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TESTS ON THE LATERAL STRENGTH OF CONCRETE BLOCK WALLS AT AULTSVILLE

by

W. G. Plewes

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PREFACE

Advantage was taken of the opportunity to carry out structural tests on buildings to be removed or demolished in the area to be flooded by the construction of the St. Lawrence Power Project. Only one building representative of current construction practices was finally selected for study. The results of loading tests on the concrete block masonry walls of this building are now reported. They are revealing in that they illustrate both the limitations and benefits to be derived from field loading tests on actual structures.

The work was carried out by the staff of the Building Structures Section. The author, a civil engineer, is a research officer with the Division having special interests in concrete structures.

Ottawa
November 1961

N. B. Hutcheon
Assistant Director

TESTS ON THE LATERAL STRENGTH OF CONCRETE BLOCK WALLS AT AULTSVILLE

by

W. G. Plewes

This report deals with the results of load tests on selected wall panels of a concrete block building at Aultsville, Ontario. The opportunity to use this building for tests presented itself with the construction of the St. Lawrence Power Project, when the flooding of lands necessitated the razing or removal of all buildings in several villages and farms.

It appeared at first that load tests might be made on a number of these buildings and useful information obtained which would assist with the planning of a proposed laboratory study of the strength of masonry walls and the design of load test apparatus. Tests on actual structures would also, it was thought, be of some help in correlating laboratory tests with field conditions.

Permission was obtained from the Hydro Electric Power Commission of Ontario to use some of the buildings for this purpose, and a survey was made to see what might be done.

The buildings scheduled for demolition were houses, churches, schools, and an occasional commercial building varying from 50 to 150 years old, the majority of them 80 or 90 years of age. Many of them originated as modestly-sized buildings but had received one or more additions over the years, often of different construction. Examination of the buildings revealed interesting information concerning the construction methods of former days. Walls were generally 8-in. solid brick or 10- to 12-in. cavity walls. Lime mortar was the rule. Rubble walls and hand-hewn beams were found in almost all basements. Joist and rafter sizes and spacings were quite irregular, and connections were made with wrought iron square nails or even wooden pegs. The inner wythe of cavity walls frequently had wooden members or furring strips embedded in the brickwork, and at least one wall had a layer of sheathing in the cavity, presumably an intuitive attempt at insulation.

Because of the outdated designs of most of the structures, it was thought that the concentrated effort and probable expense involved in load tests would not be warranted by the amount of useful information obtained. One building, however, a former cheese factory in Aultsville, Ontario, was of concrete block construction, about 30 years old, and in good condition. Since its construction was typical of some present day buildings, it was decided to carry out a limited number of simple tests on its walls using equipment immediately at hand.

DESCRIPTION OF THE BUILDING

The building chosen for the test is shown in Fig. 1. The wall seen in the figure consisted of a series of concrete block panels separated by window openings. It was 10 ft 4 in. high (15 courses), made of 8-in. hollow block with a ½ in. of plaster rendering on the inside face. The far side of the building was of similar construction with the addition of a small office extension and loading platforms.

The walls of the building rested on a low concrete foundation wall which was in good condition and showed no signs of weakness or movement throughout the tests. Roof rafters and ceiling joists were carried on a plate bolted to the top of the wall with ½-in. bolts at 7-ft centres (Fig. 2).

The concrete blocks used in the wall were 8- by 8- by 16-in. two-celled units having 2-in. face shells and 1½-in. webs. Probably from age, the units had a high compressive strength averaging 5610 psi on the net area. Mortar joints were ½ in. thick; the mortar appeared to be a mixture of cement and lime, very hard, and generally strongly bonded to the units.

Numerous cracks were noted in the plaster on the inside of the walls that followed the pattern of the mortar joints behind. Since there were no signs of structural movement, shrinkage was presumed to be the cause.

SELECTION OF PANELS

Short masonry walls supported on four sides undoubtedly derive their resistance to lateral forces from a plate action that is difficult to assess due to the non-homogeneity of the materials. In the case of a long wall, however, the strength of the centre portion can be taken as the sum of the strengths of vertical strips supported top and bottom, which are easier to analyze.

To keep the tests simple and to facilitate execution and interpretation, it was decided to test four panels only, and to isolate the panels numbered 1, 2 and 3 in Fig. 1 by knocking out the blockwork above and below the windows. This resulted in three vertical strips of a typical concrete block wall (Fig. 3). The edges were left saw-toothed to avoid damaging the walls by an attempt to trim the edges of the panels. It was thought that the projecting blocks would not affect the tests very much, except by virtue of a little extra weight.

Panel 4 (Figs. 1 and 4) was much larger and represented a different condition. This wall was part of an extension to the main building and was not plastered on the inside. The edge of the panel abutting the original building was not bonded to it, i.e. it was freestanding. The other end was integral with the end wall of the extension. It thus represented a wall supported on three edges, and it was thought that a useful comparison might be made between the strength of this wall and a similar one supported only top and bottom using the strength data obtained from Panels 1, 2 and 3.

The dimensions of the panels are given in Fig. 5.

LOADING METHOD

As previously noted, the tests were done using the material and equipment readily available. For this reason the loading system shown in Figs. 5 (a) and (c) was adopted.

Two horizontal line loads were applied at the third point of each panel 1 to 3 through $2\frac{1}{2}$-in. diameter pipes. These were yoked together by 8- by 8-in. timbers and a steel beam. A 2-in. rod was bolted to the steel beam, passed through a small hole at the centre of the test panel and carried across the building to the far wall. It was passed through the far wall and through a 100-ton centre-hold Simplex jack. Since the deflections of the test panels were expected to be greater than the jack extension, a steel stool was provided below the jack so that the load could be "tied off" with a nut while the jack was retracted and reset. Further 8- by 8-in. timbers below the stool spread the jack reaction over the reaction wall.

At the test panel the loading assembly was supported on rollers to avoid any addition to the normal vertical wall loads (Fig. 6).

For wall panel 4 the line loads were not continuous but nearly so (Figs. 5(b) and 7). In this case they were applied in 5-ft lengths spaced about 1 ft apart along the length of the wall. Four such loads were yoked to one jack. Thus, two jacks were required. The reaction and jacking arrangements on the far wall were the same as previously described for the other panels.

MEASUREMENTS OF DEFLECTION

Deflections were measured by means of two dial gauges (reading to .001) straddling the centre line of the wall on the inside (Figs. 8 and 9). In addition, wires were attached at 6 points on the outside of the wall along the vertical centre line for panels 1 to 3, and at 17 points along horizontal and vertical lines for panel 4 (Fig. 8). These wires with a 1-lb weight at the end of each were carried by a system of pulleys to a marking board (Fig. 7) so that the deflections could be recorded (Scale 1:1) on a chart.

LOADING SEQUENCE

The load was applied to wall panels 1 and 2 in increments of approximately 100 lb at the jack. The load at each increment was held for about 3 minutes while both the dial gauges and deflection board readings were recorded. Loading was discontinued when the load reached a maximum and remained constant, or decreased with increasing deflection. The load was then decreased in three equal steps to zero.

Panel 3 was tested in essentially the same way except that loading was continued to actual collapse. Fig. 10 shows panel 3 under load at an advanced state of deflection.

Panel 4 was loaded through two jacks. The loads on the jacks were kept equal so that the two line loads were at all times essentially uniform along the wall. Loading increments were 100 lb at each jack and deflection readings were taken at each increment. The test was discontinued after the load had reached a maximum.

RESULTS AND DISCUSSION

Panels 1, 2 and 3

Close agreement was obtained between the test results for panels 1, 2 and 3 as can be seen from the load-deflection curves of Fig. 11. (Load is expressed in terms of total load, applied at 1/3 points of the vertical span, per ft length of panel). Cracking occurred at an average load of 254 lb per ft length and the accompanying deflections averaged 0.03 in.

Inspection of the curves shows that the test on panel 2 should have been carried further for a better assessment of the average maximum load between the three panels. The curves for all panels show that after cracking, deflections increased rapidly and although the loads continued to increase, the erratic nature of the curves indicate that there was an intermittent yielding of structural parts. The best estimate of the average maximum load appears to be about 335 lb at a deflection of better than 1 in.

Loading of panel 3 was continued until collapse occurred at a very large deflection (8.93 in.). The portion of the curve beyond maximum load is influenced partly by the characteristics of the loading system which did not apply a constant load for all deflections and is, therefore, perhaps not too significant. It shows that beyond the maximum load equilibrium was possible under successively reduced lateral loads until a limiting deflection was reached. Figure 12 shows vertical centre line profiles at selected loads for panel 3 and is typical as well of panels 1 and 2. The principal significance of these curves is the confirmation of the cracking loads that were otherwise observed visually. Immediately prior to cracking, the deflections at points 3 and 4 were equal as would be expected, and the maximum deflection was, therefore, at the centre line. After the appearance of the crack, however, the deflection at 4 became greater than at 3 and the maximum deflections thereafter occurred at the crack.

In every case cracking of the panels occurred at a joint at the mortar-block interface, i.e., by bond failure. Taking the wall weight (70 psf) and the reaction of the roof on the top of the wall (155 lb per lin ft) into account in the calculations, the average apparent modulus of rupture (f_r) for all wall panels was found to be 31.7 psi (Appendix A). If this value of f_r had been known or assumed in advance, and if the strength of the wall had been calculated without taking into account the vertical loads, a value of 211 lb per lin ft would have been obtained as compared to 254 lb per lin ft as found in the tests. This shows that for an accurate assessment of the strength of a wall, it is not sufficient merely to know the modulus of rupture and the lateral loads; the vertical loads must also be included in the calculations.

On the other hand, if a panel is assumed to have cracks at the same location as those which occurred in the tests, then the maximum lateral load calculated for conditions of stability would be 169.5 lb per lin ft (Appendix B). This is less than the cracking strength obtained experimentally; for the long term strength of a wall, therefore, one must consider whether or not bond capable of resisting tensile stresses can be expected to persist for the life of the building or whether weathering or accident is likely to cause a crack. In this concrete block building the bond appeared to be good after 25 years life.

Since for the given vertical loads on one of the test panels, the lateral loads causing cracking can be shown to be greater than the lateral stability load of a cracked panel, collapse of the panels should have followed immediately after cracking. Nevertheless, they continued to resist increased loads up to a maximum about 30 per cent greater than the cracking load or about double the above-mentioned stability load. This can only be accounted for by the presence of external forces not previously taken into account in the calculations. The only apparent source of such a possible force is that the torsional resistance of the roof, plate and anchor bolt combination exerted a negative movement at the top of the wall. Whether or not the moment resistance of the connections is a practical matter or reliable enough to be taken into account in design needs further study. In planning further tests, however, the apparatus to be used must provide for its evaluation to assist in the interpretation of results.

Panel 4

As would be expected, the cracking of wall panel 4 began at the free end of the wall (Fig. 8) and progressed towards the end supported by the abutting wall. In this case cracking occurred in the joint below the fifth course, from the top rather than below the sixth as in panels 1, 2 and 3. In Fig. 13 the load-deflection curves are plotted for sections A, B, C and D located at the free end of the wall, the quarter points and the centre line.

The first crack appeared at section A at a lateral load of 244 lb per ft of wall as indicated by 'x' on the curve. This is only slightly less than the cracking load for the previous panels showing that the deflection at this section was practically unaffected by the abutting wall at the far end. Sections B, C and D were progressively stiffer, since they were successively nearer to the support; when cracking occurred at A the deflection at D was about 25 per cent of that at A.

After the first cracking the crack progressed towards D reaching that section when the load reached the value 'y' shown on the curves. Finally at load 'z' the complete crack pattern shown in Fig. 8 was developed and the load began to decrease. When the test was discontinued the deflection at D was about 80 per cent that at A. The gradual equalization of deflections is also illustrated by the horizontal wall profiles plotted in Fig. 14 for selected loads.

If the effect of the abutting cross-wall were not considered, it would be expected that panel 4 would reach a maximum load per ft of 335 lb per lin ft, taking the results of panels 1, 2 and 3 as the criteria (actually about 5 per cent less since panel 4 was not rendered on the inside). The actual average maximum load from the test was 348 lb per lin ft which is only a small increase.

The above observations indicate that an abutting wall may influence the stiffness and cracking strength of a wall over a horizontal distance about three times the wall height, but that the maximum strength is likely to be little affected except near the cross-wall. It appears from this that the practical maximum spacings of cross-walls or abutments will depend on whether cracking or maximum load is assumed as the criterion.

CONCLUSION

The tests on the concrete block building at Aultsville indicate that the strength of the vertical strips of the walls was greatly influenced by three factors: the strength of the mortar-block bond, the effect of the vertical loads and the moment resistance of the connections. If comprehensive field or laboratory tests are undertaken, they should be planned to evaluate the relative importance of these sources of strength over a wide range of practical cases. Consideration should also be given to determine the probable reliability of bond over long periods.

The tests also indicate that cross-walls or abutments may not greatly increase the maximum strength of some walls, but their influence on the cracking strength may be greater.

While the results of this investigation may not appear unexpected, they are a reasonably clear starting point for the Divisions' future investigations into the lateral strength of masonry walls, a subject on which more information is needed despite the long use of this type of construction.

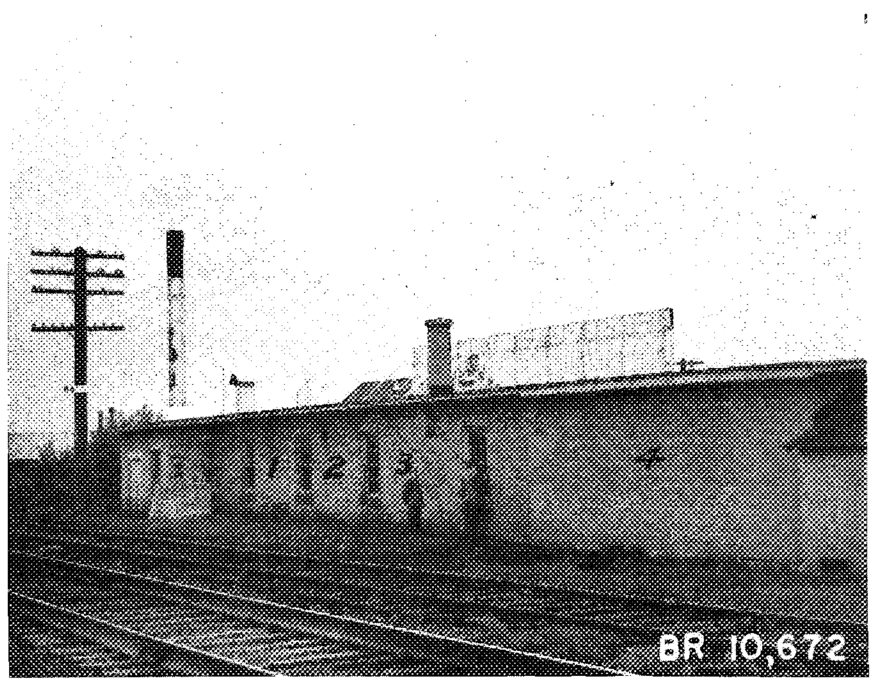

Figure 1 Cheese factory, Aultsville, Ontario. (Wall chosen for tests with panel numbers shown)

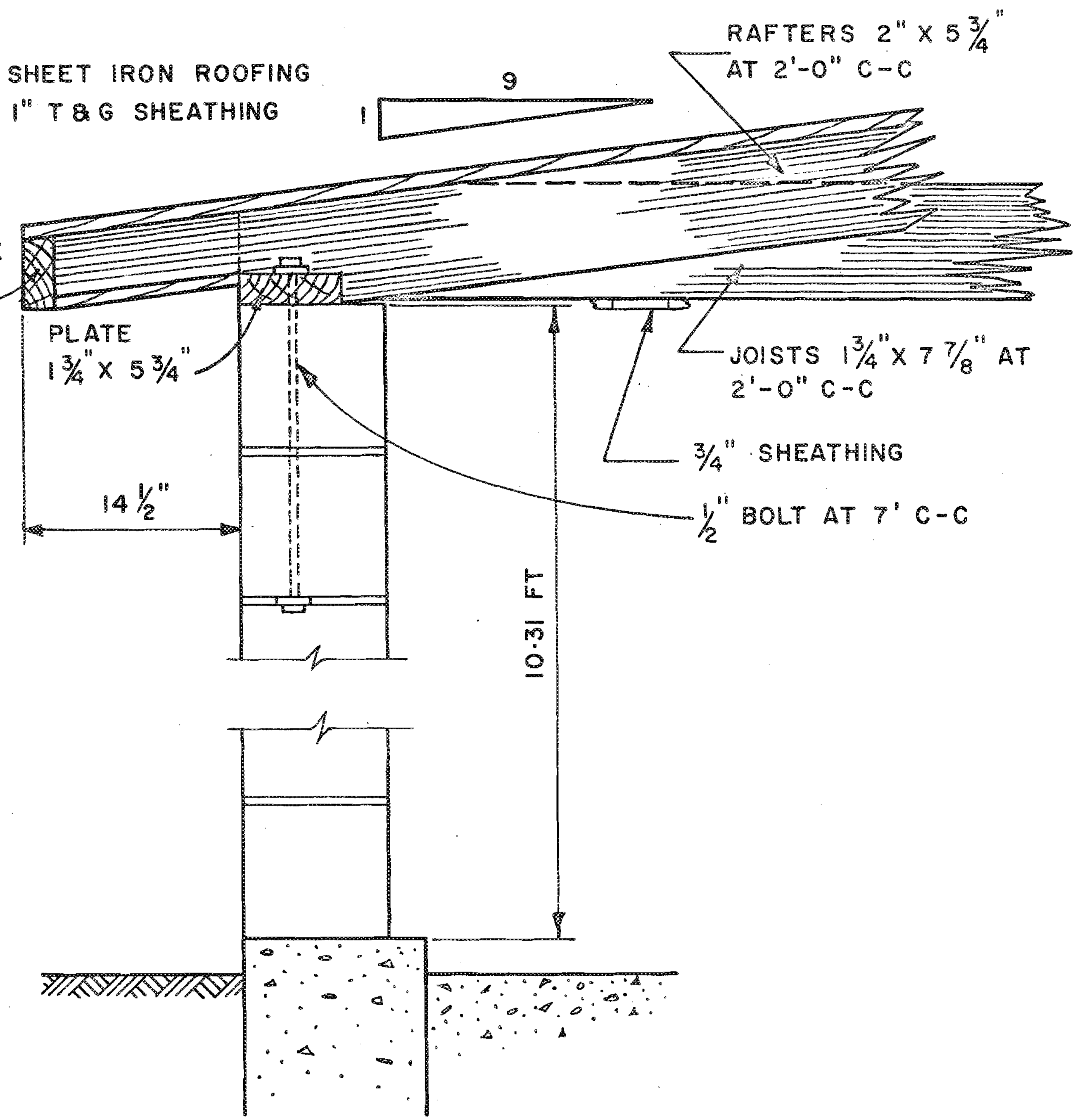

FIGURE 2

TYPICAL CROSS-SECTION OF TEST WALL

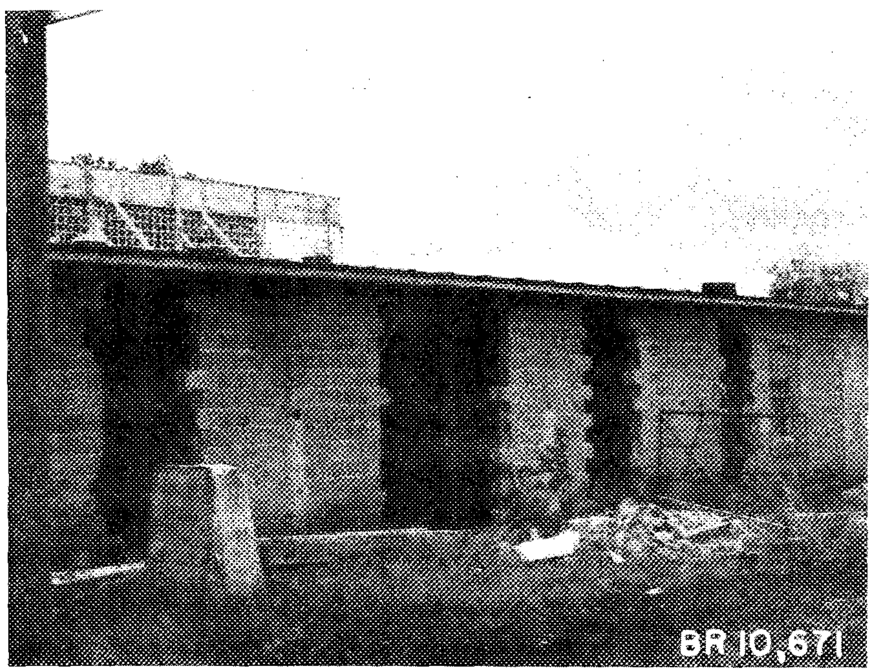

Figure 3 Test panels 1, 2 and 3 after isolation from remainder of wall

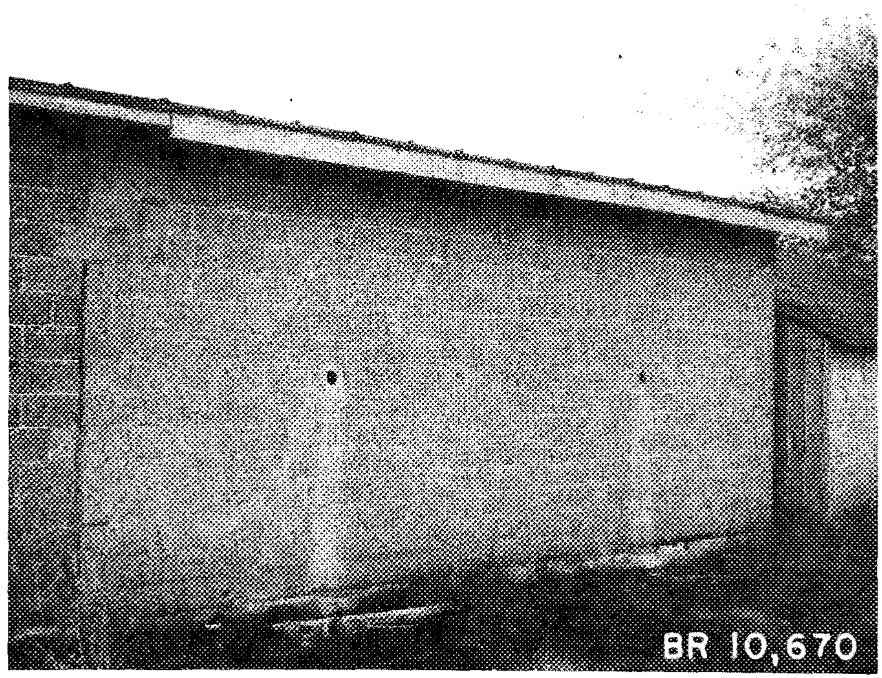

Figure 4 Wall panel 4

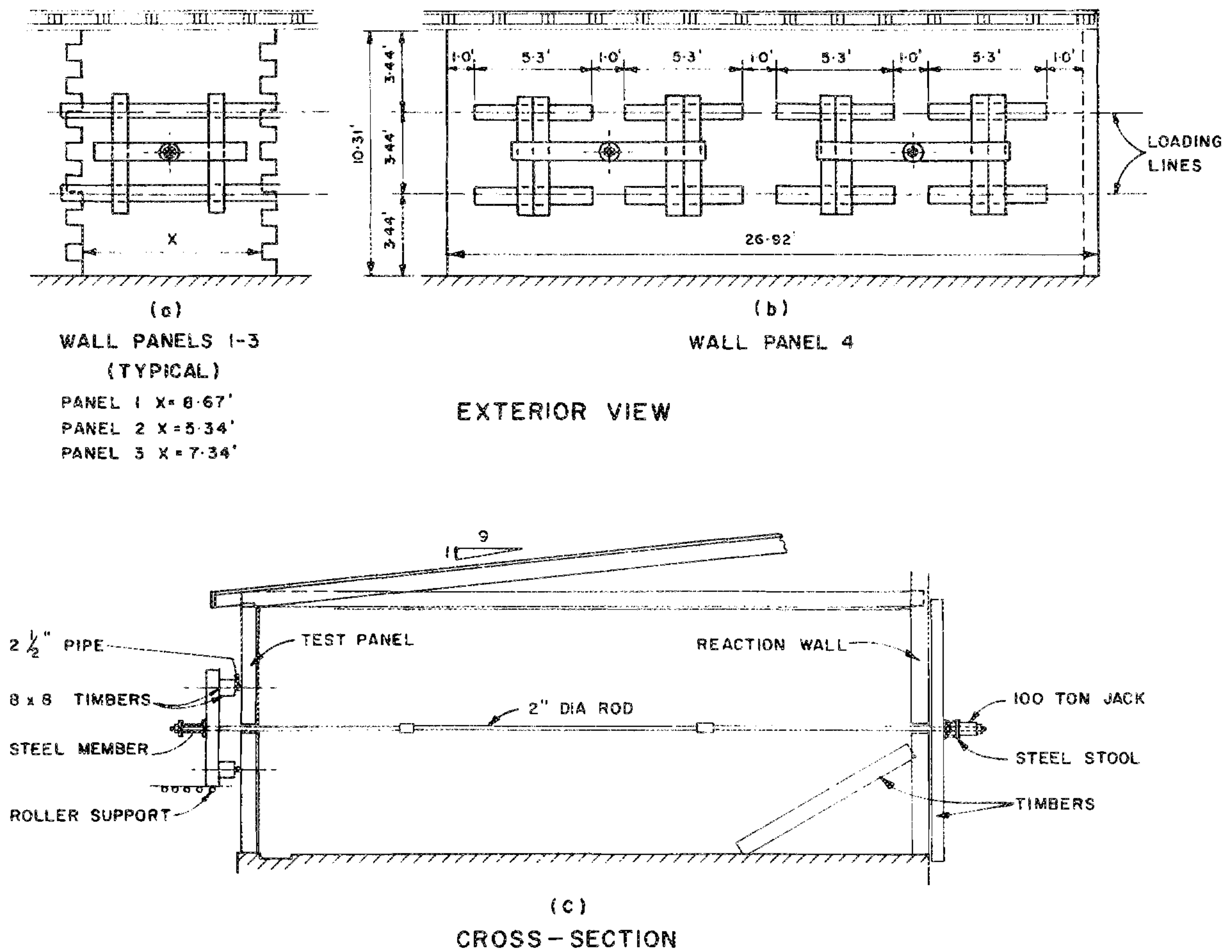

FIGURE 5
WALL PANEL LOADING ARRANGEMENT

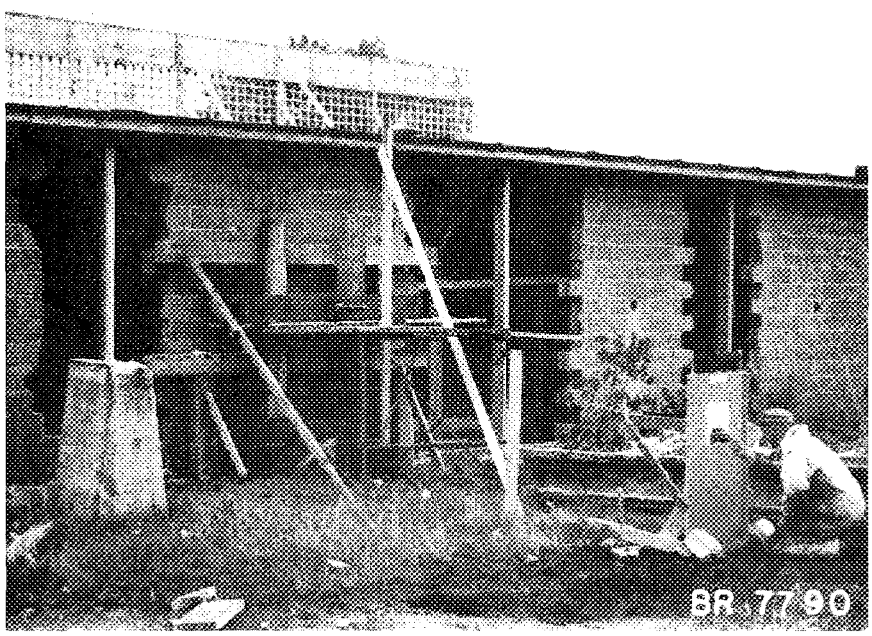

Figure 6 Wall panel 1 showing loading and deflection apparatus

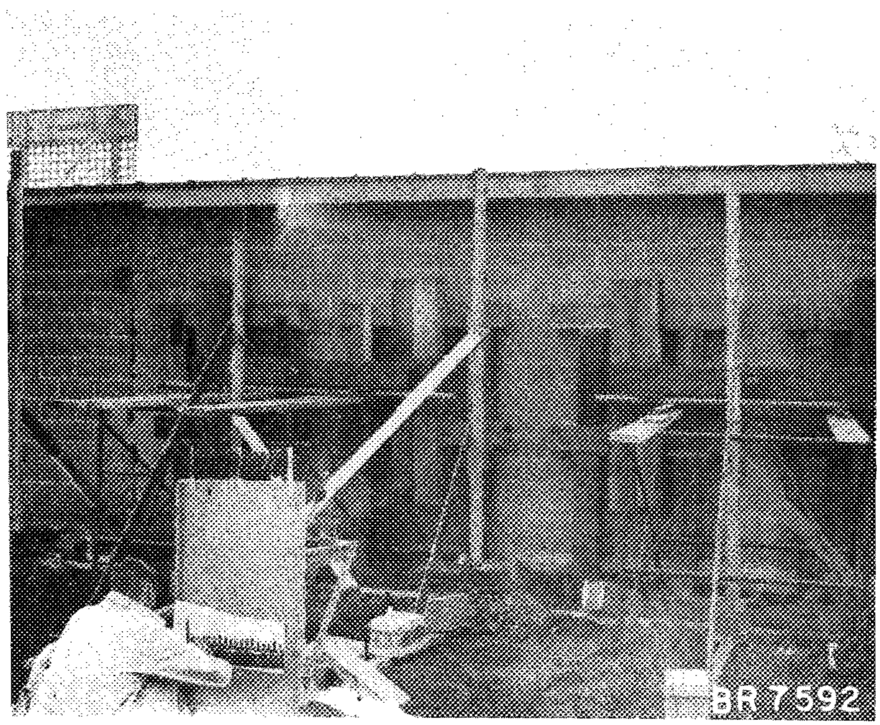

Figure 7 Wall panel 4 showing loading and deflection apparatus

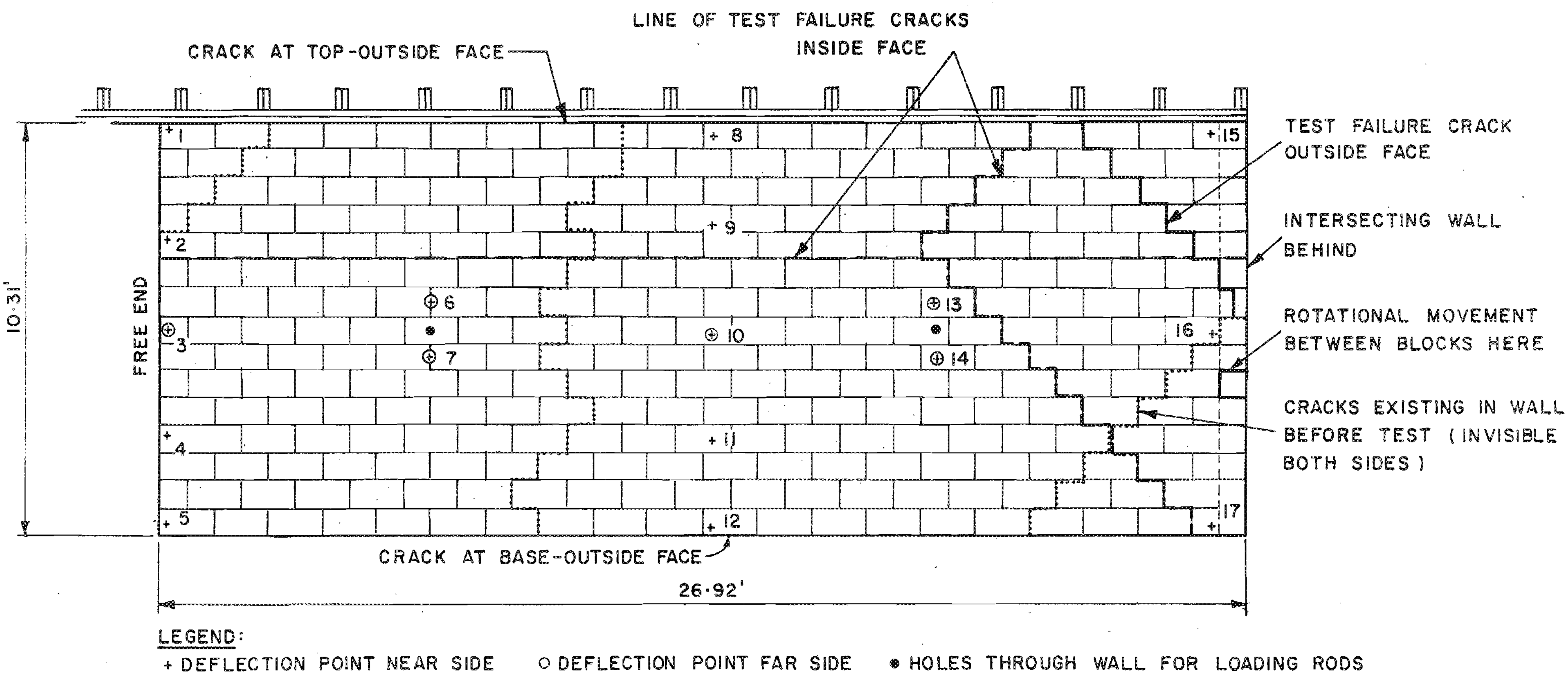

FIGURE 8

TEST WALL PANEL NO. 4, EXTERIOR VIEW SHOWING DEFLECTION MEASUREMENT POINTS AND CRACK PATTERN AFTER LATERAL LOAD TEST

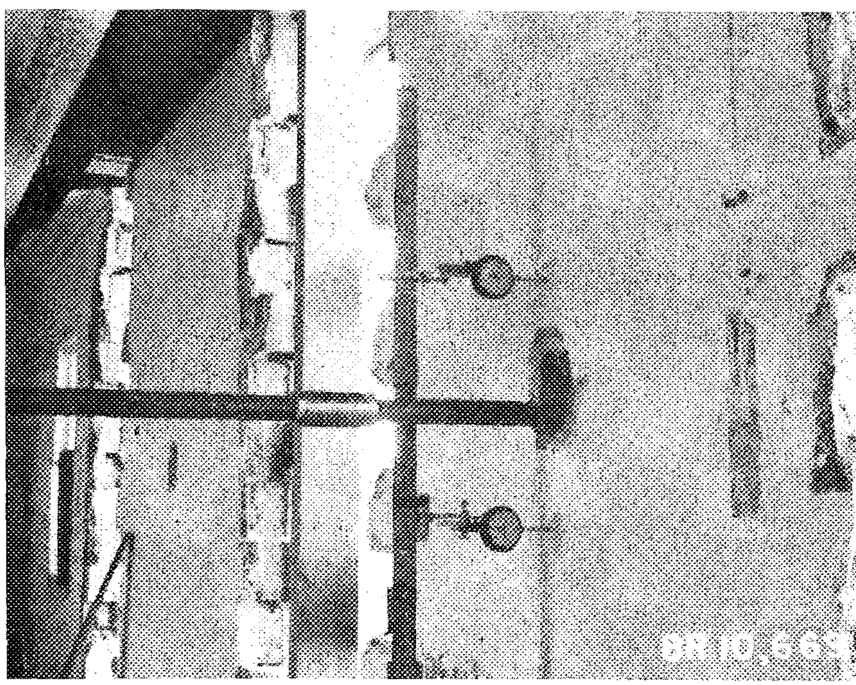

Figure 9 Dial gauges on inside face of a test panel

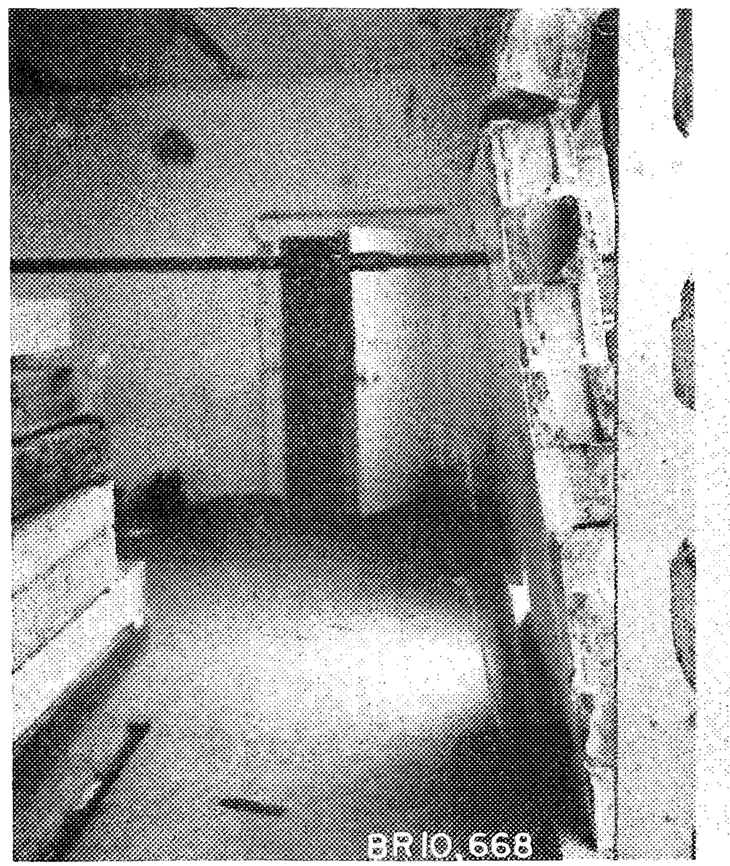

Figure 10 Wall panel 3 just before collapse

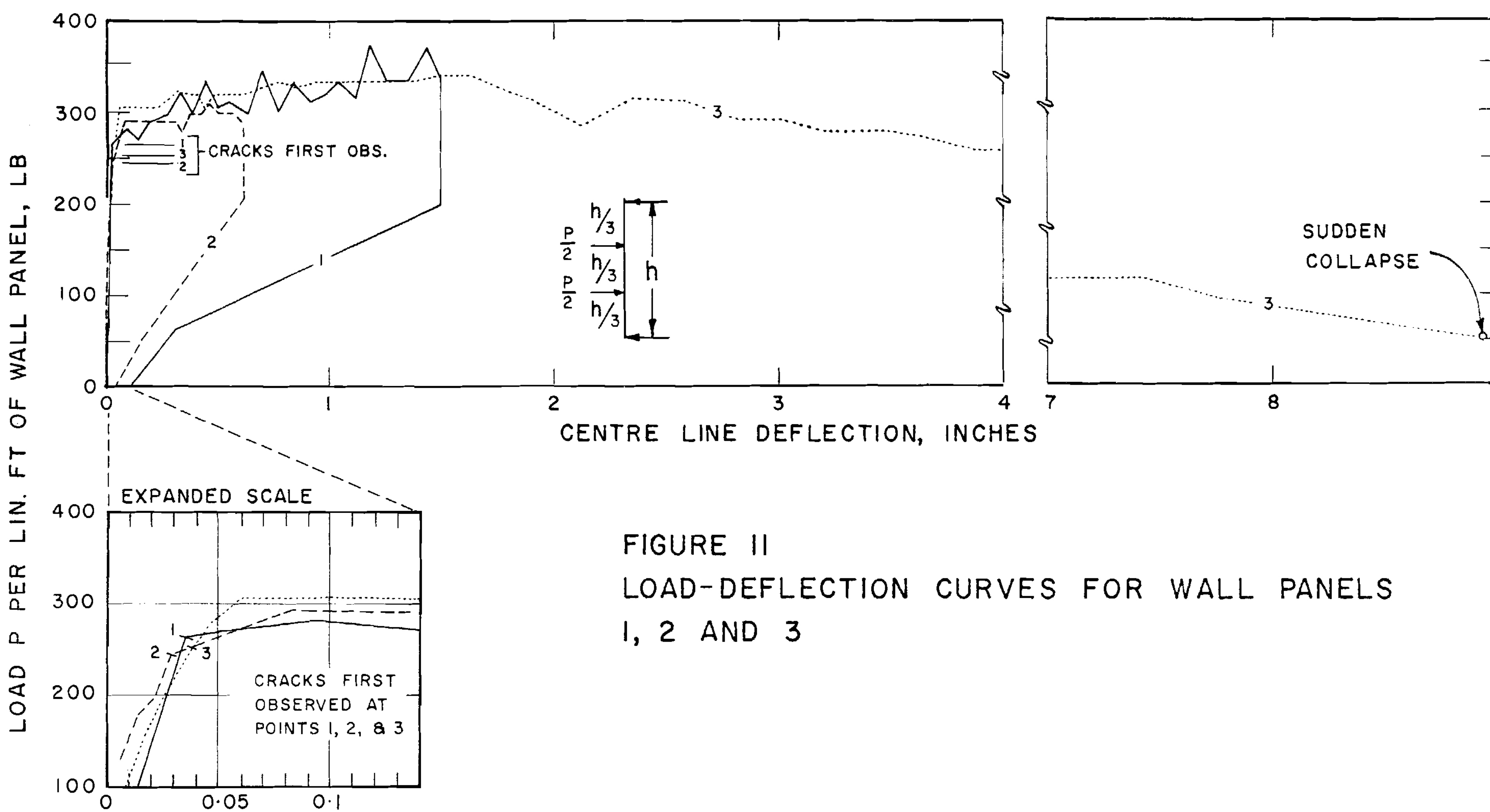

FIGURE 11
LOAD-DEFLECTION CURVES FOR WALL PANELS 1, 2 AND 3

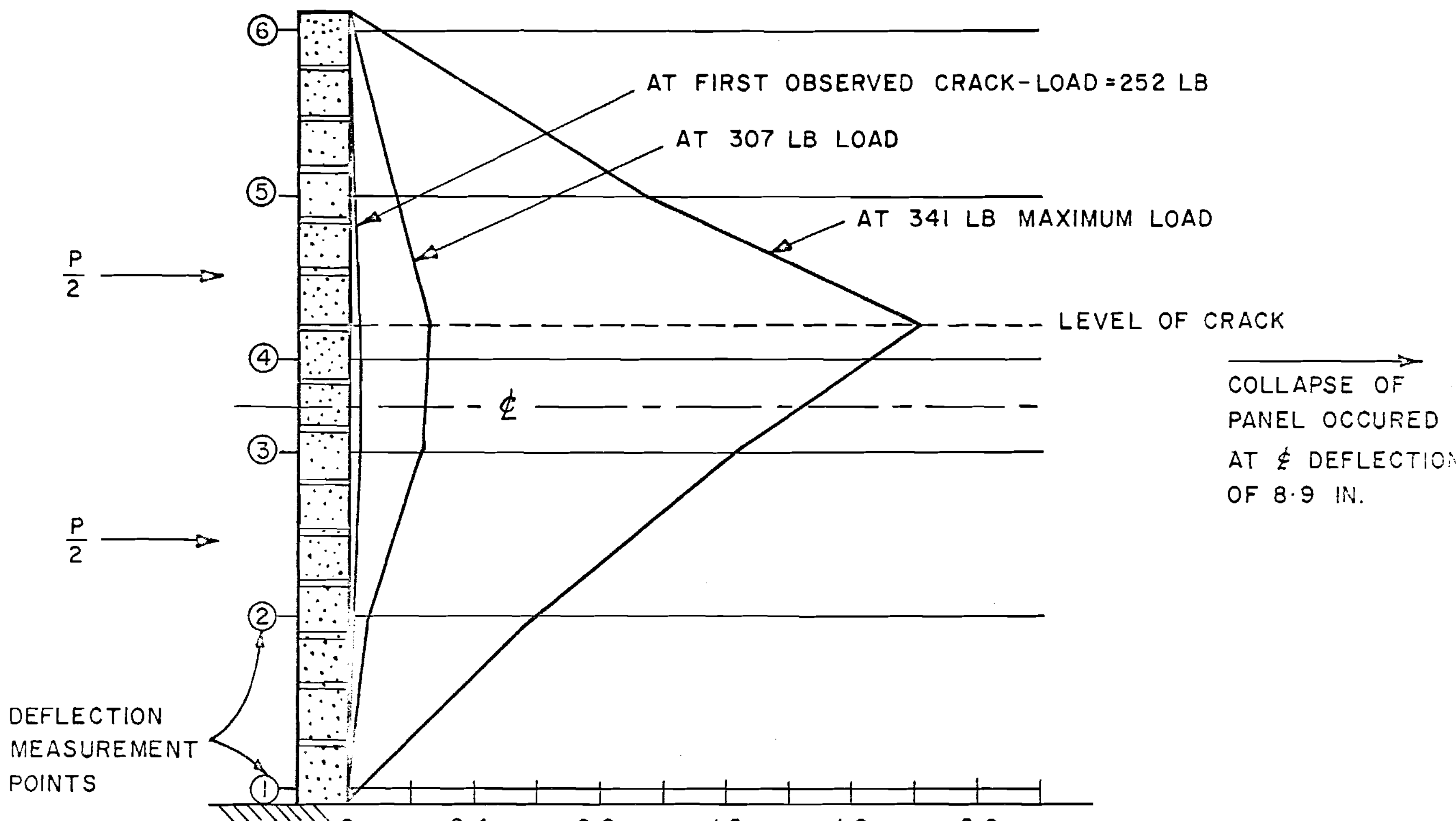

FIGURE 12
PROFILES OF WALL PANEL NO. 3 UNDER TEST

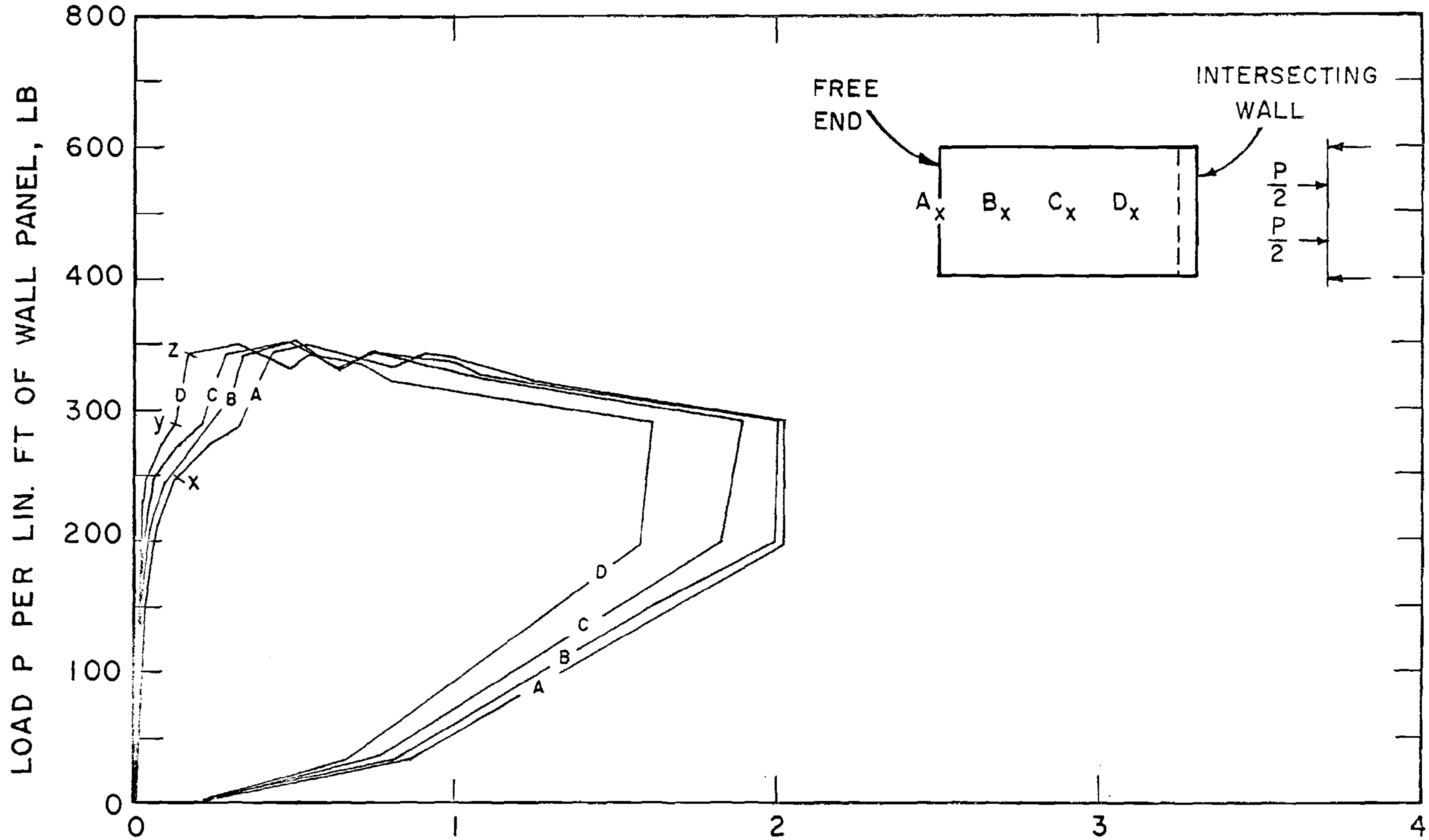

FIGURE 13

LOAD-DEFLECTION CURVES FOR WALL PANEL NO. 4

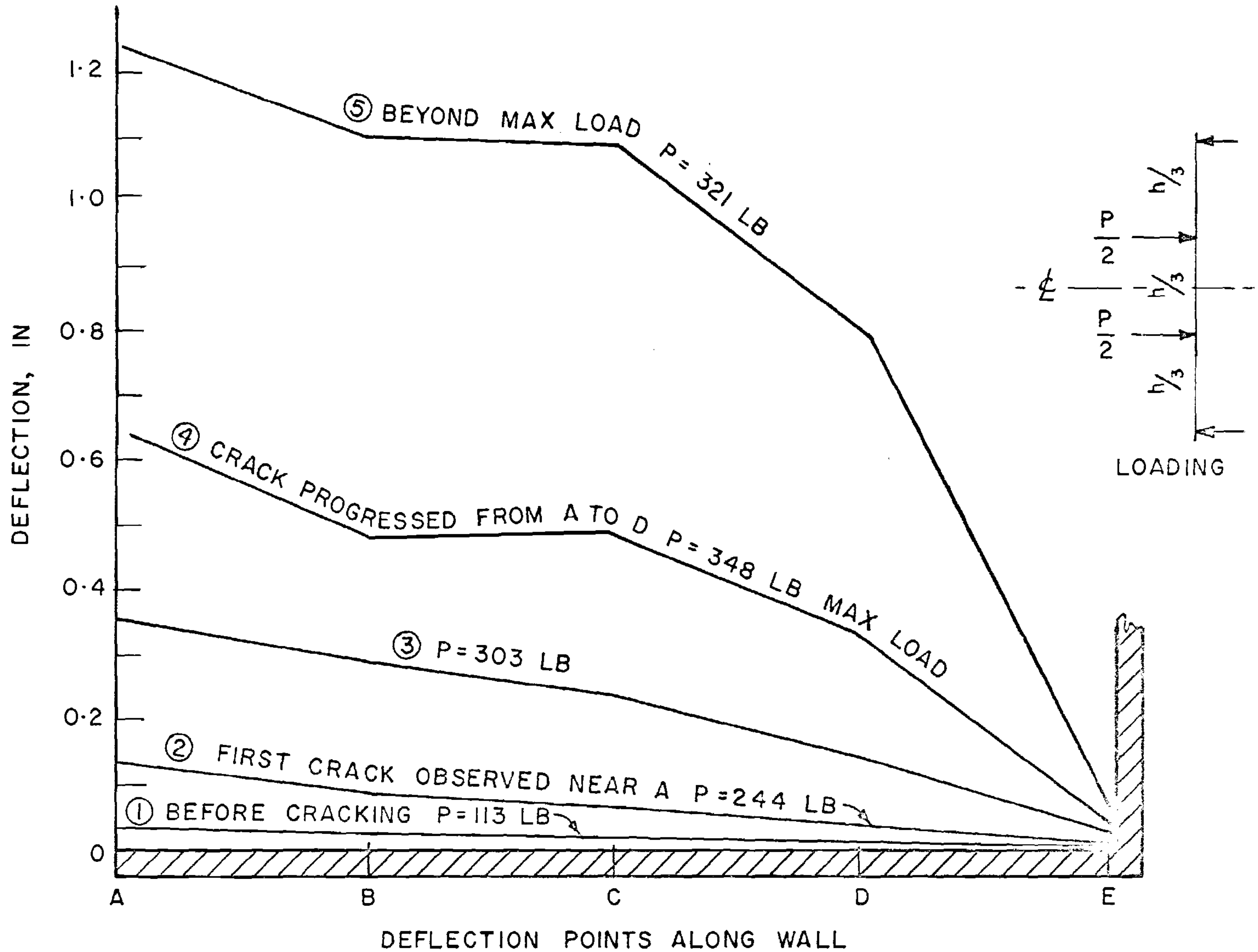

FIGURE 14
HORIZONTAL PROFILE ALONG VERTICAL ℄ OF WALL 4 AT VARIOUS LOADS

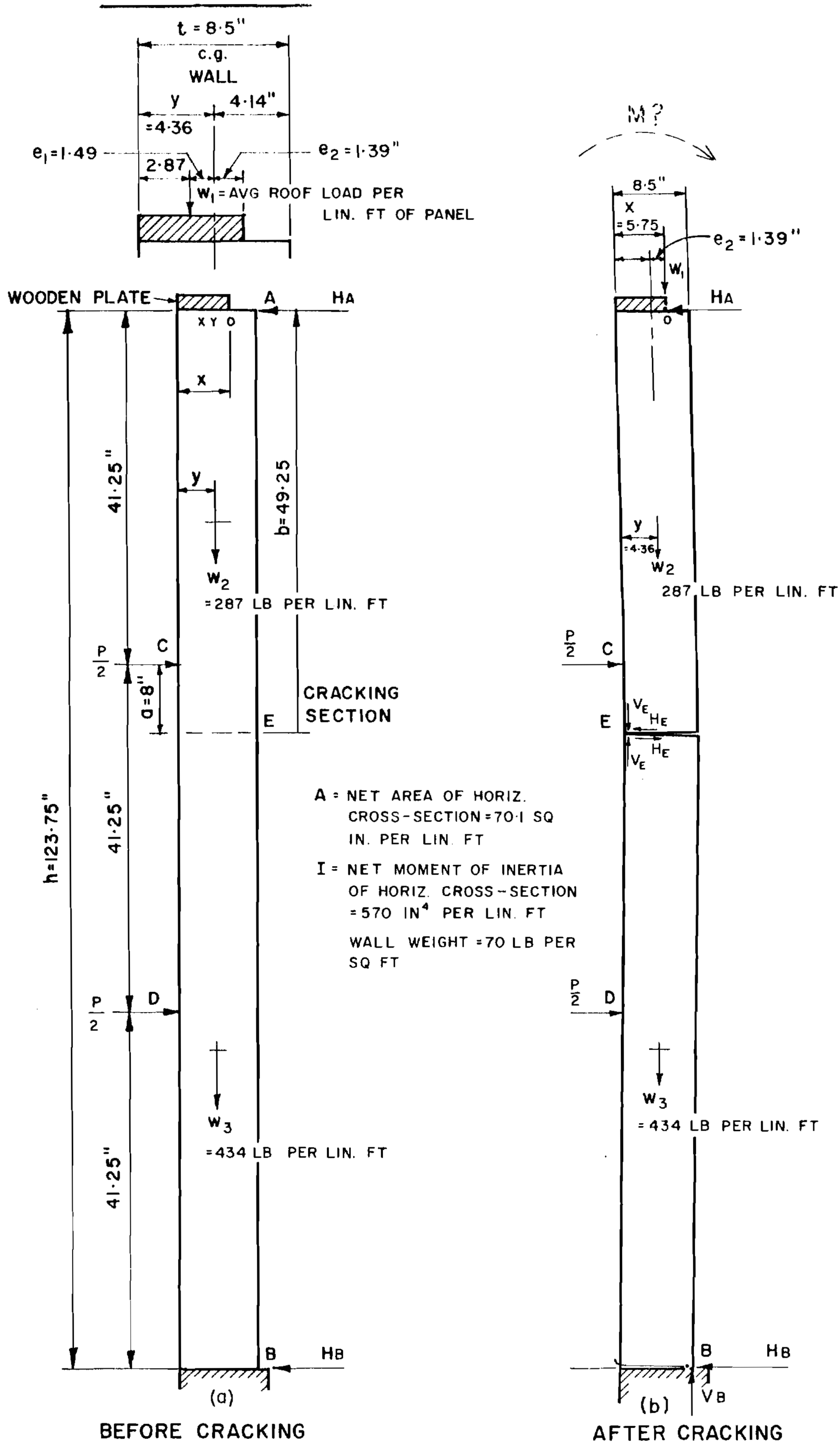

FIGURE 15

LOADS AND REACTIONS ON WALL PANELS 1, 2, AND 3

APPENDIX A

Wall Panels 1, 2 and 3 - Calculations at Cracking Load

Cracking of the wall panels occurred at small deflections of about 0.03 in. For stress calculations the geometry of the wall and the loading system prior to cracking may be considered unchanged except for the location of the vertical loads and reactions at the top and bottom of the panels.

Rotation of the top of the wall will cause the roof reaction to move from its theoretical original position 'x' (Fig. 15(a)) at the centre of the plate towards the inner edge of the plate at 'o'. Its exact position is not known but the error will be small if for the purposes of these calculations it is assumed to have moved to the centre of gravity of the wall at 'y'.

Notation:

w_1 = the reaction of the roof on the top of the panel

w_2 = the weight of the panel above the section considered

A = the net area of the horizontal cross-section of the panel

c = distance to the outer fibre of the panel from its centre of gravity

I = the moment of inertia of the net cross-sectional area of a panel

M = the moment acting on a horizontal cross-section of the panel

P = the total load applied to the panel at 1/3 points

Other symbols are defined in Fig. 15.

Assuming no bond between the base of the wall and the foundation, the tensile stress (f_t) at any cross-section may be expressed by

$$f_t = - \frac{(w_1 + w_2)}{A} + \frac{Mc}{I}$$

Maximum tensile stress will occur where the maximum moment is combined with the least vertical load, i.e. at the upper loading point. In the tests, cracking occurred at the first mortar joint below the upper load, i.e. at E.

At section E,

$$M = \frac{Ph}{6}$$

and

$$f_t = - \frac{(w_1 + w_2)}{A} + \frac{Phc}{6I}$$

The average reaction w_1 of the roof on the wall panels was calculated to be 155 lb per lin ft of panel. The weight of the wall was found by measurement to be 70 lb per sq ft including $\frac{1}{2}$ in. of rendering on the inside face. Hence

$$w_2 = b \times 70 = \frac{49.25}{12} \times 70 = 287 \text{ lb per lin ft}$$

of panel. The average cracking load for the panels was 254 lb per lin ft. At cracking, f_t became equal to the modulus of rupture f_r and

$$f_r = - \frac{(155 + 287)}{70.1} + \frac{254 \times 123.75}{6} \times \frac{4.14}{570}$$

$$= - 6.3 + 38.0$$

$$= 31.7 \text{ psi.}$$

Assuming that the value of f_r were known in advance and that the vertical loads were neglected, then on the basis of bending strength only the load required to crack the wall would be calculated as follows:

$$P = \frac{6\ f_r\ I}{hc} = \frac{6 \times 31.7 \times 570}{123.75 \times 4.14} = 211 \text{ lb per lin ft}$$

The estimate, therefore, would have been about 17 per cent too low.

APPENDIX B

Wall Panels 1, 2 and 3 - Calculations Regarding Stability After Cracking

After a wall panel has cracked, further rotation of the upper and lower segments will take place immediately. Vertical reactions w_1 and V_B will shift to their extreme positions at 'o' and B (Fig. 15 (b)) and will be concentrated over a small enough area to be considered as acting along a line or a hinge. A "hinge" will also occur at E.

Overturning of the two segments should occur immediately following cracking, unless the shifts of w_1 and V_B are sufficient to restore the equilibrium lost by the disappearance of the moment of resistance at section E or unless other forces come into play.

For panels 1, 2 and 3, the lateral load that would cause collapse of a cracked wall may be calculated as follows:

$$\Sigma M_B = 0$$

$$\frac{P}{2}\frac{(h)}{(3)} + \frac{P}{2}\frac{(2h)}{(3)} - w_1\ (t-x) - w_2\ (t-y) - w_3\ (t-y) - H_A\ (h) = 0$$

$$\frac{123.75\ P}{6} + \frac{247.5\ P}{6} - 155\ (8.5 - 5.75) - 287\ (8.5 - 4.36)$$
$$- 434\ (8.5 - 4.36) - 123.75\ H_A = 0$$

$$H_A = .5\ P - 27.5 \qquad (1)$$

Similarly, taking the free body of the upper segment of the wall panel,

$$\Sigma M_E = 0$$

$$\frac{P}{2}\ (a) + w_2\ (y) + w_1\ (x) - H_A\ (b) = 0$$

$$\frac{8P}{2} + 287\ (4.36) + 155\ (5.75) - 49.25\ H_A = 0$$

$$H_A = 0.0813\ P + 43.5 \tag{2}$$

Solving (1) and (2)

$$P = 169.5 \text{ lb per lin ft of panel.}$$

This load is less than the load causing cracking of the panels, and on this basis collapse should have occurred immediately following cracking. Since the panels did not fail, an explanation can be made only on the basis of an additional force or moment that was not taken into account in the bond calculations. In these tests the torsional resistance of the top plate, the anchor bolts and the roof must have applied a negative resisting moment at the top of the panel.