DIAPHRAGM ACTION OF DIAGONALLY SHEATHED WOOD PANELS

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Introduction

Wood structures hold up well under blasts, winds, and earthquakes, according to surveys of damage caused by such forces. As a result, engineers and architects have shown increased interest in the use of wood and are seeking more design information regarding its use. In particular, there are few data on the behavior of wood diaphragms under stress. Tests were made 25 years ago on a few models of timber walls. Later, other tests were made by the Oregon Forest Products Laboratory on 15 quarter-scale models of 20- by 60-foot panels. Test of models, however, did not furnish adequate information, primarily because there is no satisfactory method of correlating the results with test results of full-scale structures. Joint fastenings cannot be accurately scaled down to give data comparable to those obtained from tests of joints made with full-size members and fastenings. The testing of a full-scale unit as it would normally be employed in use is the preferred method of procedure.

In 1952, the Forest Products Laboratory, in cooperation with the Agricultural Engineering Research Branch of the U. S. Department of Agriculture Research Branch, tested the diaphragm action of full-scale, hayloft-floor panels. Three series of tests were made. Two were on 6- by 16-foot panels, and one was on an 18- by 48-foot panel. While these tests provided valuable information, they did not furnish information for use in the design of all types of wood structures. The Oregon Forest Products Laboratory and the Douglas Fir Plywood Association have more recently conducted tests on full-size diaphragms.

\[1\text{-Maintained at Madison, Wis., in cooperation with the University of Wisconsin.}\]
This report presents the results of later tests on full-size floor and wall panels made of wood. The tests were conducted at the Forest Products Laboratory with funds provided jointly by the Division of Architecture of the State of California, the Army Corps of Engineers, the Timber Engineering Company, and the Forest Products Laboratory. The type of construction, the schedule of tests, the variables to be considered, and the manner of testing were for the most part established by the cooperators.

**Part I: Tests of Floor or Roof Panels**

Part I of the report presents the results of lateral bending tests, conducted on full-size roof or floor panels made of wood.

**Description of Material**

The material used in the panels consisted of 1- by 6-inch No. 1 S4S Douglas-fir sheathing and 2- by 6-inch and 2- by 10-inch No. 1 S4S Douglas-fir framing and joists, graded under Sections 187 and 204 of Standard Grading and Dressings Rules for Douglas-fir No. 14, Rev. No. 1, 1948. The 3- by 10-inch and 4- by 10-inch joists and the 2- by 14-inch stiffeners used in some of the panels were laminated from the 2- by 10-inch lumber.

The moisture content of the sheathing lumber averaged about 15 percent, and the moisture content of framing lumber averaged about 20 percent at the beginning of the panel construction. At the time of test, the sheathing had dried to 8 to 12 percent, and the framing had dried to 10 to 13 percent moisture content.

**Description of Test Panels**

Eleven series of tests were made on 20- by 60-foot panels. Various details of sheathing and framing were incorporated in these panels. The distinguishing characteristics of the panels are shown in figures 1, 2, and 3, while construction details are shown in figures 4 to 11. The tests were made out-of-doors on a paved lot (fig. 12). Eight tests were made on panels with 2- by 10-inch joists spaced 2 feet apart, two tests were made on panels with 4- by 10-inch joists spaced 6 feet apart, and one test was made on a panel with 3- by 10-inch joists spaced 4 feet apart. The joists were continuous over the 20-foot span. The sheathing was laid at an angle of 45° to the framing members and nailed at each bearing with common wire nails. Three nails were driven at the ends of each sheathing board, and two were driven at each intermediate joist. To simulate service conditions resulting from shrinkage, the
sheathing boards were spaced 1/8 inch apart, and a separation of 3/64 inch was provided between the sheathing and the framing. Eightpenny nails were used to nail the 1-inch sheathing, and sixteenpenny nails were used to nail the 2-inch sheathing.

Testing Procedure

The panels were tested in bending under fifth-point loading (fig. 12). The load was applied to the panel by a system of I-beams, which were loaded through a wire rope that was attached to the movable head of a testing machine. The load was measured by a hydraulic capsule, mounted between the cable and the outermost I-beam. The loads are expressed in pounds per foot of panel width. This value is the reaction load divided by the panel width.

An initial load of 12.5 pounds per foot of panel width was placed on the panels. The panels were then loaded at intervals of 25 pounds per foot. After each load interval had been applied and the deflection readings had been made, the load was reduced to the initial load, and the deflection was again read to provide a measure of the set retained in the panel.

The lateral deflection of the panels was measured at a number of points along each end joist and along the unloaded header. Measurements were also made of the joint slip in the header splices and at the ends of a number of sheathing boards. The deformation in the plane of the panels was also obtained.

Discussion of Results

Panel stiffness is compared in table 1 and by the load-deflection curves shown in figure 13. The curves show the deflection at the center of the unloaded edge of the various panels. The table presents the deflection at the center of the unloaded edge and at the center of the end joists for each 250 pounds per foot of panel width and at the end of the test. The residual deflection after each of the above load increments is also listed in table 1.

The test results show that the panels FA1, FF1, FF2, and FF3, constructed with let-in blocking at the ends of the joists, deflected more at a given load than the panels with continuous headers. They also carried a smaller maximum load or smaller load at the termination of the tests than the panels with continuous headers. The reinforcement of the corners of panel FF2 and the additional nailing in the outer 4 feet of the perimeter of panel FF3 improved their stiffness over that of panel FF1 but not enough to equal that of the poorest panel with continuous headers and end stiffeners. The tests of panels FF2, FF3, and FG2 were, however, a continuation of the loading of panels FF1, FF2, and FG1, respectively. Previous tests of conventionally nailed panels have shown that during the early portion of the tests, the deflections that result from a subsequent loading of the same magnitude as
the initial loading will be approximately the same as the initial deflection minus the residual set. Because of the modifications of the panels between tests, however, the deflection that results in the subsequent tests may not conform to previous panel tests, and therefore the results of the tests are not directly comparable within the series or with the results of the other test panels.

Panel FC1, with both stiffened end and longitudinal chords, was the stiffest and strongest of the panels with 1-inch sheathing on 2- by 10-inch joists spaced 2 feet apart. The stiffness of panel FC1 was exceeded only by that of panels FH1 and FG2 with thicker sheathing and heavier joists. The panels whose headers showed the least amount of joint slip at their ends and splices were generally stiffer than panels whose headers lengthened during test, particularly those with let-in headers. Panel FE1 with continuous headers and V or herringbone sheathing acting in compression was stronger and stiffer than the similarly framed panel F1B with diagonal sheathing, whereas panel FF1 with let-in headers and V or herringbone sheathing acting in compression was not so strong or as stiff as the similarly framed panel FA1 with diagonal sheathing. The sheathing applied in the herringbone pattern provided symmetry in panel action and deflection. The panels with 1-inch sheathing showed a decrease in stiffness with a decrease in the length of sheathing boards, but the decrease was not in proportion to the decrease in board length. The panel with short sheathing boards (extending over two joist spaces) emitted load noises during test.

Panel FH1 with 2- by 6-inch sheathing on 3- by 10-inch joists spaced 4 feet apart was the strongest and stiffest panel tested. The similarly constructed panel FG1 with 2- by 6-inch sheathing on 4- by 10-inch joists spaced 6 feet apart showed considerably less rigidity for shear loads below 400 pounds per foot than panel FH1 and was less rigid than panels FB1, FC1, and FE1 with 1- by 6-inch sheathing on 2- by 10-inch joists spaced 2 feet apart. The stiffness of panel FG1 was improved and corresponded generally with that of panel FH1 after 2- by 6-inch pieces were nailed flatwise to the underside of the sheathing midway between the joists as on panel FG2. Before reinforcement the sheathing of panel FG1 was very springy, and some pieces of sheathing deflected more than an inch under the weight of a man. The shape that the panels assumed under the applied load varied in accordance with the action of the sheathing. The deformation that occurred along the perimeter of panel FH1 with 2-inch sheathing is shown in figure 14, and the deformation at two planes across the width of the same panel at given intervals of load is shown in figure 15.

Digest of Test Results

The testing program of roof or floor diaphragms indicated the following results:

(1) The floor diaphragms framed with continuous headers were more rigid than those with solid blocking let in at the ends of the joists.
(2) The fastenings that permitted a minimum of nonelastic slip at the header ends and splices promoted greater panel rigidity.

(3) The floor panels with perimeter stiffeners were stronger and more rigid than the panels with only end stiffeners.

(4) Sheathing applied in V or herringbone pattern provided symmetry in panel action and deflection.

(5) The stiffness of the diagonally sheathed floor panels increased as the length of the boards increased but not in proportion to the board length.

(6) Additional nailing along the perimeter of the panel increased the panel stiffness.

(7) The floor panel framed with 3- by 10-inch joists spaced 4 feet apart and diagonally sheathed with 2- by 6-inch lumber provided greater rigidity than the similarly constructed panel with 4- by 10-inch joists spaced 6 feet apart. It was also more rigid than the diagonally sheathed floor panels with 1- by 6-inch sheathing on 2- by 10-inch joists spaced 2-feet apart.

(8) The floor panel framed with 4- by 10-inch joists spaced 6 feet apart was not as rigid within the limits of the test (400 pounds per foot of panel width) as the similarly constructed panel with 1- by 6-inch sheathing on 2- by 10-inch joists spaced 2 feet apart.

(9) The stiffness of the panel with 4- by 10-inch joists spaced 6 feet apart corresponded generally with that of the panel with 3- by 10-inch joists spaced 4 feet apart after 2- by 6-inch pieces were nailed flatwise to the underside of the sheathing midway between the joists.

(10) The diagonal sheathing of S4S 2- by 6-inch lumber on 4- by 6-inch joists spaced 6 feet apart was springy and deflected more than an inch at midspan between joists under the weight of a man. Much of the springiness was removed after the 2- by 6-inch pieces were nailed flatwise to the underside of the sheathing midway between the joists.

(11) In the diagonally sheathed panels, the accumulated slip in the sheathing joints was greater for a particular member acting in tension than for a corresponding member at the opposite end of the panel acting in compression.

(12) The greatest joint slips of the sheathing boards occurred at the intersection of the long diagonal boards and the corners of the panel.

(13) The accumulated slip of the several joints in the long diagonal boards that intersected the corners of the panel was from 24 to 85 percent greater than that of their immediately adjacent sheathing boards at a shear load of 250 pounds per foot of panel width. The relative difference in the joint slip
of adjacent boards decreased with an increase in load, and, at 1,000 pounds per foot, the corner boards showed only 9 to 29 percent more slip than their adjacent boards.

**Part II: Tests of Wall Panels**

The tests on wall panels were conducted on a dual panel that represented two sections of frame wall of a building with a story height of 12 feet. Seven series of tests on wall construction that incorporated various details of sheathing and framing were included in the testing program. The two sections of each panel were constructed of full-size members and bolted to a center beam. The sections were laterally loaded simultaneously through the center beam in the direction of their length. An investigation of the effect of the height-length ratio was made by cutting the initial 17-foot 4-inch panel to a 12-foot panel and later to a 5-foot 4-inch panel.

**Description of Material**

All lumber used in the wall panels was Douglas-fir graded under Sections 187 and 204 of the Standard Grading and Dressing Rules for Douglas-fir No. 14, Rev. Nov. 1, 1948. Panels WA and WB were framed with No. 2 lumber, and the remaining panels were framed with No. 1 lumber. The sheathing on all panels was No. 1. At the time of test, the moisture content of the framing varied from 10 to 12 percent, and that of the sheathing varied from 8 to 10 percent.

**Description of Test Panels**

Seven series of tests were conducted on wall constructions that incorporated various details of sheathing and framing. The tests were made on a dual panel shown in figure 16. The panel represented two sections of a frame wall of a building with a story or ceiling height of 12 feet. Figures 17, 18, and 19, show the important structural differences in the panels, while table 2 gives more detailed information on distinguishing characteristics.

The two sections of the test panel were constructed of full-size members and in general represented the construction used in frame buildings. The panels were framed with 2- by 6-inch studs and plates and 3- by 6-inch sills. The studs were spaced 16 inches on center. Double studs and double plates were used along the edges of the panels. The framing was nailed with sixteenpenny and twentypenny nails. The sills of the two panel sections were bolted to a common 8- by 8-inch timber.
All panels were reinforced at their corners with corner brackets of heavy angle iron except for the initial test of panels WA, WB, and WT. Solid blocking was offset and end nailed between the studs of each panel.

The sheathing was laid diagonally and nailed with eightpenny nails. Three nails were driven at the ends of each sheathing board, and two were driven at each intermediate bearing. The sheathing was not nailed to the solid blocking or stays. A 1/8-inch crack was left between the sheathing boards and a 3/64-inch separation was left between the sheathing and framing. Continuous sheathing boards and duplex-head nails were used to construct panels WA and WB.

An investigation of the effect of the height-length ratio was conducted on some of the panels by cutting the initial panel that was 17 feet, 4 inches long to a panel 12 feet long and later to a panel 5 feet, 4 inches long. For each reduction in size, the panels were taken apart, the frame reassembled, the sheathing trimmed to remove the nail holes, and the boards replaced in the same general order that they occupied in the larger panel.

Panel WD (fig. 17) was used as the control panel. The sheathing on both sections of the panel was arranged to act in compression, and all corners of the panel were secured with the heavy steel angle brackets.

Panel WA was constructed the same as panel WB. They were of conventional construction with sheathing boards that spanned the full length of the frame diagonally and sloped in the same direction on both sections of the test panel. This method of application placed the sheathing in tension on one section and in compression on the other. For test runs 1 and 2, corner brackets were applied only at the corners on the loaded edge of the panel. During test, the compressive action of the sheathing caused the double plate to pull from the studs at the lower corner of the section on which the sheathing was in compression. For test run 3, a corner bracket was attached to the lower corner of the section where the framing members had separated about an inch.

In panel WC, the sheathing was applied so that it acted in compression on each panel section. Horizontal stays or ties, of 1- by 6-inch boards were let in at the third points of the studs. During the test, when the distortion had become appreciable but before any failure occurred in the members, commercial framing anchors of 18-gage, zinc-coated sheet steel were attached to the unsheathed edge on the ends of all intermediate studs.

Panel WE was diagonally sheathed on both faces with the sheathing on one face acting in compression and on the other face in tension. Corner angle brackets were attached at all interior corners and on the outer edges of the double studs at the center post.

Panel WF was tested only as a 12- by 12-foot panel. It was constructed in the same manner as the control panel except that the sheathing was applied to act in tension rather than in compression.

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Panel WT was tested only as an 18- by 18-foot panel. It was the same as the control panel except that two commercial framing anchors of 18-gage, zinc coated, sheet steel were placed at the end of each stud. The corner angle brackets were not used for the initial test but were attached after the 3- by 6-inch sill had started to split.

**Testing Procedure**

The dual panels were tested in bending under center loading (fig. 16). For convenience in testing, the panels were placed in the testing machine with the studs in a horizontal position. The load was applied to the panels through the 8- by 8-inch center post to which the panel sections were attached. A hydraulic weighing capsule measured the load at one reaction of the panel. The loads are expressed as shear load in pounds per foot of panel length. This value is the reaction load divided by the panel length.

An initial load of 58, 85, and 94 pounds per foot of length was placed on the 18-, 12-, and 6-foot panels of the WC, WE, and WT series. For panels WA, WB, WD, and WF, an initial load of 29, 42, and 47 pounds per foot of length was placed on the 18-, 12-, and 6-foot panels. The subsequent loading intervals were the same as the initial load intervals. After each load interval had been applied and the deflection readings had been made, the load was reduced to the initial load, and the deflection was again read to provide a measure of the set or residual deflection in the panel. The deflection of the panel was measured at the center post and at several points along the perimeter of the panel (figs. 17, 18, and 19).

**Discussion of Results**

A comparison of the strength and stiffness provided by panels of various constructions and of various height-length ratios is given in table 3 and by the family of curves in figure 20. For panels WA, WB, WC, and WT they are composite curves for two or more runs and were obtained by taking the curve for the first run to its termination, then the portion of the second run beyond the first, and finally the third run beyond the second. The residual deflection of each run was the starting point for each subsequent run. The table presents a comparison of the strength and stiffness of control panel WD with strength and stiffness of the panels of other constructions. Panel WD was taken as 100 percent.

A comparison of the load-deflection curves for the three height-length ratios of the control panel is given in figure 21. The broken lines show the residual deformation or set that remained in the panels between each load interval. The deformation that resulted along the perimeter of panel WD-18 is illustrated in figure 22.
Panels WA and WB, in which a large joint slip occurred at the outer lower corner of the panel with the sheathing in compression, were less rigid than the panels with the corners secured with steel angles. The addition of the corner angles to these panels strengthened them, so that their maximum test loads were about equal to those of the control panels.

Panel WC, with let-in ties or restraining members and corner angles, showed very limited distortion of the panel frame and sustained higher loads at a given deflection than the other single-sheathed panels. The application of sheet-steel framing anchors to the ends of the studs on the unsheathed face of the panels provided a slight increase in stiffness in the larger panel but added very little to the stiffness of the smaller panels. Anchors on the unsheathed face of the panels did not function effectively, since they did not prevent separation in the joints of the framing members on the sheathed face.

Panel WF, with the sheathing acting in tension, was about 50 percent better in load-carrying capacity at a given deflection than the control panel with the sheathing acting in compression. All sizes of WE panels with both faces sheathed sustained from 2 to 4 times more load at a given deflection than the control panels. All sizes of WC panels with the tied frame and the panels sheathed on both sides increased in load-carrying capacity over the control panels as the panel length increased.

For the control panels, the 12-foot panels gave about 5/8, and the 6-foot panels gave about 1/3 of the load-carrying capacity of the 18-foot panels.

Panel WT with sheet steel framing anchors attached to the framing at the ends of all studs deflected in the same manner as the control panel to a load of 430 pounds per foot of panel length. From 430 to 605 pounds per foot, the panel showed greater deflection for a given load than the control panel. At 605 pounds per foot, the sill split, and the wood tore loose at the anchors.

For effective use, anchors should be placed at all framing joints on the sheathed face of the panel including joints at the ends of the solid cut-in blocking. The effectiveness of the framing anchors at the junction of the studs and double plate would also be increased by a modification of the sheathing nailing pattern to provide more nails in the inner-plate than in the outer plate.

The test results indicate that greater strength and stiffness could be obtained by more secure nailing or fastening of the long sheathing boards at and near the corners of the panels. The largest joint slip occurred at the ends of the long sheathing boards at the corners of the panels, and the farther the adjacent sheathing boards were from the corners of the panel, the less joint slip occurred at the ends of the boards.

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Digest of Test Results

The testing program of wall diaphragms indicated the following results.

(1) The conventional method of nailing the framing of a panel with brackets only at the corners adjacent to the center post permitted framing-joint slip that resulted in great panel distortion at low loads in single-sheathed panels with diagonal boards acting in compression.

(2) The angle brackets bolted at the outer lower corners of single-sheathed panels with boards acting in compression restrained the end framing members, so that the primary failure was shifted to the nails of the conventionally nailed sheathing.

(3) The let-in stays or ties applied perpendicular to the studs added restraint and stiffness to the single, diagonally sheathed panel with the boards acting in compression.

(4) The sheet steel framing anchors resisted joint slip at the ends of the studs in the single, diagonally sheathed panels with the boards acting in compression. Their effectiveness was greatest in the longer panels and when they were attached to the sheathed face of the framing.

(5) The panels with diagonal sheathing acting in tension were stiffer than the similar panels with sheathing acting in compression. The tensile action of the boards drew the framing members together.

(6) The sheathing applied in a herringbone pattern provided symmetry in panel action.

(7) The diagonal sheathing applied to both sides of the studs in herringbone pattern with the boards on opposite sides normal to one another provided 2 to 4 times greater shear strength per foot of panel length than single sheathing acting in compression. The sheathing applied to both faces of the panel subjected the framing to torsion.

(8) The shear resistance per foot of the single-sheathed wall panels increased with an increase in panel length.

(9) Up to a shear load of about 450 pounds per foot of length, the performance and stiffness of the panels with the framing secured with sheet steel framing anchors only was the same as for the control panel with corner angle brackets. Above this load, a corner connection of greater strength than that of the sheet steel framing anchors used is required for maximum strength and stiffness.
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American Society for Testing Materials

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U. S. Dept. of Defense, Atomic Energy Commission

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Whitney, Chas. S., Anderson, Boyd G., and Salvadori, M. G.  
Table 1.--Deflections and residual deflection at center and ends of panels at various intervals of load

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<th>Actual</th>
<th>Ratio</th>
<th>Longitudinal chord</th>
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<th>Ratio</th>
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1Minus sign indicates that the left end chord deflected toward the interior of the panel.
2Panel FA1, which was constructed in accordance with conventional construction methods (common to State of California), was taken as a standard of comparison and assigned a factor of 100 percent.
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**17-FOOT, 4-INCH PANELS**

- **WE**: 3,200 : 28,000 : 692 : 200 : 36,000 : 1,038 : 221 : 54,000 : 1,556 : 246 : 67,000 : 1,920 : 235

**12-FOOT PANELS**

- **WD**: 1,700 : 5,300 : 221 : 100 : 7,200 : 300 : 100 : 10,000 : 416 : 100 : 13,600 : 566 : 100
- **WF**: 1,950 : 8,000 : 333 : 151 : 11,000 : 458 : 153 : 15,000 : 625 : 150 : 19,400 : 808 : 143

**5-FOOT, 4-INCH PANELS**

- **WD**: 930 : 1,300 : 122 : 100 : 1,600 : 150 : 100 : 2,200 : 206 : 100 : 2,800 : 262 : 100
- **WE**: 1,300 : 4,500 : 422 : 346 : 6,000 : 563 : 375 : 8,400 : 788 : 382 : 12,000 : 1,125 : 430

1 Weight as measured by weighing capsule.
Figure 1. --Schematic panel details for panels FAI, FB1, FC1, and FD1 with 1- by 6-inch sheathing on 2- by 10-inch joists.
Figure 2. - Schematic panel details for panels FE1, FF1, FF2, and FF3 with 1- by 6-inch sheathing on 2- by 10-inch joists.
Figure 3. -- Schematic panel details for panels FG1, FG2, and FH1 with 2- by 6-inch sheathing on 3- by 10-inch and 4- by 10-inch joists.
Figure 4. -- Construction details of panel FA1. Distinguishing characteristics are the let-in headers and blocking.
Figure 5. -- Construction details of panel FB1. Distinguishing characteristics are continuous headers and stiffened end chords.
Figure 6.--Construction details of panel FC1. Distinguishing characteristics are continuous header, stiffened end chord, and stiffened longitudinal chord.
Figure 7. Construction details of panel FD1. Distinguishing characteristics same as FD1 except for short sheathing boards and bolted header splice.
Figure 8. --Construction details of panel FE1. Distinguishing characteristics same as FB1 except for herringbone sheathing and fewer nails in splice.
Figure 9. -- Construction details of panels FF1, FF2, and FF3. Distinguishing characteristics same as FA1, but FF1 has herringbone sheathing, FF2 has herringbone sheathing and reinforced corners, and FF3 has herringbone sheathing, reinforced corners, and additional nailing along perimeter.
Figure 10. -- Construction details of panels FGI and FG2. The distinguishing characteristics are the continuous headers with bolted splices, stiffened end chords, 2- by 6-inch diagonal sheathing, and 4- by 10-inch joists spaced 6 feet apart. In panel FG2, 2- by 6-inch pieces were nailed flatwise between the joists.
Figure 11. --Construction details of panel FHI. The distinguishing characteristics are continuous headers with bolted splices, stiffened end chords, 2- by 6-inch diagonal sheathing, and 3- by 10-inch joists spaced 4 feet apart.
Figure 12. A 20- by 60-foot floor or roof panel under test. Steel I-beams on the far side of the panel transmitted the loads at the reactions to the columns of the Laboratory building. The testing machine within the Laboratory applied the load through a 3/4-inch wire rope to the series of I-beams arranged to apply load at the fifth points of the panel. The load was measured by a weighing capsule mounted between the wire rope and the large I-beam, and it was recorded on a dial gage located within the building. A tape recorder was used to record notes during the test.
Figure 13. -- Load-deflection curves showing deflection at the center of the longitudinal chord on the unloaded edge of the roof or floor panels.
Figure 14.--Deformation at 3 points along each end and along the unloaded edge of panel FH1 at shear load increments of 250 pounds per foot of panel width and at the termination of the tests. The dashed lines indicate the residual deformation or set following a shear load of 250 pounds per foot of panel width.
Figure 15. -- Curves showing the relative deformation at 2 planes across the width of panel FH1 at shear load intervals of 250 pounds per foot of panel width. The measurements were taken at the numbered points in the diagram.
Figure 16. -- General test setup showing panel WA-18 at the beginning of the initial test. The two sections of frame wall, bolted together through a center post and with opposite ends resting on solid supports, were tested simultaneously by applying load to the center post.
Figure 17. --Schematic details of panel designs WD and WE. Control panel WD, diagonally sheathed with boards in compression. Panel WE, diagonally sheathed on both faces of panel with boards in compression on one face and in tension on the reverse face. Dotted lines indicate the direction of sheathing on the back side.
LEGEND:

- DEFLECTION MEASURED BY SCALES
- DEFLECTION MEASURED BY DIALS
- DEFLECTION MEASURED BY CLIP GAGES
= SHEATHING IN TENSION
= SHEATHING IN COMPRESSION

Figure 18. -- Schematic details of panel designs WA, WB, and WF. Panels WA and WB, diagonally sheathed with boards on panel a in compression and on panel b in tension. Corner angle bracket added at lower corner of framing of panel a before final test. Panel WF, diagonally sheathed with boards in tension. Test included only a 12- by 12-foot panel.
Figure 19.--Schematic details of panel designs WC and WT. Panel WC, diagonally sheathed with boards in compression; framing reinforced with let-in 1- by 6-inch boards. Sheet steel framing anchors were added at the ends of studs before the final test. Panel WT was the same as WD, except the sheet steel framing anchors were attached to the framing at the ends of all studs. The corner angle brackets were omitted during the initial test. Test included only a 12- by 18-foot panel.
Figure 20. -- Curves showing the relationship of the shear per foot of panel length to the deflection for all panels. The curves for panels WA, WB, WC, and WT are composite curves made up of two or more test runs. The arrows above show whether the sheathing acted in tension or compression.
Figure 21. --Curves representing the relationship between the shear per foot of panel length and the deflection for panels WD-18, WD-12, and WD-6.
Figure 22. -- Deformation resulting along the perimeter of panel WD-18 for several increments of load.