NATIONAL BUREAU OF STANDARDS REPORT

10 164

TEST AND EVALUATION OF THE PREFABRICATED LEWIS BUILDING AND ITS COMPONENTS
PHASE I PART 2
FULL-SCALE BUILDING TESTS

NBS

U.S. DEPARTMENT OF COMMERCE
NATIONAL BUREAU OF STANDARDS

NATIONAL BUREAU OF STANDARDS

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TEST AND EVALUATION OF THE PREFABRICATED LEWIS BUILDING AND ITS COMPONENTS
PHASE I PART 2
FULL-SCALE BUILDING TESTS

By
Thomas W. Reichard and Edgar V. Leyendecker
Structures Section
Building Research Division
Institute for Applied Technology
for
Commanding Officer
Naval Civil Engineering Laboratory
Port Hueneme, California

IMPORTANT NOTICE

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Abstract

A two bay modification of the three bay Mark III Lewis Building was tested in the laboratory to determine gross structural inadequacies in the building. Measured vertical and lateral load capacities are reported for the two bay structure and estimated for a three bay structure.

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TEST AND EVALUATION OF THE PREFABRICATED LEWIS BUILDING AND ITS COMPONENTS
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by
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and
E. V. Leyendecker

1. Introduction

1.1 Objective

The primary objective of this study was to evaluate the performance of a Mark III model of the Lewis Relocatable Structure under simulated wind and snow load. A secondary objective was to evaluate the relocatable feature of the structure.

1.2 General

The Mark III model building was prefabricated in Florida using 4 ft wide sandwich panels described in the Phase I Part 1 report[1][1/] and was made by the manufacturer of the Brand A panels described in that report.

[1/] Numbers in brackets refer to references in the bibliography.

The basic design of the building was detailed in NAVFAC Drawing No. 065-012, dated November 1, 1968.

The erection and structural details for the building were given in a set of 8 drawings entitled "Relocatable 20 ft 6 in x 48 ft 9 in Lewis Building for the Department of the Navy." This set of drawings, dated February 17, 1969, was prepared by the manufacturer of the building.

The structure was designed as a 16 ft long module, 20 ft wide. Three such modules made up the 20 ft x 48 ft building. Because of space limitations in the test laboratory a two module, 20 ft x 32 ft building was erected for these tests.

1.3 Scope

The purpose of the test program was to discover any gross strucutral inadequacies in the building design and not to develop complete data on all aspects of the structural performance.

Two types of loads were imposed on the building. One type was a simulated snow load on a 12 ft long section near the center of the building. The second type was a simulated wind load applied to one long wall of the building.

1.4 SI Conversion Units

In view of present accepted practice in this country in this technological area, commom U.S. units of measurement have been used throughout this paper. In recognition of the position of the USA as a signatory to the General Conference on Weights and Measures, which gave official status to the metric SI system of units in 1960, we assist readers interested in making use of the coherent system of SI units, by giving conversion factors applicable to U.S. units used in this paper.

Length	1 in =0.0254* meter
	1 ft =0.3048* meter
Area	1 in^2 =6.4516* x 10^{-4} $meter^2$
	1 ft^2 =0.09290 $meter^2$
Force	1 lb(lbf) = 4.448 newton
	1 kip = 4448 newton
Pressure, Stress	
	1 psi = 6895. newton/$meter^2$
	1 ksi = 6.895×10^6 newton/$meter^2$
Mass Volume	
	1 lb/ft^3 (lbm/ft^3) = 16.02 kilogram/$meter^3$
Moment	
	1 kip-in = 113.0 newton-meter

*Exactly

2. Components and Erection of the Test Building

2.1 Components

The major components of the building were:

1. 4 ft x 8 1/2 ft wall panels.
2. 4 ft x 20 1/2 ft floor panels.
3. 4 ft x 11 1/2 ft roof panels.
4. 16 ft long ridge beams.
5. 20 ft long, gable shaped, transverse beams.
6. Triangular 20 ft 6 in gable end member.
7. Eight different aluminum extrusions.
8. Aluminum "Eye" beams (extruded).
9. Extruded rigid vinyl "locking cleat."

2.1.1 Panels

The wall, floor, and roof panels were three in thick, symmetrical sandwich panels. The sandwich core was 11 percent phenolic impregnated, 99 lb kraft paper fabricated into a 3/4 in cell size honeycomb. The skins on the floor panels were .050 in (nominal) mill-finish, sheet aluminum. The skins on the wall and roof panels were 0.024 in prefinished, stucco embossed, coiled sheet aluminum. The skins were

3105-H264 aluminum with a yield strength of 24,000 psi and an ultimate strength of 26,000 psi. The skins were bonded to the honeycomb with a synthetic rubber adhesive.

The floor panels were made with a special aluminum channel, edge-member surround (extrusion G, figure 2.2). The paper honeycomb was exposed at the edge of the roof and wall panels.

The lengthwise edges of the wall and roof panel skins were bent into a "J" shaped, internal lock groove during fabrication. Figure 2.1 indicates the use of the rigid vinyl extrusion to connect the "J" shaped edges of two panels together.

2.1.2 Aluminum Extrusions

The purpose, shape and scaled size of the extrusions used in the building are shown on figure 2.2. Extrusions A, B, and G were used as parts of the gable end member, and the ridge and transverse beams in addition to the uses indicated on figure 2.2. The extrusions were aluminum alloy 6063-T6.

2.1.3 Ridge and Transverse Beams

The ridge and transverse beams were fabricated from long pieces of the sandwich panel material described in section 2.1.1. Extrusion A was used as the bottom flange of both types of beams as well as the top and end flanges of the transverse beam. Extrusion B was used as the top flange of the ridge beam. Extrusion G was used at the ends of the transverse beam.

The extrusions on the beams were connected to the sandwich panel material with 1/8 in diameter aluminum pop rivets at about six in on centers.

2.1.4 Gable End Member

In this model of the building the wall panels are the same height at the ends as on the sides. To close in the gable ends full width, 20 ft 6 in long, triangular members were fabricated to fit between the top of the walls and the roof. These members were fabricated from the 3 in thick sandwich panel material (described in section 2.1.1) and edged with aluminum extrusion A (figure 2.2). Triangular parts made by welding 3 pieces of 1/8 in thick aluminum plate together were used at the eave ends of these members. The individual

parts were fastened to the sandwich panel material with 1/8 in diameter aluminum pop rivets.

2.1.5 Aluminum "Eye" Beams

Aluminum "Eye" beams served as the foundation for the building. These beams were extrusions obviously made for some other purpose as they had several special purpose projections on the web and one flange. Neglecting these projections the dimensions of the beams were:

Depth - 10 in

Width - 4 in

Web thickness - 3/32 in

Flange thickness - 5/32 in

Weight - 2.56 lb/ft (Including projections)

Aluminum type - 6063-T5

2.2 Building Erection

2.2.1 General

The building was prefabricated in Florida and trucked to Gaithersburg, Maryland in five crates (figure 2.3). The crates and their contents were received in what appeared to

be good condition. During the testing of the building some moisture was observed in the floor panels, consequently three floor panels were cut open following the structural tests. Water ran out of all three when cut open, but the core was obviously wet in only one. This core was saturated with water, but only on one end of the panel. The source of the water is not known since the floor panel cores could not be inspected as received due to the factory installed closure extrusions.

The 20 ft x 32 ft portion of the building was erected by laboratory personnel on a reinforced concrete test frame cast on the floor of a large environmental test chamber (figure 2.4). The aluminum "Eye" beams were connected to nine steel-plywood plates anchored to the concrete test frame (figures 2.4 and 2.5). The plates simulated piers called for by the building design.

The floor panels were placed, starting at one end of the building, after the "Eye" beams were connected to the "pier" plates. Six 1/4 in aluminum stove bolts were used to fasten the first floor panel at each intersection with the longitudinal "Eye" beams (three bolts on each 20 1/2 ft side). Subsequent panels were laid in order by connecting the edges of the factory installed extrusion G (figure 2.2) in a

tongue and groove fashion. Three 1/4 in bolts were used to connect the exposed side at the floor beam intersections as shown in figure 2.6.

After the floor panels were in place extrusion D was fastened to the floor with stainless steel No. 8 and No. 10 sheet metal screws. These screws were placed in a double row along the extrusion at six in on centers in each row.

The wall panels were set into extrusion D starting at a building corner (figure 2.7). The rigid vinyl extrusion was inserted, from the top, into each vertical wall joint except at the corners where extrusion H was used. Stainless steel No. 8 sheet metal screws at six in on centers fastened both faces of each panel to extrusion D (figure 2.8).

Extrusion A was placed on the top of the end wall panels and extrusion B on the top of the side walls.

The gable end members were then placed on top of the end walls. These members were fastened to the end wall extrusion with 1/4 in aluminum stove bolts at 12 in on centers, inside and out (figure 2.9).

The transverse beam and the longitudinal ridge beams were then placed and connected to each other and the gable end members with 1/4 in stove bolts (figures 2.24a and c).

The roof panels were then installed using extrusion E at the ridge and extrusion F at the eave. No. 8 sheet metal screws were used at 6 in on centers (inside and out) to fasten the extrusions to the panels.

Five windows and two doors were used in this building. These components came prehung in special wall panels.

Figure 2.10 is a picture of the south end of the completed building through the exterior doorway of the test chamber. Figure 2.11 is a picture of the building after disassembly and removal from the test chamber.

2.2.2 Caulking of the Joints

All through-wall joints and connections were liberally caulked prior to placing two parts together with the exception of the lengthwise joint between the wall panels and between the roof panels. These lengthwise panel joints were sealed only by the extruded vinyl locking cleat (figure 2.1). A fine seam of caulking was also placed on many of the panel-to-aluminum extrusion connections after assembling.

The caulking material came in gun cartridges and was a white "vinyl latex caulk" soluble in water until set.

2.2.3 Problems in Erecting the Building

1. Tolerances - Many of the problems encountered in erecting the building were due to dimensional incompatibility, some of which could have been eliminated by providing larger tolerances.

a) Floorlength - The length of the floor was greater than the length of the wall panels. This would not have been a problem except that the skirt of the floor extrusion (extrusion D) was designed to extend down over the floor panels. In this instance the skirts on the end wall extrusions had to be cut off.

b) Gable End Member - Another example of a dimensional problem was the triangular gable end member. Figure 2.9 illustrates this problem encountered when making the roof-wall corner connections. As can be seen the gable end member was so high at the corner that the extrusion along the top of the side wall had to be moved up about 3/4 in to line up with the gable end. This meant that the vertical loads from the roof were transmitted by the sheet metal screws instead of by bearing to the wall skins.

c) Wall Panels - Another example was in the length of the wall panels. The building was designed so that vertical roof loads would be transmitted through the extrusions (A or B) to the wall panel skins by bearing. Note the shoulders on the inside of these extrusions (figure 2.2) where the skins are supposed to bear. Actually the wall panels were unequal in length (Some as much as 3/8 in longer than others) so that the loads were transmitted through the screws at many locations.

d) Ridge Beam to Gable Connection - A time consuming problem was found when connecting the ridge beams to the gable ends. Numerous trial fittings and cuttings were made before the final connections were made.

2. Production errors - Although some of the problems listed above could be termed production errors the following problems were caused by human error in fabrication.

a) Slip Sheeting - The most time consuming problem encountered in erecting the building was caused by a production error termed "slip sheeting." This term means that one skin of a sandwich panel is not lined up with the other skin. It could be a linear or angular translation. Angular translation was the more difficult problem here. Sometimes the vinyl locking cleat could not be forced all the way in without heavy hammering. This procedure tends to split the

vinyl cleat. In one instance the cleat could not even be hammered in all the way. When it was decided to replace the bad panel the two partially connected panels could not be separated without sawing.

b) Cut Outs - Poor production procedure was particularly evident in the cut-outs in the panels containing factory installed windows and for the duct system through the transverse beams. The window cut-outs were not always square with the panels and were oversize. One window cut-out was so large that some of the window-fastening screws did not even contact the panel skins.

In making the cut-outs for the duct through the transverse beams the saw cuts were continued past the cut-out corners. This resulted in stress-concentrating notches at each corner of the cut-out. The transverse beam fractured at one of these corners in a load test described in a later section.

c) Delaminated Panels - Some of the floor panels were defective which caused some erection hold ups. Poor production procedures in making the panels caused delamination or loss of bond between the skins, extrusions and the core. The panel skins were subsequently fastened to the edge extrusions with sheet metal screws.

d) Core Moisture - Three floor panels were cut open following the building tests. Two of these were considered

defective (delaminated) and the other appeared to be satisfactory. Water ran out of all three when cut open, but the core was obviously wet in only one. This core was saturated with water, but only on one end. A piece of the saturated core when dried in a 220°F oven lost 55 percent of its dry weight. This weight loss did not include the excess water which ran out when preparing the oven specimen.

e) Predrilling - In erecting this building approximately 3,000 sheet metal screws were installed. Approximately three man-days would have been saved if the extrusions had been predrilled for the screws.

f) Core Discontinuity - At least one roof panel was produced with a discontinuity in the core. This was caused when the honeycomb was not long enough for the panel. To complete the panel another piece of core was placed in, but with no attempt to splice the two pieces of core.

2.2.4 Erection and Disassembly Time

Erection was started Tuesday, April 1, when the crates were opened and an inventory was made. The inventory was made because it appeared that some items might have been lost or stolen from one of the five crates. Although no packing list was available it appeared that only the asphalt tile and cement for the floor were missing. Subsequently it

was learned that these items had not been crated and nothing was missing. Erection was completed Wednesday, April 9, although a small amount of finish-caulking was performed April 11.

Overall, 36 man days (7 1/2 hr days) were used in erecting the building. This time could have been reduced considerably if the problems noted in section 2.2.3 had not arisen and if extreme care had not been taken in erection.

The disassembly of the building was accomplished with 10 man-days labor. This included the time for stacking the components in preparation for crating.

The building could be erected again at another location without difficulty if some of the components had not been damaged during testing. Reassembly should, in fact, be easier since screw holes were already drilled in the various extrusions.

3. Loading, Instrumentation and Performance Criteria

3.1 Loading

The Mark III was load tested under vertical roof load and simulated lateral wind load. The vertical roof design load

was 35 psf and the wind design pressure was 25 psf. These load values were obtained from the Mark III drawings.

The design wind pressure of 25 psf may be related to wind velocity by the use of the formula

$$P_o = 0.00256\ C_h V^2 \tag{3.1}$$

where:

P_o = pressure, psf

C_h = shape factor = 1 for the Mark III

V = wind velocity, mph

as obtained from NAVFAC's DM2 [2]. Thus 25 psf corresponds to a 99 mph wind since $C_h = 1$ for the Mark III [2].

The critical wind load pattern [2] for the building is shown in figure 2.12a. It was assumed that the roof suction pressures were not critical for the simulated wind loading, hence a simplified loading pattern is shown in figure 2.12b. The total wind load on the building is the sum of the windward and leeward pressures times the wall area or 1.3H (where $H = P_o$ x Wall Area). Thus one design load is 1.3x25 = 32.5 psf considering the pressure on both walls. Laboratory equipment was not available for applying a uniform load over such large wall areas, hence it was assumed that

pressures could be replaced by concentrated loads on one wall only. For convenience it was decided to apply the simulated wind load to the leeward wall. Two loading patterns were considered as shown in figures 2.12c and d. Condition (c) provides the same base reactions as condition (b), however it was felt that this loading could lead to a local wall failure. Loading (d) provides the same external eave reaction as loading (c) and lessens the chance of a local failure. The base load of 1.3H/2 was eliminated for laboratory loading, leading to the applied load shown in figure 2.12d. The loading shown in figure 2.12d provides the same base bending moment but only one-half the base shear shown in figure 2.12b.

The simulated wind loading is shown schematically in figures 2.13a and 2.13b. Note in the figure that the eave load was applied at four points using ten ton capacity hydraulic rams. The four identical rams were operated from a common manifold to insure equal force at each load point.

The design vertical roof load was 35 psf. Laboratory equipment was not available for loading the entire roof surface uniformly. Therefore, the actual roof load was applied over six roof panels as shown in figure 2.14. Note that the panel directly over the transverse beam was loaded.

This loading was selected to provide information on transverse beam, longitudinal ridge beams, and roof panels. The six panels were loaded by four 4 ft x 12 ft air bags as shown in figure 2.15.

The pressurized air bags applied load normal to the roof. However, the vertical roof pressure is approximately 98% of the normal roof pressure for the 2.5 on 12 slope of the roof, hence the vertical pressure is assumed equal to the normal pressure.

3.2 Instrumentation

3.2.1 Lateral Deflection Measurements

The lateral movements of the side walls in the plane of the end walls were measured at twenty locations using LVDT linear displacement gages (2-in. measuring range). Figure 2.16 is a schematic of the gage layout for these measurements.

The analog signals from the gages were fed into a 100 channel data processor and recorded on perforated paper tape as well as a printed copy.

3.2.2 Vertical Deflection Measurements

Vertical movements of the ridge line were measured with a surveyors level and leveling rods bonded to the ridge. Figure 2.17 indicates the position of the five leveling rods on the ridge line of the building. The leveling rods were made by gluing paper centimeter scales to wooden rods. These rods were held upright by a horizontal taut wire fastened to the walls of the laboratory.

3.2.3 Load Measurements

a) Roof loads - The roof loads were developed by the pressure in four air bags connected in parallel. The bag pressure was measured with a low pressure, strain gage pressure transducer.

b) Racking loads - The racking loads were developed by hydraulic rams connected in parallel to a hydraulic power supply. The ram pressure in the hydraulic system was measured with 10,000 psi, strain gage pressure transducers. The load on the building was calculated from the ram pressure.

3.3 Performance Criteria

Establishing acceptable performance criteria for the Lewis Building is beyond the scope of this report. Items such as load factors and acceptable deflections must be determined. These criteria are dependent on the use of the building.

The Mark III model tested (two bay building) was a modification of the actual designed structure (three bay building). The load response characteristics of the two structures are different. Thus the two bay laboratory building cannot be used to determine acceptability for a three bay building under a performance criteria. It can however be used to determine structural defects and approximate load capacities for a three bay structure.

4. Test Results

4.1 General

Four load tests were conducted on the building. These were, in chronological order, (a) racking test No. 1, (b) roof load test, (c) racking test No. 2 and (d) racking test No. 3.

Racking test No. 1 was performed primarily to determine the effects of lateral loads on air leakage up to about one design load. The roof load test was performed to determine the behavior and strength of the roof system under vertical load. Racking test No. 2 was performed to determine the behavior of the building under lateral loads larger than one design load. Racking test No. 3 measured the ultimate capacity of the building under lateral load.

4.2 Racking Test No. 1

The purpose of this test was to observe the effect on air leakage of one design lateral load (one design load = 32.5 psf). The air leakage results are described in another report [3]. Structural data are described in this section.

The lateral load was cycled ten times to 15.7 psf, the load was then increased monotonically in about three psf increments from 0.0 to 31.5 psf (0.97 design loads). At 31.5 psf the test was halted due to large horizontal movement of the floor panels with respect to the "Eye" beam foundation. The movement is shown in figure 2.18 by an end view of the deformed central longitudinal "Eye" beam. The intersection of the web with the top flange moved about 0.3 in from the original centerline of the web. This movement is discussed below.

A plot of lateral load versus midlength lateral deflection of the west wall eave and base is shown in figure 2.19. The total deflection at the base of the wall is shown dotted while the total deflection at the top of the wall is shown solid (total deflection with respect to the reaction floor). The load-deflection curves do not start at zero due to residual deflections obtained when cycling the load ten times to 15.7 psf. An undetermined portion of the residual deflections is due to slack in the building system. The base deflection in figure 2.19 is initially due to the deformation of the longitudinal "Eye" beams as shown in figure 2.18. Between 18.9 and 22.0 psf a bolt connecting the north floor panel to the "Eye" beam in the northwest corner of the building sheared. This accounts for the large base deflection of about 0.15 in during this load increment. Loading was continued up to 31.5 psf (0.97 design load) and the test was then halted. One full design load was not reached due to a small error in calibration of the load measuring transducer.

The total deflection at the top of wall (figure 2.19) reflects the base movement plus additional top movement. Net top wall deflections for all lateral gage points are shown in figure 2.21 and are discussed later. The net deflections were unaffected by the large base deflections.

A free body diagram for lateral forces is shown in figure 2.20. The lateral load is transferred into the "Eye" beam foundation by shear through the 1/4-in bolts (neglecting friction) attatching the floor panels to the "Eye" beams. Due to the relative stiffness of the side walls, end walls, and roof the lateral loads are transmitted by diaphragm action through the roof out to the end walls and then into the floor panels. These lateral loads are then transferred into the foundation "Eye" beams almost entirely by the 1/4-in bolts attaching the two end floor panels to the "Eye" beams. That is, almost all of the lateral load is carried by six-1/4 in bolts (three on each end of the building), assuming that no shear is transferred between adjacent floor panels. Note also that the base lateral load in figure 2.12c was not applied during this test so that the total base shear carried by the bolts in this test was only one half of what it would be under the wind load shown in figure 2.12a.

The panels were attached to the foundation beams by aluminum tie-down bolts placed only in the longitudinal beams, as called for on the building plans. The lack of connection between the transverse foundation "Eye beams and floor panels permitted free movement of the floor panels with respect to the transverse beams. Because of this the

longitudinal beams had large shear forces applied at the top flange, hence the deflected shape shown in figure 2.18. After unloading, eight 1/4 in tie-down bolts were used to attach the floor panels at the two ends of the building to each of the four exterior transverse foundation "Eye" beams for a total of 32 additional bolts (eight equally spaced bolts per beam). These bolts, which were not called for on the building plans, were in place for all subsequent tests. These bolts increased the lateral load capacity of the foundation by about 6.3 times. The additional bolts also decreased the shearing load causing twisting in the longitudinal beams by transmitting some load into the transverse beams. It is shown in section 4.4 that the relative deflection of the top of the wall with respect to the base was not significantly affected by the large base movements during this test.

The relative lateral deflections at the top of the wall are shown in figures 2.21 a through e. Two curves are shown for each of five plots. The west wall (loaded wall) deflections are shown solid while the east wall deflections are shown dashed; both curves of each plot are for the same building cross section. Except for the curve shown in figure 2.21d, the general trend of the deflection data is for the loaded wall to have a slightly larger lateral

deflection than the unloaded wall. While loading from 18.9 to 22.0 psf a tie-down bolt connecting the building to the aluminum "Eye" beam foundation sheared off in the northwest corner of the building. This is reflected in figure 2.21 by a change in the load-deflection relationship for most of the deflection data shown. This change is usually in the form of an increase in the rate of deflection with increasing load. The east wall deflections (shown dotted) in figures 2.21b and c show a decrease in deflection with loads larger than 18.9 psf. Examination of the original test data indicates that this is probably due to the deflection instrumentation binding and not following the actual movement of the building. The binding may have been caused by the bolt shearing off but, in any case, the two curves probably reflect faulty instrumentation rather than the actual load-deflection curves.

Vertical ridge deflection data are not shown for this test since they were so small as to be negligible.

There was no visible damage to the building during this test other than the slightly bent longitudinal foundation beams (bent in the shape shown in figure 2.8) after unloading.

Wind velocities may be obtained from the measured lateral loads by using equation 3.1

$$P_o = 0.00256\ C_h V^2 \tag{3.1}$$

It was shown in section 3.1 that the total lateral load on the Lewis Building at a given wind velocity is 1.3 P_o (0.8 P_o and 0.5 P_o on the windward and leeward walls respectively). Thus the wind velocity may be computed as

$$V = \sqrt{\frac{P_o}{0.00256\ C_h}} \tag{3.1a}$$

where:

V = wind velocity, mph

P_o = measured load per sq ft/1.3

C_h = 1

or:

$$V = \sqrt{\frac{\text{measured load per sq. ft.}}{1.3 \times 0.00256}} \tag{4.1}$$

The maximum lateral load capacity for racking test No. 1 was 18.9 psf (75 mph wind), assuming the tie-down bolt fracture constitutes failure. However, even with the fractured bolt the test structure withstood 31.5 psf (97 mph wind) before loading was arbitrarily halted. Since actual load conditions produce twice the base shear as in the tests, the test structure should be limited to 9.4 psf

(53 mph wind). Assuming a load factor of 2.0 the service load is 4.7 psf (36 mph wind).

Similar loads may be computed for a three bay structure assuming the same failure mode occurs. It is possible however that a different type of failure could occur in the longer three bay building, depending on the stiffnesses and strengths of the various components. Based on the assumption above, the failure load is 2/3 of two bay load (9.4 psf) or 6.3 psf (43 mph wind). Assuming a load factor of 2.0 the service load is 3.2 psf (31 mph wind).

4.3 Roof Load Test

The design live load for the roof was 35 psf as obtained from the building plans. Only three of the nine roof panel sections were loaded, one of which was located symmetrically over the transverse beam at the midlength of the building as shown in figure 2.14.

The load versus the midlength vertical and horizontal roof deflections are shown in figure 2.22. The vertical ridge deflection is shown in the right hand portion of the figure. The west eave lateral deflection is shown in the left hand portion of the figure. The east eave (not shown) and west

eave lateral deflections were equal, within a few percent. The maximum recorded load was 38.2 psf (vs 35 psf design load for the entire roof loaded) with an estimated (discussed below) deflection of 0.93 in at midspan. At 38.2 psf the load suddenly dropped to 25.0 psf accompanied by a loud cracking noise before deflections could be recorded. The estimated load-deflection path is shown dashed. The estimated path is the projection of a straight line through the last two known data points up to the known maximum load of 38.2 psf, then a line was drawn from the maximum point to the 25 psf data point. The load was increased to 32.5 psf with a vertical deflection of 1.33 in. At this point the test was halted since the maximum load had been reached. It was felt (based on the load-deflection curve in figure 2.22) that further loading could further damage some portion of the building and invalidate load data obtained for future racking tests.

Ridge deflection profiles are shown in figure 2.23 for each stage of loading except for the maximum load of 38.2 psf for which deflections were not obtained.

A general view of the building interior before testing began is shown in figure 2.24a with arrows indicating areas damaged during the roof load tests. Close up views of these

damaged areas are shown in figures 2.24b, c, and d. Inspection of the interior of the building indicated that the west longitudinal ridge beam between the transverse beam and south end wall slipped down at the connection to the transverse beam (about 1/4 in relative to the transverse beam and roof), pulling the screws out of the roof panels near the center of the building as shown in figures 2.24c and d. Slight buckling of the transverse beam aluminum skin at the transverse beam to wall connection was also observed; this is shown in figure 2.24b.

The roof was loaded as shown in figure 2.25. The ridge beams must transmit a large reaction from the roof load through the connection into the transverse beam. Close examination after all testing was complete showed that the rivets connecting extrusion A to the ridge beam had been sheared. Although this was not found until after all testing was completed, it accounts for the observed failure. The failure pattern is shown in figure 2.25b. As the rivets sheared, the reaction was transferred from the connection into roof panel P by means of the screws connecting ridge beam B to roof panel P. These screws could not carry the load so they pulled loose from the roof skin as shown in figures 2.24d and 2.25b.

An approximate uniform load over the entire roof may be computed from these test results although the system is indeterminate. The following assumptions are made (refer to figure 2.14).

1. There is no transfer of shear between roof panels.
2. The reaction of roof panels directly over the end walls is carried only by the end walls.
3. The reaction of roof panels directly over the transverse beam is carried only by the transverse beam.
4. The ridge beams are simply supported by the end walls and transverse beam.
5. Roof panels other than those in 2 and 3 are simply supported by the ridge beams and side walls.
6. There is no change in the failure mode.

Based on this set of assumptions a uniform load can be computed which gives the same reaction at the ridge beam to transverse beam connection which caused failure during this load test at 38.2 psf. This computed uniform load over the entire roof surface is about 28 psf.

These assumptions are certainly not exact; however the degree of error may be checked by comparing the computed reaction at the transverse to ridge beam connection and comparing this with the connection capacity. The computed

reaction at 38.2 psf is 955 lb (based on the assumptions above). The measured capacity of one rivet is 102 lb. Thus the ten rivets which failed in the connection have a capacity of 1,020 lbs. This is about seven percent larger than the computed failure reaction. This indicates that the assumptions are reasonably accurate enough to compute an approximate uniform load.

The ultimate roof load on the two bay laboratory structure was 38.2 psf. Assuming a load factor of 2.0 the service is 19.1 psf. The load-deflection plot in figure 2.22 indicates the service load should be between 15.3 and 23.3 psf (departure from linearity). Using the same load factor of 2.0, the service load for a uniformly loaded roof is 14.0 psf. The two bay laboratory structure and the three bay field structure would both have the same ultimate and service loads providing the same failure mode occurred in each and the load responses were similar.

4.4 Racking Test No. 2

It was indicated in section 4.2 that insufficient bolts were provided near the ends of the building to transmit the shear forces in the end walls into the "Eye" beam foundation. At the end of racking test No. 1, 32 additional

bolts were installed to prevent large horizontal movements between the floor panels and aluminum "Eye" beam foundation. The effect of these bolts on lateral movement is shown in figure 2.26 by comparing the base deflections and net eave deflections between racking test No. 1 and No. 2 at the midlength of the building. Base deflections are compared at the left hand portion of the figure. Residual deflections were eliminated for comparison purposes. Racking test No. 2 shows much smaller base movement when compared with racking test No. 1. The net eave deflections are compared in the right hand portion of the figure. The load-deflection responses are almost identical even though the base deflections are far different from each other. This indicates that the additional 32 bolts increased the base shear resistance but did not significantly affect the net load-deflection characteristics of the wall eave deflections with respect to the floor.

The deflected shape of the building at the beginning of the test is taken as the zero position for plotting the net lateral deflections in figures 2.27 thru 2.31. The lateral load was cycled six times during racking test No. 2 with each succeeding load being higher than the preceeding load. The maximum load applied for cycles one through six are as follows: 22.9 psf, 26.9 psf, 46.0 psf, 55.1 psf, 72.7 psf

and 92.7 psf (2.85 design loads). All six load cycles are shown in each of the five figures. The plot in the left hand portion of the figure shows the west wall (loaded wall) deflection at a cross section, the plot in the right hand portion of the figure shows the corresponding lateral deflection for the east wall at the same cross section. Figures 2.27 and 2.30 have portions of the curves labeled as "estimated" in the right hand plot during cycle No. 6. During this load increment lateral deflections exceeded the measuring range of the LVDT's. The actual deflections at a load of 92.7 psf are not known, but values were estimated by extrapolating with a straight line from the previous two known load-deflection points. The unload deflection point is also a known value since deflections decreased to values within the capacity of the LVDT's upon unloading. In figure 2.29 the sixth cycle load-deflection curve for the west wall is also labeled as estimated. Deflection values were obtained for each increment of loading. However, as shown in the figure, the measured deflection decreased when loading from 0 to 35.8 psf. Apparently during this load increment the LVDT was disturbed, offsetting the instrument from its true zero. A corrected zero was obtained by extending the load-deflection curve through the deflections at 35.8 and 56.1 psf until this line intersected the zero load level. This extropolated value was then placed at the residual deflection

from cycle five and the entire load deflection curve transferred to agree with this new zero.

Maximum lateral deflection profiles of the east and west eave line during load cycle No. 6 are summarized in figure 2.32. The deflections at 92.7 psf are shown by solid circles and connected by solid lines, open circles indicate the estimated deflections at 92.7 psf. Residual deflections after loading cycle No. 6 are shown as triangles and are connected by dotted lines. The estimated unloaded values are shown by open triangles.

Structural damage from racking test No. 2 is shown in figure 2.33. Figure 2.33a is an interior view of the building before testing. Two locations of structural damage are indicated by arrows. Close up views of these locations are shown in figures 2.33b and c. The buckling in the transverse beam shown in figure 2.33c was observed after racking test No. 2 was complete. The fracture of the skin shown in 2.33b was not observed after racking test No. 2. This fracture was not found until after all testing had been completed. However, it is felt that this damage occurred between 83.9 and 92.7 psf in cycle No. 6. During this load increment a loud cracking noise was heard which could not be accounted for after the test. Examination of the deflection

data shown in figure 2.29 indicates that during cycle No. 6 the east wall deflection actually decreased during the last load increment. It is felt that if the fracture occurred during this loading increment that the fractured transverse beam would decrease the stiffness of the roof system so that the east wall might not follow the west wall as closely as on previous cycles, thus the deflection might drop off somewhat. The midspan deflection of the east wall in figure 2.32 is smaller than any of the other east wall deflections, whereas the west wall deflection is larger than any of the other west wall deflections. Thus the data substantiates the conclusion that the beam fractured during the last load increment.

The maximum lateral load capacity during racking test No. 2 was 83.9 psf (159 mph wind), assuming the transverse beam fracture between 83.9 and 92.7 psf as failure. The similar load on a three bay structure is 55.9 psf (2/3 x 83.9) provided no other component fails prior to this load. This is equivalent to a 130 mph wind. However, even with the fractured beam the test structure withstood 92.7 psf before arbitrarily halting the loading. The similar load on a three bay structure is 61.8 psf (2/3 x 92.7). This is equivalent to a 136 mph wind on a three bay structure.

Assuming a load factor of 2.0, the service load on the two bay structure is 42.0 psf (112 mph wind) and 28.0 psf (92 mph wind) for a three bay structure.

4.5 Racking Test No. 3

After examination of the load-deflection data from the roof test and racking test No. 2 it was decided to determine the ultimate lateral load capacity of the building under lateral load. Instrumentation included the five vertical ridge deflections and the lateral eave (and base) deflection of the west wall (loaded wall) at midlength of the building. The structure was loaded in increments of 10 psf up to a maximum load of 100 psf. The vertical deflection profile of the roof is shown in figure 2.34 for each load increment. The vertical ridge deflection and net lateral eave deflection of the west wall at midlength of the building are shown in figure 2.35. Ridge deflection data are shown by solid triangles connected by a dashed line, eave deflections are shown as solid circles connected by a solid line. The deflections shown in figure 2.34 and 2.35 are relative to the deflected shape of the building at the beginning of the test. The load-deflection curve shown in figure 2.35 indicates that 100 psf was the ultimate lateral load capacity of the building. Further load-deflection data was not taken since this

data indicated that the maximum load had been reached, and the sponsors of the project had indicated that they wanted the building back after tests had been completed for possible salvage.

The midlength west wall lateral deflection is shown in figure 2.36, considering the residual deflection from racking test No. 2. The load deflection data from racking test No. 2 is shown by a series of straight lines indicating only significant changes in the load-deflection curves. The zero position for racking test No. 3 is then assumed to be the unloaded position from cycle No. 6 of racking test No. 2. The load-deflection data for racking test No. 3 are shown by a solid line, also indicating only significant changes in the load-deflection response. The load-deflection response for racking test No. 3, indicated that the building was more flexible than it was during racking test No. 2. This is one more indication that the transverse beam fractured during racking test No. 2 (section 4.4). A view of interior damage after racking test No. 3 is shown in figure 2.37. The general interior view of the building before racking test No. 3 is shown in figure 2.37a with the major structural damage area identified by an arrow. A closeup of the damage is shown in figure 2.37b. As shown in figure 2.37b the rivets connecting the bottom extrusion to

the transverse beam sheared off. The relative movement between the extrusion and aluminum skin was about one in.

Most of the load in the transverse beam is carried by the extrusions in bending. After the rivets sheared, the load was transferred into the aluminum skin which fractured due to stress concentration at the corner of the cut out. The fracture of the beam reduced the stiffness of the roof system and probably accounts for the flexible load-deflection relationship shown in figure 2.36 for racking test No. 3.

Exterior damage is shown in figure 2.38. Much of this damage was concentrated around the doors of the north and south end of the building. Damage around the north door is shown in figures 2.38d and e. Both faces of the skin on the panel containing the door wrinkled immediately above the door as shown in figure 2.38d. The screws connecting the door panel to the bottom extrusion tore through the skin on the tension uplift side of the door panel as shown in figure 2.38e. Similar damage is shown around the south end door in figures 2.38b and c. The screws shearing through the door panel skin had relative movement on the order of 5/8 in. Typical panel to panel buckling of two end wall panels is shown in figure 2.38f. Damage of the

aluminum "Eye" beams is shown in figures 2.38g and h. The transverse "Eye" beam sheared the web from the bottom flange of the longitudinal "Eye" beam (figure 2.38g).

Uplift deflection was recorded at the base of the east wall (southeast corner of the building) after the test was underway. The uplift is shown in figure 2.38h. The data are shown in figure 2.39 by solid circles and connected by solid lines. The zero value was obtained by extropolating the first two readings. Total deflection values were then obtained by using the extropolated zero. The maximum uplift value was not obtained. An estimated value was determined by connecting the unloaded deflection with the last known deflection by a straight line and extending this line up to 100 psf.

The maximum lateral load capacity during racking test No. 3 was 100 psf. This corresponds to a 173 mph wind. Similar values for a three bay structure are 66.7 psf and a 142 mph wind. Service loads for a load factor of 2.0 are 50 psf and a 123 mph wind for the laboratory building and 33.3 psf and a 100 mph wind for a three bay building.

5. Conclusions and Recommendations

5.1 Conclusions

5.1.1 Erection

The erection of the Mark III building required 36 man days. The erection was significantly hindered by poor quality control in prefabricating the building as evidenced by the following problems encountered during erection.

(1) Floor length greater than wall length

(2) Gable end member cut inaccurately

(3) Wall panel lengths unequal

(4) Ridge beam to gable ends required trial and error fittings

(5) Wall panels out of square (slip sheeting)

(6) Window cut-outs oversized and out-of-square

(7) Floor panel delamination

(8) Vinyl cleats splitting

Approximately 3000 sheet metal screws were used to erect the building. Drilling extrusions to receive these screws required three man days; these extrusions would be pre-drilled when relocating the building.

5.1.2 Structural

The bolts connecting the floor panels to the aluminum "Eye" beams were insufficient to transmit shear forces into the foundation, especially near the end of the building. As detailed, these bolts were installed only in the longitudinal "Eye" beams. The beams provided were not stiff enough to prevent movement at one half the base shear design load. An insufficient number of bolts were provided from the standpoint of strength since one sheared at about one half of one design load. Installation of additional bolts in the transverse floor beams allowed the application of 1.55 of the design base shear without serious sliding.

The strength of the building under simulated wind load was 3.1 times the total horizontal load applied at the eaves by a 99 mph wind (as computed using NAVFAC's DM2). (This is only 1.55 times the design base shear). At the maximum load numerous structural failures were apparent. These failures are listed below:

(a) Wrinkling of panel skins around the doors

(b) Tearing of screws through the skin of end wall panels

(c) Fracture of the "bottom chord" of the transverse beam.

Failure mode (c) limited the maximum lateral load. The fracture occurred in a reentrant corner of the duct cut-out in the transverse beam. The saw cuts extended as much as 1/4 in past the corners of the duct cut-out. These cuts caused stress concentrations which led to a premature fracture.

The ultimate load capacity of the roof system with 33 percent of the roof area loaded was 38.2 psf. At this load three structural failures were evident as listed below:

(a) Failure in the longitudinal to transverse beam connection.

(b) The screws connecting the longitudinal ridge beams to the roof panels pulled out of the roof panel near the transverse beam.

(c) The transverse beam buckled at its connection into the wall panels.

The roof system load capacity cannot be stated conclusively since the entire roof was not loaded, however, the ultimate load capacity of the roof was about 28 psf based on extrapolation of test data. This is only 80 percent of the design value.

Ultimate and service load capacities are summarized in table 2.1 for each load test for the two bay test structure

and a three bay field structure. Service loads are based on a load factor of 2.0 this load factor appears reasonable based on load-deflection response for the four tests. The loads for a three bay structure were determined by extrapolation of the data for the two bay test structure. The data was extrapolated based on the assumption that the same failure mode occurs for both structures and that their load response is similar. Racking load capacities are given in terms of the wind velocity in mph. The roof load capacities are for a uniformly loaded roof. The ultimate load for each test was based on the failure shown in the third column of the table although larger loads might have been imposed during a test. The following conclusions were reached based on the previous discussion and the summary in table 2.1:

a. Racking test No. 1 indicates that a three bay structure will carry only a 31 mph service load wind (43 mph ultimate) due to a base shear failure. This is only 31 percent of the design wind load.

b. Racking test No. 3 indicates that with adequate tie-down bolts a three bay building will carry a 100 mph service load wind (142 mph ultimate). This is equivalent to the design wind load of 100 mph.

c. The roof service load capacity is 14 psf (28 psf ultimate) due to an inadequate longitudinal to transverse beam connection. This is only 40 percent of the design value.

5.2 Recommendations

5.2.1 Erection

It is felt that the erection of the Mark III would be considerably simplified by establishing closer fabrication tolerances and reducing the production errors such as those listed in section 5.1.1. Greater attention to details to insure that all parts fit is necessary if this building is to be widely used as a relocatable building. It is recommended that at least two detail changes be made to allow greater erection ease. These are (1) predrilling extrusions to accept sheet metal connection screws and (2) elimination of the triangular end gable. The first of these recommendations would save approximately 10 percent of the erection time. Replacing the triangular end gable by running the end wall panels from the floor to the roof would eliminate time consuming trial and error fitting encountered in the building erection.

5.2.2 Structural

Tie-down bolts should be used to connect the floor panels to the transverse as well as the longitudinal "Eye" beam system. Stiffer "Eye" beams are also desireable.

Duct cut-out corners in the transverse beam should be fillets rather than right angles. If possible the cut-out should be eliminated.

The connection of the ridge beam to transverse beam should be improved.

6. Acknowledgments

The work described herein was carried out in the structural laboratories of the Building Research Division at the National Bureau of Standards. The program was sponsored by the Naval Facilities Engineering Command, liason was provided by Mr. Paul Dettor. The Naval Civil Engineering Laboratory (NCEL) at Port Huenene, California was responsible for program coordination. Monitoring was provided by W. A. Keenan, NCEL. Mr. Kennan reviewed this report and made several helpful suggestions.

7. References

1. Reichard, T. W. and Leyendecker, E. V., "Test and Evaluation of the Prefabricated Lewis Building and Its Components, Phase I Part 1, Evaluation of Sandwich Panel Components," National Bureau of Standards Report 10163, March 1970.

2. "Structural Engineering, NAVFAC DM-2," Department of the Navy, Naval Facilities Engineering Command, Washington, D.C., December 1967.

3. Phillips, C. W. and Kuklewicz, M. E., "Thermal Performance of a Mark III Relocatable Lewis Building," National Bureau of Standards, NBS Project 4214442, June 1969.

Test	Load Condition	Ultimate Load Limitation	Load Capacities[d] Two Bay Test Structure	Load Capacities[d] Three Bay Field Structure[c]
Racking No. 1	Service[a] Ultimate	Base shear (Bolt fracture)	36 mph wind 53 mph wind	31 mph wind 43 mph wind
Racking No. 2	Service[a] Ultimate	Transverse beam fracture	112 mph wind 159 mph wind	92 mph wind 130 mph wind
Racking No. 3	Service[a] Ultimate	Building system	123 mph wind 173 mph wind	100 mph wind 142 mph wind
Uniform Roof Load[b]	Service[a] Ultimate	Beam connections	14 psf 28 psf	14 psf 28 psf

a. Service load is based on a load factor of 2.0

b. Estimated uniform load on two bay test structure

c. Loads are extrapolated from two bay test data assuming the same failure modes and similar load response.

d. Racking design load = 100 mph wind
Uniform roof design load = 35 psf

TABLE 2.1 Summary of Load Capacities.

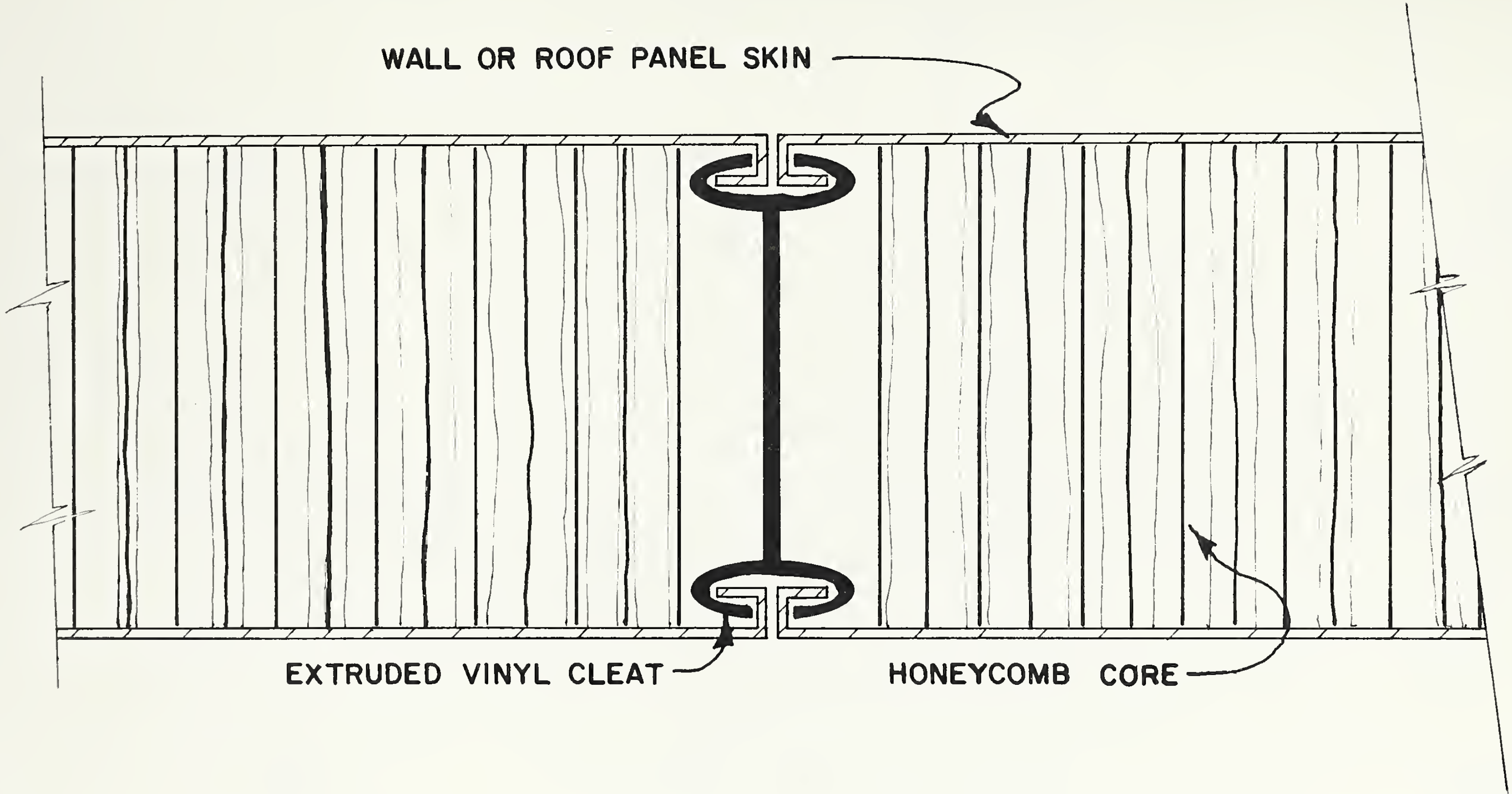


Figure 2.1 Method of using vinyl cleat to connect panels.

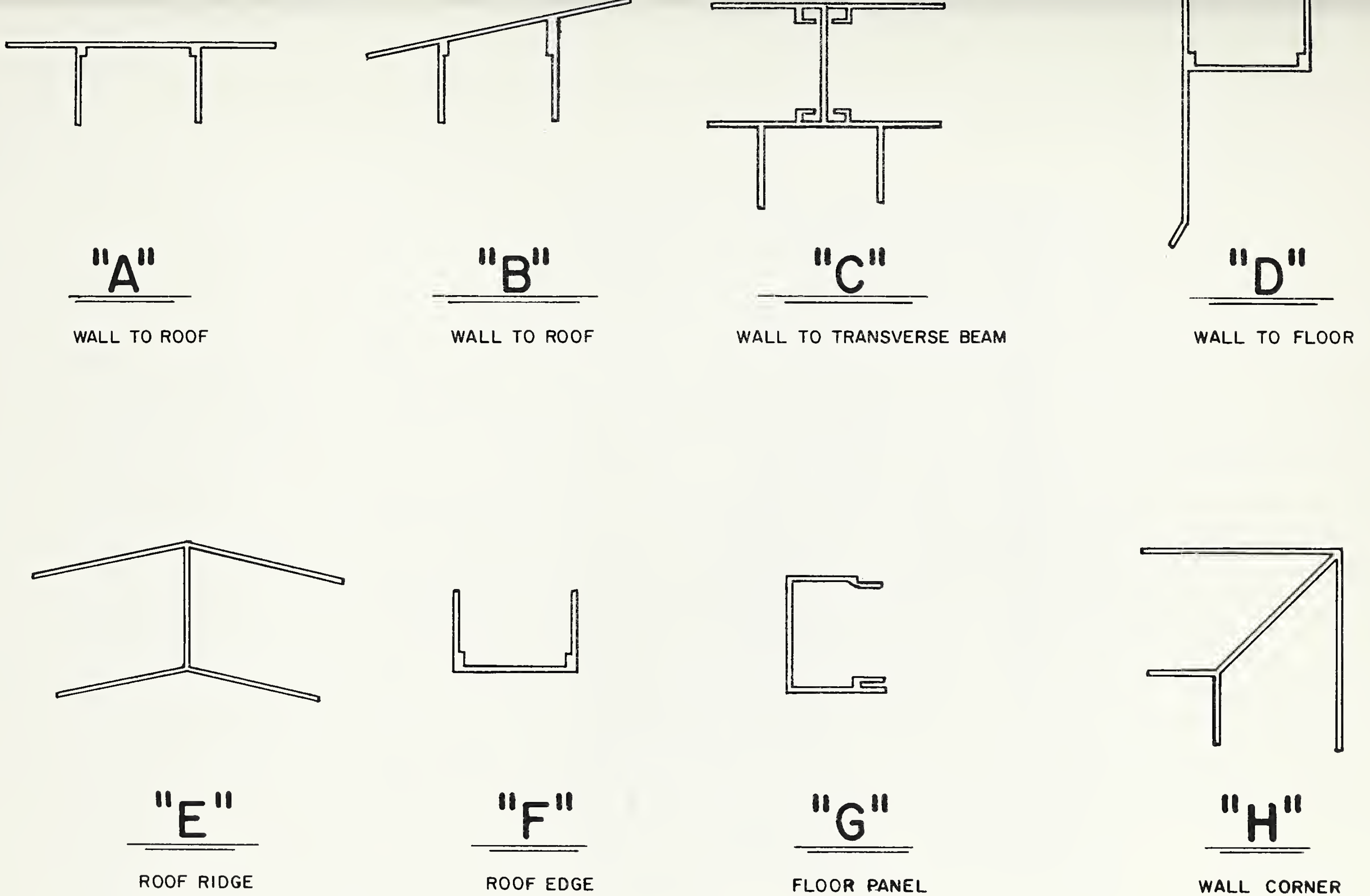


Figure 2.2 Extrusions used in building (1/4 size).

Figure 2.3 Building as it was delivered.

Figure 2.4 Connecting "eye" beams to
concrete test frame and simulated

Figure 2.5 Method of connecting "eye" beams to simulated piers.

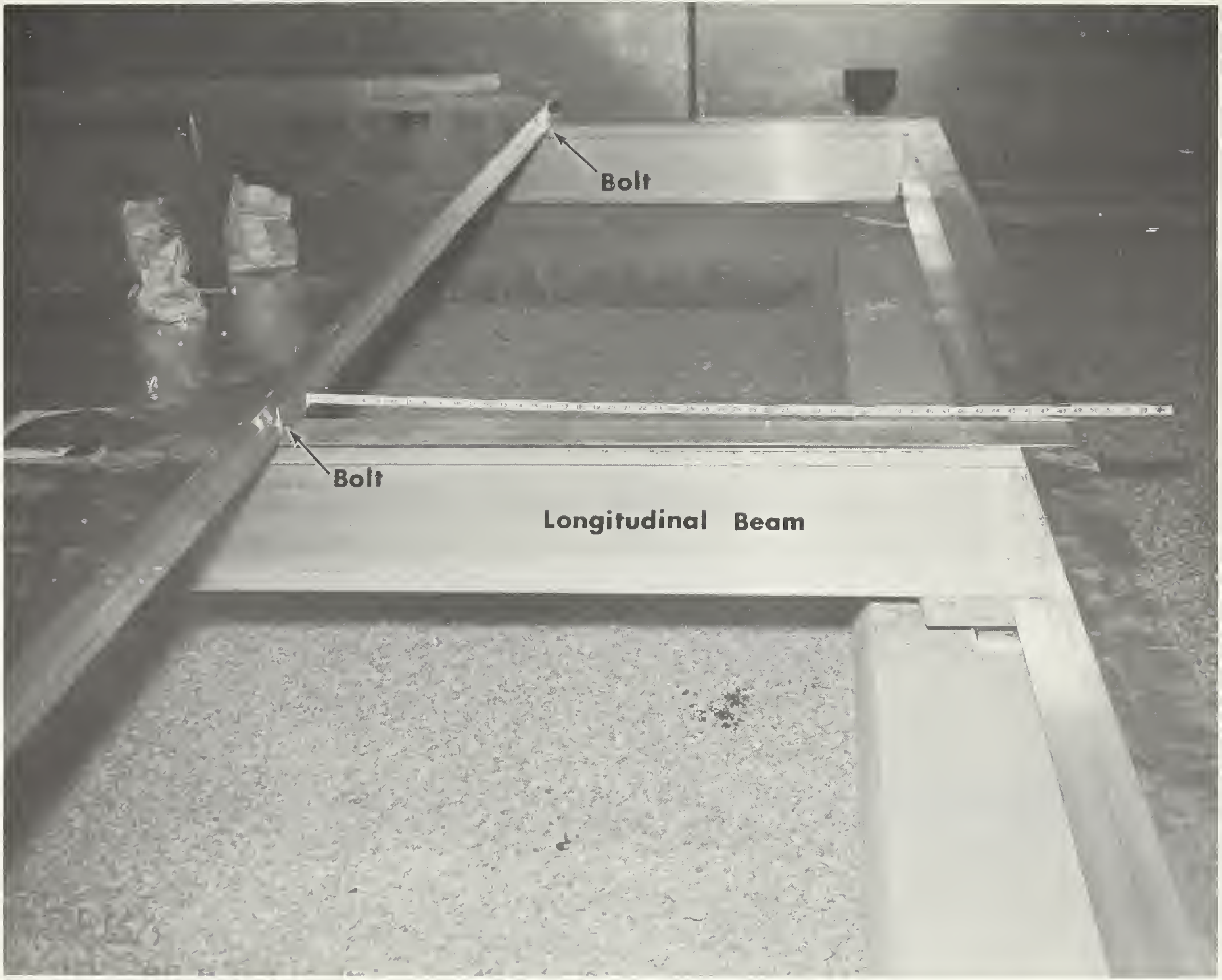


Figure 2.6 All but last floor
panel in place.

Figure 2.7 Placing a wall panel

Figure 2.8 Placing one of the
3000 screws.

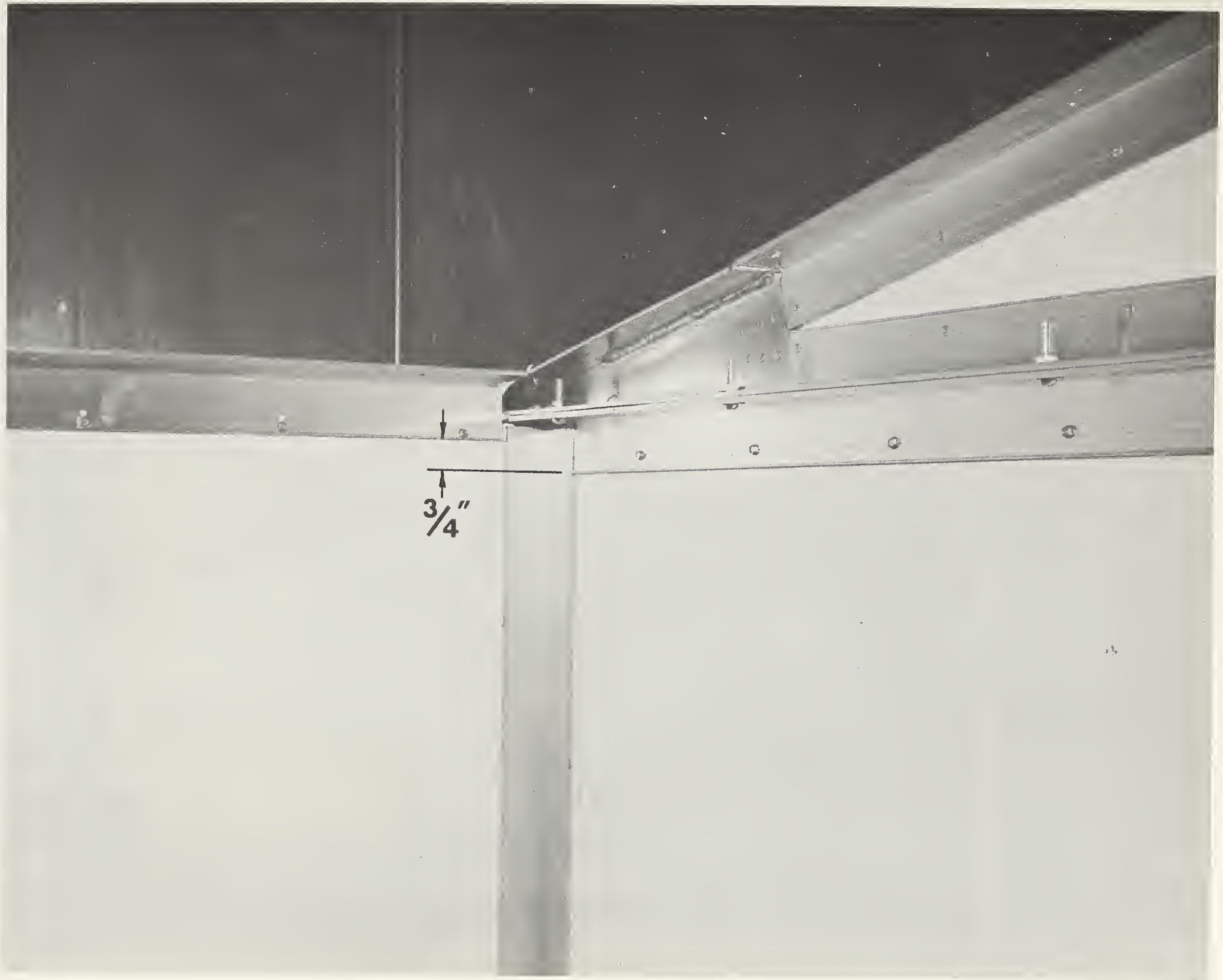


Figure 2.9 Detail of corner connection

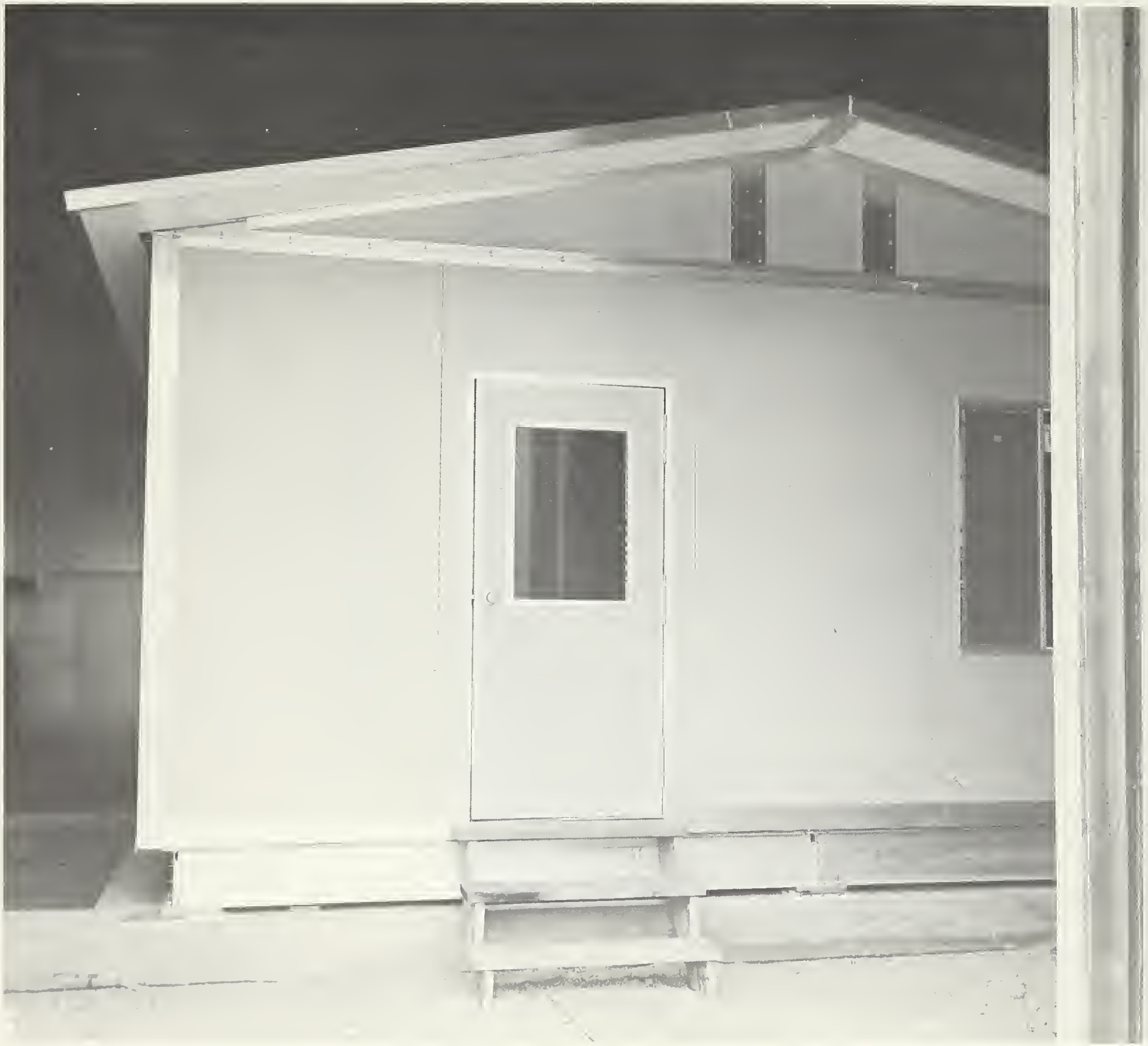
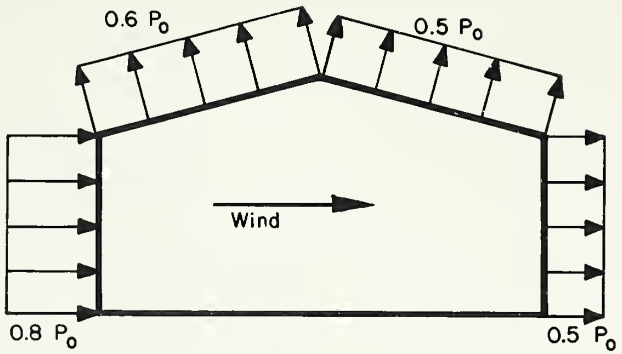
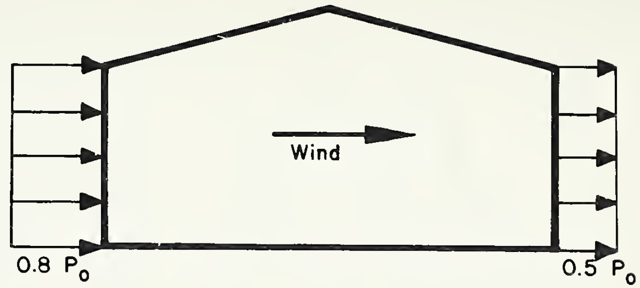


Figure 2.10 Completed building
through the doorway of testing chamber

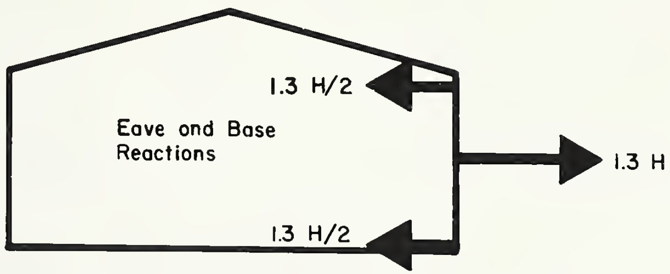
Figure 2.11 Building ready for relocation after testing.



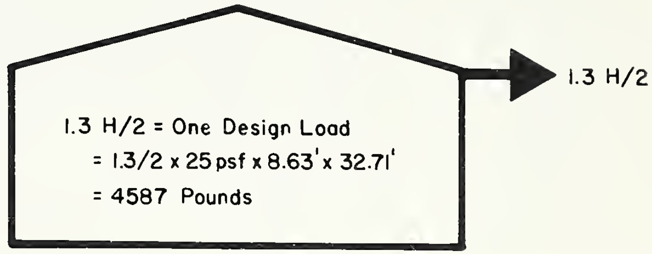
(a) Critical Wind Load Pattern, NAVFAC DM2



(b) Critical Wind Load Pattern, Neglecting Roof Pressures

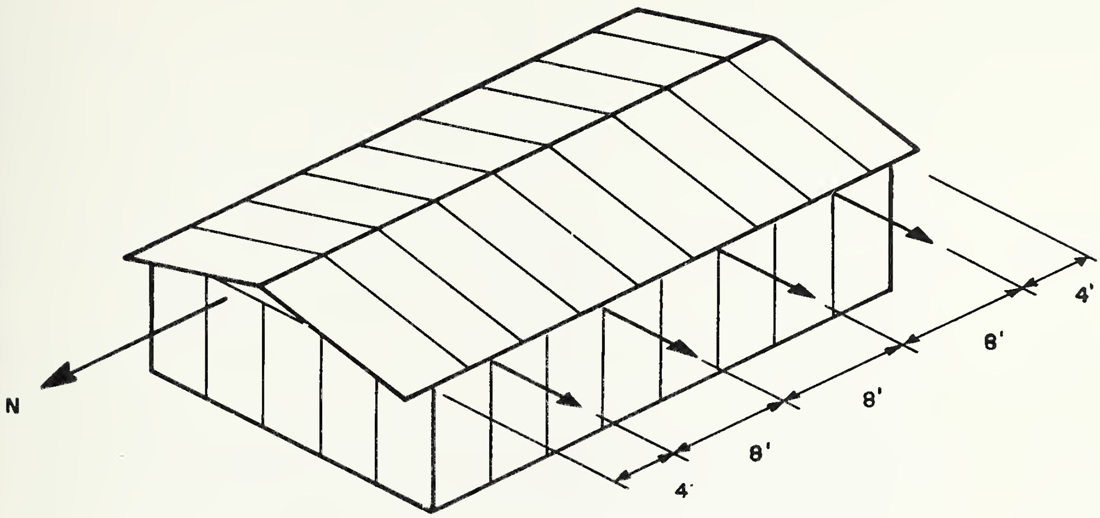


(c) Critical Wind Load Pattern, Replacing Lateral Pressures by One Lateral Load

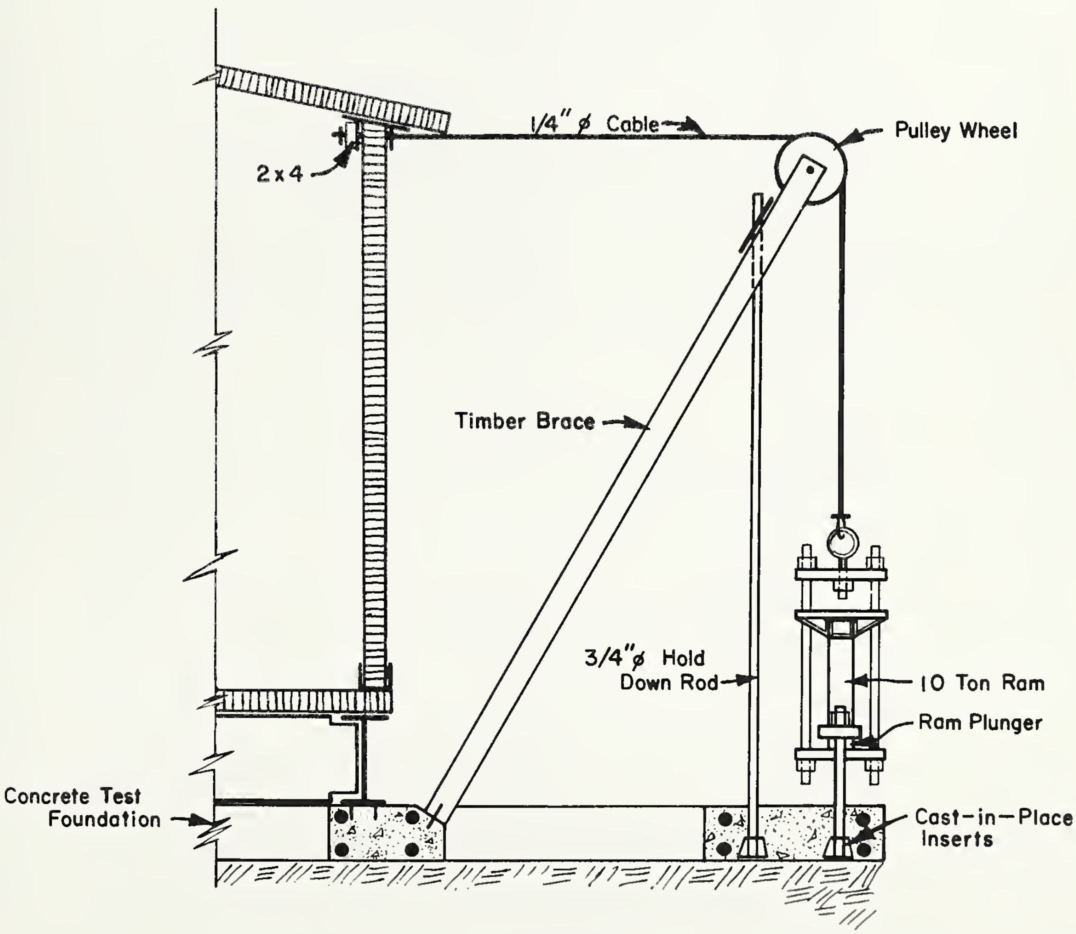


(d) Critical Wind Load, Laboratory Loading

Figure 2.12 Development of Lateral Loadings.



(a) Concentrated Lateral Loads



(b) Schematic of Lateral Load System

Figure 2.13 Simulated Wind Loadings.

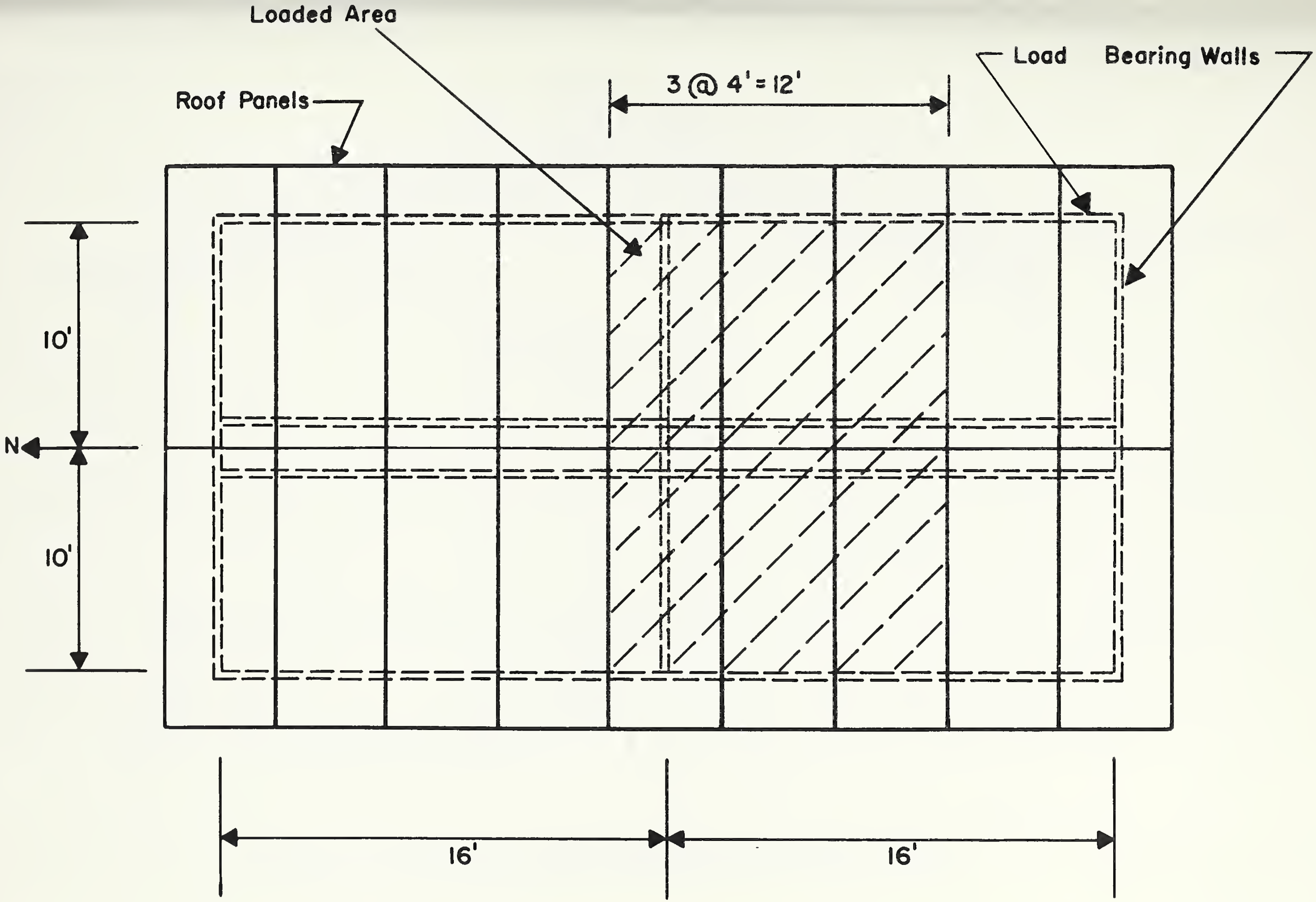


Figure 2.14 Roof loading

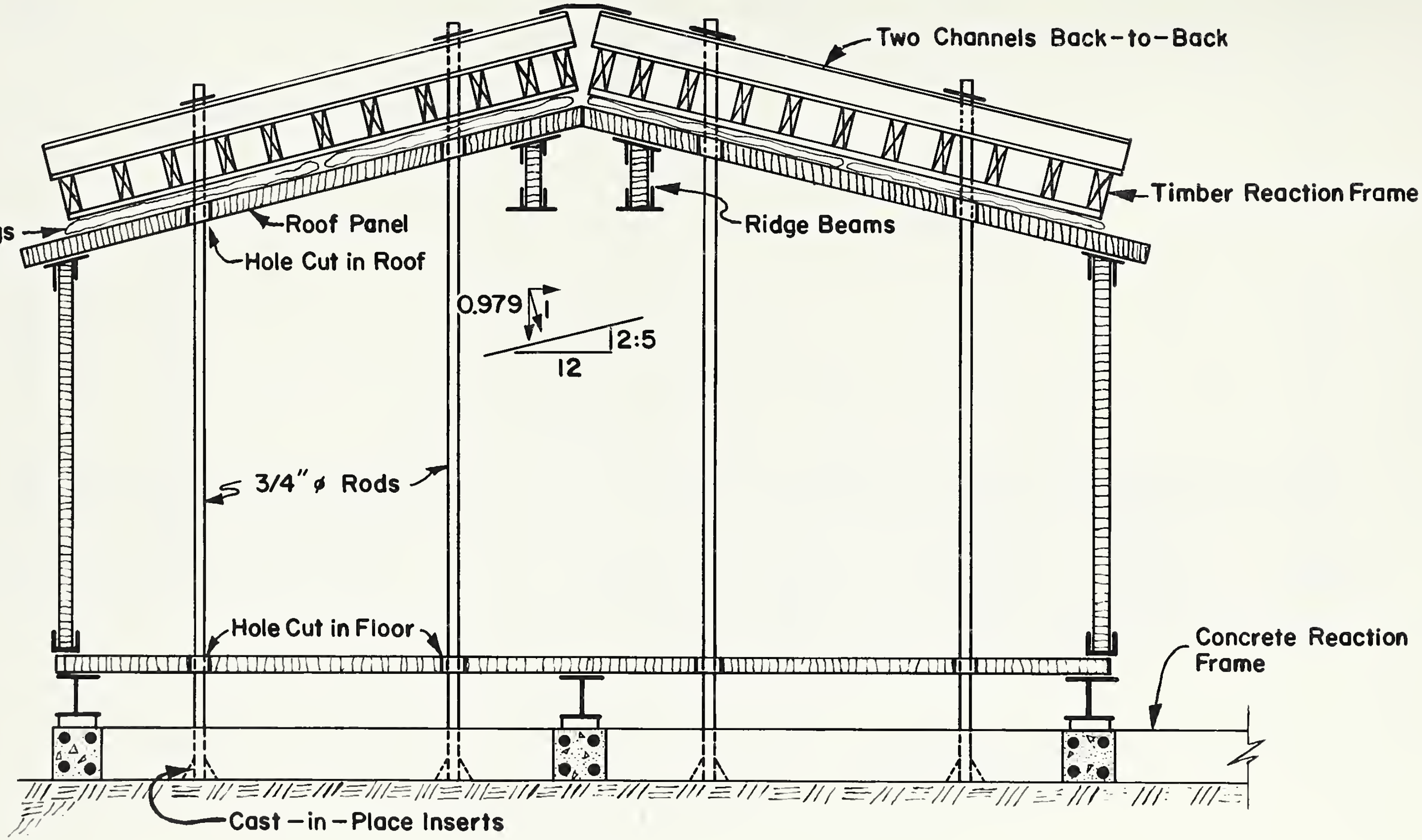
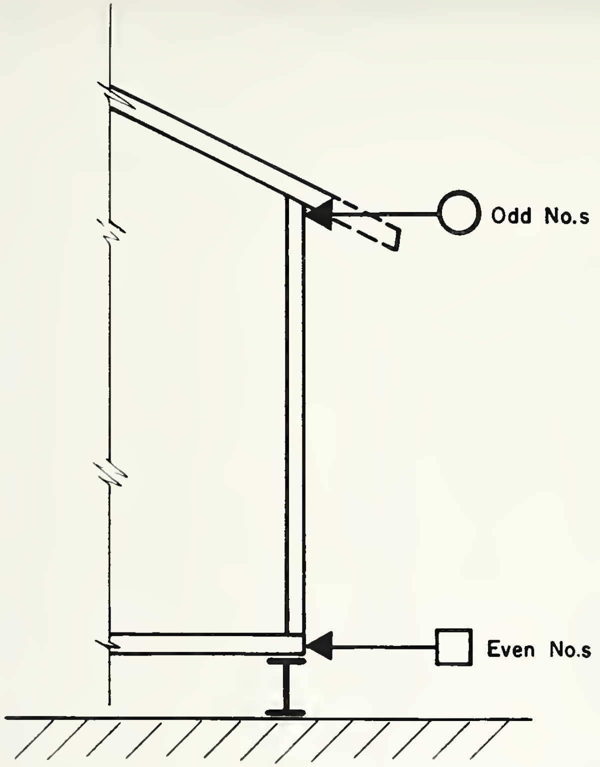
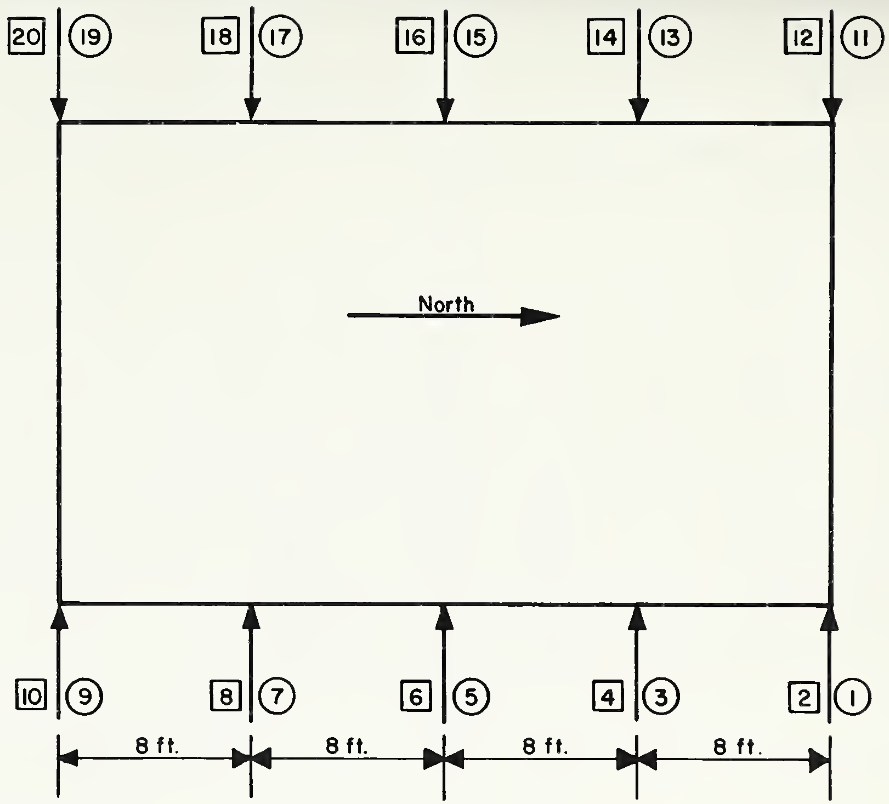


Figure 2.15 Cross Section Showing Schematic of Roof Loading System

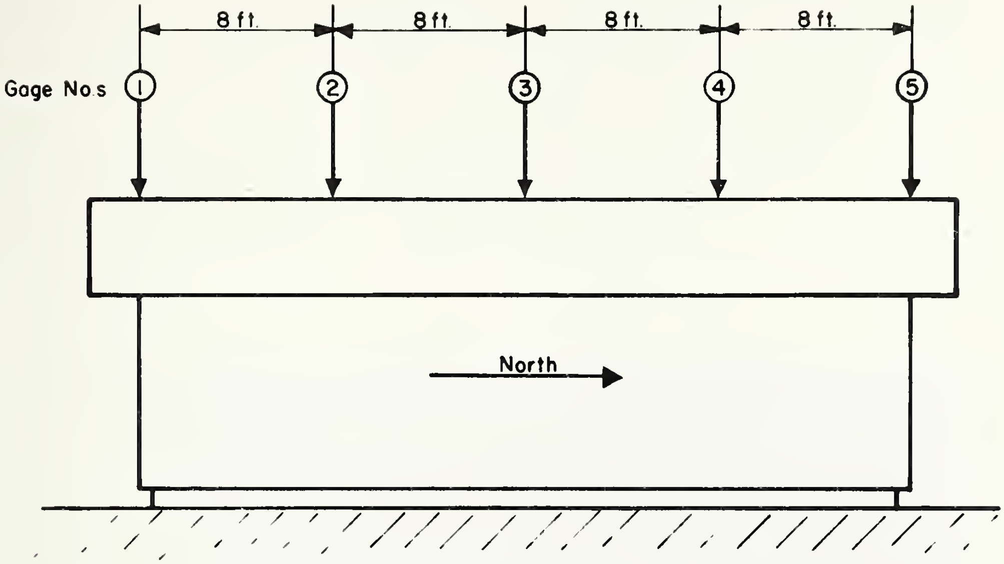


Side Elevation

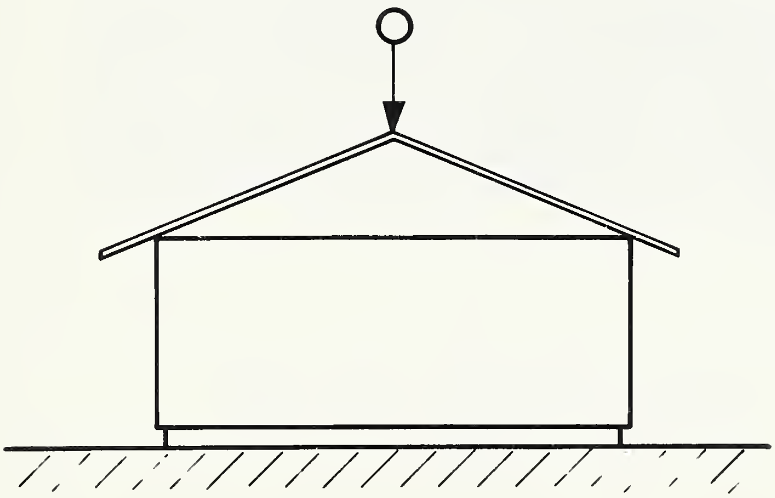


Plan of Building

Figure 2.16 Schematic Showing Gage Location and Identification Numbers for Lateral Deflection Measurements.



Side Elevation
Scale 1" = 8 ft.



End Elevation

Figure 2.17 Schematic Showing Position of Scales Used for Measuring Vertical Deflections.

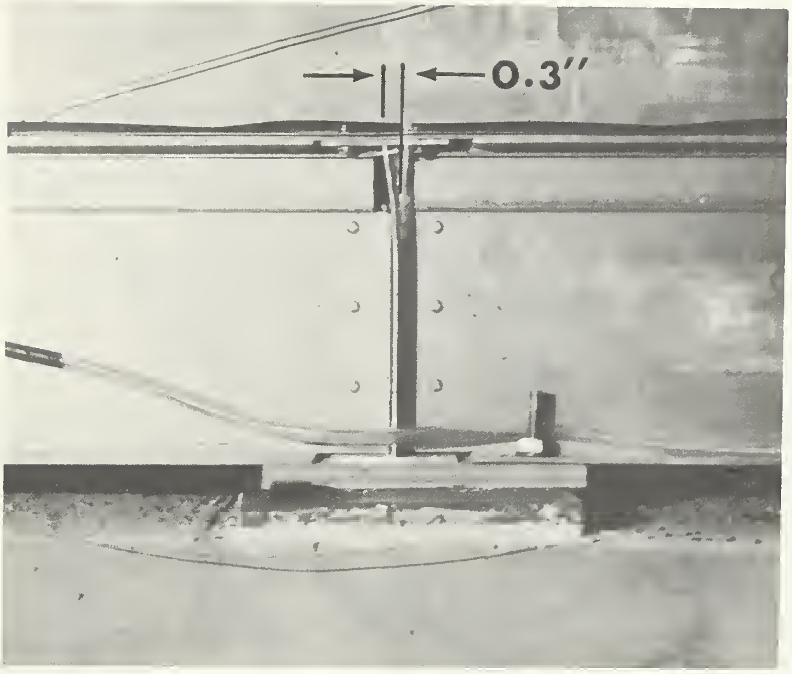


Figure 2.18 Horizontal movement of building on foundation, racking test No. 1

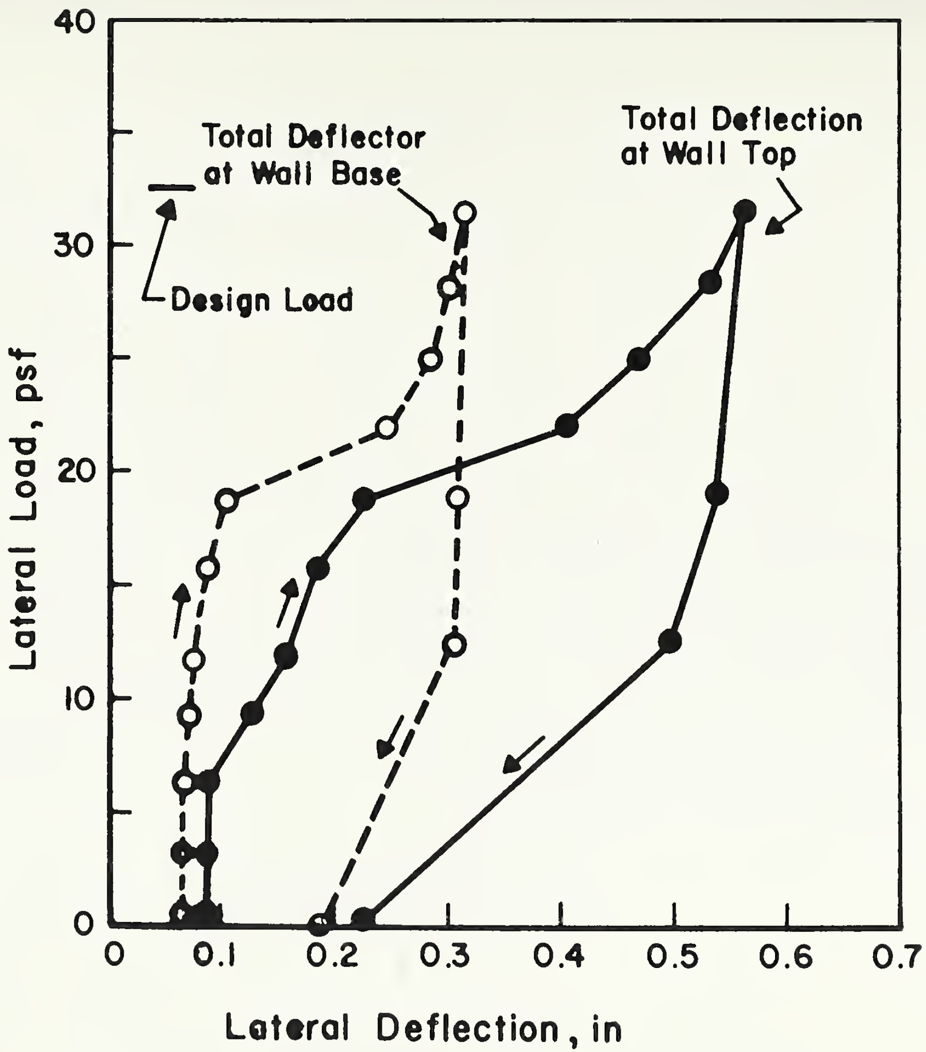


Figure 2.19 West wall lateral deflection at mid-length, racking test No. 1

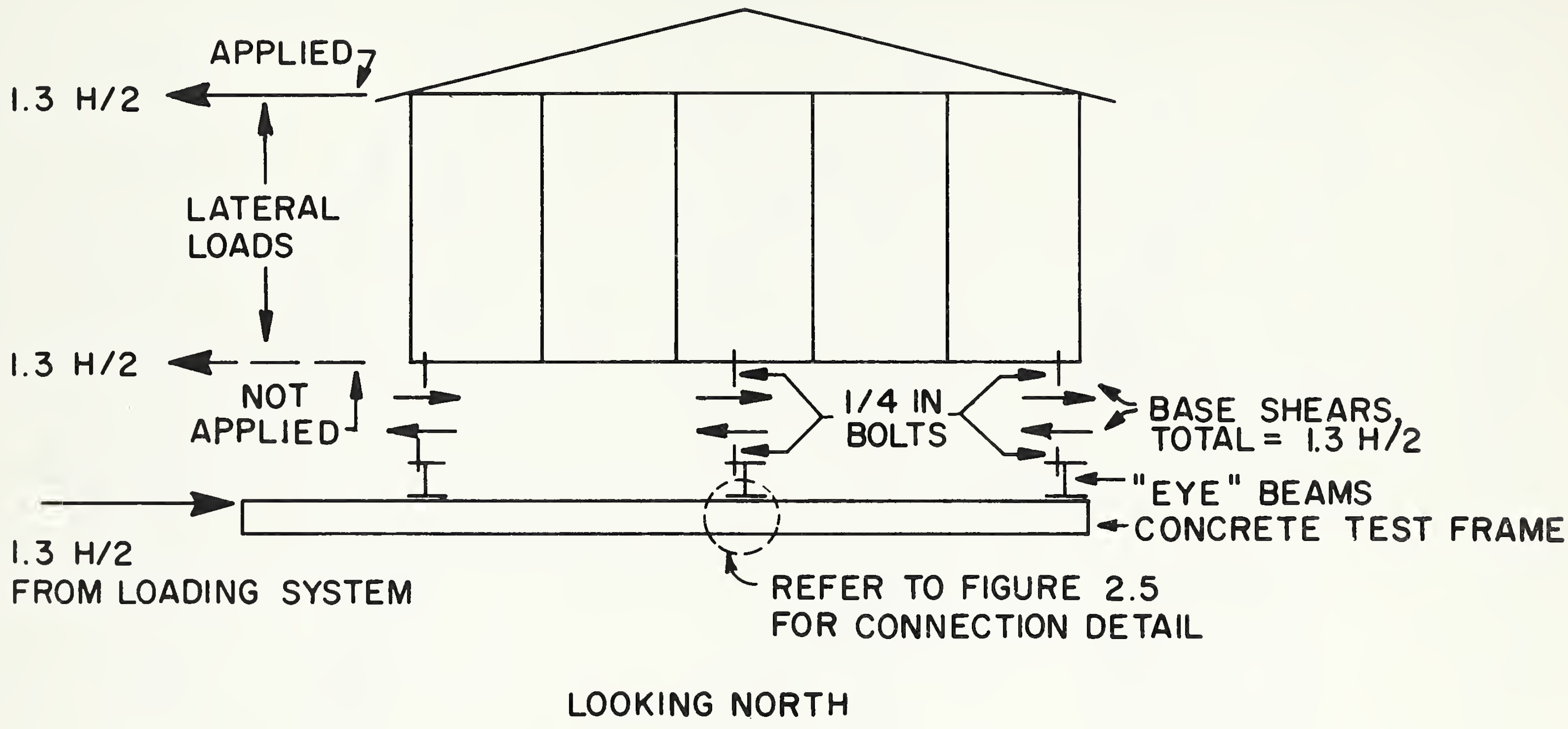


Figure 2.20 Free body diagram showing lateral loads and lateral reactions

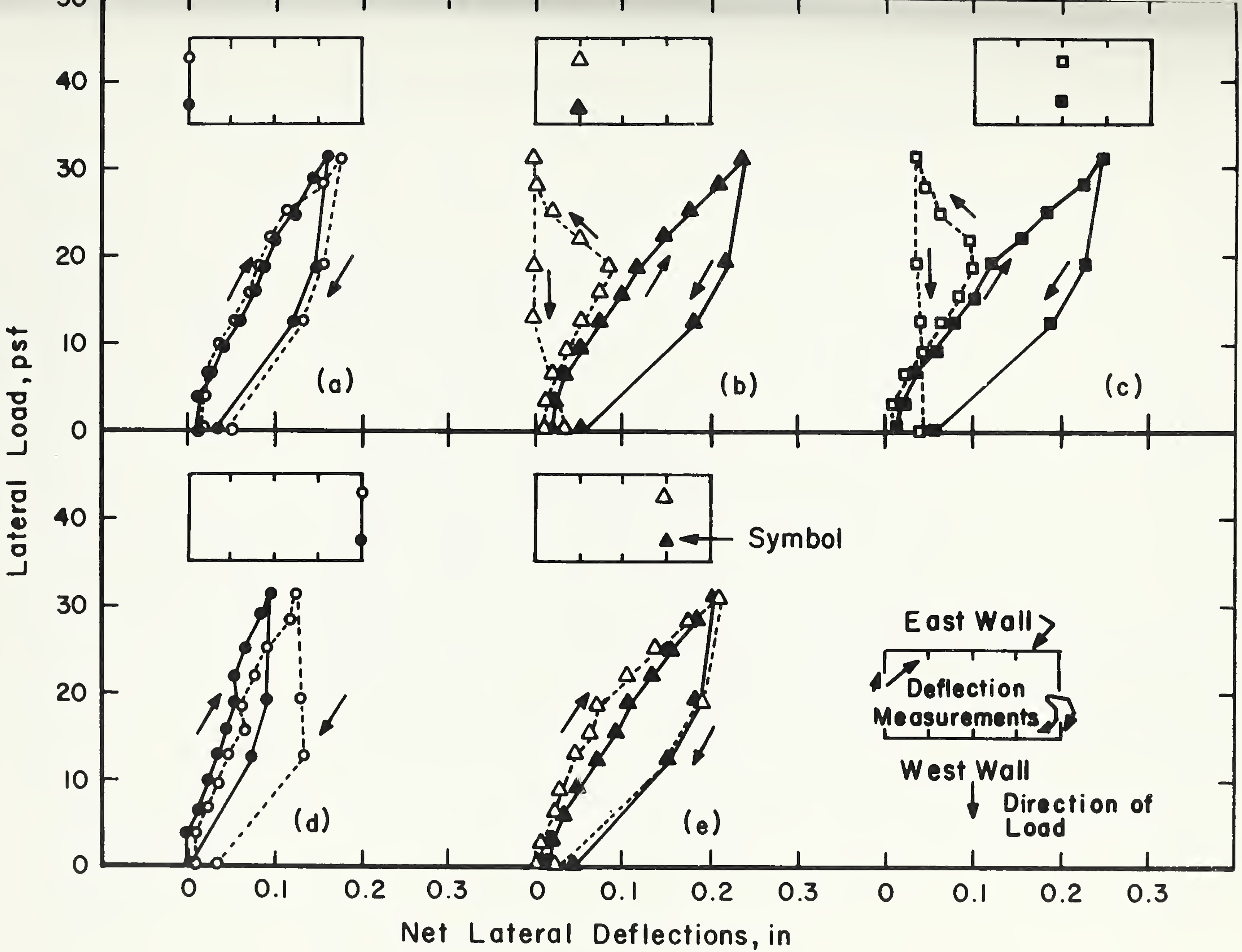


Figure 2.21 Lateral load-deflections during racking test No. 1

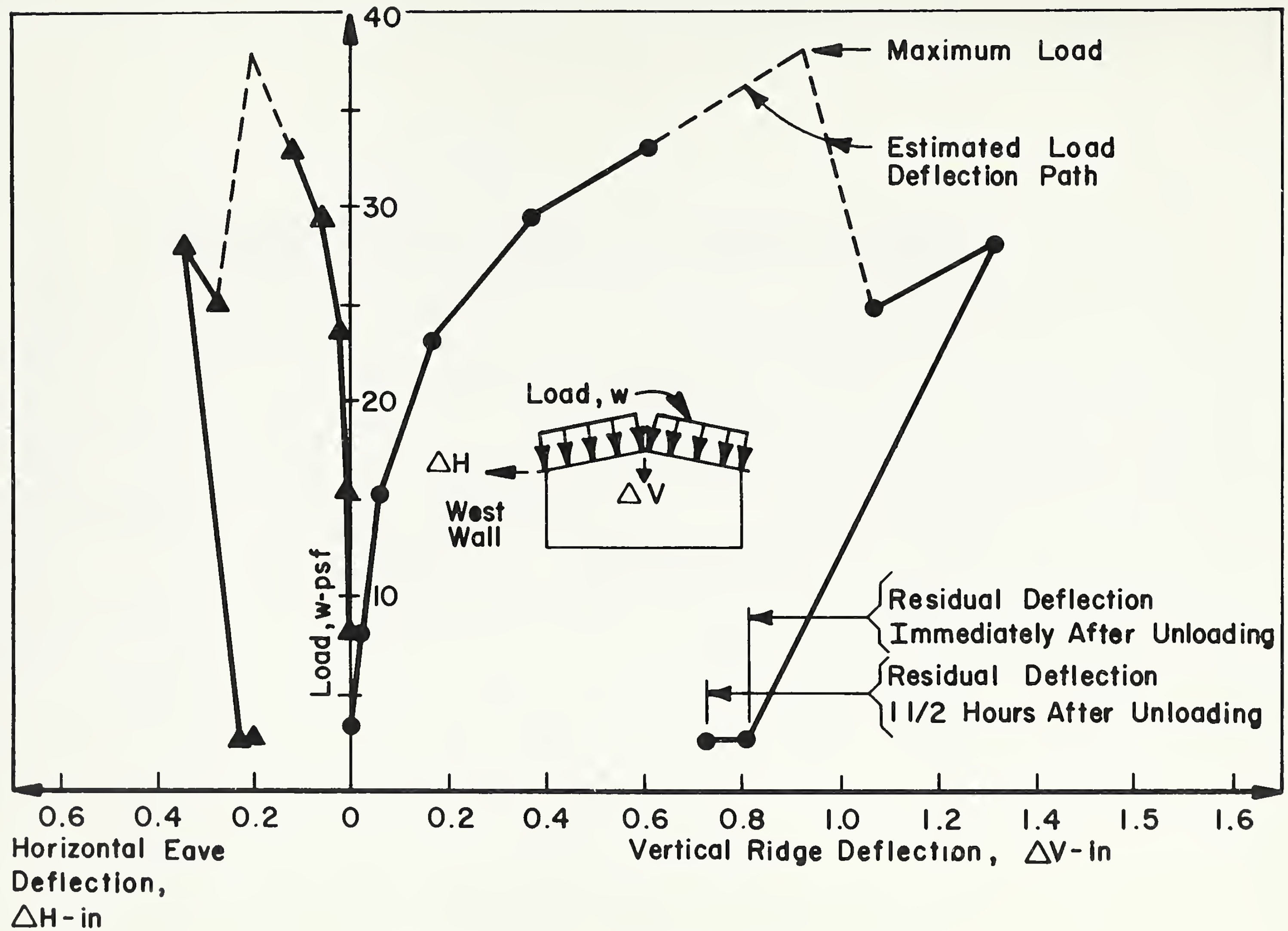


Figure 2.22 Midlength vertical an horizontal load-deflections under roof loading

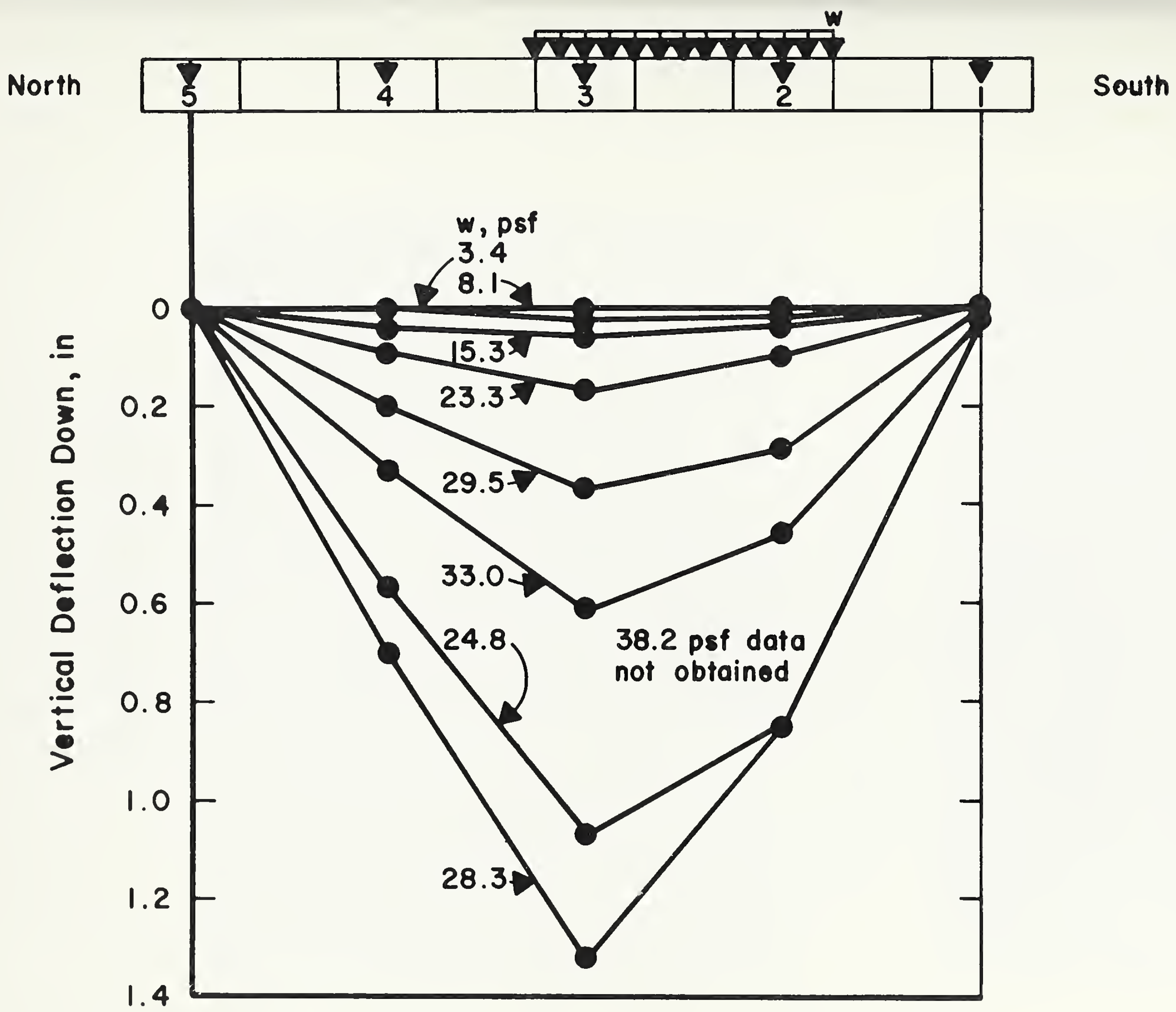


Figure 2.23 Vertical ridge deflections profile under roof load

(a) General interior view before testing

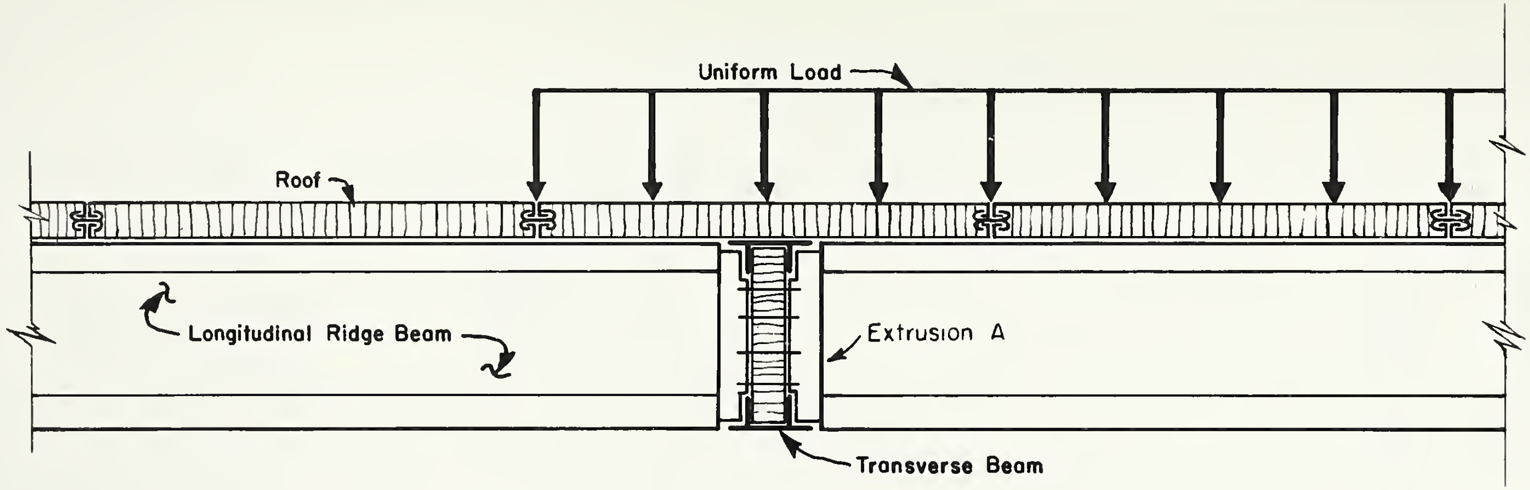
(b) Damage area No. 1, Buckle
at beam to East wall connection

Figure 2.24 Roof load damage areas

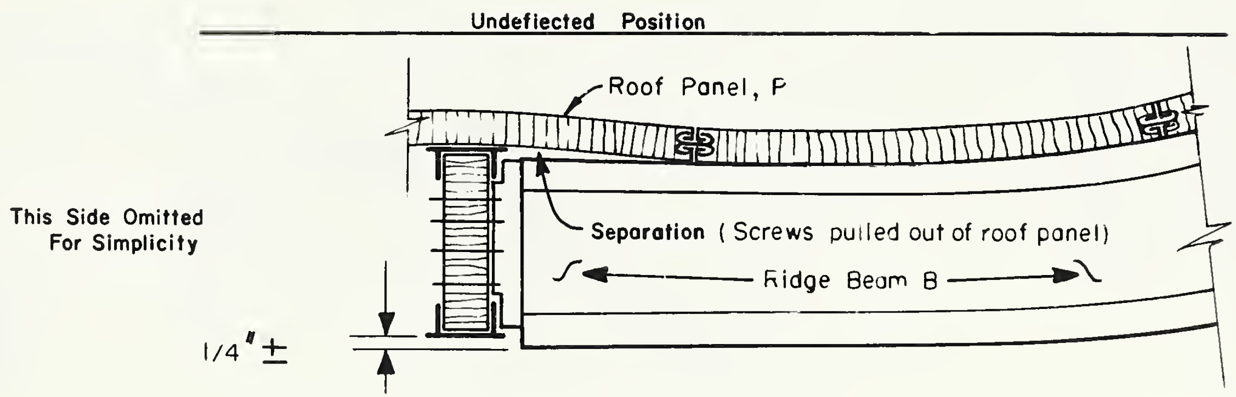
(c) Damage area No. 2, Screws pulled of roof panel at ridge beam to transverse beam connection

(d) Closeup of separation at damage area No. 2

Figure 2.24 Roof load damage areas



a) Longitudinal Section Near Ridge



b) Exaggerated Deflection Shape

Figure 2.25 Separation of Ridge Beams and Roof Panels Under Roof Load.

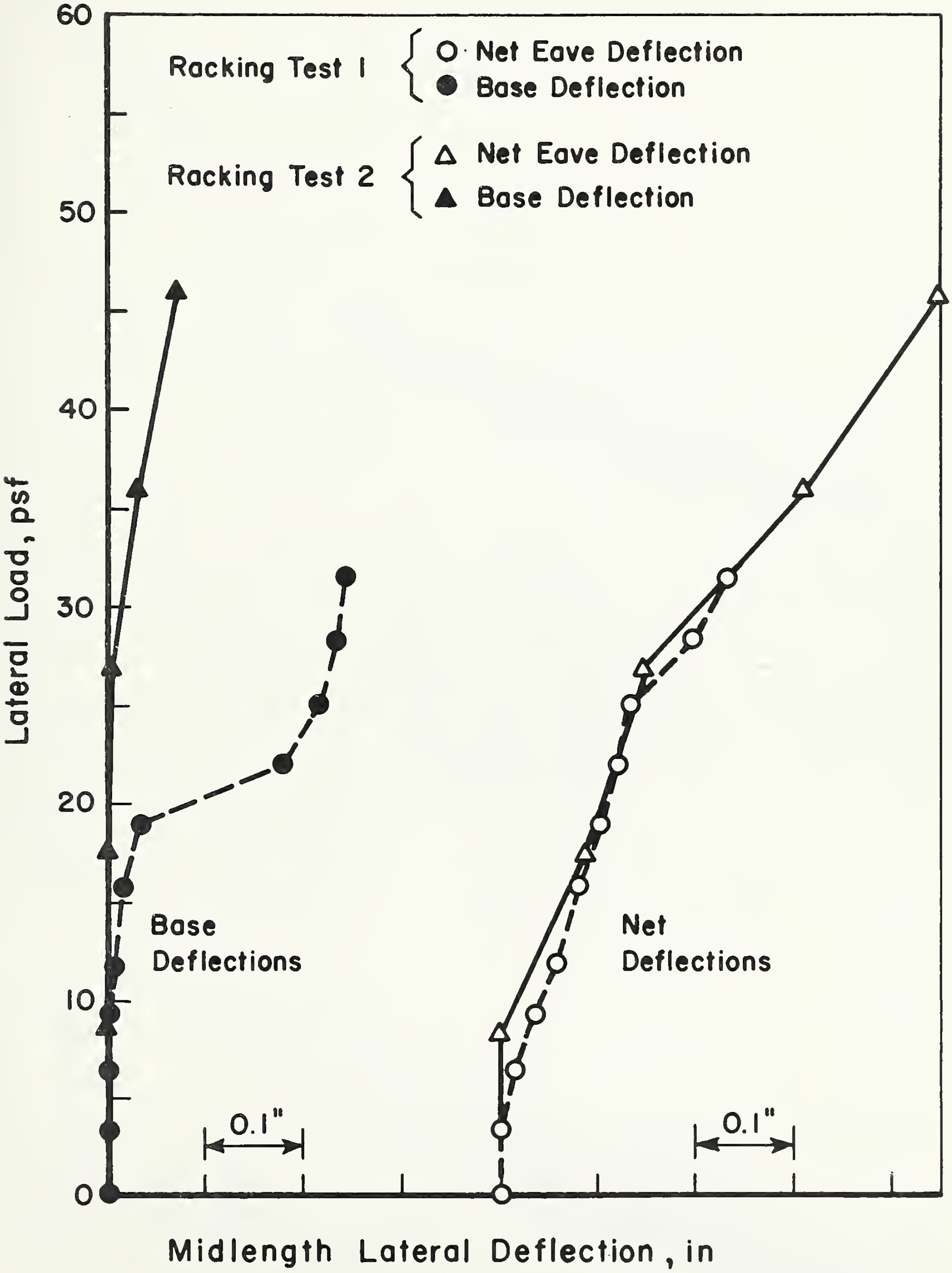


Figure 2.26 West wall load vs. lateral deflection at midlength, racking tests 1 and 2

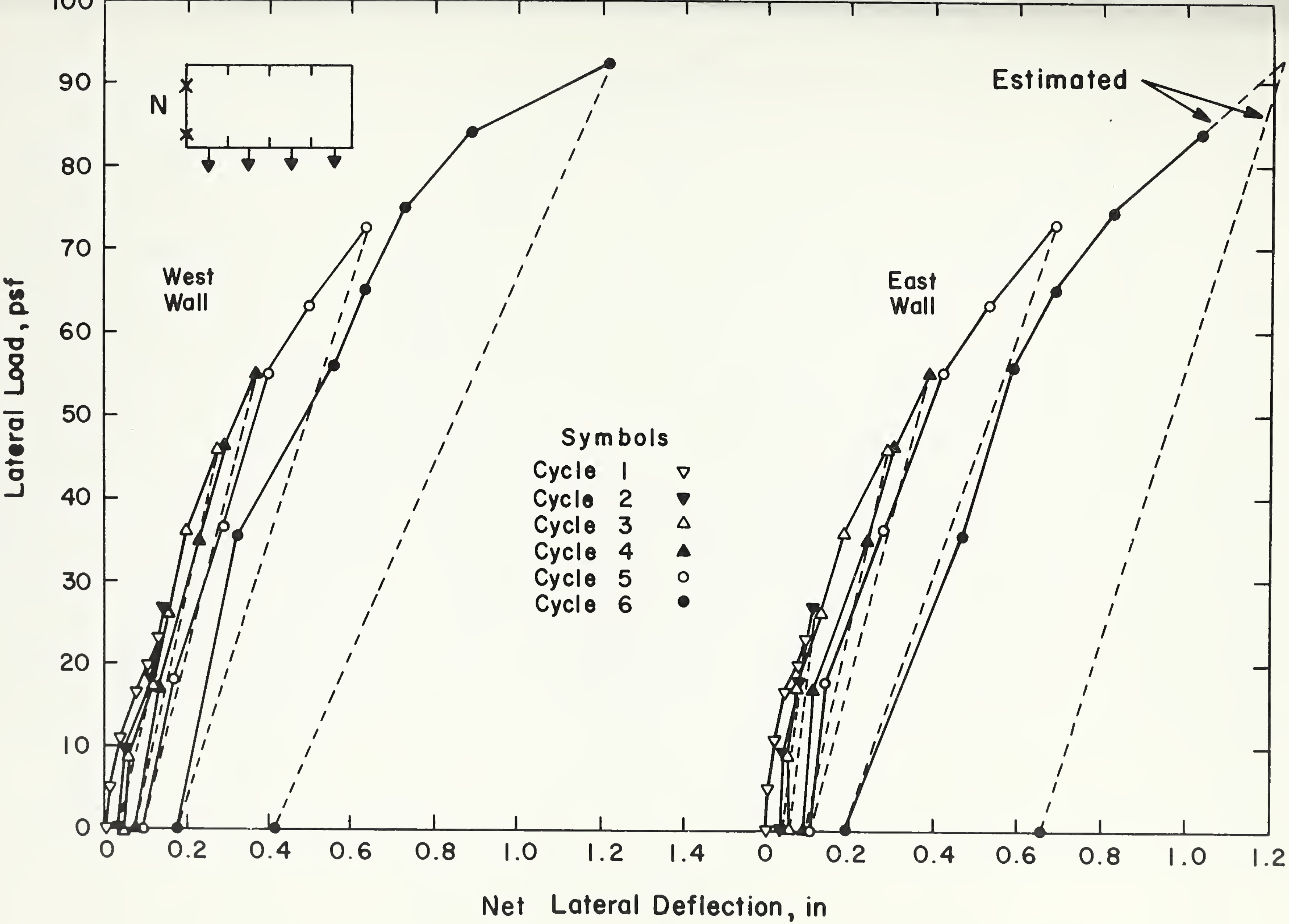


Figure 2.27 Racking test No. 2, load vs net lateral eave deflection at north end

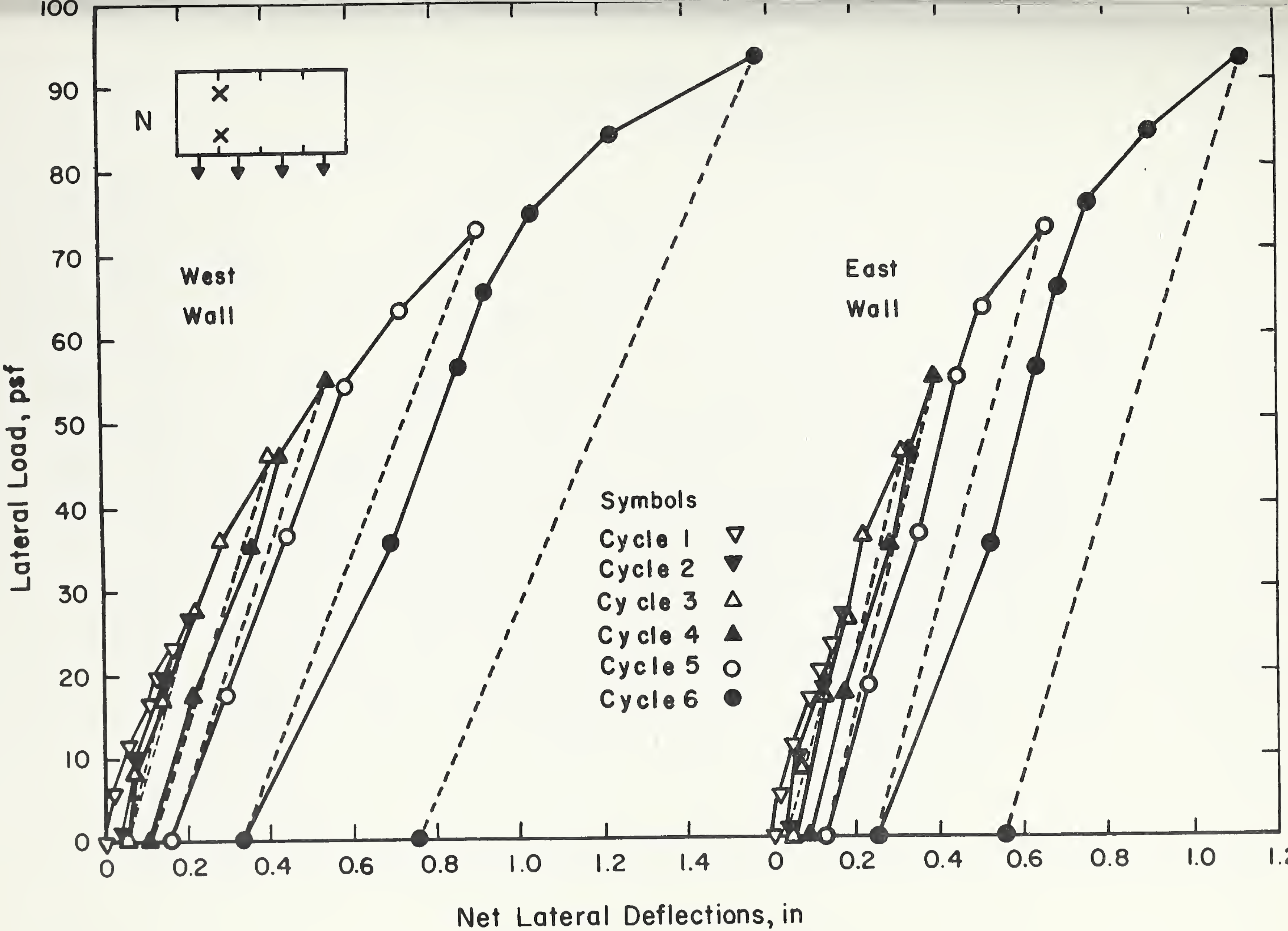


Figure 2.28 Racking test No. 2, load vs net lateral eave deflection at 1/4 length (north)

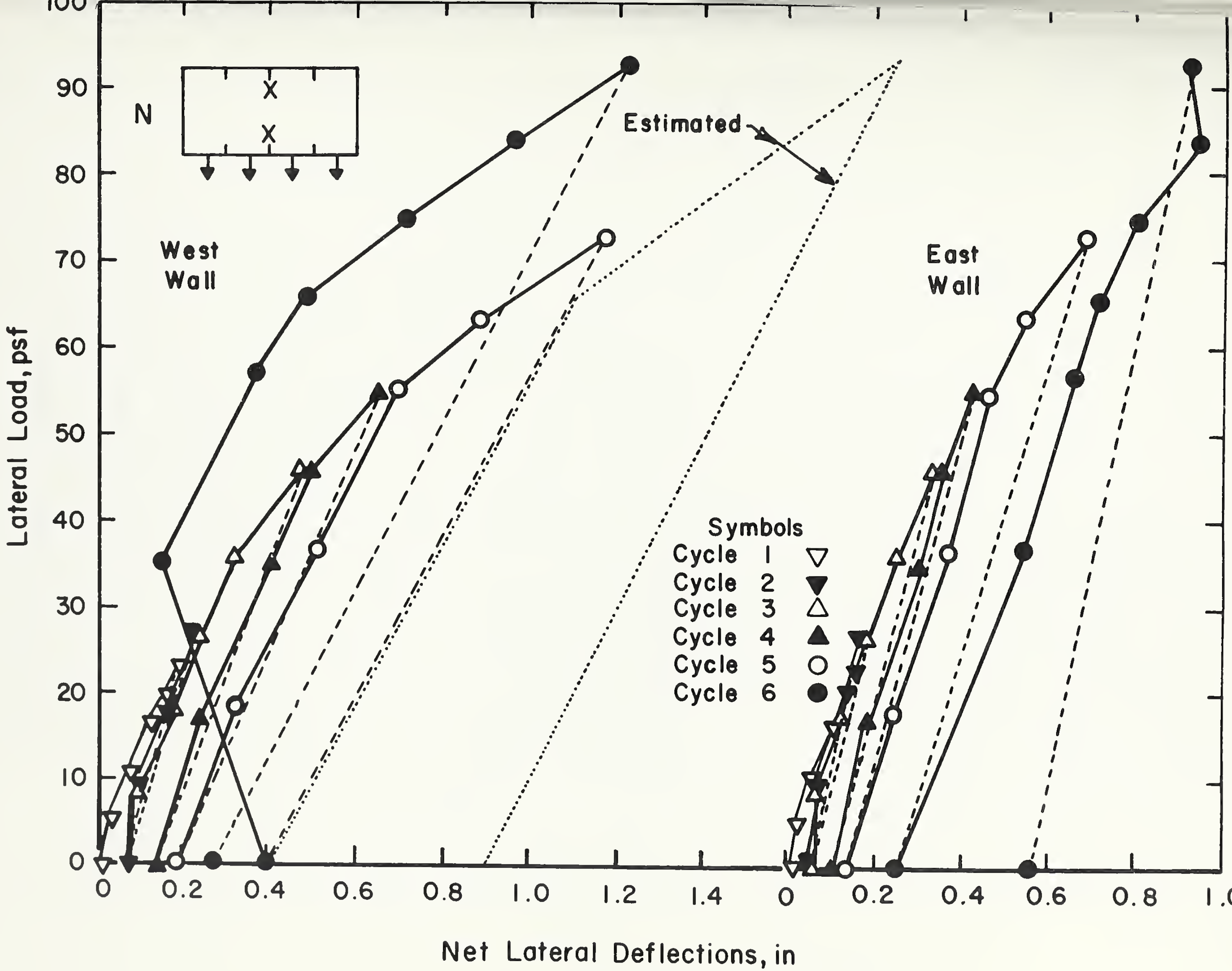


Figure 2.29 Racking test No. 2, load vs net lateral eave deflection at midlength

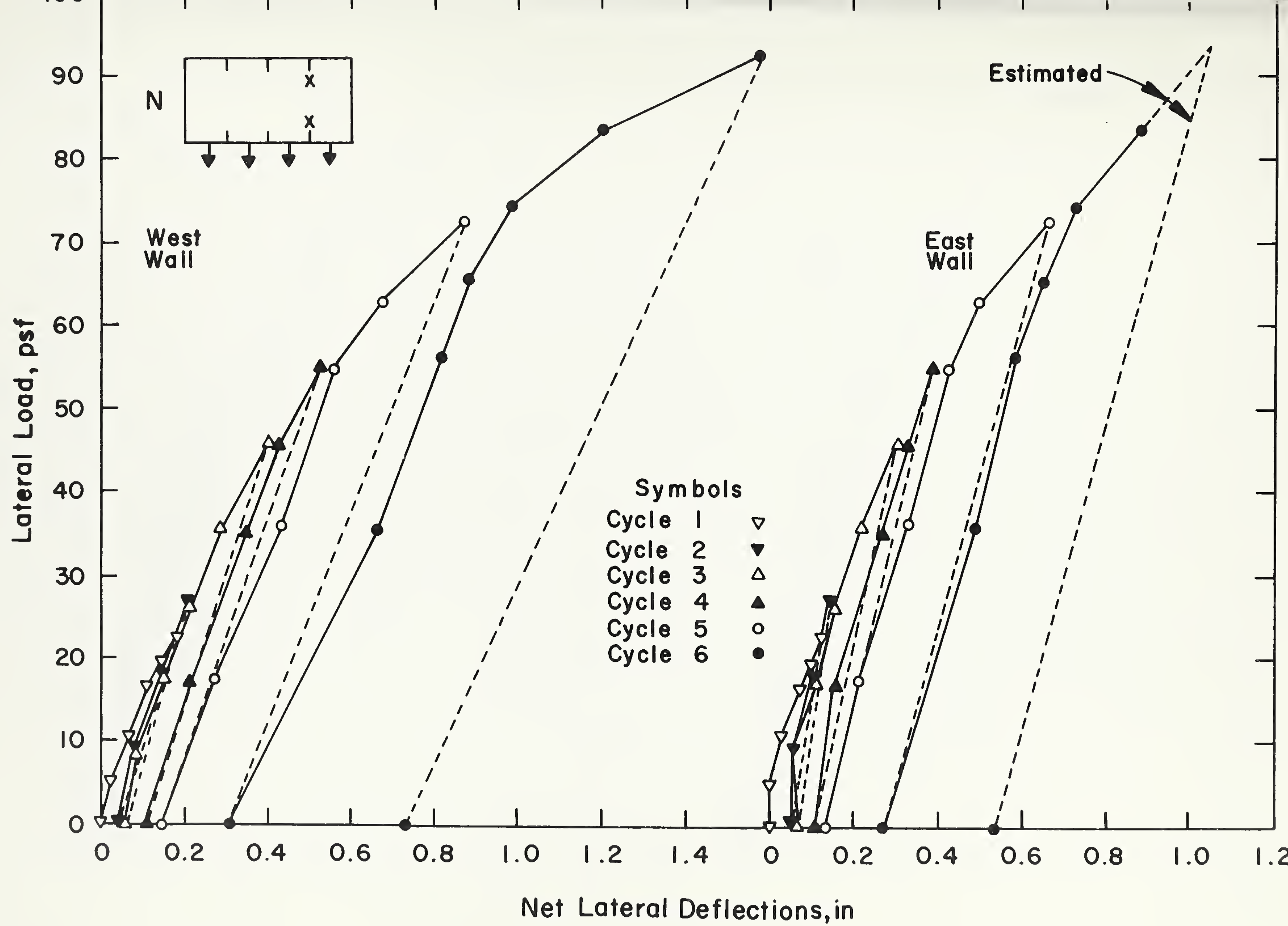


Figure 2.30 Racking test No. 2, load vs net lateral eave deflection at 1/4 length (south)

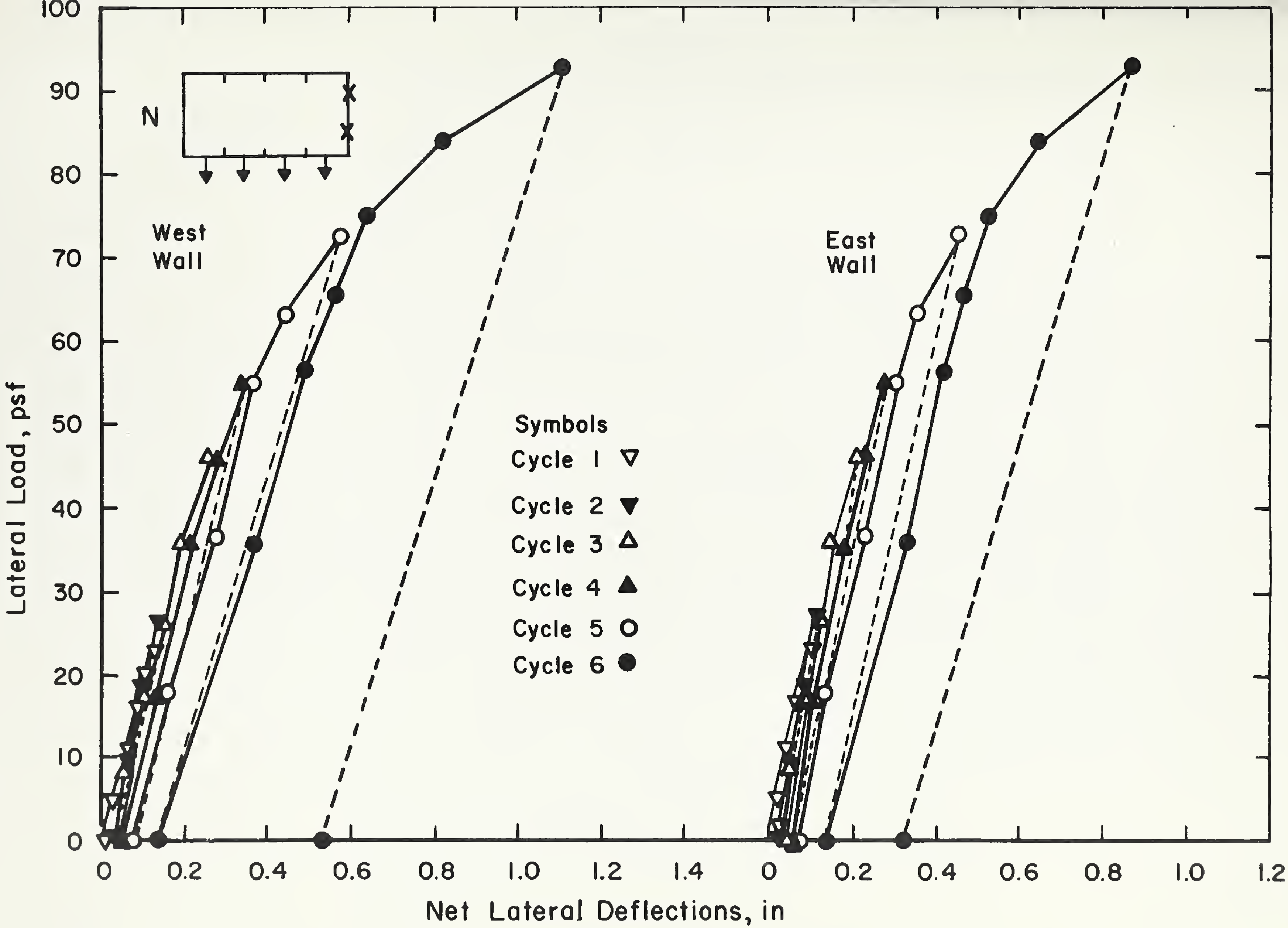


Figure 2.31 Racking test No. 2, load vs net lateral eave deflection at south end

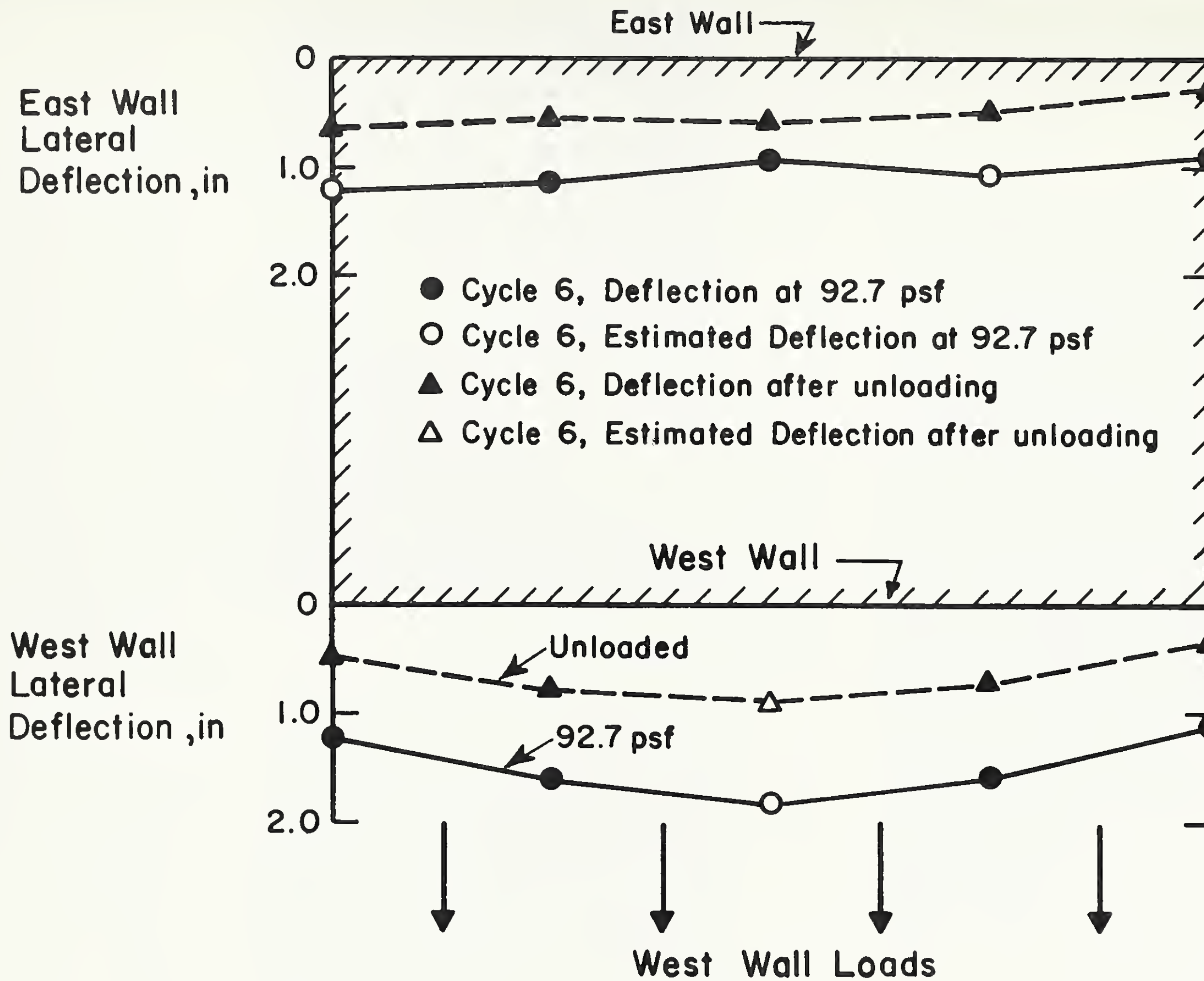
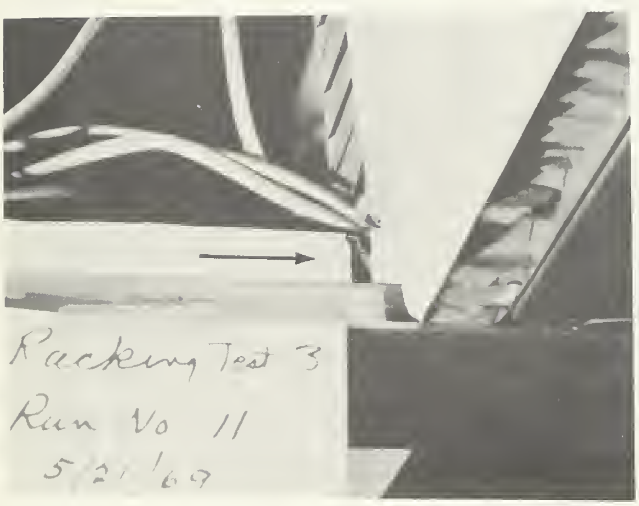


Figure 2.32 Deflected shape of eave line under maximum lateral load, racking test No. 2

(a) General interior view before testing



(b) Damage area No. 1,
fractured bottom chord

(c) Damage area No. 2,
Buckle at transverse beam to
West wall connection

Figure 2.33 Racking test No. 2
damage areas

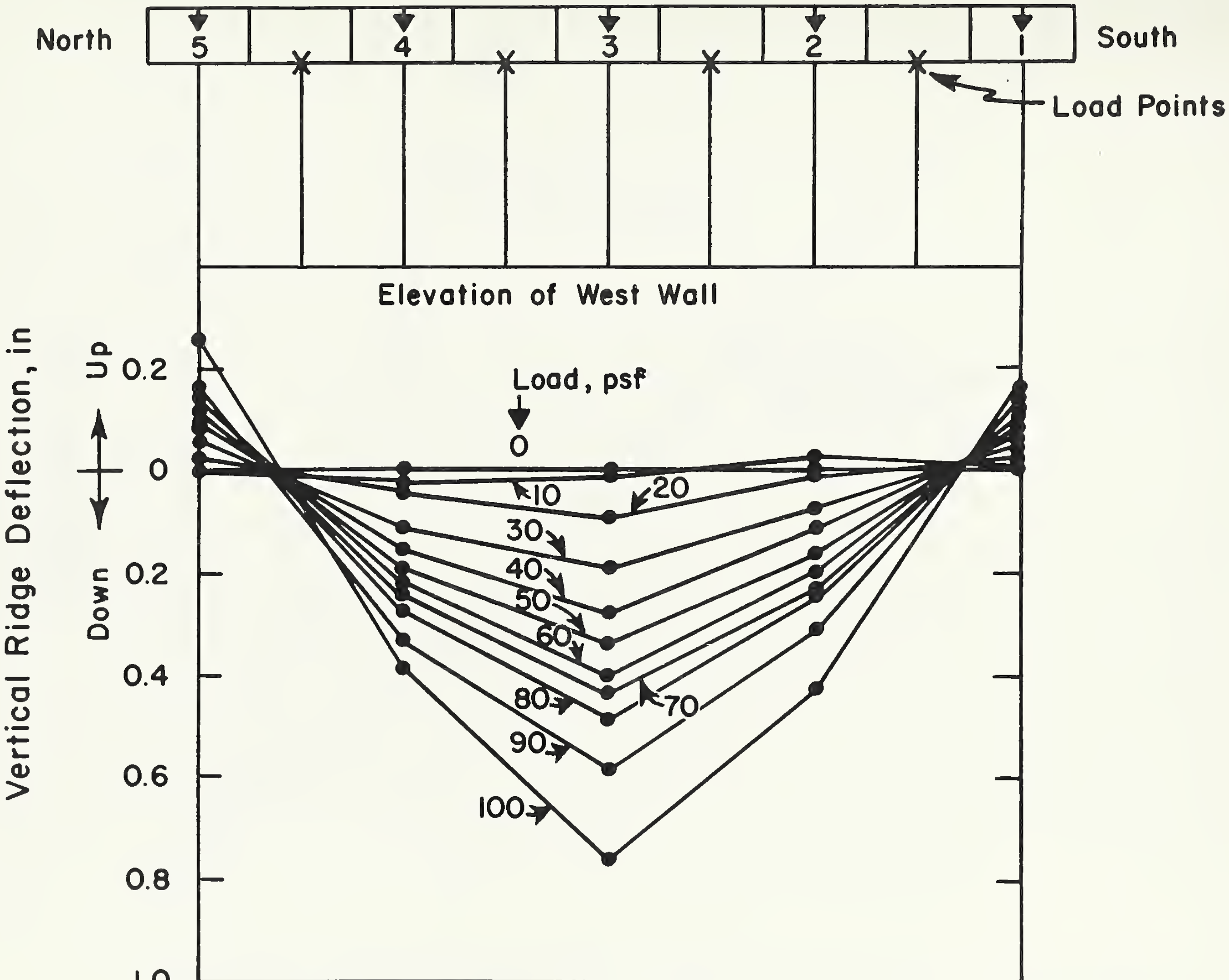


Figure 2.34 Vertical ridge profiles racking test No. 3

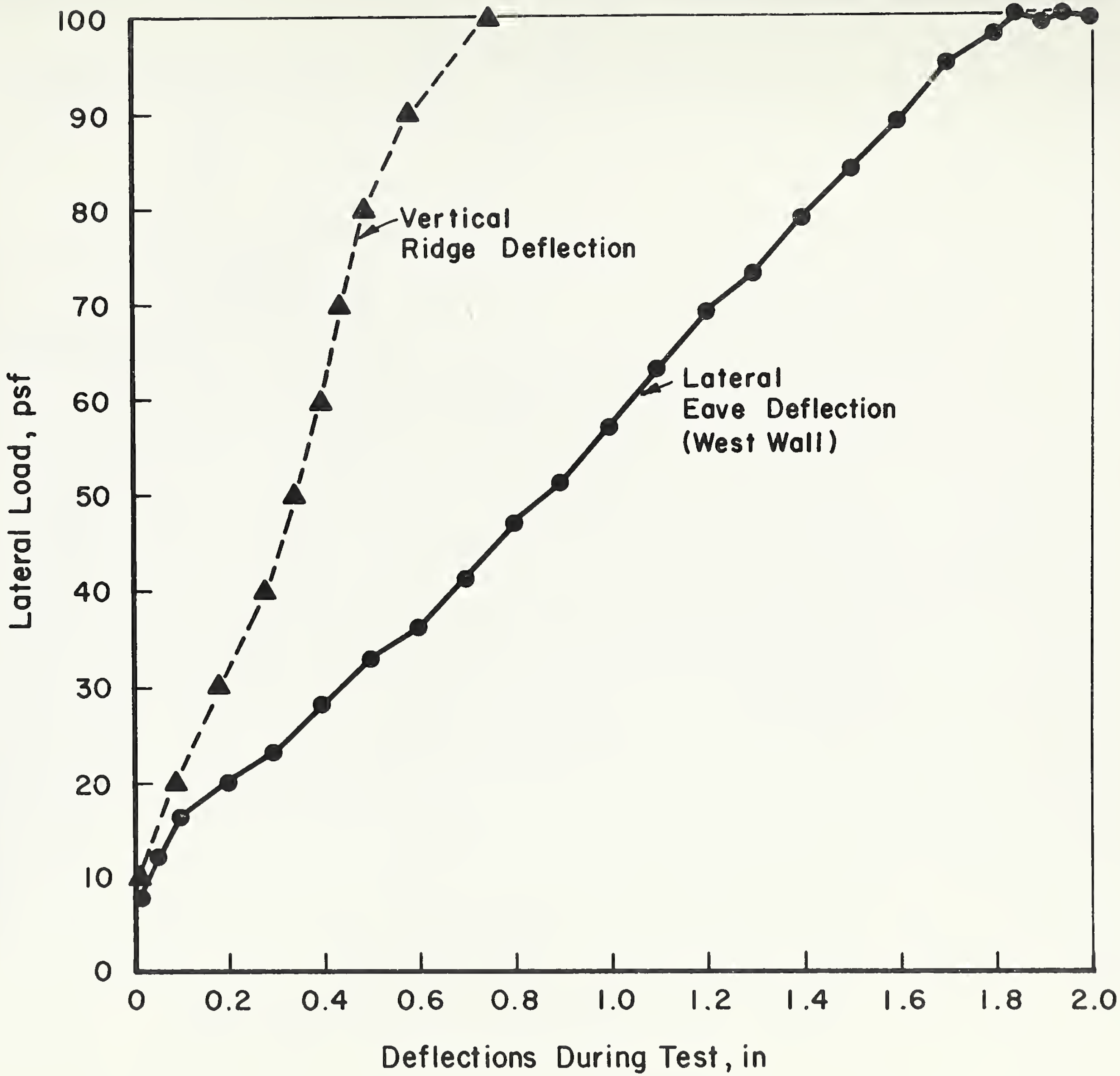


Figure 2.35 Midlength ridge and eave load-deflections, racking test No. 3

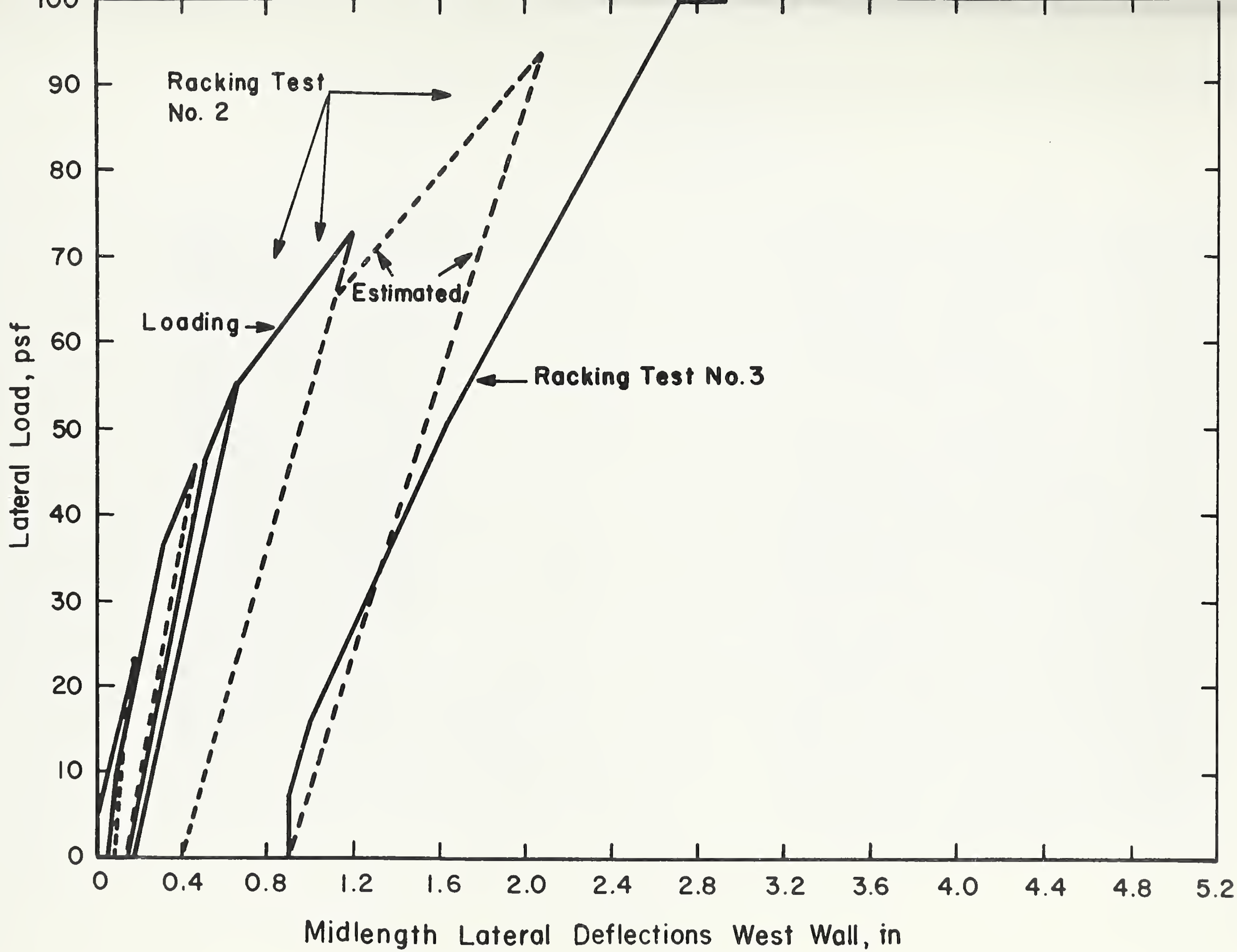


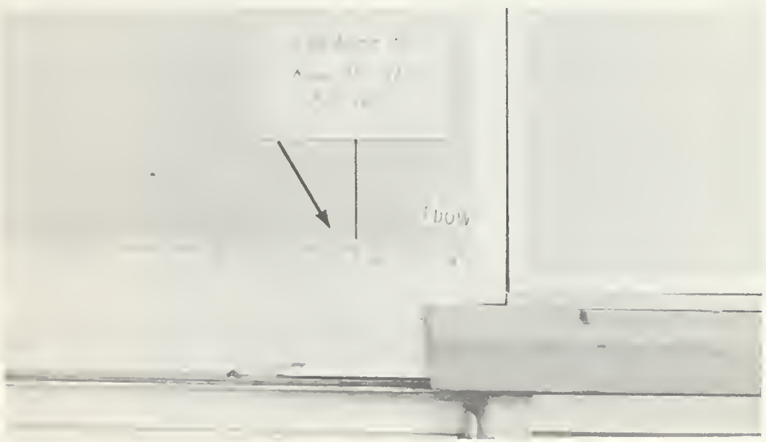
Figure 2.36 Midlength load-deflections for racking tests No. 2 and No. 3

(a) General interior view before testing

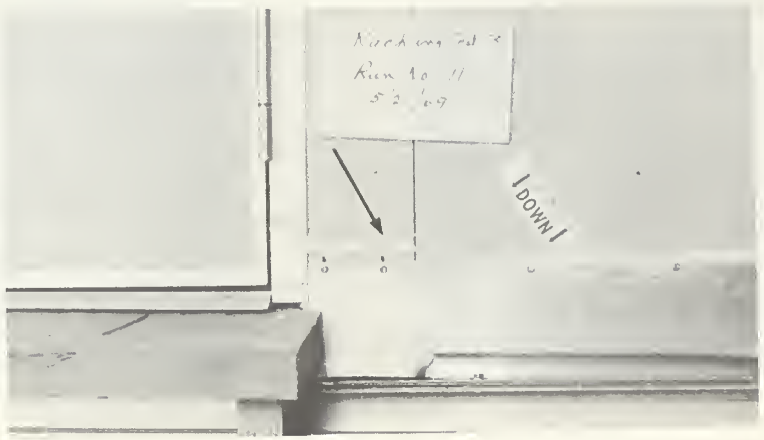
(b) Closeup of damage area

Figure 2.37 Racking test No. 3,
Interior Damage

(a) Exterior view of South end before testing



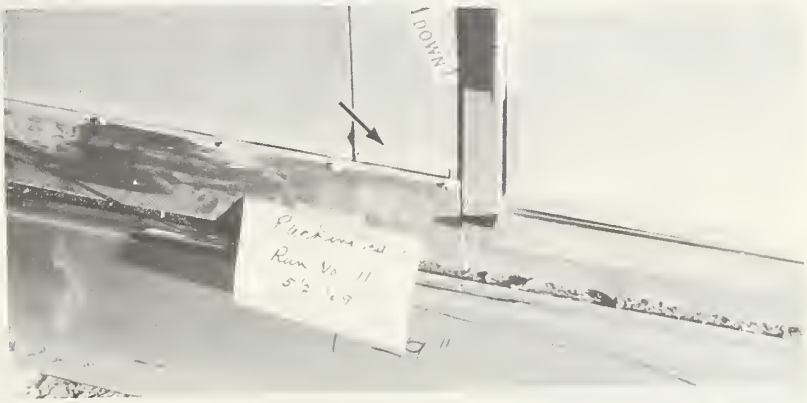
(b) Lower left hand side of South door



(c) Lower right hand side of South door

Figure 2.38 Racking Test No. 3,
Exterior damage

(d) Buckling at top of North door

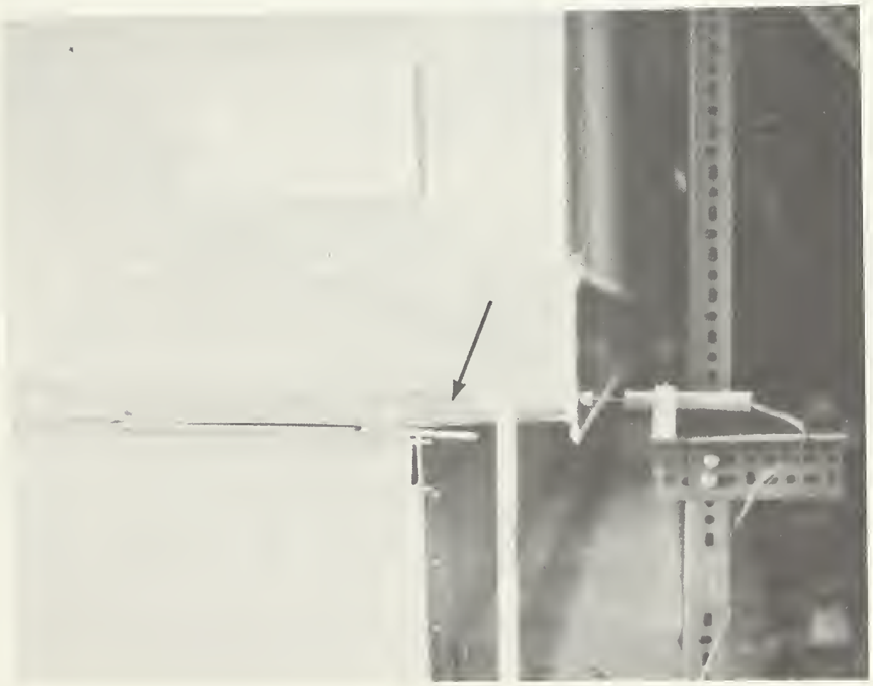


(e) Bottom of North door

(f) Typical buckling at end wall panels

Figure 2.38 Racking Test No. 3,
Exterior Damage

(g) Sheared web of longitudinal beam
at Southwest corner



(h) Uplift at Southeast corner

Figure 2.38 Racking Test No. 3
Exterior damage

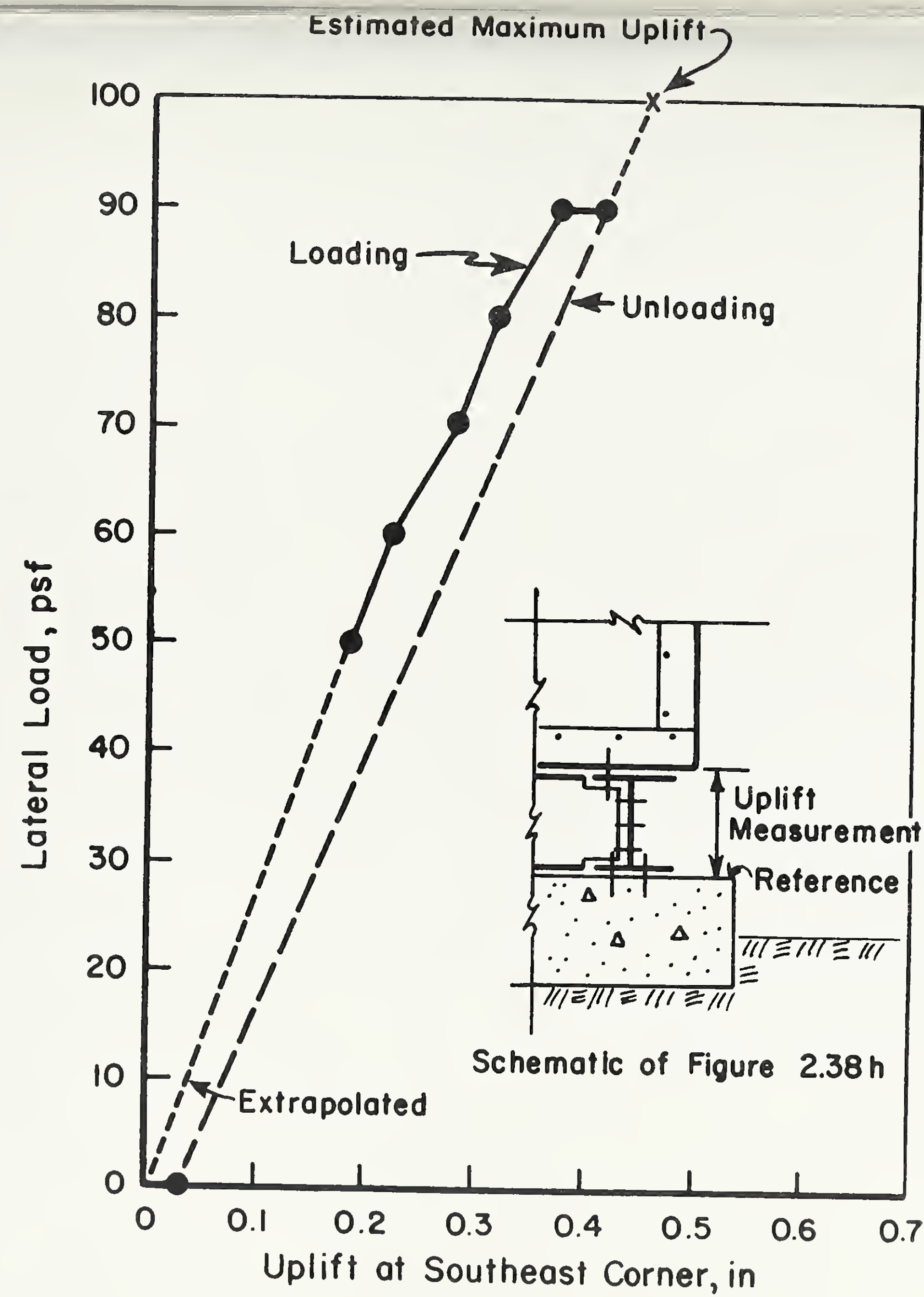


Figure 2.39 Load vs uplift at southeast corner

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