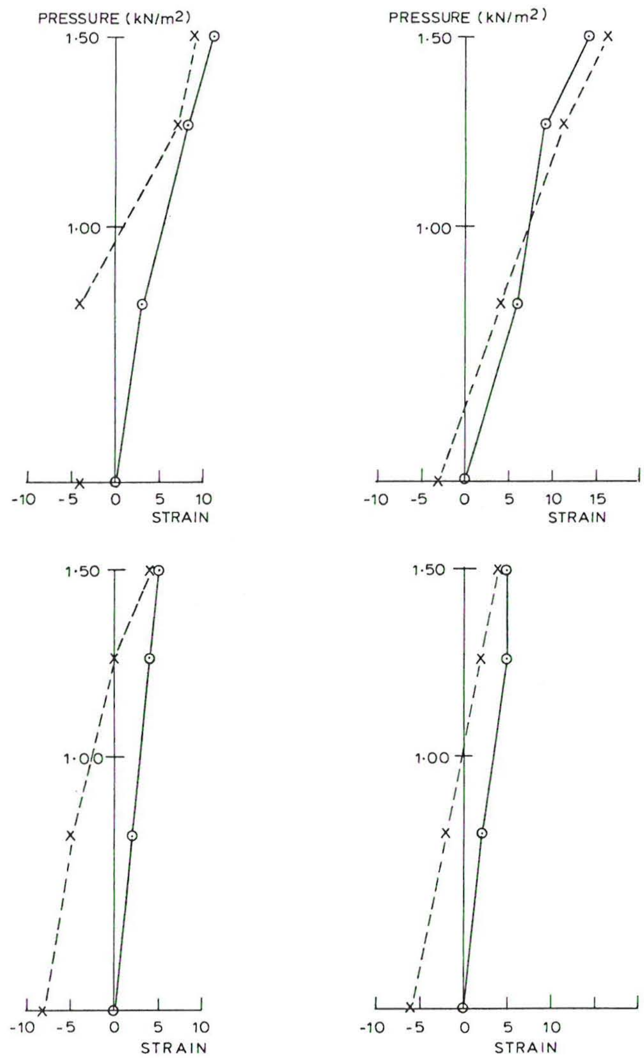

Figure 13. Liverpool Wall—Pressure strain graphs under lateral load

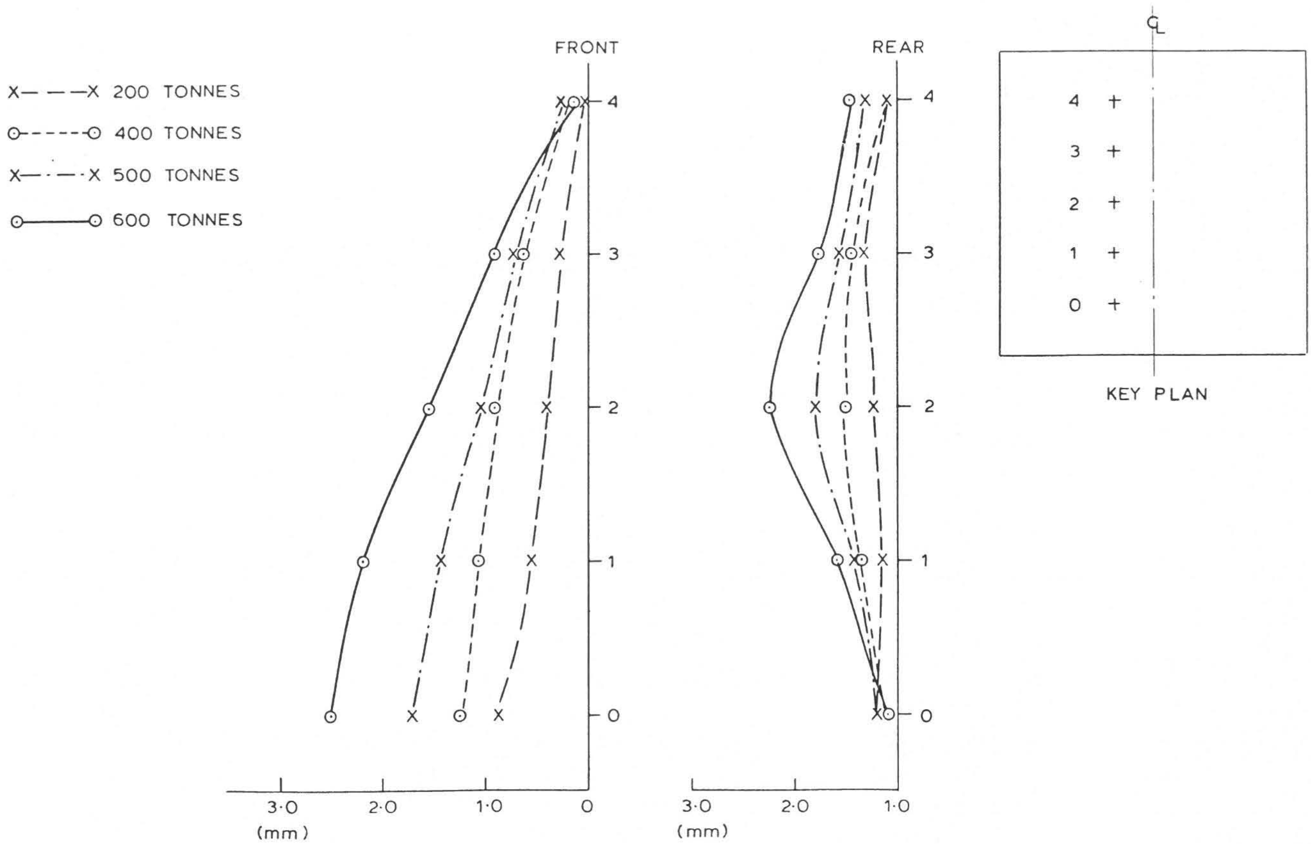

Figure 10. Second Redland Wall—Load Deflection diagram

Figure 15. Liverpool Wall—Part elevation of wall under ultimate lateral pressure

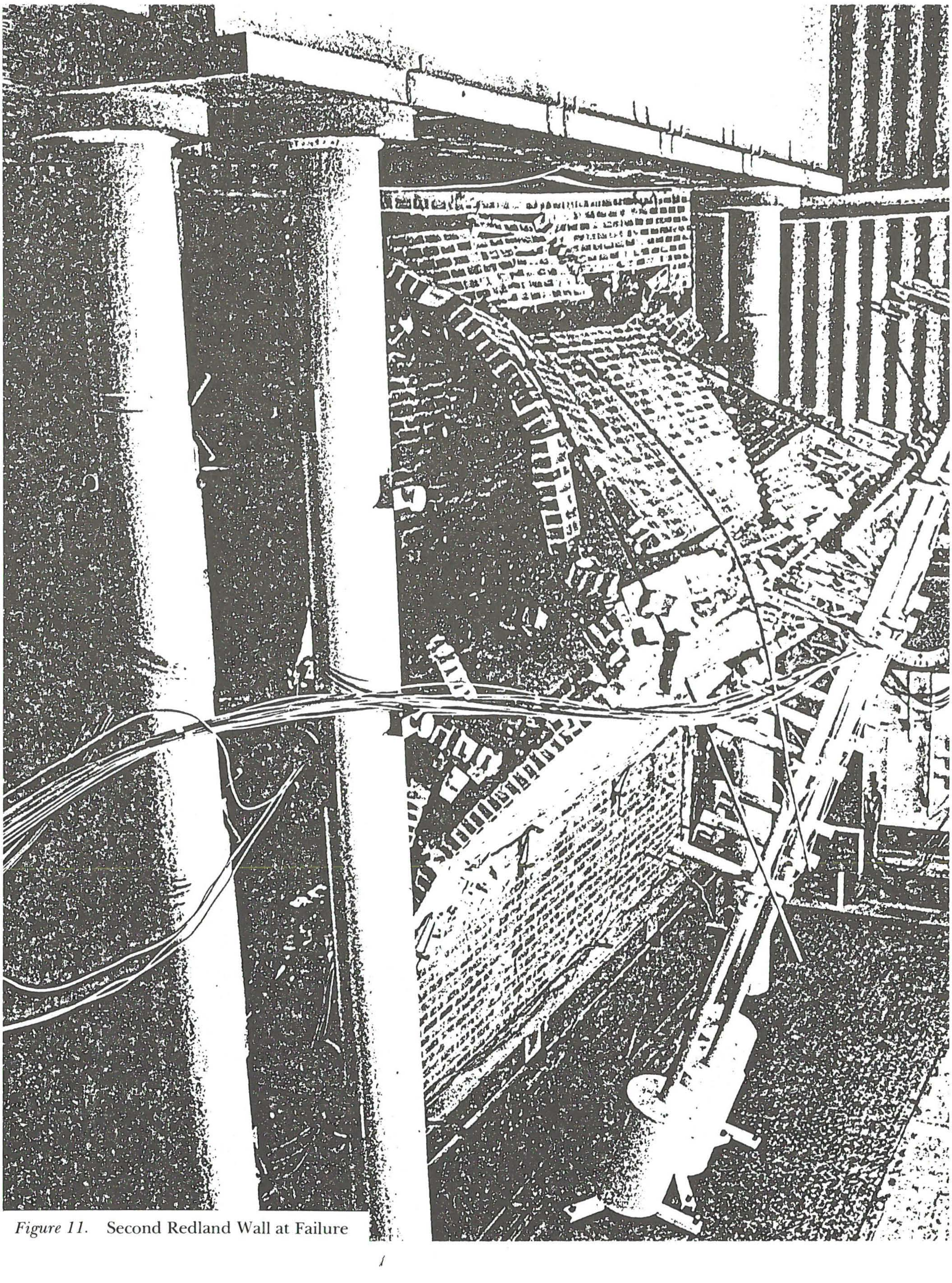

Figure 11. Second Redland Wall at Failure

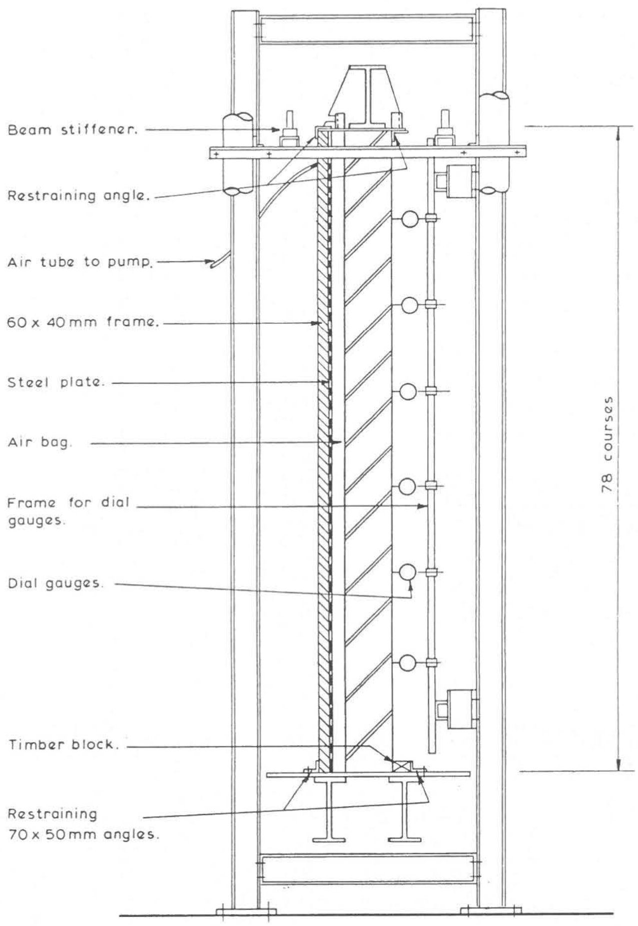

Figure 12. Experimental set up for lateral load test

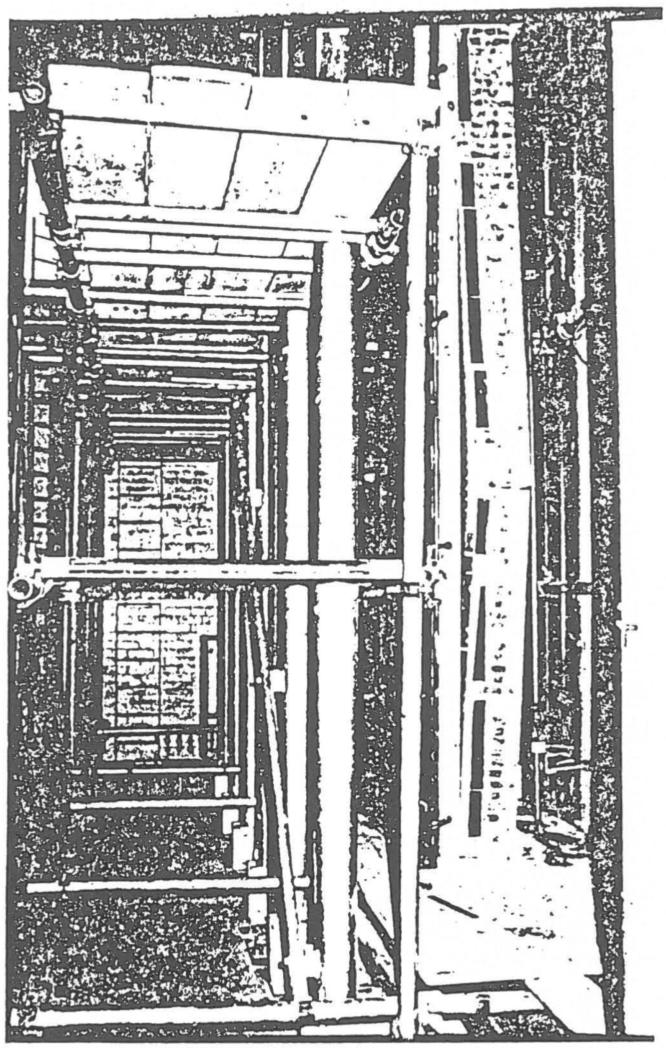

Figure 16. Liverpool Wall—End elevation of Wall after ultimate lateral pressure

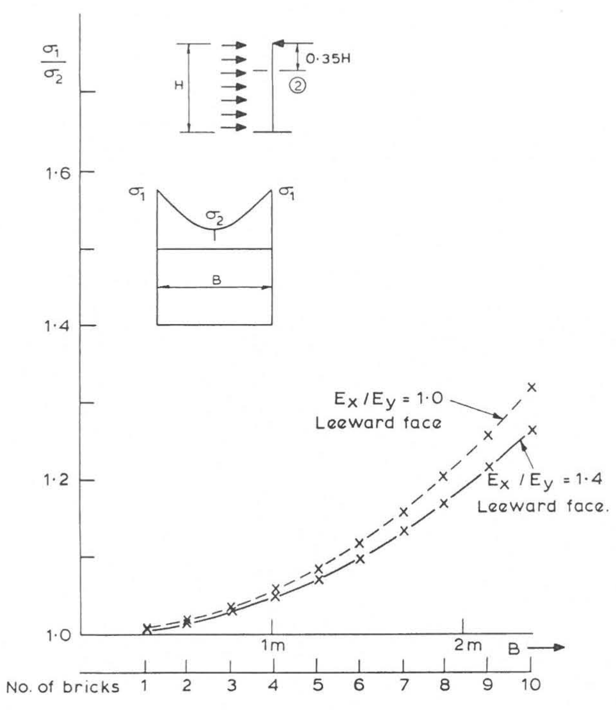

Figure 18. Extrapolation of brickwork strength for Redland Walls

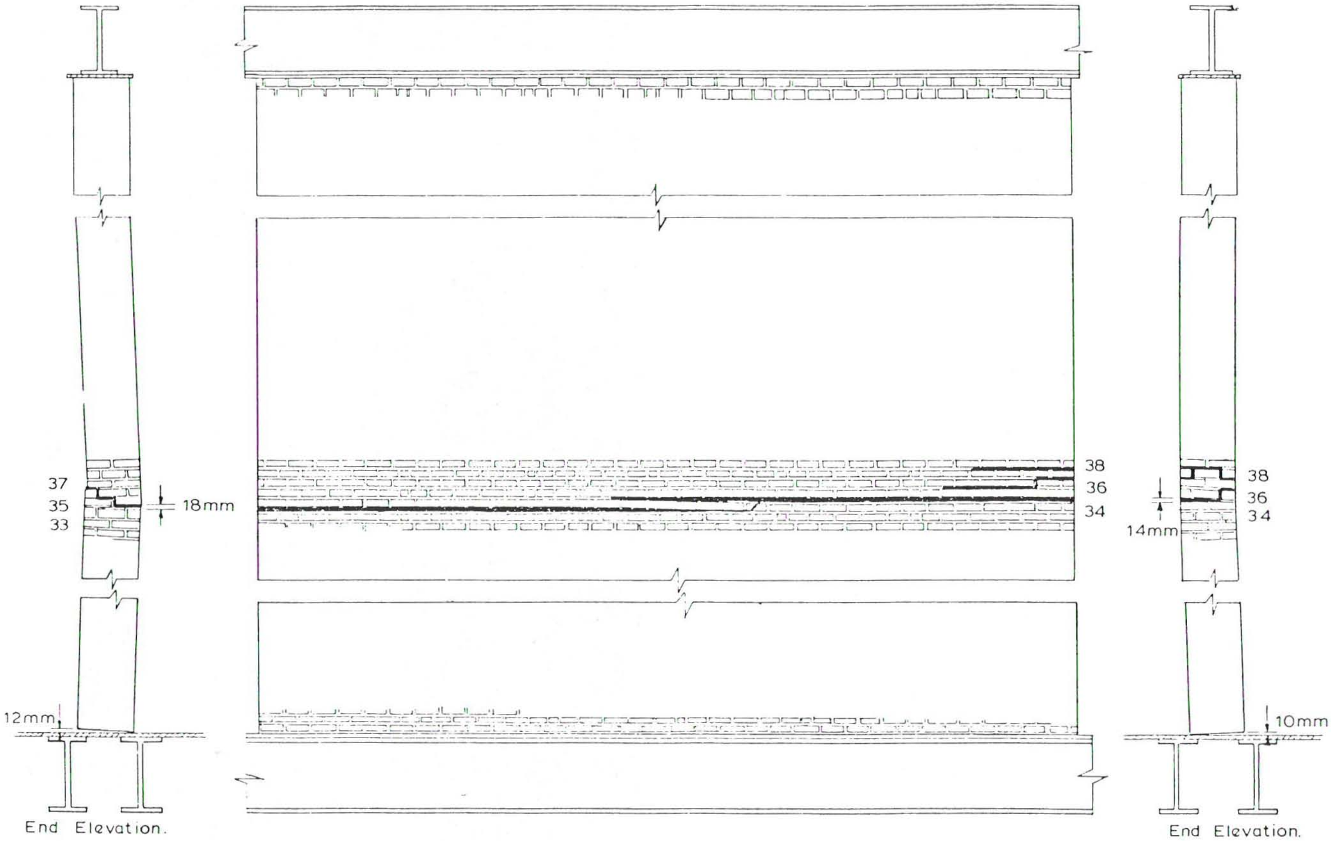

Figure 14. Liverpool Wall—Failure under lateral pressure

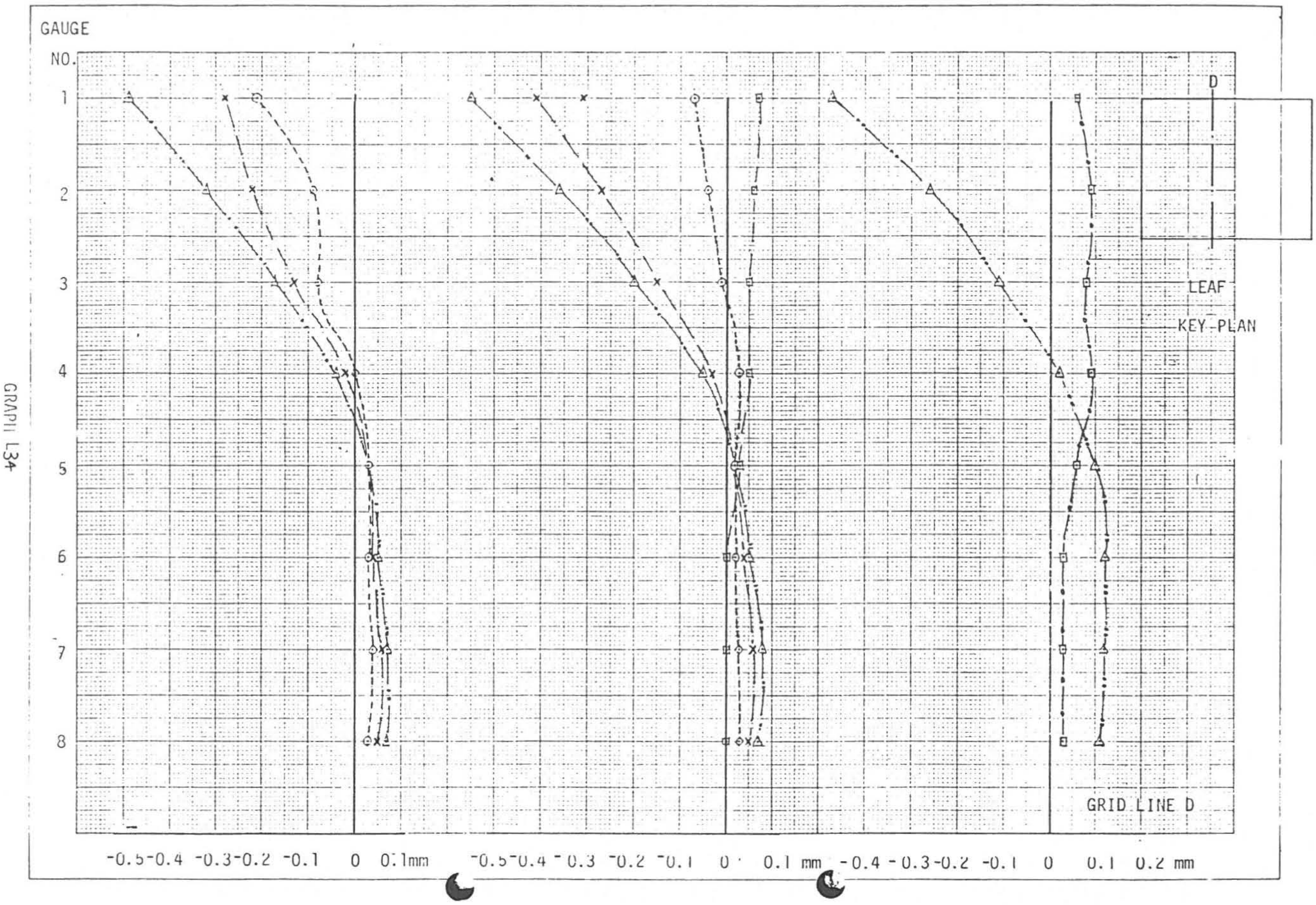

Figure 17. Liverpool Wall—Load deflection curves under lateral pressure