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DESIGN OF CASTING AND TESTING APPARATUS FOR REINFORCED CONCRETE PANELS

by

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SYNOPSIS

This thesis presents the design, construction, and testing of the apparatus for casting and testing reinforced concrete rectangular panels. The panels, which were 8' x 4', were simply supported on all four edges, and were uniformly loaded in uniaxial compression along the short edge.

Two panels, one inch thick, with different reinforcing steel configurations, were constructed and tested in compression. The test frame was modified by stiffening and securing during, and subsequent to, the test of the first panel.

The experimental results showed that the panels buckled with biaxial curvature and failed in a panel mechanism mode. Based on these results, the modified test frame performed adequately in that the theoretical support and load conditions were satisfied to an acceptable degree.

INTRODUCTION

The purpose of this project was to design and construct the apparatus to be used in determining the buckling load and post-buckling strength of rectangular, reinforced concrete panels. The panels were to be simply supported on all four edges and subjected to uniaxial compression.

The first objective was to design and build a form in which to cast the panels. One requirement was that provision be made for varying the percentage of reinforcing steel from 0.2% to 1.0%. This steel will be a two-way mesh in either one or two layers. Thicknesses of panels to be cast in this form will be one inch (1"), one and one quarter inches (1 1/4") and one and one half inches (1 1/2"). All of the panels to be cast will be four feet (4') by eight feet (8'). The panels with which this thesis is particularly concerned were one inch (1") thick with a ratio of steel reinforcement of 0.005 in two layers.

The second objective was to design and build a test frame to be utilized in a Tinius Olsen testing machine with a 200,000 pound capacity. The major criterion for the test frame was that of fulfilling the assumption that the panel would be simply supported on all four edges. A second requirement was that the load be applied uniaxially and distributed uniformly over the loaded (short) edges of the plate. The edge restraints had to be built so that they could be adjusted to accommodate the different plate thicknesses.

Two panels were cast and tested for the purpose of determining the adequacy of the casting and testing system.

REVIEW OF LITERATURE

Little experimental work has been done in the area of buckling of reinforced concrete panels, simply supported or otherwise, and no work has been done on panels of this size [four feet (4') by eight feet (8')]. Ernst (3)* used mortar panels forty inches (40") by forty inches (40") and forty inches (40") by twenty inches (20"), with the forty inch (40") edges being loaded on both sizes of panels. These panels were reinforced only in the center (middle surface), with the same amount of steel in each panel. The panels ranged from one half inch (1/2") to one and one half inches (1 1/2") in thickness. The unloaded edges of the panels were not supported but were merely stiffened with steel members. The loaded sides were supported on knife edges. Eccentricity of load was minimized by adjusting the top edge of the plate in the testing machine during loading; however, eccentricity due to curvature at the edges was not removed. The type of buckling which occurred was plate buckling (double curvature).

Tests have recently been conducted on masonry wall panels by Yokel (6). These panels were supported only along the loaded sides. The panels were subjected to uniaxial compression and buckled essentially as columns (single instead of double curvature).

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* Numbers in parentheses refer to references listed in Bibliography.

Timoshenko (5) has a lengthy discussion of the buckling of thin plates which is restricted to plates having a constant modulus of elasticity and exhibiting ductile behavior. Experiments on buckling of plates, with particular attention paid to the problems which arise in attempting to fulfill assumed boundary conditions, are also discussed. A description of the use of Southwell's Method for determining the critical value of the load from the observed deflections is given.

THEORETICAL BACKGROUND

The buckling load, P_{cr}, for a thin, homogeneous, simply supported plate is given as (5):

$$P_{cr} = \frac{\pi^2 D}{b} \left(\frac{mb}{a} + \frac{a}{mb}\right)^2 \quad (1)$$

where

$$D = \frac{Eh^3}{12\,(1 - \nu^2)}$$

a = plate width

b = plate length

E = Young's Modulus

h = plate thickness

m = number of half waves of buckled surface in direction parallel to applied load

ν = Poisson's Ratio

For the case of a = 2b, the plate buckles into two half waves (m = 2) and P_{cr} is:

$$P_{cr} = \frac{4\pi^2 D}{b} \quad (2)$$

It should be re-emphasized that in order for the above equations to be valid, the theoretical support conditions must be satisfied.

This type of buckling is different from that observed in columns in that the plate bulges (biaxial curvature) and generally failure does not occur at the buckling load, particularly for ductile materials.

For the case of concrete panels, the value of E in the expression for D is not constant, but rather, decreases with increasing stress. Therefore, the buckling equation given above should be modified to account for this. However, the deflection behavior of the panel should agree with that predicted by theory.

Southwell's Method (5) provides a means for determining the critical value of the load from the observed deflections. This method accounts for the initial curvature of the panels and eccentricities in application of the load, which cause buckling to begin at a load level less than the theoretical buckling load. This method is applied by plotting the observed deflections, δ, against the values of δ/P at the point of maximum deflection, where P is the load corresponding to an observed δ. This plot should be a straight line, the slope of which gives the true value of the critical load.

DESIGN OF EQUIPMENT

The panel casting form was as shown in Fig. 1, with the panel cast flat. The sides of the form were 2" x 4" soft pine boards slotted to accept the two way reinforcing wire mesh. Inspection of the form sides showed that it would not be feasible to use them in casting more than two panels, due largely to the fact that the panels were cured in the form in a surface-wet state for three days. It was decided to make new form sides for each ratio of reinforcing steel. The form bed was three quarter inch (3/4") exterior plywood attached to a framework with 25, two inch, No. 10 flat head wood screws, which were countersunk. Five coats of satin varnish were applied to the plywood in order to seal it against moisture seepage. It was decided not to treat the supporting members of the form framework, since they would not be exposed directly to moisture. The form was checked for flatness with a level after each panel was cast and no warping was found in the framework.

The panel transportation and storage rack was as shown in Fig. 2. This device was used, as the name implies, to move the panel to the moist-cure room, to store it there, and to move the panel to the testing machine. This reduced the number of times the panel was handled to an absolute minimum, thereby reducing the chances of the panel's being damaged prior to testing.

The panel testing apparatus was originally as shown in Fig. 3. The theoretical support and load requirements to be satisfied were:

1. No deflection normal to the panel surface along the edges;
2. Free rotation (no moment) along the edges;
3. The load should be applied uniaxially at the middle surface of the panel and should be distributed uniformly over the loaded edge.

The first two of these requirements were to be met by the side and top clamps shown in Figs. 4 and 5. The steel rods running through the side clamps and the milled grooves in which the bottom and top clamps rode were to prevent movement other than rotation along the edges. The steel rods were restrained in bending by the lugs bolted to the side channels. The side channels were originally supported only at the base so that the upper I-beam (the load distributing beam) could move vertically relative to the side channels. In this way, the panel could be compressed without any of the applied load being carried by the side channels. The third of these requirements was to be met by the load distributing beam and the panel support beam, both of which were W 10x29. The load distributing beam was four feet (4') long and the panel support beam was eight feet (8') long. The load distributing beam was attached to the movable head of the testing machine with threaded rods and the panel support beam had lugs welded to it which rode in a slot in the base of the testing machine. The method of aligning the panel in the testing machine was to locate the load distributing beam directly over the panel support beam using a plumb bob, plumb the side channels, and therefore the panel, with a level, and align the milled groove on the rollers of the top clamps. However, when the first panel was tested there was excessive relative movement between the side channel and the load distributing beam, such that the web of the load distributing beam exhibited excessively large elastic bending deformations. Therefore, it was necessary to stiffen and secure the beams and channels as shown in Figs. 6, 7, and 8. Additional clamps at the tops of the side channels were installed due to the unexpectedly high warping forces in the upper corners of the panels. This problem was not encountered at the lower corners, probably due to the rigid connection of the side channels to the panel support beam. The side channels were designed and built using Ernst's (3) work as a guide in determining the appropriate stiffness. These side channels were finally supported at both ends and at the midpoint, as shown in Fig. 8.

The original dial gage supports were as shown in Fig. 9. However, there was some ambiguity in applying Southwell's Method (5) in that the dial readings at the extreme ends of the dial gage supports were not taken at the center of rotation of the edge of the panel. These dial gages were to be used to measure the movement of the panel relative to the load frame but instead, measured this movement plus an additional movement caused by the rotation of the panel. This problem was overcome by placing the extreme dial gages on the horizontal line over the steel rods which run through the side clamps, and by placing the extreme dial gages on the vertical line at the centers of the rollers on the top and bottom clamps. This change is shown in Fig. 10.

DESIGN AND CASTING OF PANELS AND CYLINDERS

A major criterion for the concrete used in casting the panels and cylinders was the ease of placing the concrete in the form. The coarse aggregate was Bayer Stone, graded to 3/8" minus, the sand was Kaw River Sand, also graded to 3/8" minus, and the cement was Monarch Type 1 Portland Cement. The fineness modulus of the sand was 3.05. The mix design used for the first trial panel consisted of a water-cement ratio of 0.71 and equal portions by weight of sand and coarse aggregate. A total of eight gallons of water and one sack of cement was used in the mix, which had a total volume of 3.5 cubic feet. This design gave a very stiff mix (zero slump) with a poor surface quality. Therefore, for the second trial panel the mix was modified to a water-cement ratio of 0.68 and a proportion of sand to coarse aggregate of 1.71 with total quantities as follows:

weight of water	=	45.3#
weight of cement	=	66.7#
weight of sand	=	242.0#
weight of coarse aggregate	=	141.4#

This second mix, which had a slump of six inches (6"), gave satisfactory workability.

The reinforcing steel for the first trial panel was a commercially available welded wire fabric, no. 10 wires at 6" centers. The mesh was placed in the form in two layers on wire chairs designed to provide 1/8" of cover. Flattening of the welded wire fabric was difficult, and not enough variations in wire sizes and spacings were readily available, so it was determined that some other type of reinforcing steel was required.

The concrete was shoveled into the form and rodded with a 5/8" diameter rod. The form sides were vibrated with an electric vibrator in an attempt to remove any trapped air, but it was decided, after inspection of the form side of the panel, that vibration was relatively ineffective in a form this shallow. Excess concrete was screeded off, and the surface was worked with a steel trowel.

There were two test cylinders cast for each panel. These were standard six inch (6") by twelve inch (12") cylinders, cast in accordance with ASTM Standard C192. The cylinders were cured under the same conditions as those applied to the panels.

The second trial panel was reinforced with no. 14 cold drawn steel wires at 2" spacing. These wires were placed in two layers, in two directions, with one wire diameter of cover. The lower wires were supported on wires which were removed after the concrete was in place and before it was screeded. The upper wires were stapled to the form sides with the required cover. When the concrete was shoveled into the form, the upper wires tended to sag, which necessitated their being pulled up to the proper depth after the concrete was placed. It was decided that some more positive means of supporting the upper reinforcing steel wires was required. The concrete was tamped with a tamper built from seven no. 4 deformed reinforcing bars twelve inches (12") long welded to an angle iron at 2" centers. Inspection of the form side of the panel indicated that this was an acceptable means of placing the concrete. The upper surface of the panel was screeded and finished with a steel trowel.

INSTRUMENTATION OF PANELS AND CYLINDERS

The first trial panel was tested to determine the adequacy of the test frame. To this end, the dial gage support shown in Fig. 9 was not attached to the test frame.

Electrical resistance strain gages were placed on the panel as shown in Fig. 11. The strain gages were linear wire gages with a gage length of three quarters of an inch (3/4") and were placed parallel to the long direction of the panel. The gages were read electronically using three Budd-Datran C-10 switch and balance units and a Budd-Datran A-110 digital strain indicator. A Budd E-140 printer control unit and a Victor Digit Matic printer were used to record the strain readings on paper tape. These readings were used to detect strain eccentricities in the panel under load.

The test cylinders were instrumented with two diametrically located electrical resistance, two element, wire strain rosettes and a dial gage for measuring axial strain. The strain rosettes were read using a Budd SB1 switch and balance unit and a Budd HW1 strain indicator. The readings from the strain rosettes were used to determine Poisson's Ratio for the cylinders. The dial gage readings, taken at specified load levels, were used to plot the nominal stress-strain diagrams for the cylinders. An attempt was made to read the dial gage as the load decreased after ultimate to rupture.

The second trial panel had the dial gage support shown in Fig. 9 attached to the test frame. Deflections of the panel were measured with Soil Test LC10 dial gages which have a least reading of 0.001 in. and a travel of two inches. The dial gages were fitted with extensions made from paper straws to prevent damage to the dial gages should the deflection of the panel exceed the travel of the dial gages. The dial gages were read at each load level to determine deflections perpendicular to the plane of the panel. The dial gage readings were used to determine the deflected shape of the panel and to calculate the buckling load using Southwell's Method (5).

The second panel had electrical resistance strain gages placed as shown in Fig. 11.

The test cylinders for the second trial panel were instrumented in the same manner as were those for the first trial panel.

DESCRIPTION AND RESULTS OF PANEL TESTS

The following test procedure was used in testing the panels:

1. Align panel in test frame as described previously;
2. Apply an initial load to seat the panel in the test frame. Remove the load to zero;
3. Take strain and dial readings at the initial (zero) load. Note that the first trial panel was not instrumented with dial gages because it was not known how the panel and test frame setup would perform;
4. Loads were applied gradually using the lowest speed and gear ratio of the testing machine;
5. At even load increments the movement of the load head was stopped and strain and dial readings were taken;
6. As the panel neared collapse the dial gages were removed if it appeared that the panel would buckle in a manner which would damage the gages.

The 6" x 12" concrete cylinders associated with each panel were tested by measuring the applied compressive loads and corresponding strains up to failure of the cylinders.

The initial attempt to test the first trial panel was made using the panel testing apparatus shown in Fig. 3. However, a deflection of the web of the load distributing beam occurred, in part due to excessive eccentricity, forcing the test to be stopped, as explained previously. The load distributing beam and the panel support beam were stiffened as shown in Figs. 6 and 7, respectively, and the test was rescheduled. The second test of the first trial panel was successful in that the panel failed. An initial failure in the panel occurred at a load of 59,420 pounds. The failure was of a shearing type in the top corners, which indicated that there was still considerable movement between the side channels and the load distributing beam. However the panel was able to sustain an increase of load up to 75,000 pounds, at which load the panel collapsed. The crack pattern, which indicated the final failure in a panel mechanism, is shown in Fig. 12. The panel buckled as shown in Fig. 13 with biaxial curvature. The excessive movement between the side channels and the load distributing beam produced the excessive rotation shown in Fig. 14. The decision was made to secure the side channels to the testing machine at the top and at midheight. A plate was fabricated to prevent relative movement between the tops of the side channels and the ends of the load distributing beam. This was the final test setup as shown in Fig. 8.

The data from the tests of the cylinders for the first trial panel was not used, since the test cylinders were failed prior to the first attempt to test the panel. Six weeks elapsed between the first and second attempts to test the first trial panel, due in part to the required modifications of the test frame.

The second trial panel was tested using the panel testing apparatus shown in Fig. 8. The ultimate load for the panel was 100,540 pounds, which produced the crack pattern shown in Fig. 15. This crack pattern is also indicative of a panel mechanism mode of failure. The deflected shape of the panel is shown in Figs. 16 and 17 for the vertical and horizontal lines of dial gages, respectively. Note that the panel again buckled with biaxial curvature.

The Southwell plot for dial gage no. 12 is shown in Fig. 18. Southwell's method (5) calls for the use of the dial gage with the largest observed deflection. The critical buckling load, as defined by the slope of the line in Fig. 18, was 152,500 pounds. This value was larger than the experimental collapse load but should represent the theoretical buckling load if no eccentricity of loading were present. However, there was some translation of the panel relative to the edges of the test frame as indicated in Figs. 16 and 17. Therefore, the results obtained from Southwell's method for this panel were somewhat inconclusive, although the basic approach appears valid since a straight line plot was obtained. In subsequent panel tests, relative displacements will be obtained by placing dial gages directly over the side and top supports at the center of rotation. This change is shown in Fig. 10. Therefore, Southwell's method (5) may be used with the difference between the reading of the dial gage with the largest observed deflection and the readings of the extreme dial gages in that line of dial gages. In this way, any movement of the edge of the panel will be taken out of the calculation of the critical buckling load using a Southwell plot.

The critical buckling load obtained from Equation (2), using an E of 57,000 $\sqrt{f_c'}$, is 251,000 pounds. This value is much larger than the experimental collapse load but it must be noted that as the load increases, the value of Young's Modulus decreases. Further, the formula was derived for an elastic material.

A further method for estimating the buckling load is by observing the behavior of the strain gages on the buckled surface. Strain gage readings for gages located three feet from the top of the plate near the centerline are shown in Fig. 19. Both gages increase uniformly with load up to a load of about 90,000 pounds, where a marked deviation in the strain pattern occurs. This load may be considered to be a measure of the buckling load in that the strain on the "tension" side of the buckle will start to decrease rapidly as buckling initiates and the strain on the other side will increase rapidly. Note also that considerable eccentricity is still present in the panel. This will tend to lower the buckling load.

The cylinders were tested subsequent to the test of the second trial panel. The nominal stress-strain diagram for one of the cylinders is shown in Fig. 20. Poisson's Ratio was 0.17 for the cylinders and the ultimate strength was 3,890 psi. Note that the failure stress in the test panel was 2,090 psi, which is considerably lower than the cylinder strength.

CONCLUSIONS

Designs for the equipment necessary to cast and test rectangular reinforced concrete panels in uniaxial compression are presented. These panels, which were 4' x 8', were to be supported on all sides and free to rotate (hinge supports). In order to evaluate the adequacy of the system, the following criteria were used:

1. Visual inspection. The panels should fail by buckling with biaxial curvature. A panel failure mechanism should occur at failure in the buckled region.
2. Deflection readings should also indicate this buckling pattern and should show very small deflections at the support locations.
3. Using the deflection readings, a linear Southwell plot should be obtained.
4. The eccentricity of loading as indicated by strain and dial readings should be small.

Two trial panels were cast and tested. The results of the first trial panel test indicated excessive eccentricity of loading and relative movement between the side support members and the top load beam. Because of this, the design for the test frame was modified by adding stiffeners to the side and top support members and by placing additional panel clamps near the tops of the side support members.

On the basis of the results obtained from the test of the second trial panel, it is felt that the test frame is fulfilling the theoretical support and load conditions adequately. This is based upon the facts that the panel failed in a panel mechanism mode as evidenced by the crack pattern, the Southwell plot was linear, and the deflection pattern indicated biaxial bending. However, high eccentricity in the applied load was still observed, but it is felt that by aligning the panels in the test frame more accurately this can be greatly reduced. It is also felt that by placing dial gages directly over the panel supports more accurate relative panel displacements can be obtained.

The system for reinforcing the panels and casting them, as described in this report, appears to be adequate.

APPENDIX I: FIGURES

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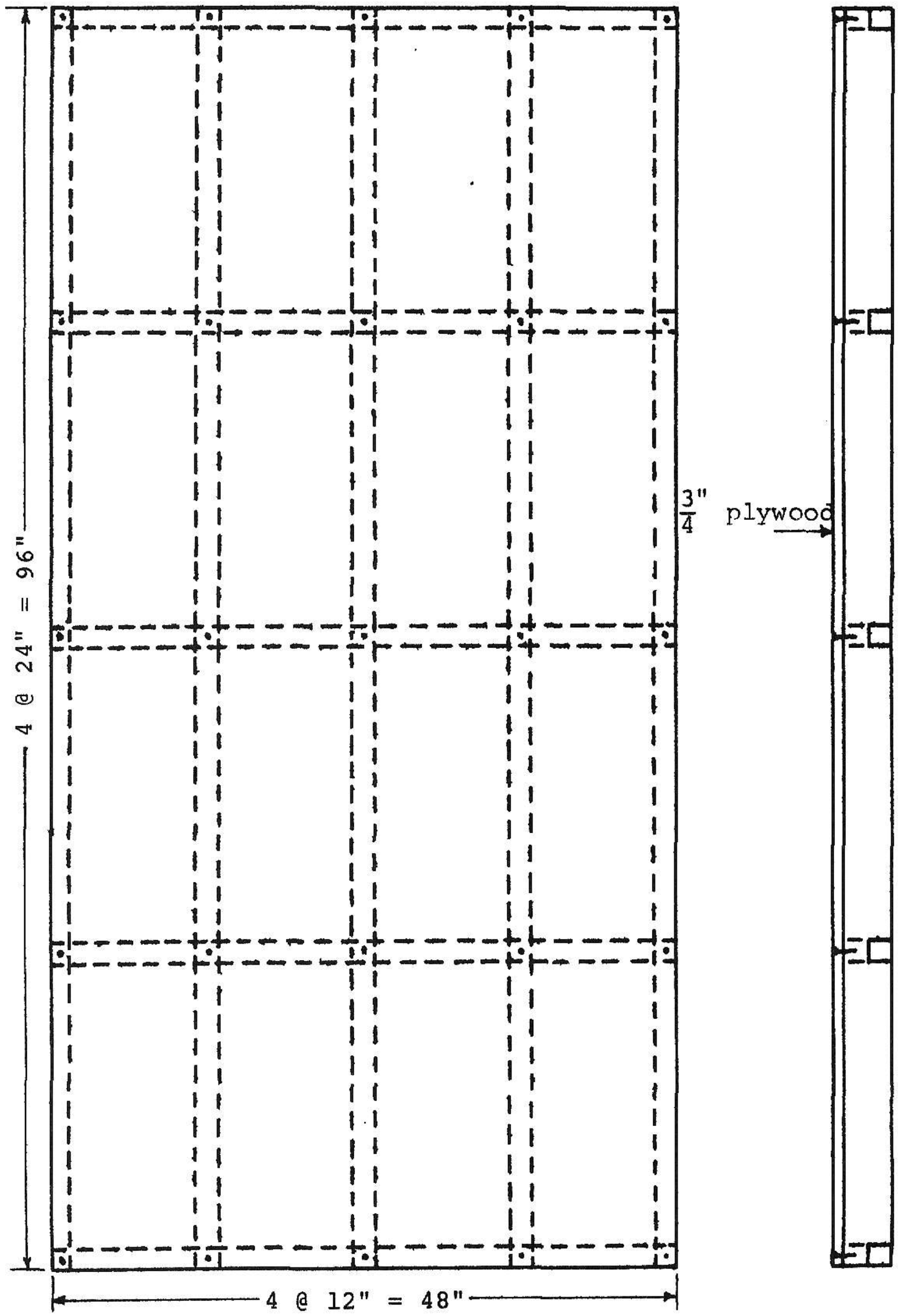

Fig. 1. Panel Casting Form

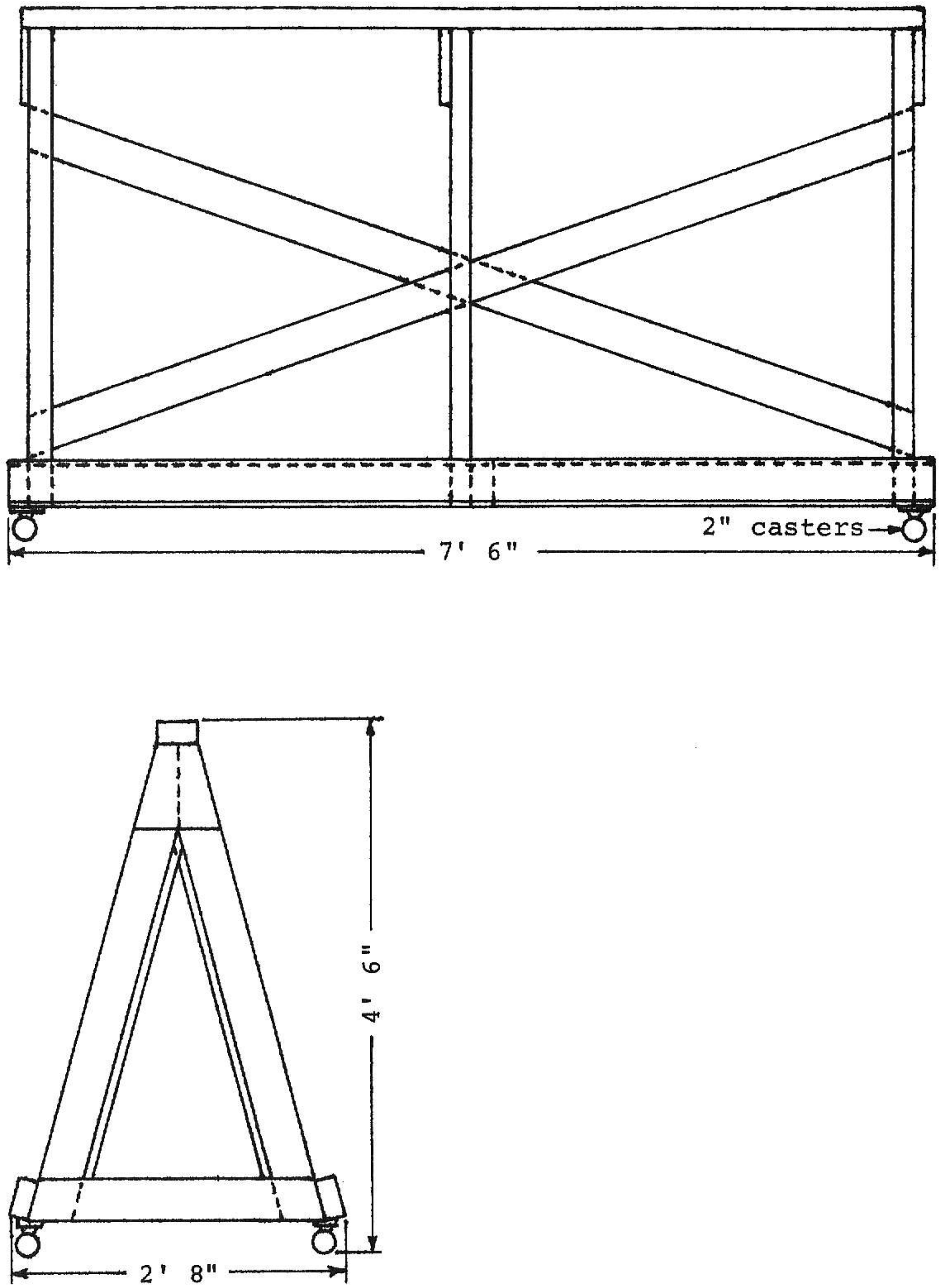

Fig. 2. Panel Transportation and Storage Rack

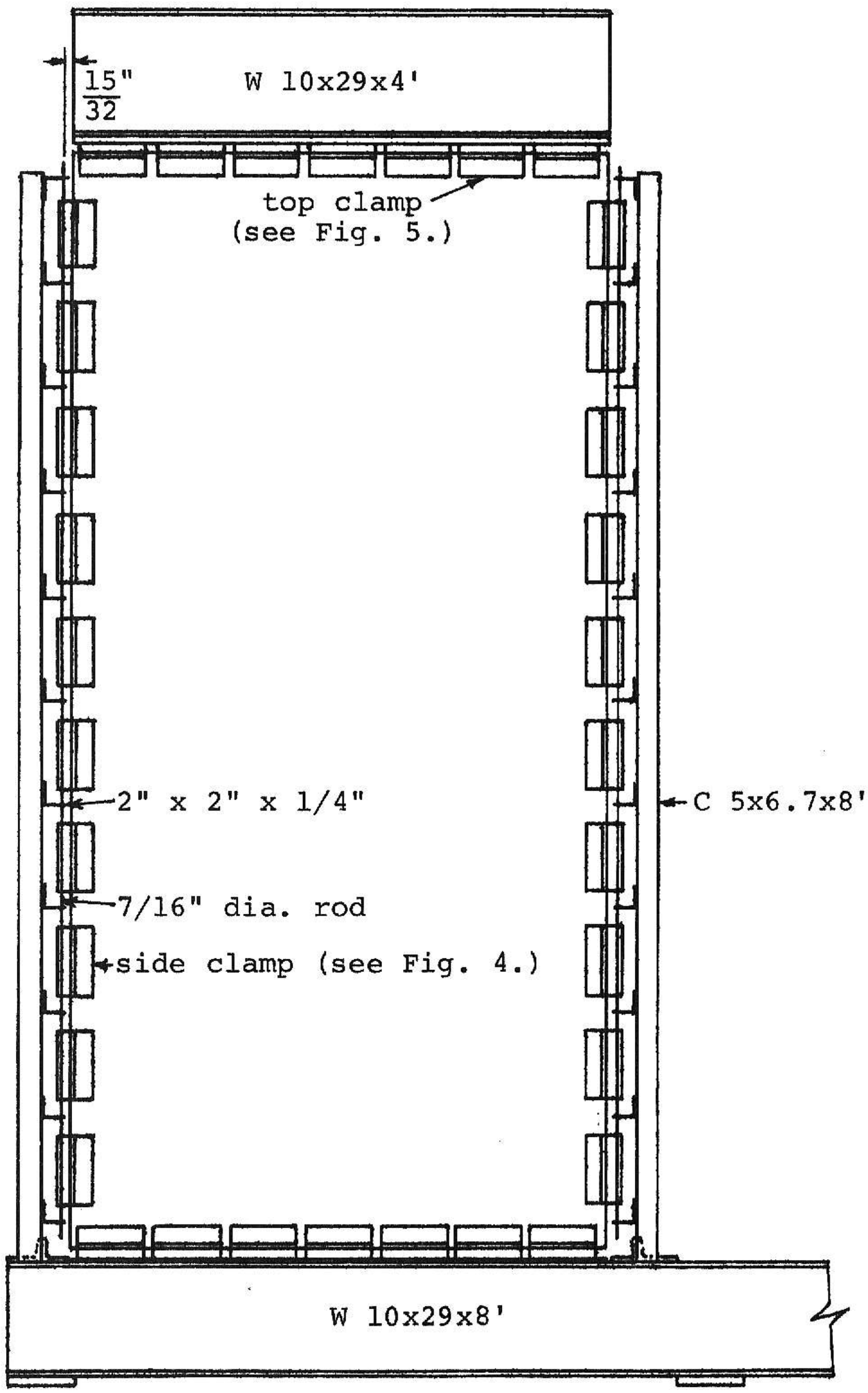

Fig. 3. Original Panel Testing Apparatus

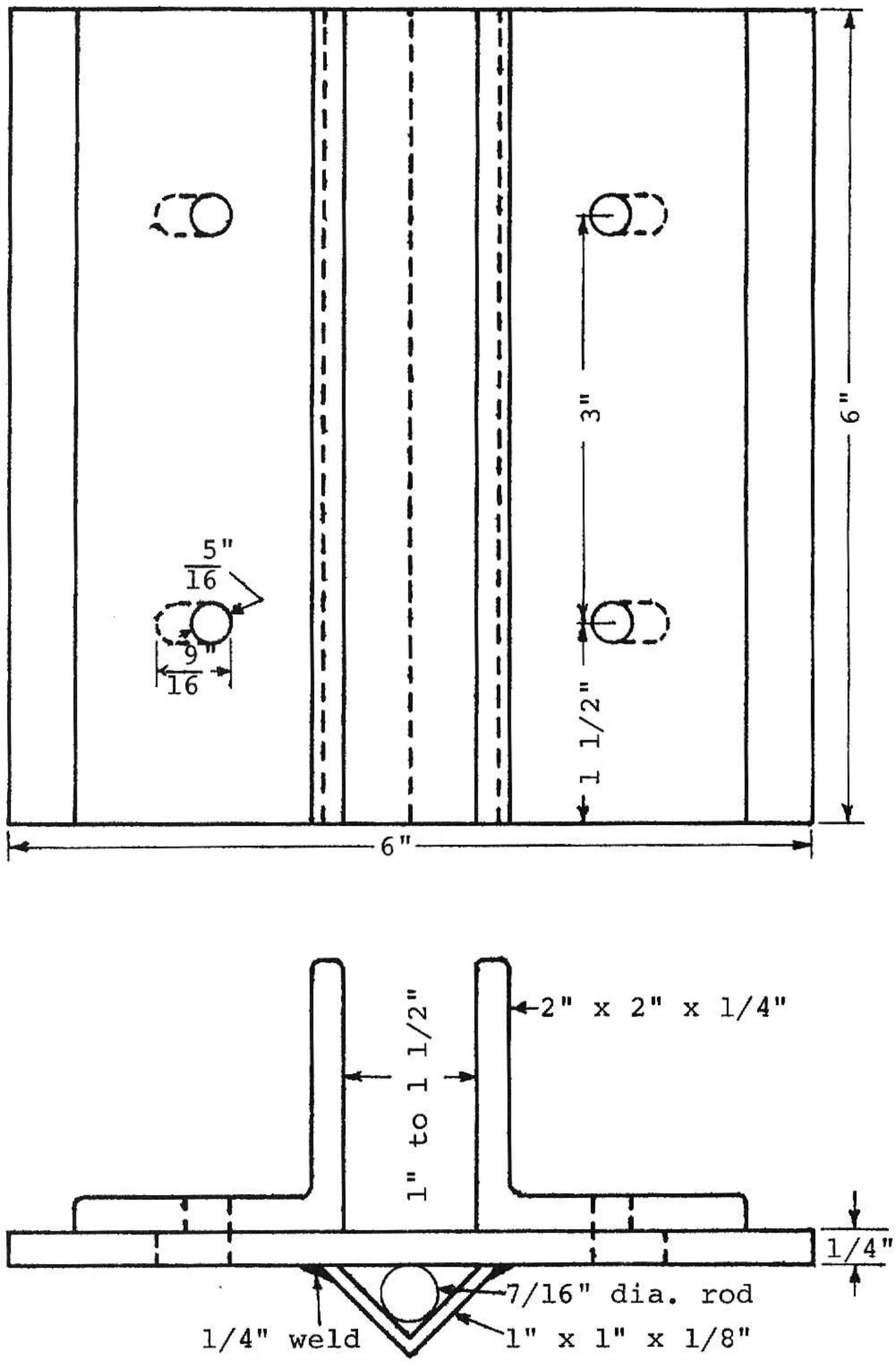

Fig. 4. Detail of Side Clamp

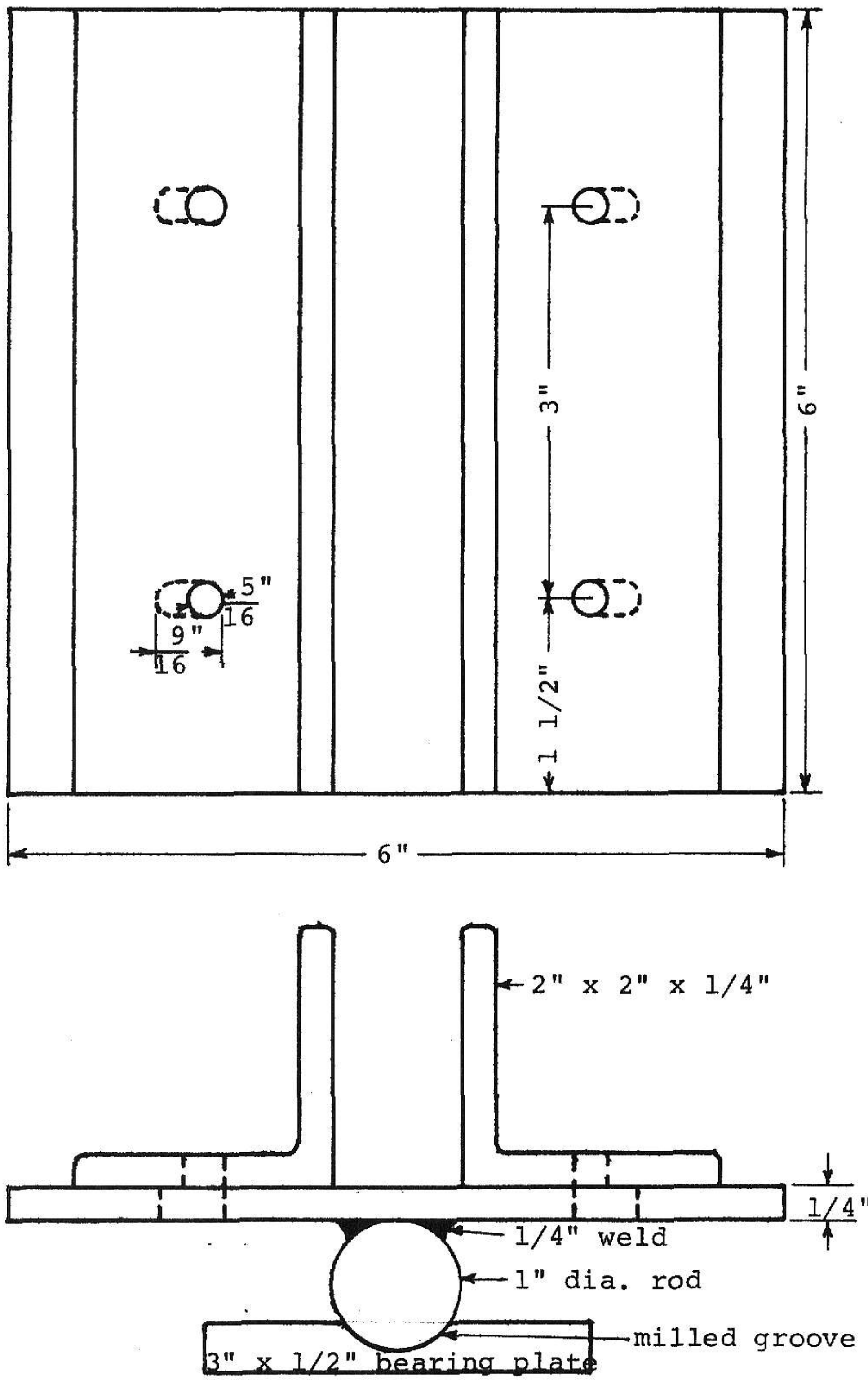

Fig. 5. Detail of Top and Bottom Clamp

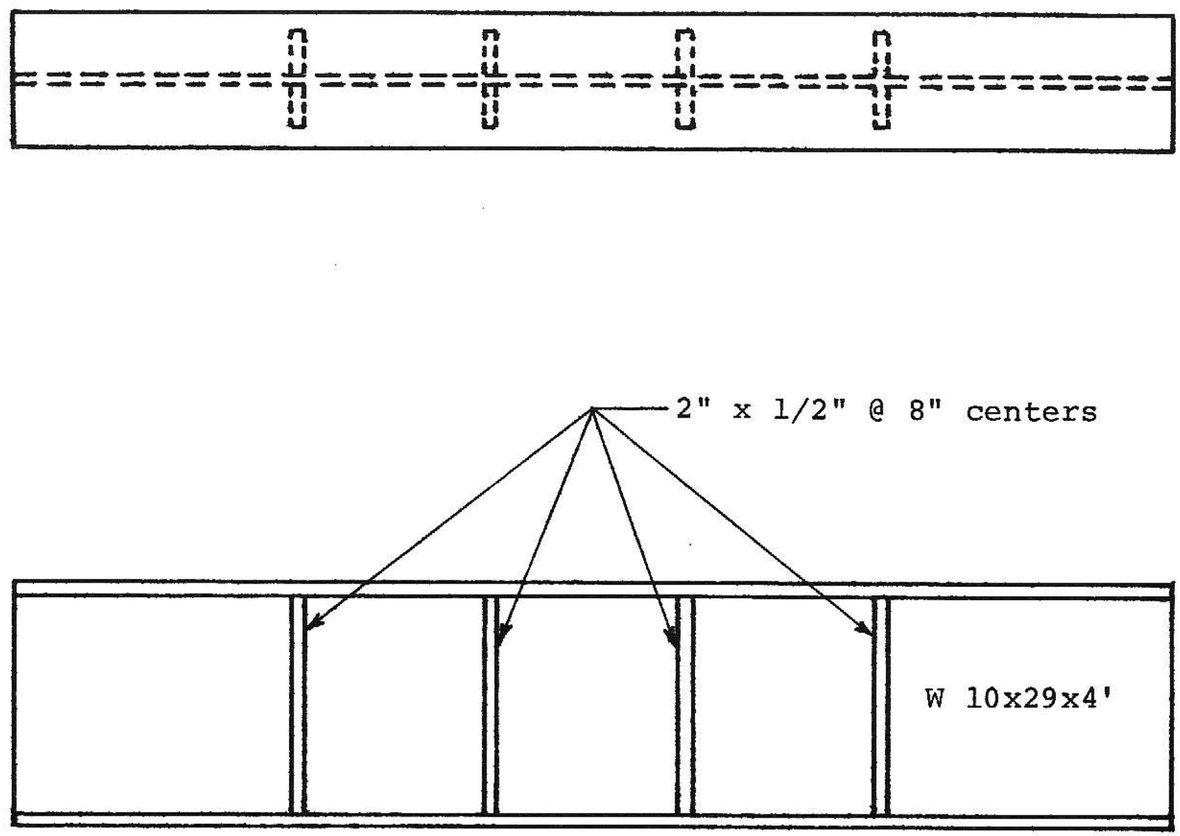

Fig. 6. Stiffening of Load Distributing Beam

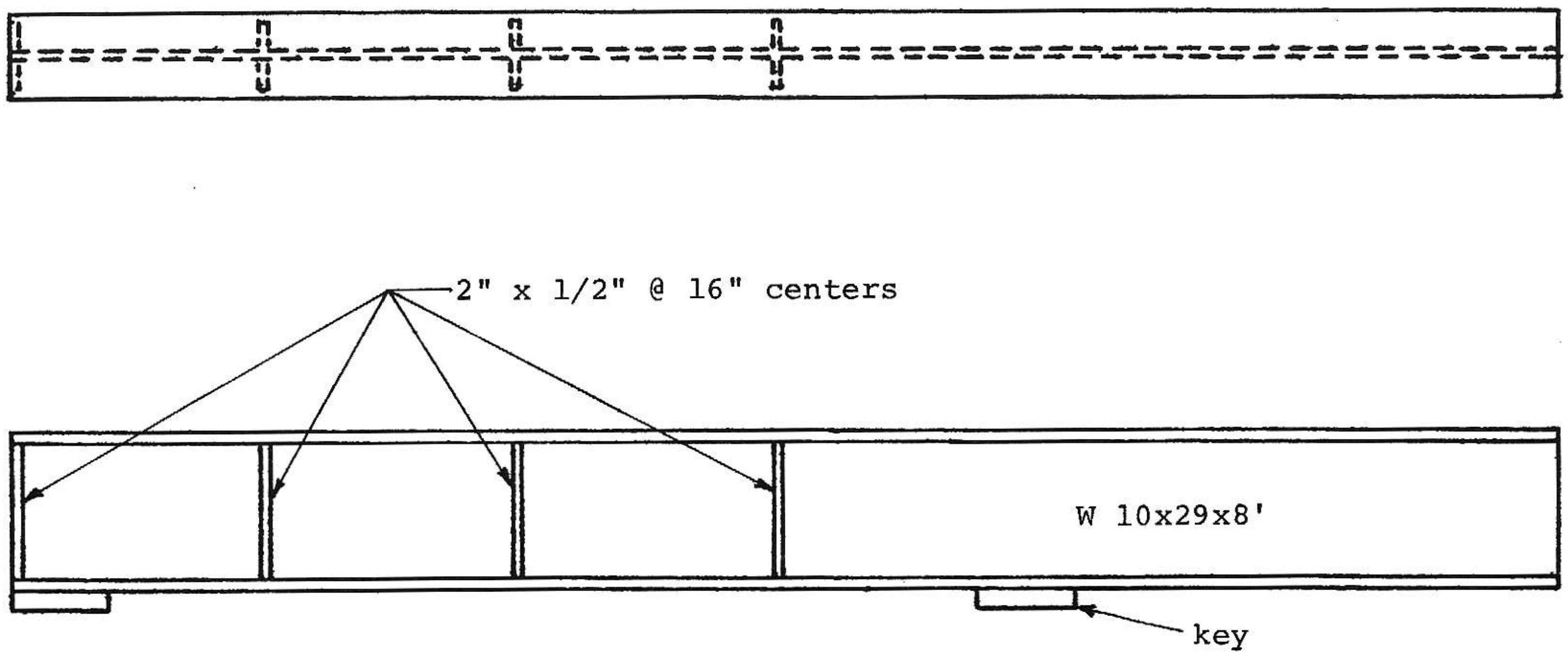

Fig. 7. Stiffening of Panel Support Beam

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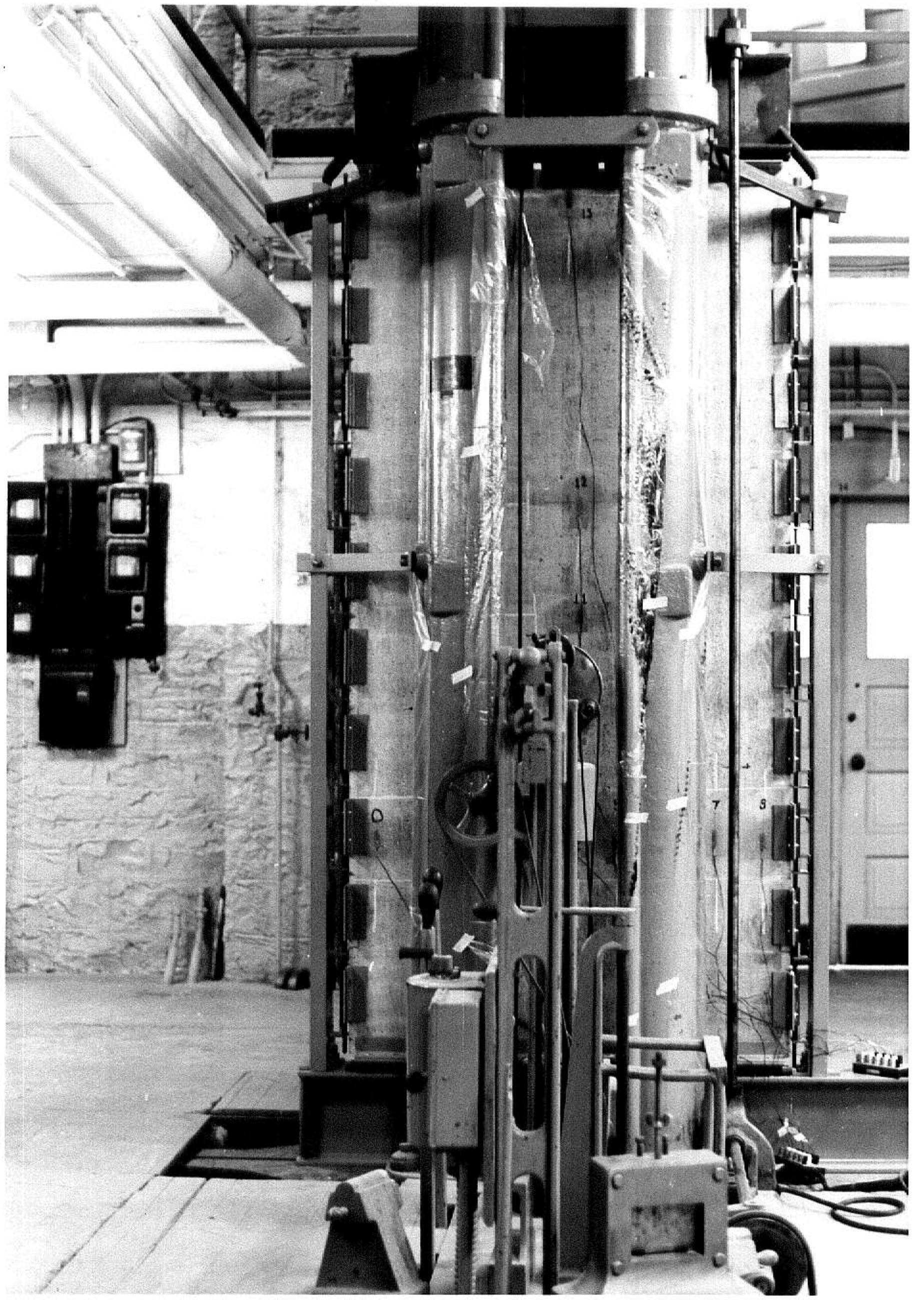

Fig. 8. Final Panel Testing Apparatus

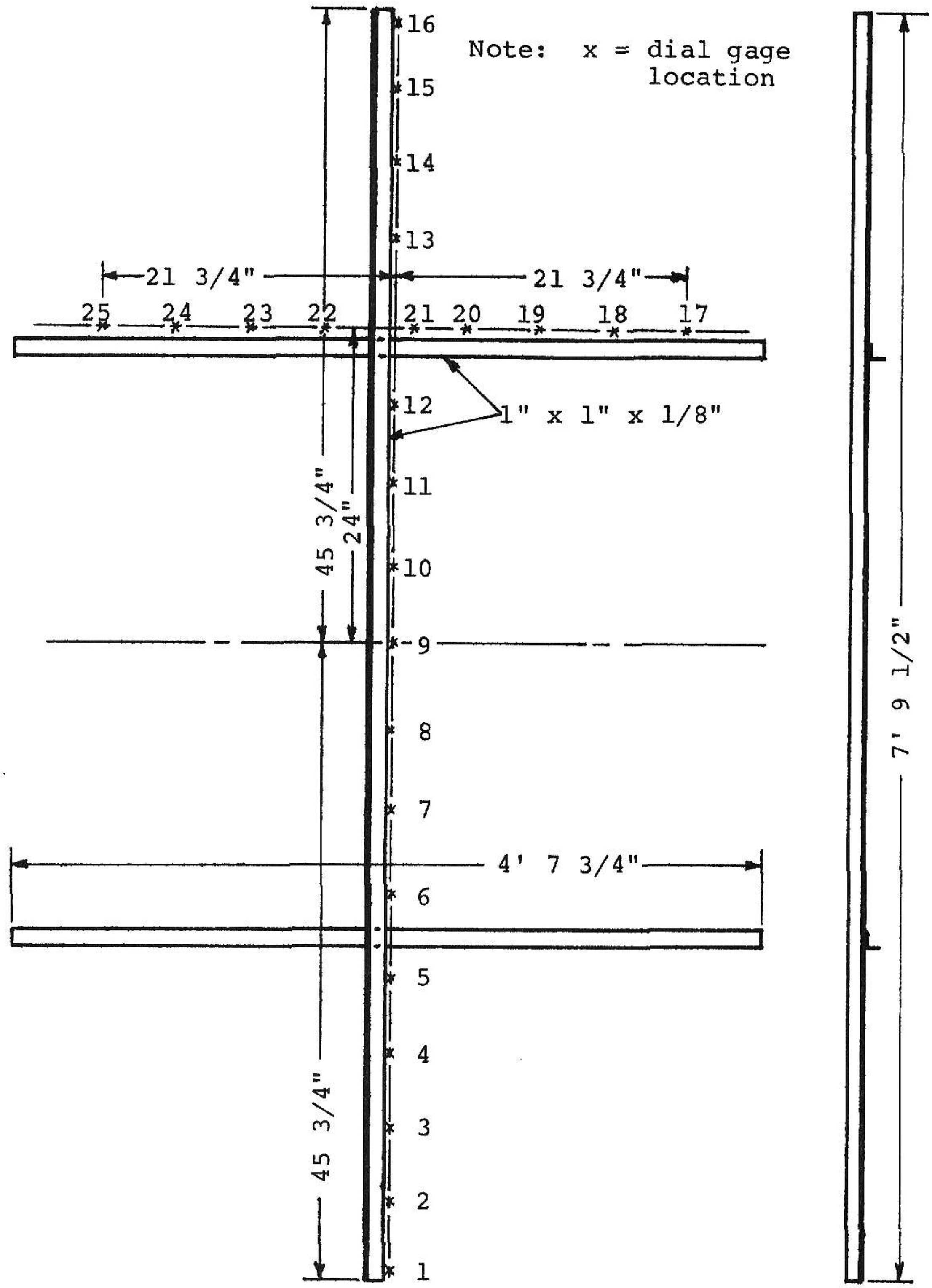

Fig. 9. Original Dial Gage Support

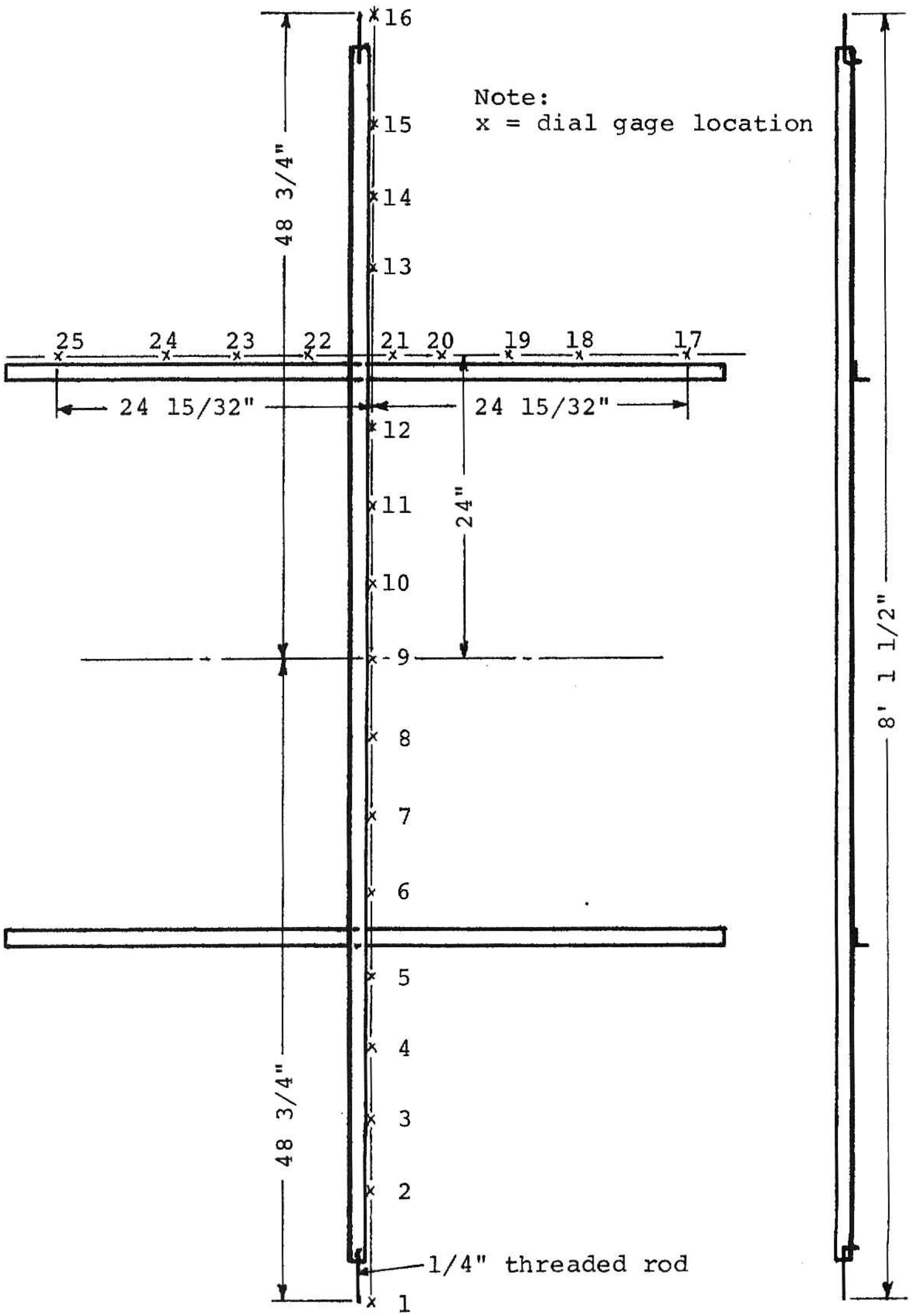

Fig. 10. Final Dial Gage Support

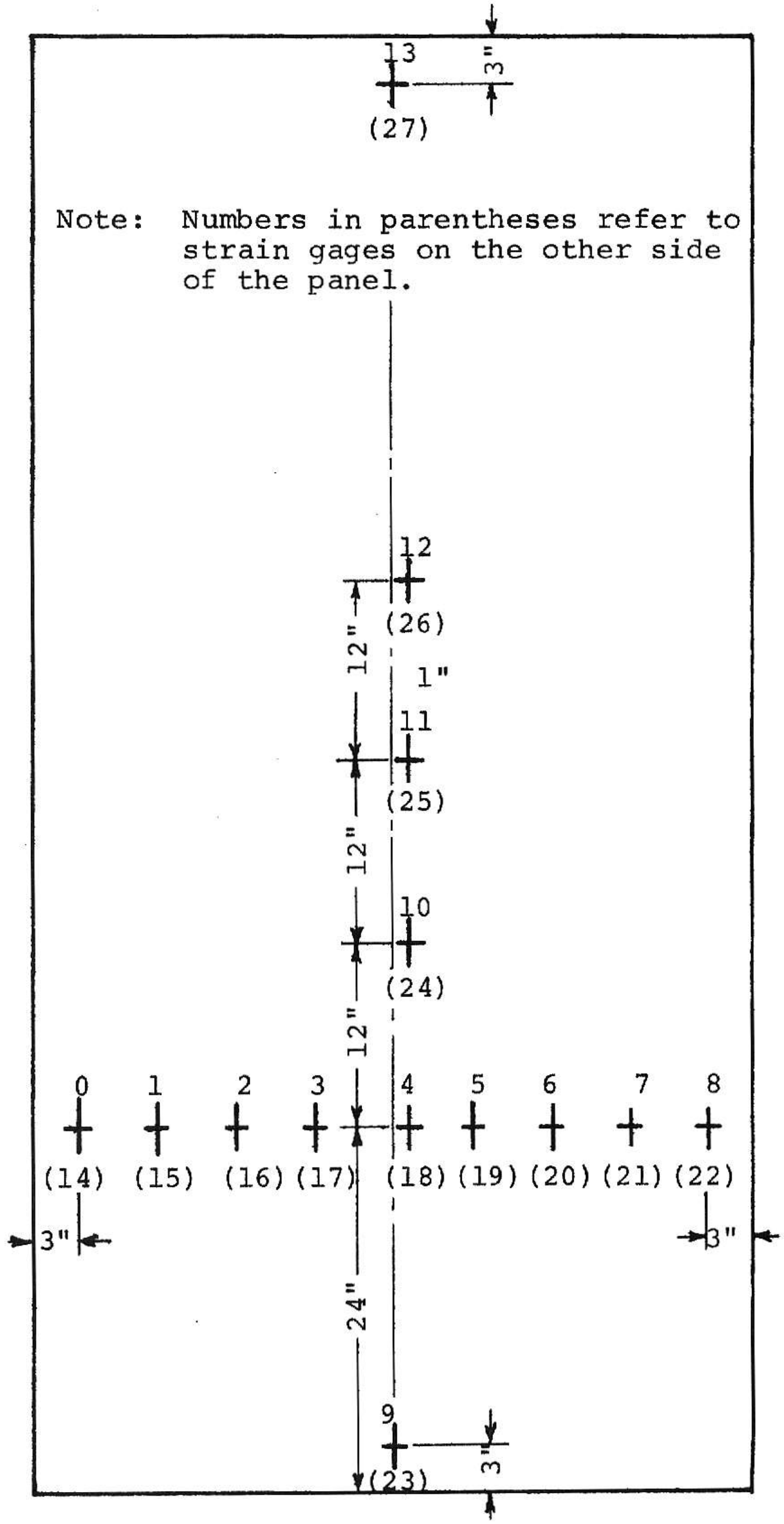

Fig. 11. Strain Gage Locations

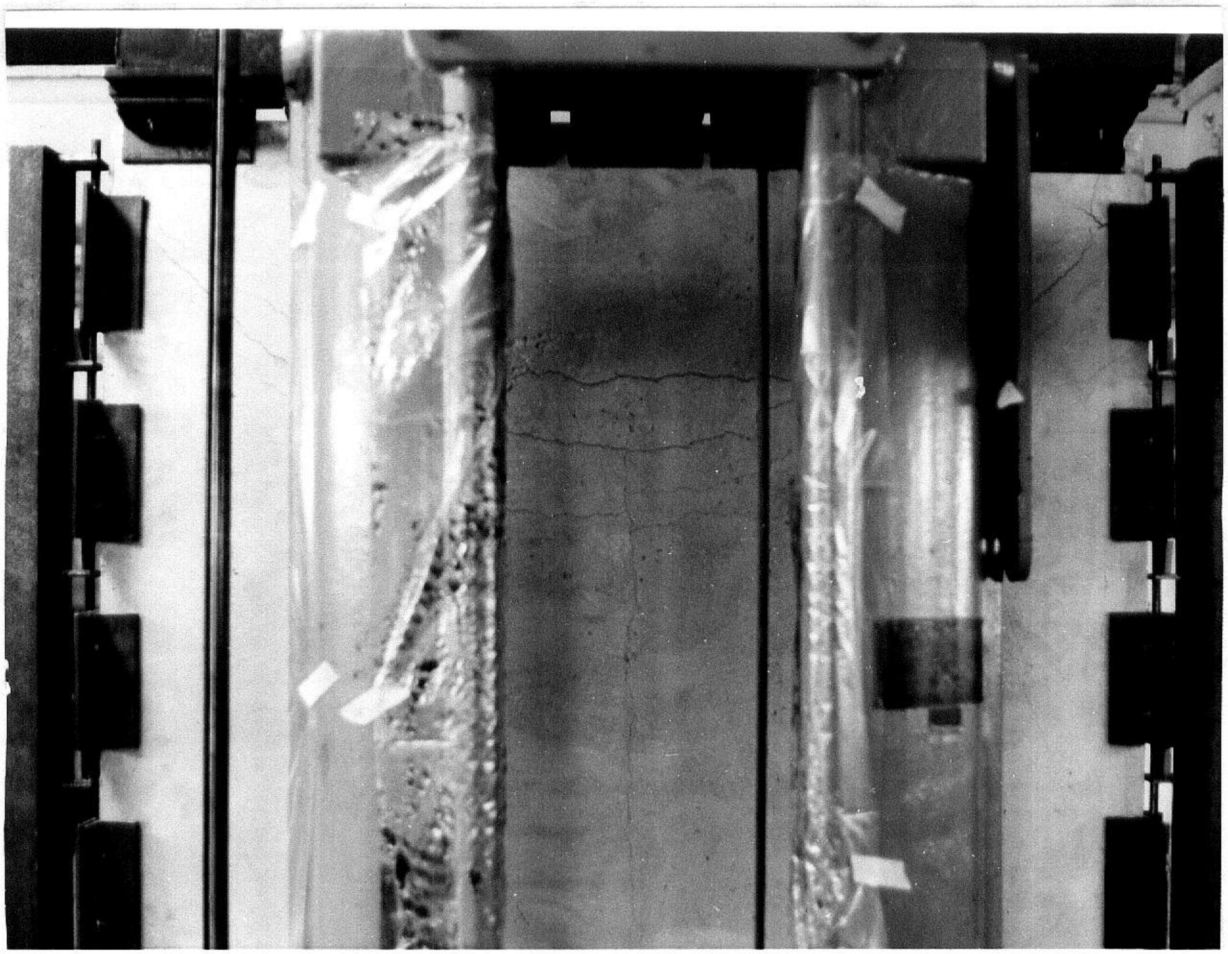

Fig. 12. Crack Pattern of First Trial Panel

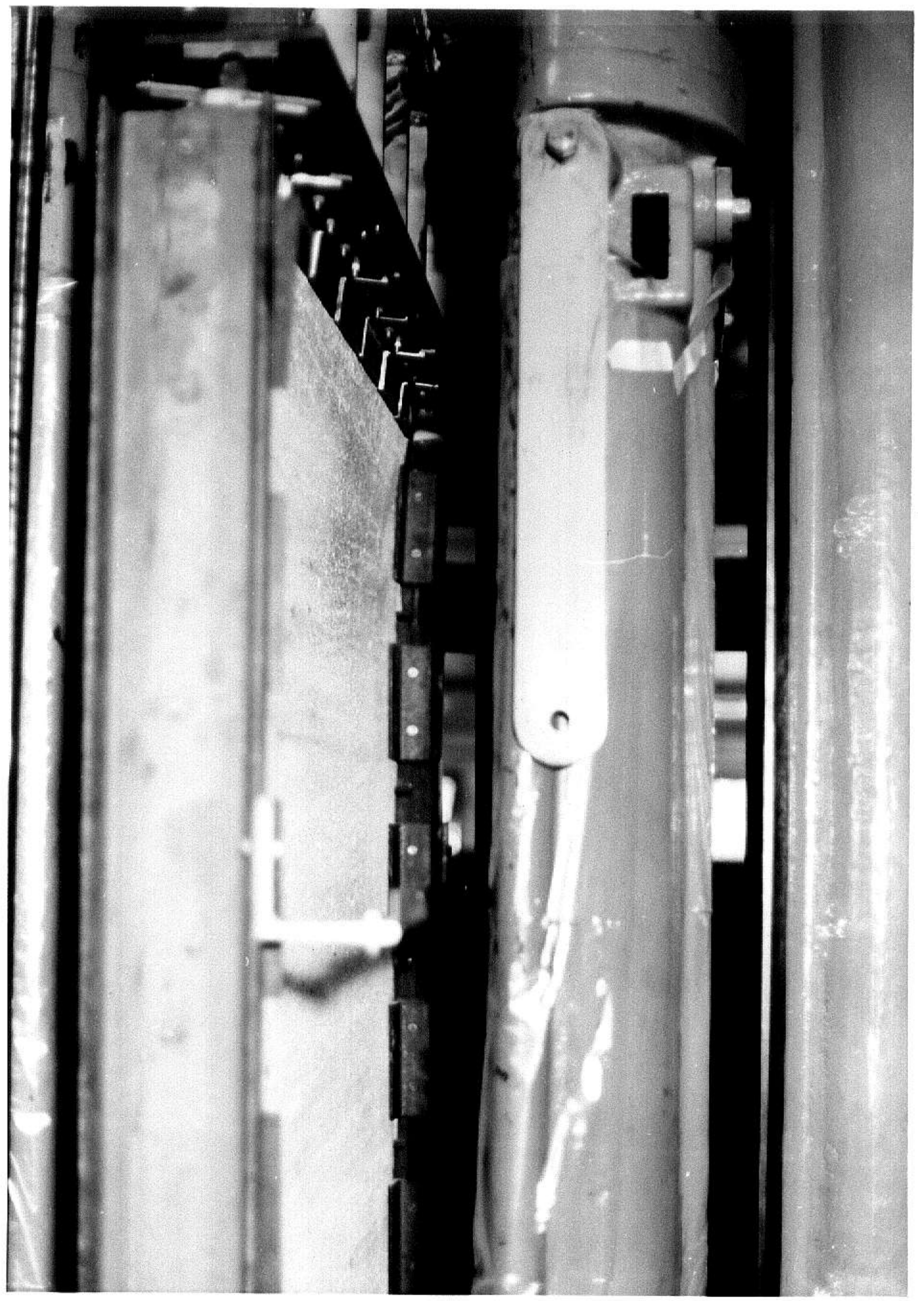

Fig. 13. Buckling of First Trial Panel

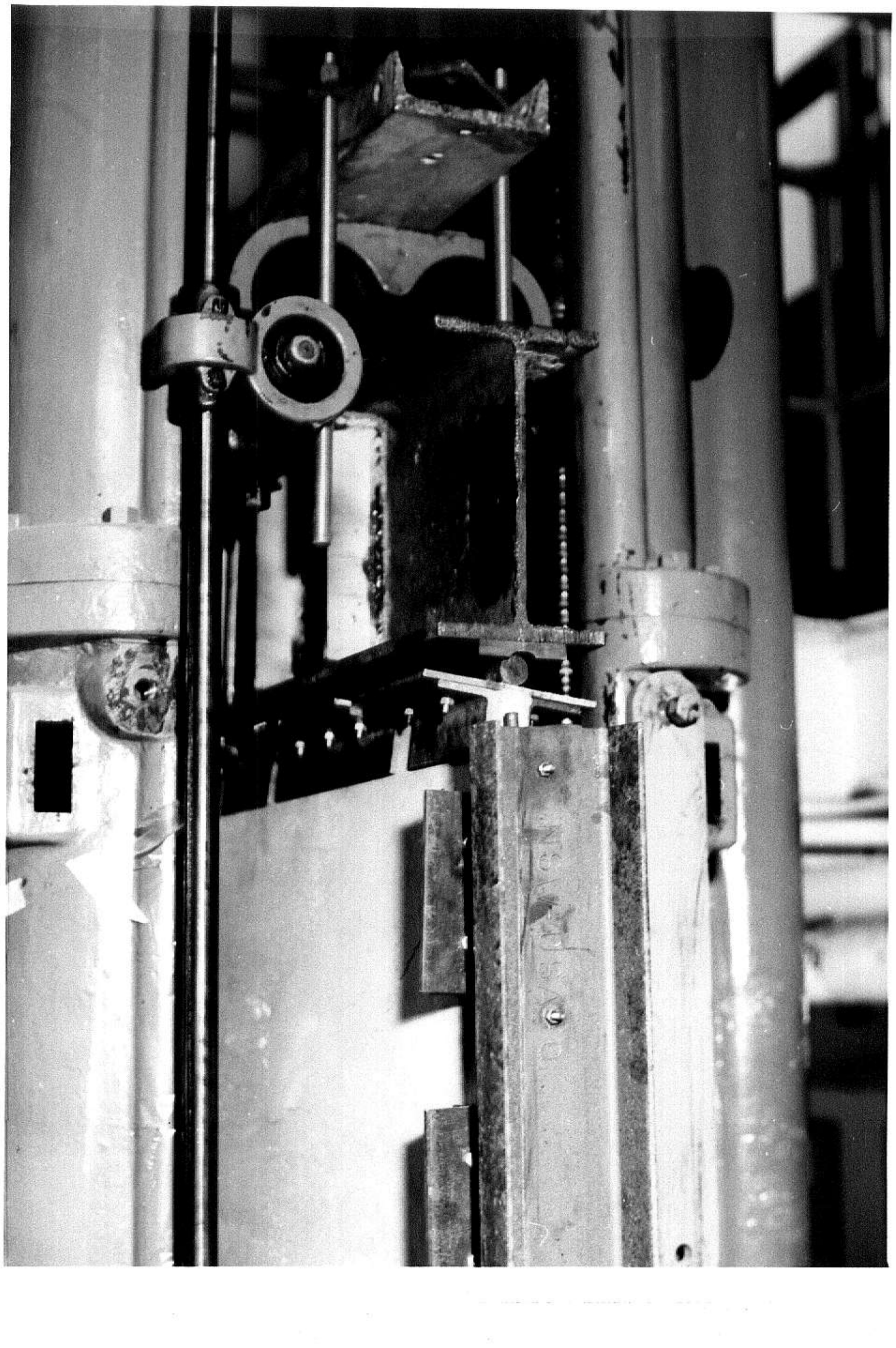

Fig. 14. Rotation of Top Clamps

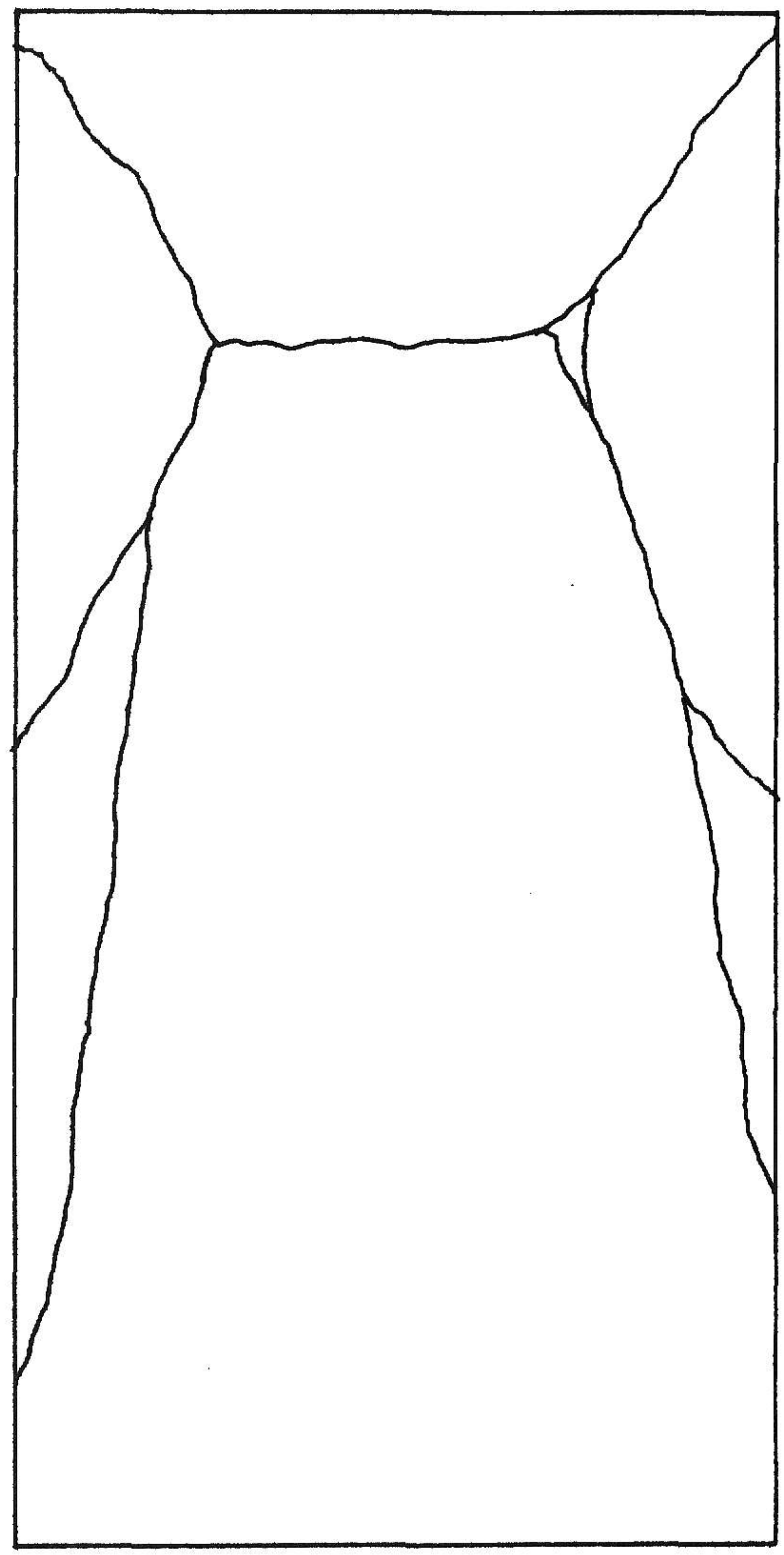

Fig. 15. Crack Pattern of Second Trial Panel

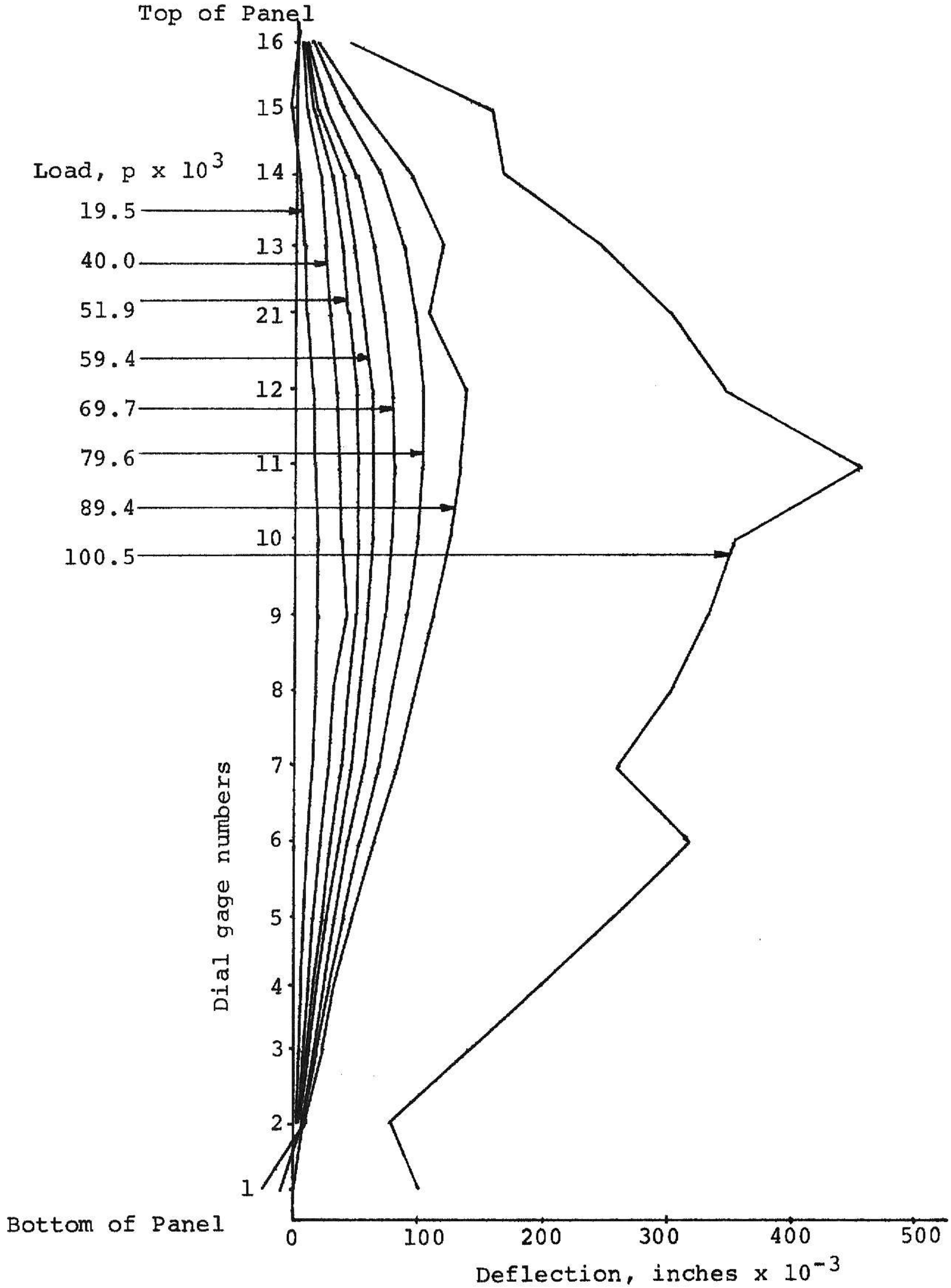

Fig. 16. Vertical Line of Deflections for Second Trial Panel

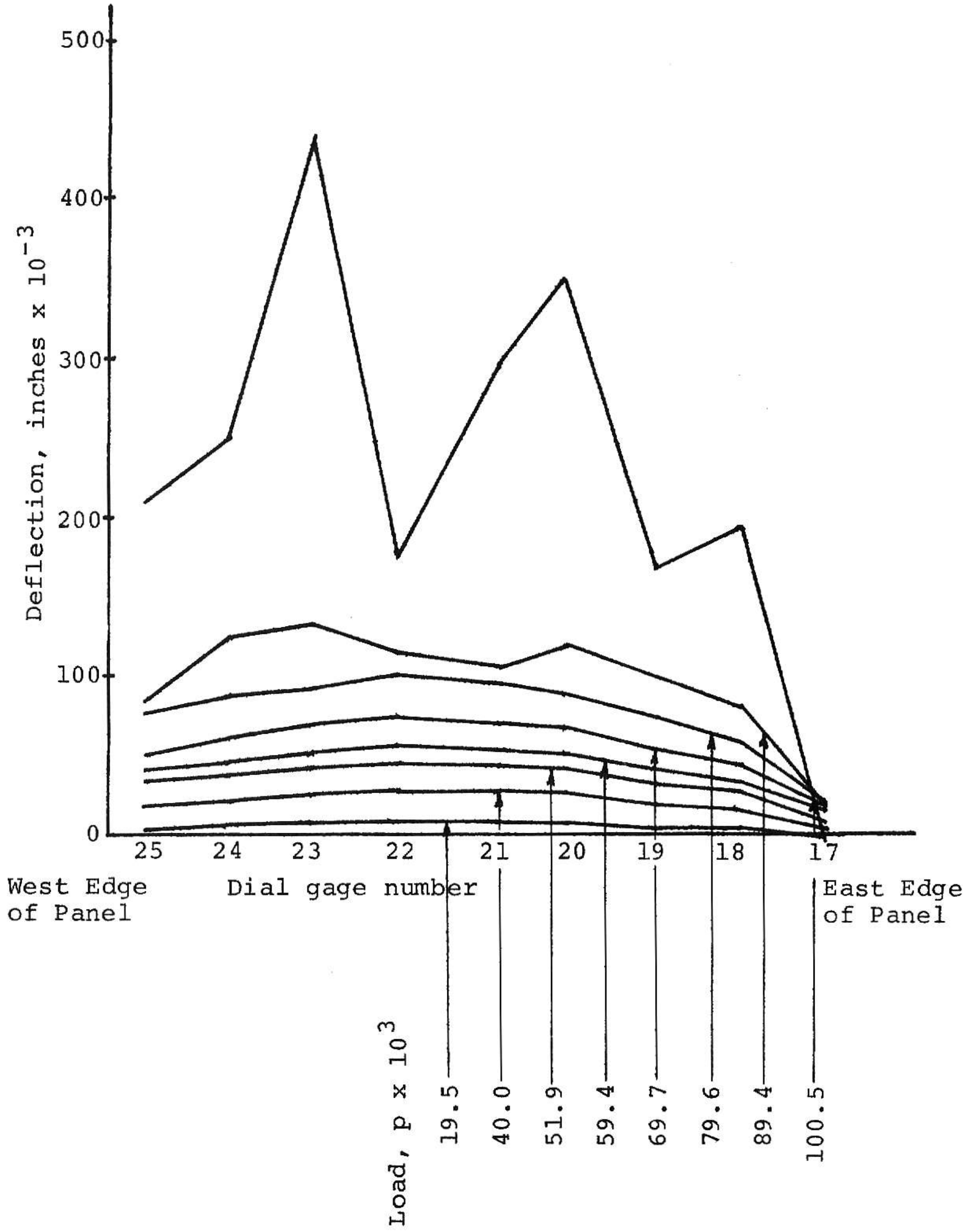

Fig. 17. Horizontal Line of Deflections for Second Trial Panel

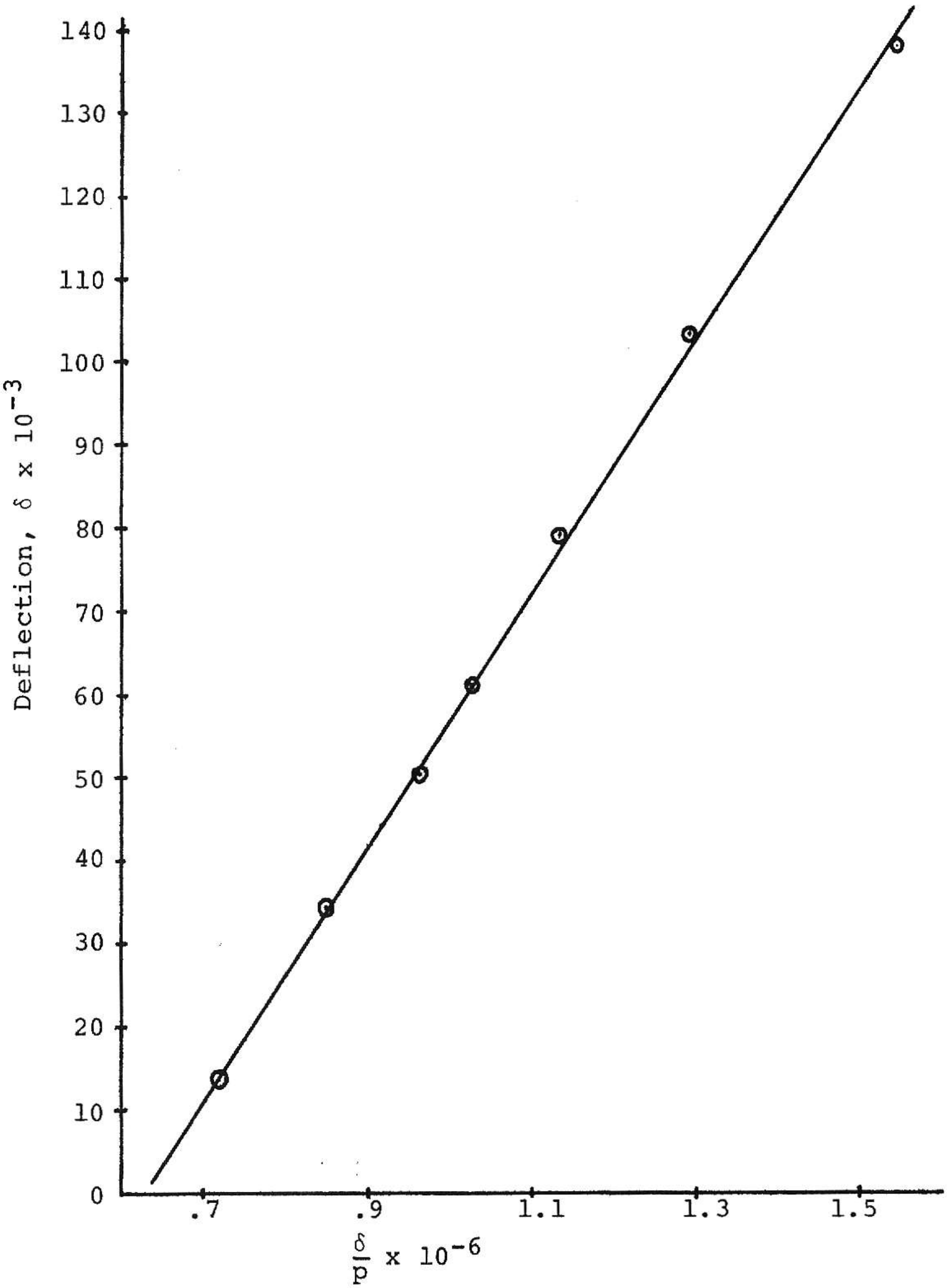

Fig. 18. Southwell Plot for Dial Gage No. 12 for Second Trial Panel

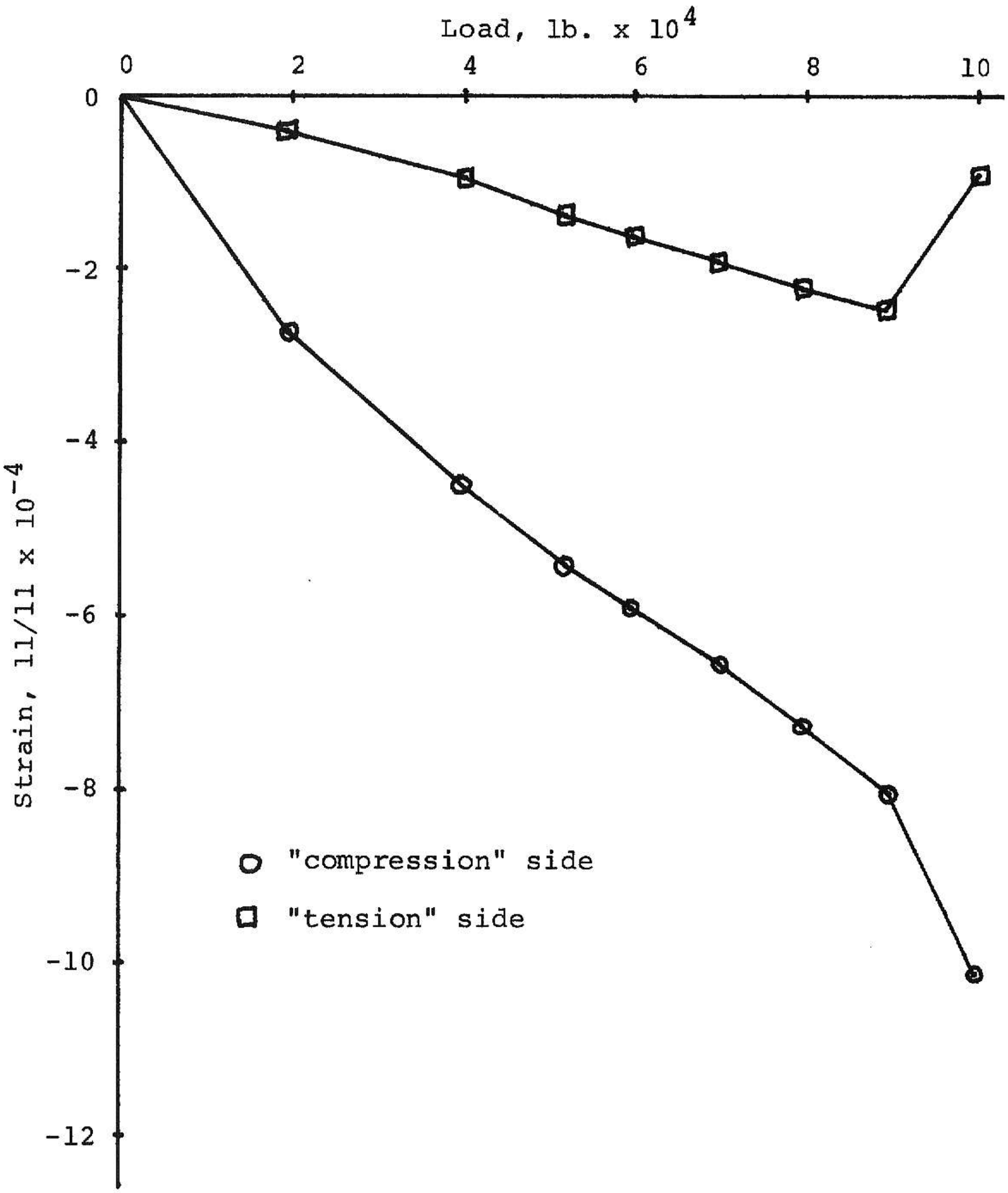

Fig. 19. Strain Gage Readings for Second Trial Panel

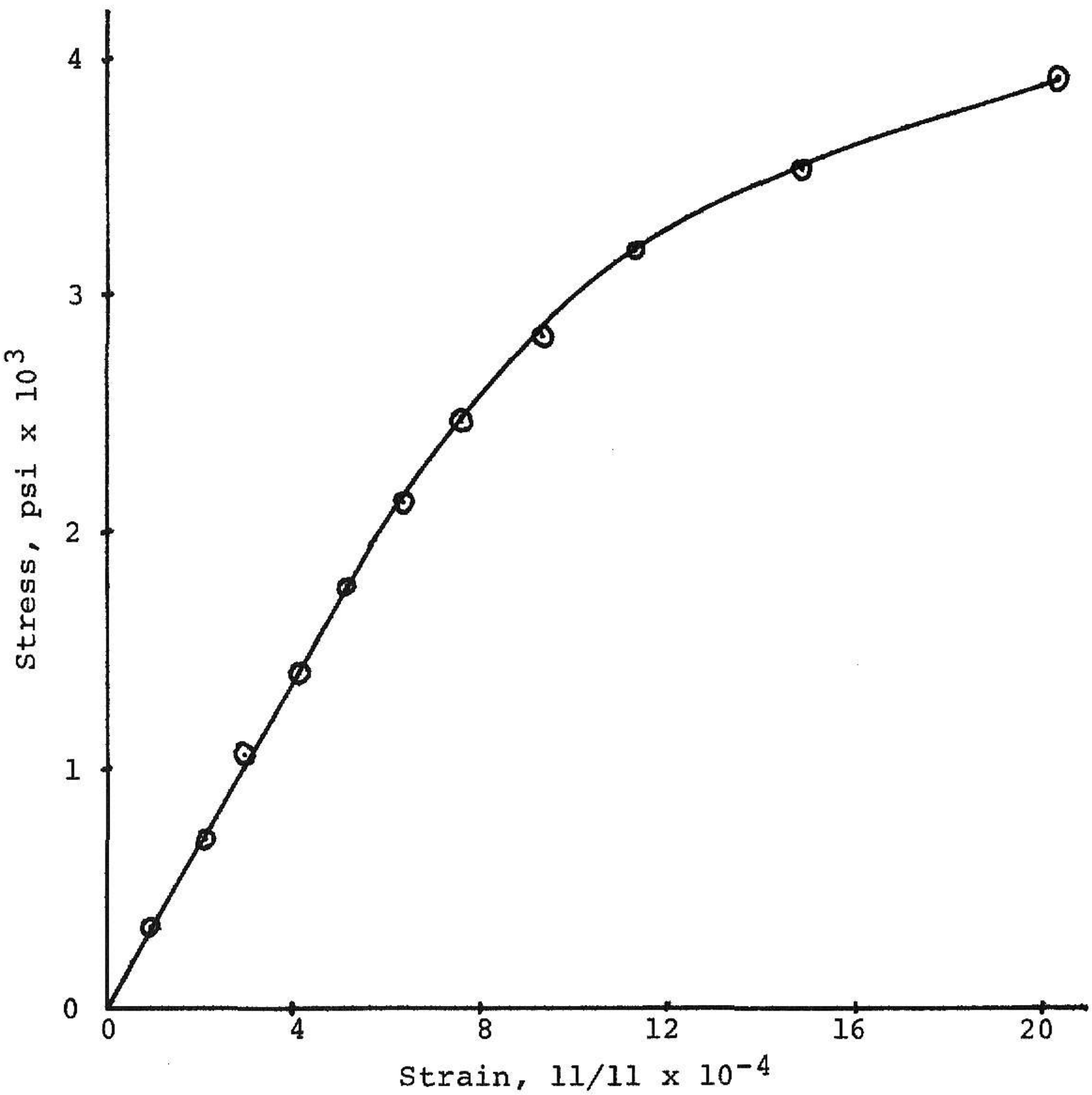

Fig. 20. Nominal Stress-Strain Diagram for Test Cylinder of Second Trial Panel

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DESIGN OF CASTING AND TESTING APPARATUS FOR REINFORCED CONCRETE PANELS

by

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ABSTRACT

This thesis presents the design, construction, and testing of the apparatus required for the casting and testing of rectangular reinforced concrete panels. These panels, which were 4' x 8', were simply supported on all four edges and subjected to uniformly distributed, uniaxial compression. The original designs are presented together with test results which indicated the need for modification of the equipment. These modifications are also presented.

The criteria which were used in judging the adequacy of the test setup in fulfilling the theoretical support and load conditions were:

1. Visual inspection. The panels should fail by buckling with biaxial curvature. A panel failure mechanism should occur at failure in the buckled region.
2. Deflection readings should also indicate this buckling pattern and should show very small deflections at the support locations.
3. Using the deflection readings, a linear Southwell plot should be obtained.
4. The eccentricity of loading, as indicated by strain and dial readings, should be small.

A test of the modified experimental setup was made, and results are presented, which indicate that the test frame is fulfilling the theoretical support and load conditions adequately.