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BRIDGE GIRDER WEBS SUBJECTED TO HORIZONTAL LOADS

BY

RANDALL FELIX PREHEIM

A thesis submitted
in partial fulfillment of the requirements for the
degree Master of Science, Major in
Civil Engineering, South Dakota
State University

1970

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BRIDGE GIRDER WEBS SUBJECTED

TO HORIZONTAL LOADS

This thesis is approved as a creditable and independent investigation by a candidate for the degree, Master of Science, and is acceptable as meeting the thesis requirements for this degree. Acceptance of this thesis does not imply that the conclusions reached by the candidate are necessarily the conclusions of the major department.

Thesis Advisor Date

Head, Civil Engineering Dept. Date

ACKNOWLEDGEMENTS

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The author also wishes to express his sincere thanks to Thomas A. Biggar, Laboratory Technician for his valuable assistance during this project.

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CHAPTER I

INTRODUCTION

A. General

Common bridge construction practice in South Dakota involves suspending the deck finishing machine and the freshly poured deck overhang from temporary metal brackets. These brackets are attached to the webs of the exterior plate girders. When loaded during construction, the brackets transmit to the web a vertical shearing force plus a couple. Since the webs are not designed for carrying horizontal loads, this type of loading could overstress the webs and appreciably lower their ultimate resistance to buckling. With specifications now permitting large depth to thickness ratios, (1,2)* deflections as well as stresses may be excessive. Rotation of the brackets, caused by deflection of the webs, lowers the paving machine and could result in undesirable thinning of the slab.

To reduce stresses and deflections, contractors are now required to place the brackets within six inches of a lateral stiffener. However, because stiffener spacing varies from bridge to bridge, standardization of formwork becomes impossible and the resulting

*Numbers in parentheses refer to entries in the Bibliography.

bracket spacing may not always be the most economical. By developing a bracket which could be used without regard to stiffener spacing, construction time and cost could be reduced. Figure 1 illustrates the manner in which construction brackets are used.

B. Historical Background

Temporary construction brackets have been used for many years. At first the brackets were built specifically for a certain depth of girder. They were attached with a bolt near the top flange and extended down to the bottom flange, thus occupying the full girder depth. The girders were generally wide flange sections of a standard depth, and therefore relatively few different bracket types were needed. However, as the plate girder came into more widespread use, it became impractical to build different brackets for the wide range of girder depths, so contractors began using one bracket type for all girders. This bracket had to be short enough to fit shallow girders, and consequently when it was used on deep girders high web stresses developed. (3)

The problem of analyzing horizontal loads on plate girder webs is very complex. To simplify the analysis several assumptions have been made. In one method of solution, the web of the girder is assumed to be a beam with the flanges acting as fixed ends. This beam is loaded with two equal and opposite concentrated loads as shown in Figure 2. The maximum moments produced by the two loads "F" are determined using standard beam formulae. The corresponding stresses

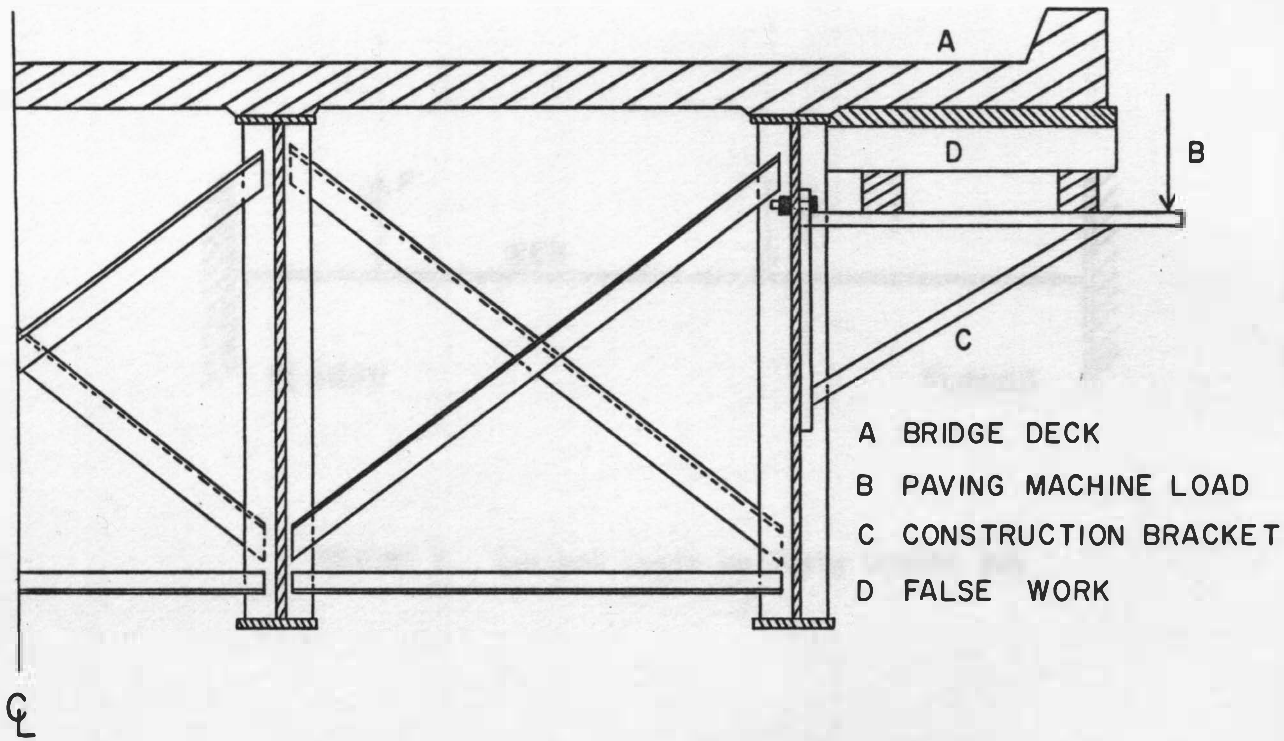

FIGURE 1. Construction Bracket Mounted on Bridge Girder Web

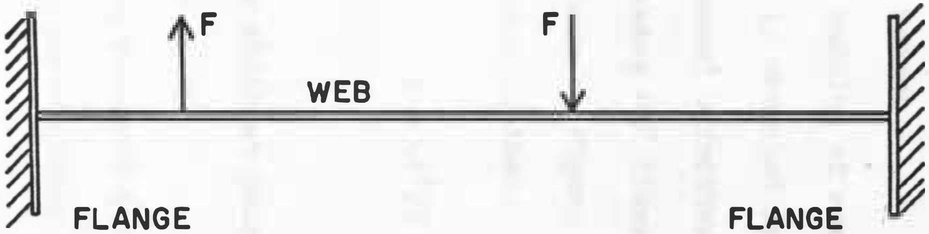

FIGURE 2. Bracket Loads on Plate Girder Web

are calculated using the flexure formula:

$$f = M/S \tag{1}$$

where

f = unit stress

M = applied moment

S = section modulus of assumed beam

The section modulus "S" is obtained for the web thickness under consideration and an assumed effective width "b". This width "b" can be determined by drawing 45° lines from the points of loading out to the flanges as shown in Figure 3.

The section modulus is then:

$$S = bt^2/6 \tag{2}$$

where

b = effective width at point being investigated

t = thickness of web

There is no record of any research done to date on the effect of construction brackets on plate girder webs.

C. Object and Scope of Investigation

The objective of this experiment was to investigate the web stresses and deflections which occur as a result of using temporary construction brackets. The results of this study should provide information on whether the short brackets now in common use function satisfactorily for all depths of plate girders.

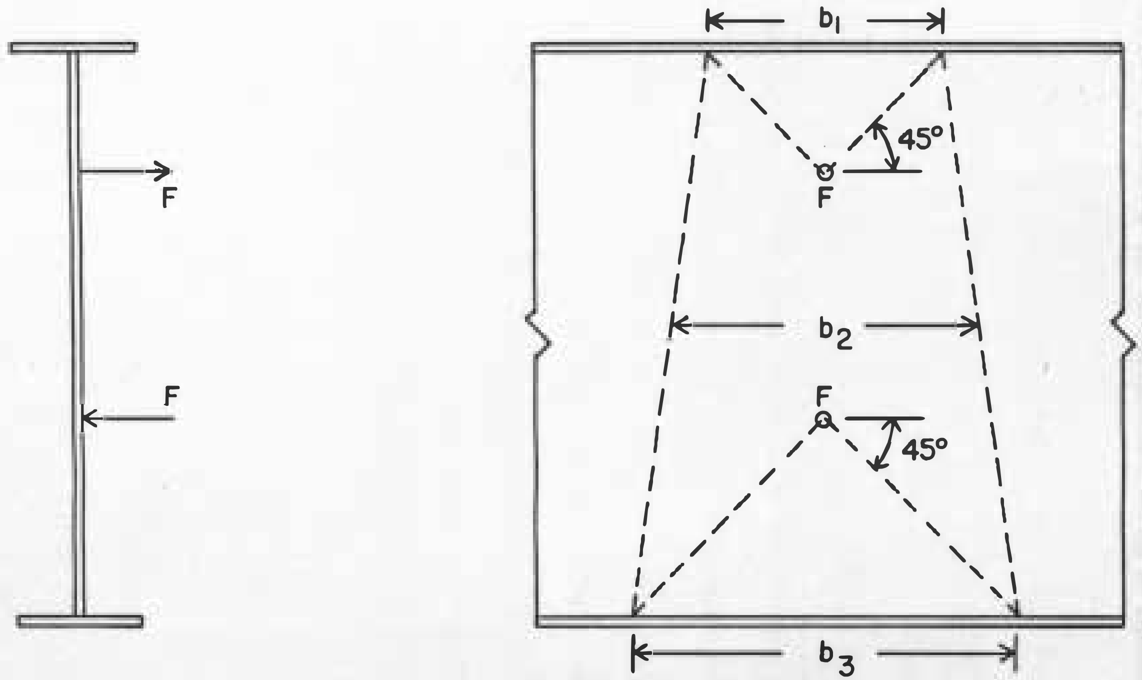

FIGURE 3. Method of Determining Effective Width b

This study was primarily concerned with the effect of bracket depth on the plate girder. Four bracket types were investigated: a short bracket similar to those in common use, a long bracket which extended down to the bottom flange, an intermediate length bracket, and a short bracket in combination with a backup angle.

CHAPTER II

TESTING PROGRAM

A. Materials and Test Specimens

Four bracket types were investigated in this study. Bracket Type I was a short bracket similar to those now in common use. Bracket Type II was an intermediate length bracket. Bracket Type III was a long bracket which extended down to the bottom flange, and Bracket Type IV was a short bracket identical to Bracket Type I except that it was backed with a steel angle. All of the brackets were fabricated out of three and four inch steel channels, and 1-1/2 inch diameter pipe. A 5 x 5 x 5/16 inch steel angle was used for the backup angle on Bracket Type IV. Details and dimensions of the brackets are shown in Figures 4, 5, 6, and 7. Figures 8 and 9 are illustrative views of the different bracket types used in this investigation.

All of the tests were conducted on two large scale laboratory model plate girders. The dimensions of these girders were limited by the capacity of the testing machine to a length of 20 feet and a depth of 44 inches. The web thickness of the girders was 5/16 inch. The top flanges measured 12 x 1/2 inch, and the bottom flanges measured 13 x 7/8 inch. Lateral stiffeners measuring 5 x 5/16 inch were placed on the back side of the girders at intervals of

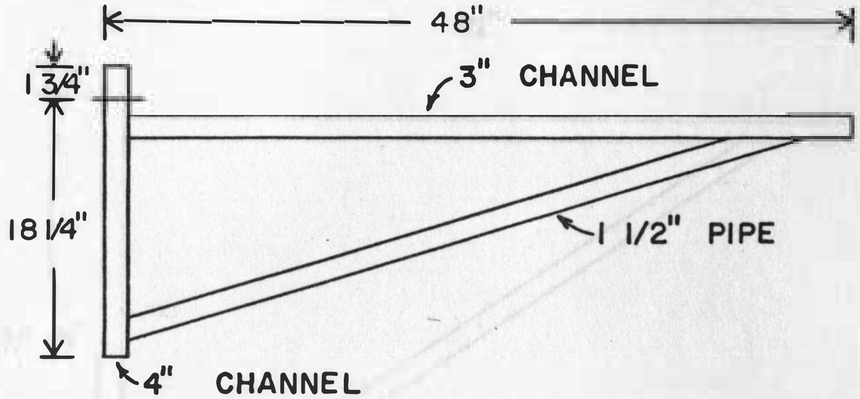

FIGURE 4. Bracket Type I Details

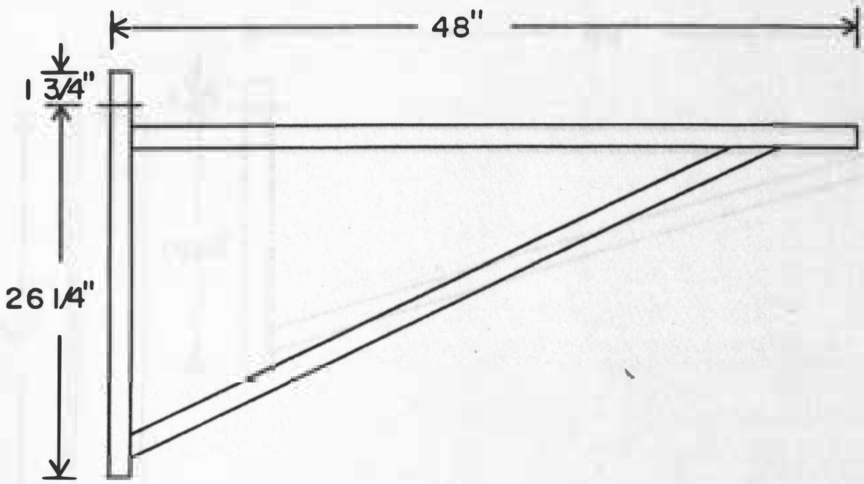

FIGURE. 5. Bracket Type II Details

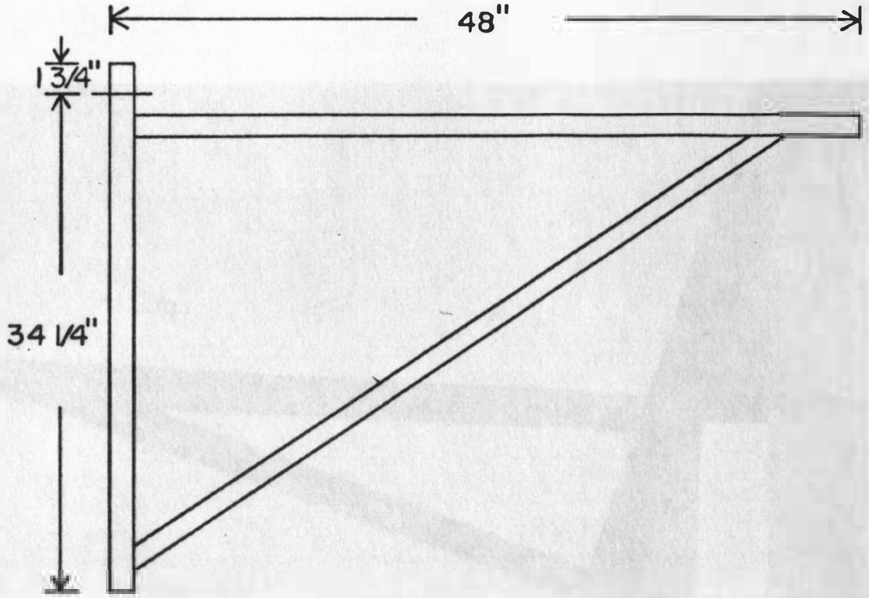

FIGURE 6. Bracket Type III Details

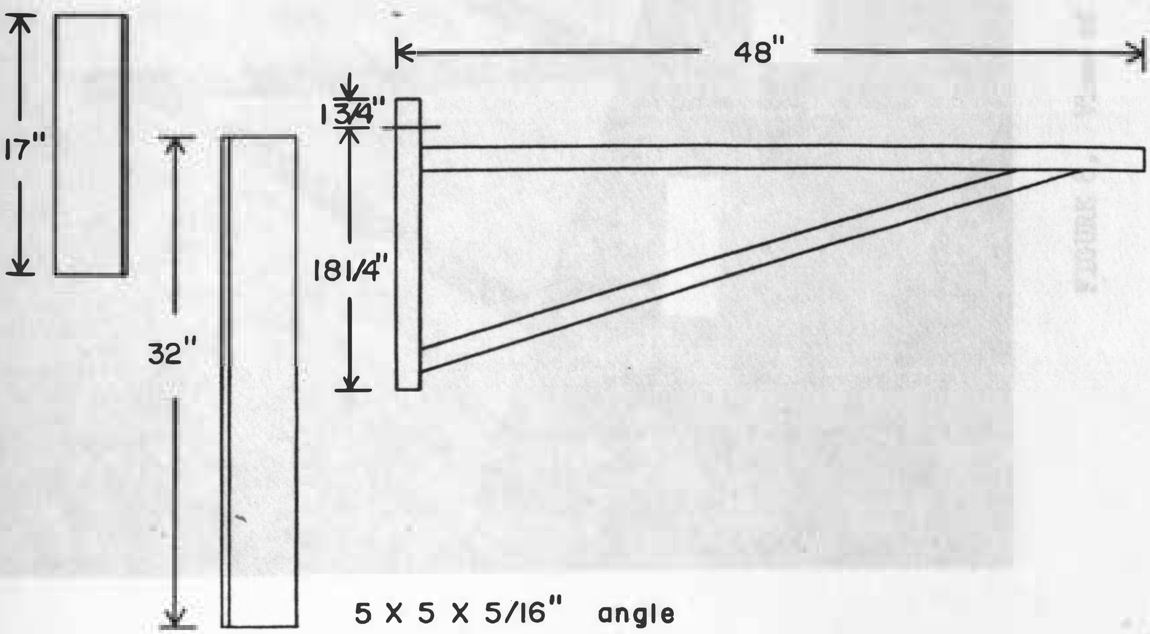

FIGURE 7. Bracket Type IV Details

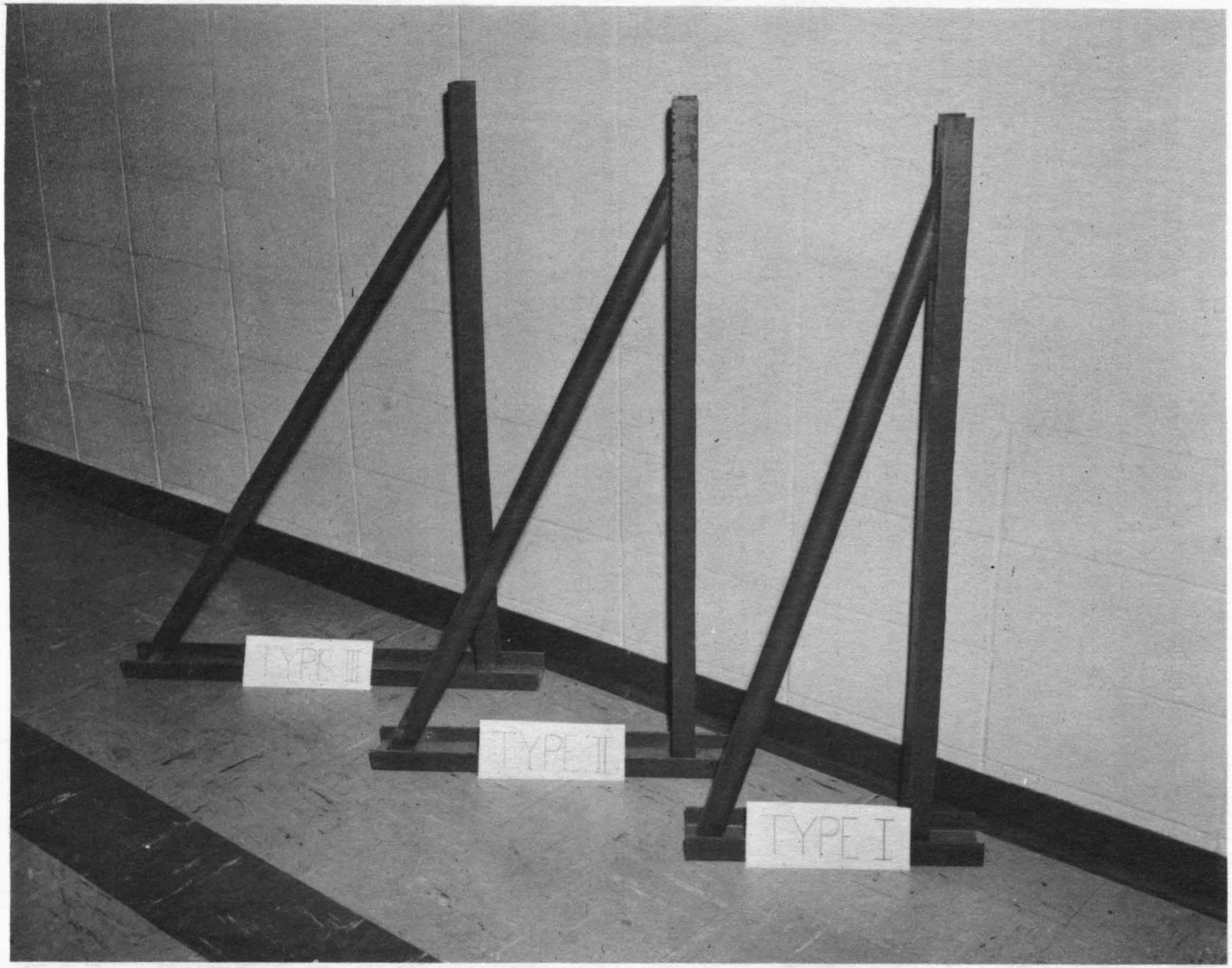

FIGURE 8. View of Bracket Types I, II and III

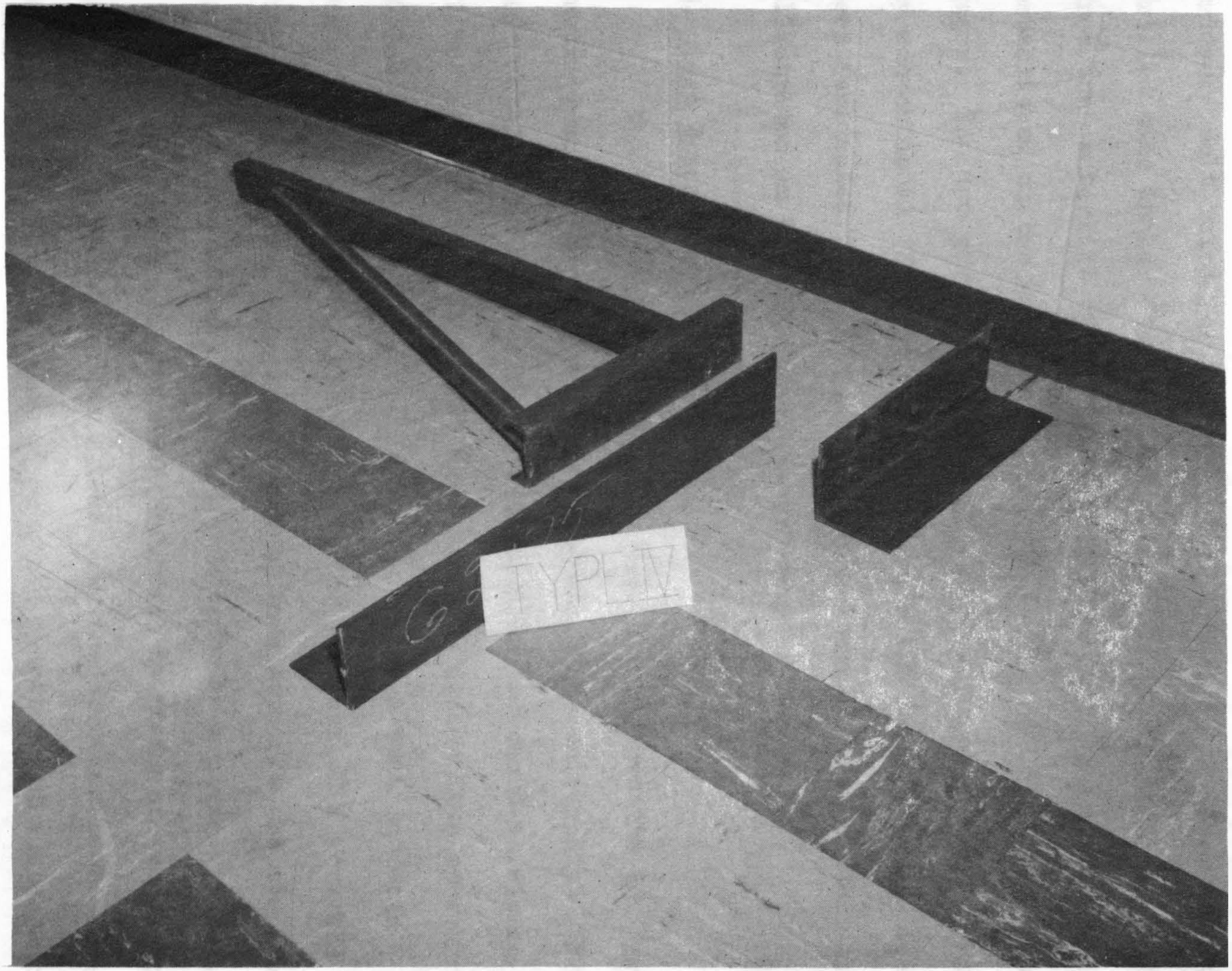

FIGURE 9. View of Bracket Type IV

44 inches from the center line. The girders were so designed in order to simulate an actual plate girder for use in a composite bridge. Bolt holes for the brackets, 13/16 inch in diameter, were drilled nine inches from the top flange at intervals of two feet from the centerline of the girder. A four inch grid system was established on the web of one girder for application of the strain gages. The two girders were simply supported, and were held together by conventional "X" type cross bracing at the two ends. All specimens were fabricated from ASTM A-36 structural steel. The girders are illustrated in Figure 10.

B. Test Apparatus

The two model plate girders were placed into the 120 ton testing machine available at South Dakota State University. This unit consists of a large steel testing frame as shown in Figure 11. It has a load capacity of 120 tons, and can accommodate specimens up to 20 feet in length, five feet in width, and four feet in depth. It was adapted for use in this study by the addition of a moveable 13 foot spreader beam. The spreader beam made possible the simultaneous application of overhead loads to brackets on both plate girders.

Loads were applied to the brackets by means of two 10 ton single acting hydraulic rams attached to the spreader beam. These rams were activated by means of manually operated hydraulic pumps. Before testing, both jacks were calibrated on a testing machine.

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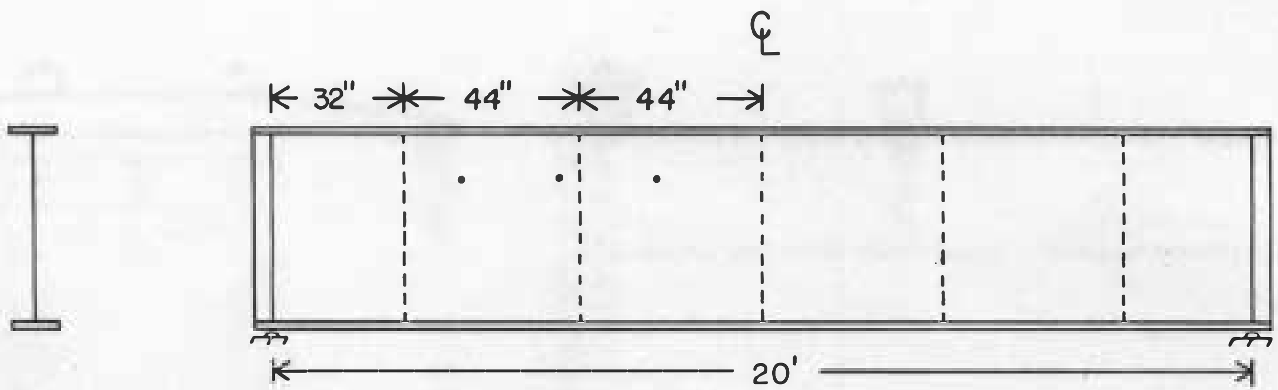

TOP FLANGE = 12X 1/2 inch
BOTTOM FLANGE = 13X 7/8 inch
WEB = 44X 5/16 inch
STIFFENERS = 5X 5/16 inch

FIGURE 10. Plate Girder Details

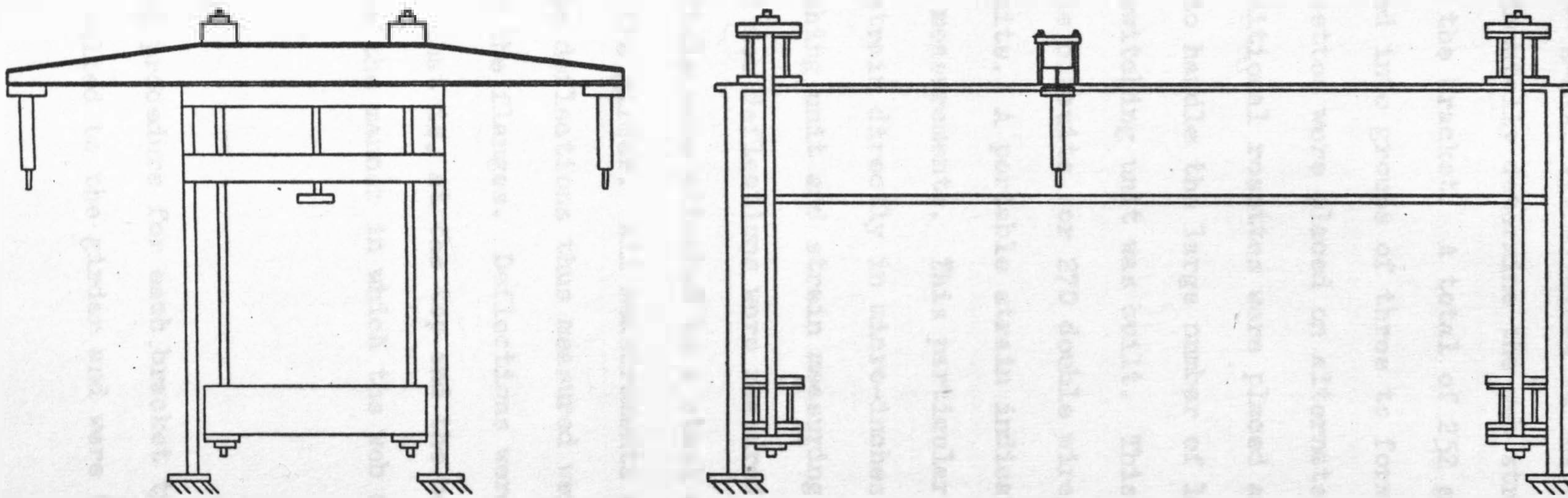

FIGURE 11. Testing Frame

SR-4 strain gages were attached to the web on one plate girder in order to experimentally determine the web stresses which developed during loading of the bracket. A total of 252 gages was used. These gages were arranged into groups of three to form 84 rectangular rosettes. The rosettes were placed on alternate grid points as shown in Figure 12. Additional rosettes were placed around the bolt holes.

In order to handle the large number of lead wires from the gages, a special switching unit was built. This unit is capable of handling 540 single circuits, or 270 double wire, temperature compensating circuits. A portable strain indicator was used for making the strain measurements. This particular instrument is designed to read strain directly in micro-inches per inch. Figure 13 shows the switching unit and strain measuring equipment.

Horizontal web deflections were measured by means of two dial indicators. The dials were attached to a steel angle which was clamped to the flanges of the girder. All measurements were taken with respect to this angle. The deflections thus measured were therefore relative to the movement of the flanges. Deflections were measured at the two points of loading, that is, at the top and the bottom of the brackets. Figure 14 indicates the manner in which the web deflections were measured.

C. Test Procedure

The testing procedure for each bracket type was identical. The brackets were bolted to the girder and were then loaded with an

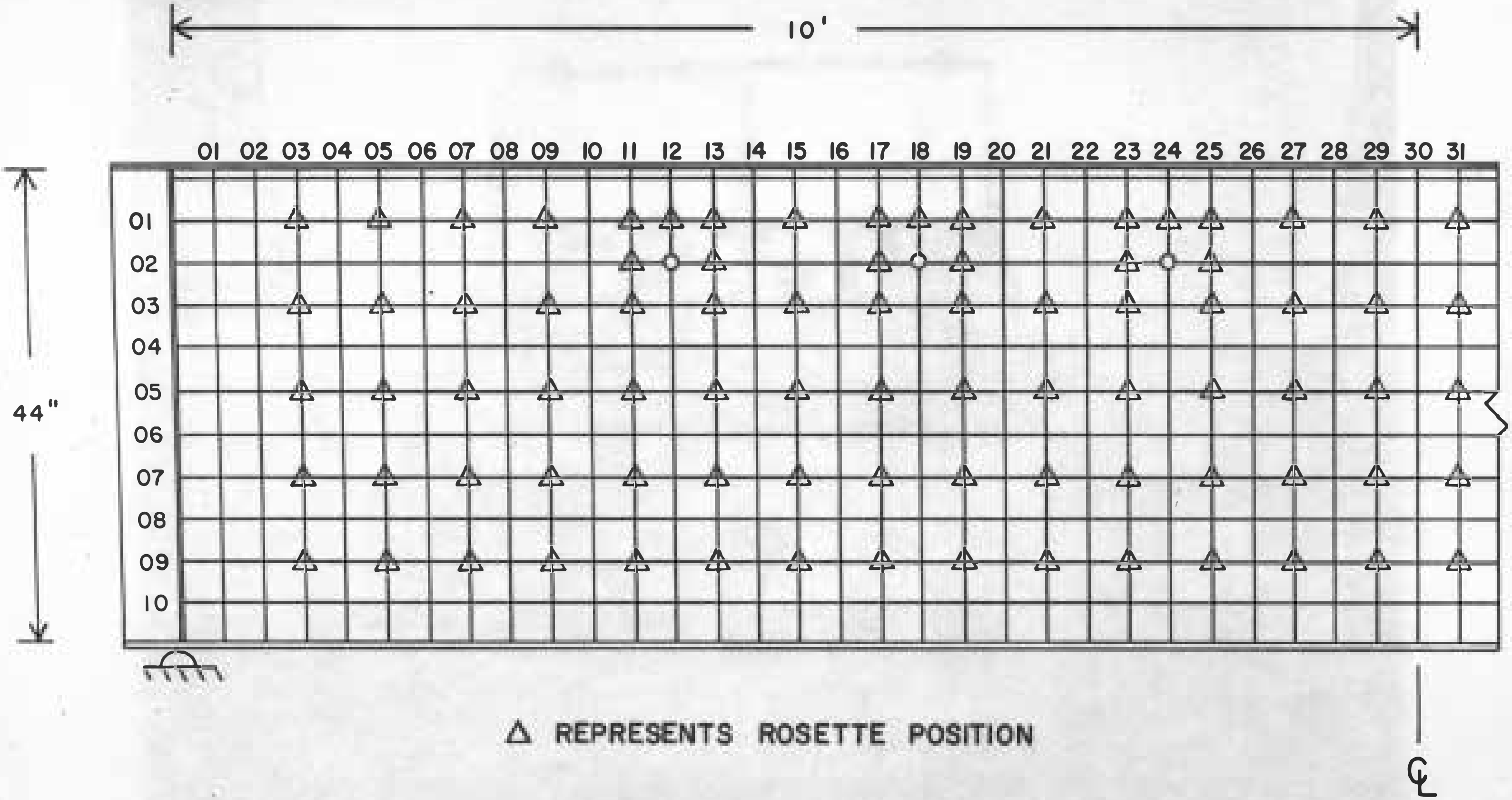

FIGURE 12. Rosette Distribution on Plate Girder Web

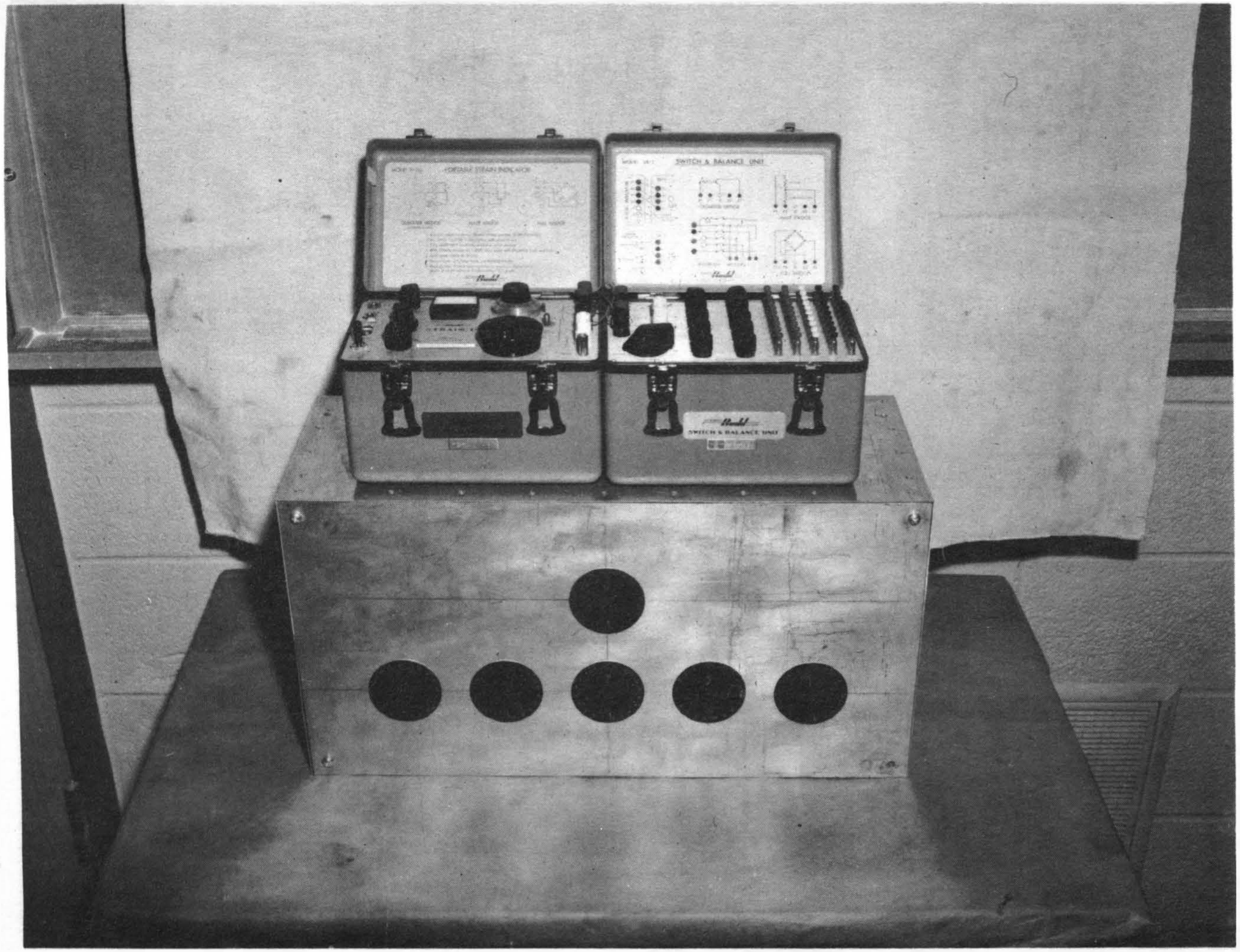

FIGURE 13. Switching Unit and Strain Measuring Equipment

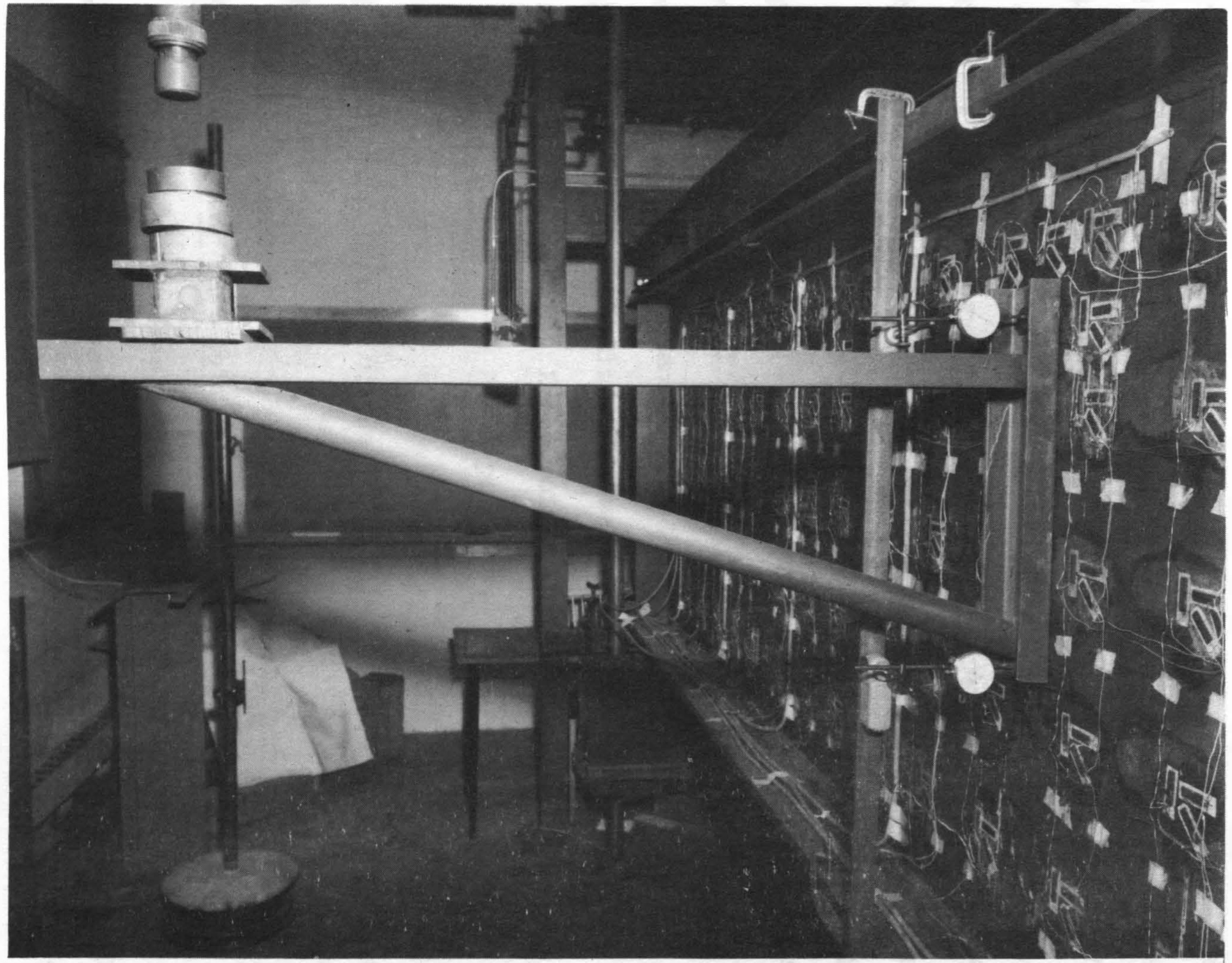

FIGURE 14. Instrumentation for Measuring Horizontal Web Deflections

initial load of 2 kips at a distance of 42 inches from the web. This was done in order to cold work any areas of high residual stress. Test loads were applied in increments of 500 pounds at distances from the web of 18, 30 and 42 inches. Yielding was avoided by checking the stresses after each increment of load. During each test, the load was held constant while the strain gages were being read. Readings were taken for the 43 rosettes nearest the panel point being investigated. Web deflections were also measured at this time.

Tests were run at three positions along the girder at panel points 12, 18 and 24 as shown in Figure 12. These three points were selected to study the effect of the lateral stiffener in reducing the web stresses. The distances from the points of loading to the nearest lateral stiffener are listed below:

Panel point 12, 16 inches

Panel point 18, 4 inches

Panel point 24, 20 inches

Figures 15 and 16 show Bracket Type I in position for testing on panel point 24.

D. Reduction of Test Data

The web stresses developed in the plate girder were experimentally determined by means of rectangular rosettes. The three element rectangular rosette employs strain gages at the 0, 45, and 90 degree positions as indicated in Figure 17. By measuring the strains in these three directions, the principal stresses can be

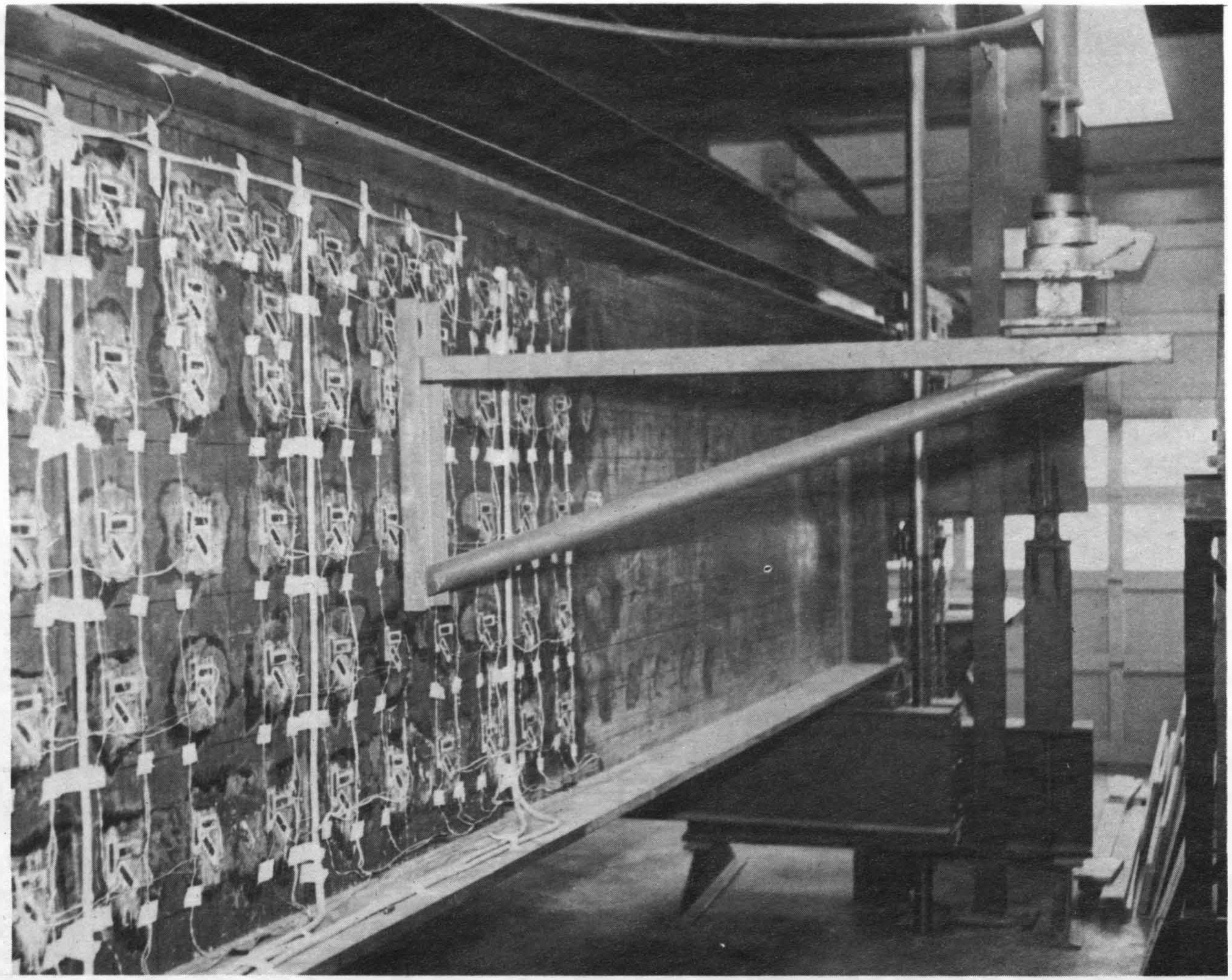

FIGURE 15. General View of Test Setup

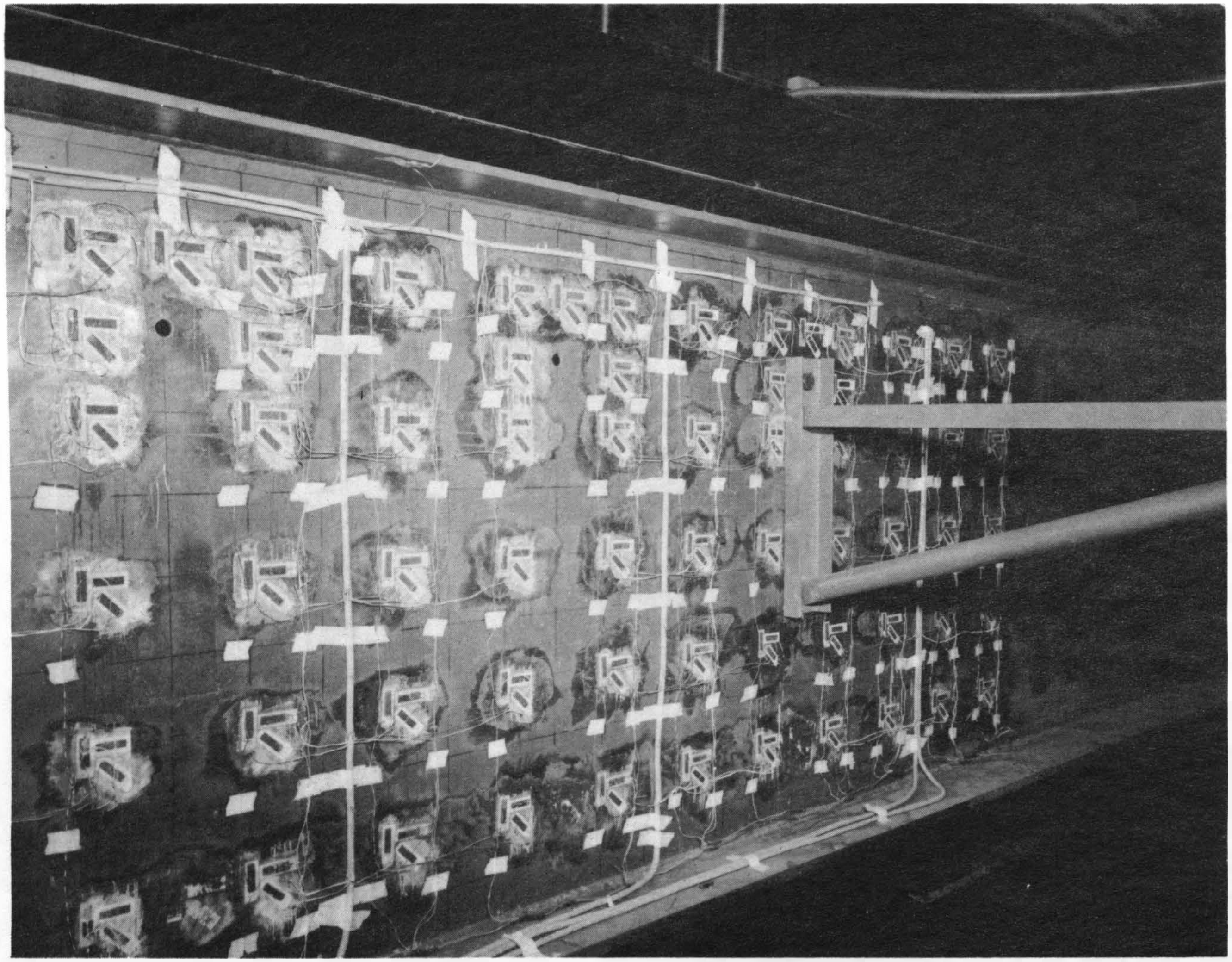

FIGURE 16. View Showing Bracket Type I in Position for Testing

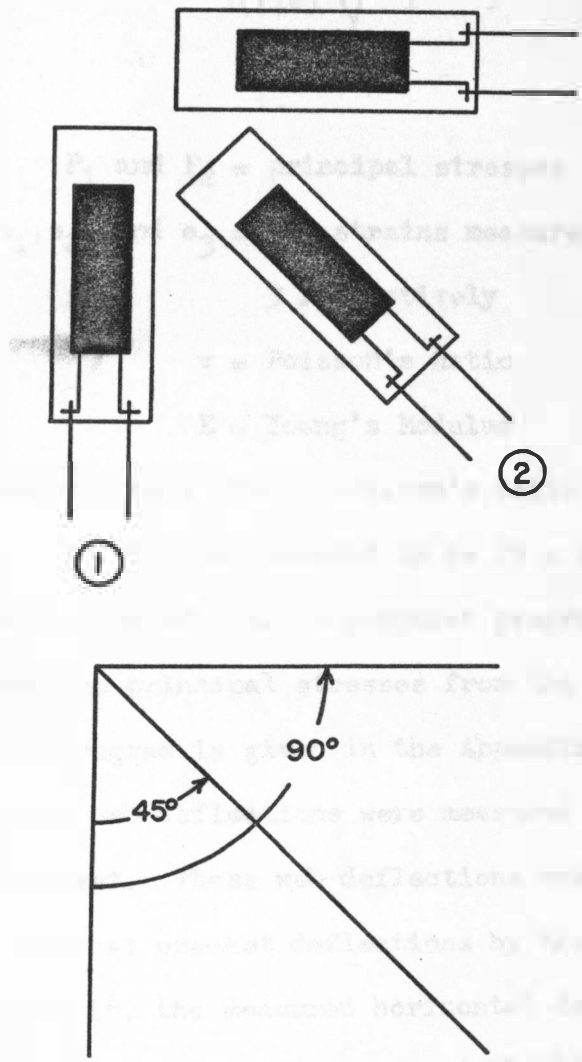

FIGURE 17. Three Element Rectangular Rosette

calculated by using the equation: (4,5)

$$P_1, P_2 = E\left(\frac{e_1 + e_3}{2(1-v)} \pm \frac{1}{2(1+v)}\sqrt{(e_1 - e_3)^2 + (2e_2 - e_1 - e_3)^2}\right) \tag{3}$$

where

P_1 and P_2 = principal stresses

e_1, e_2, and e_3 = the strains measured by gages 1, 2, and 3 respectively

v = Poisson's Ratio

E = Young's Modulus

For the specimens in this study, Poisson's Ratio was assumed to be 0.3, and Young's Modulus was assumed to be 29 x 10^6 psi. To simplify the reduction of data, a computer program was written which calculated the principal stresses from the measured values of strain. This program is given in the Appendix.

Horizontal web deflections were measured at the top and the bottom of the bracket. These web deflections were converted to corresponding vertical bracket deflections by the following method. Refering to Figure 18, the measured horizontal deflection of the web at the top of the bracket is d_t, the deflection at the bottom of the bracket is d_b. The depth of the bracket is D and its length is L. The resulting vertical deflection, $\triangle_v$, at any point, x, along

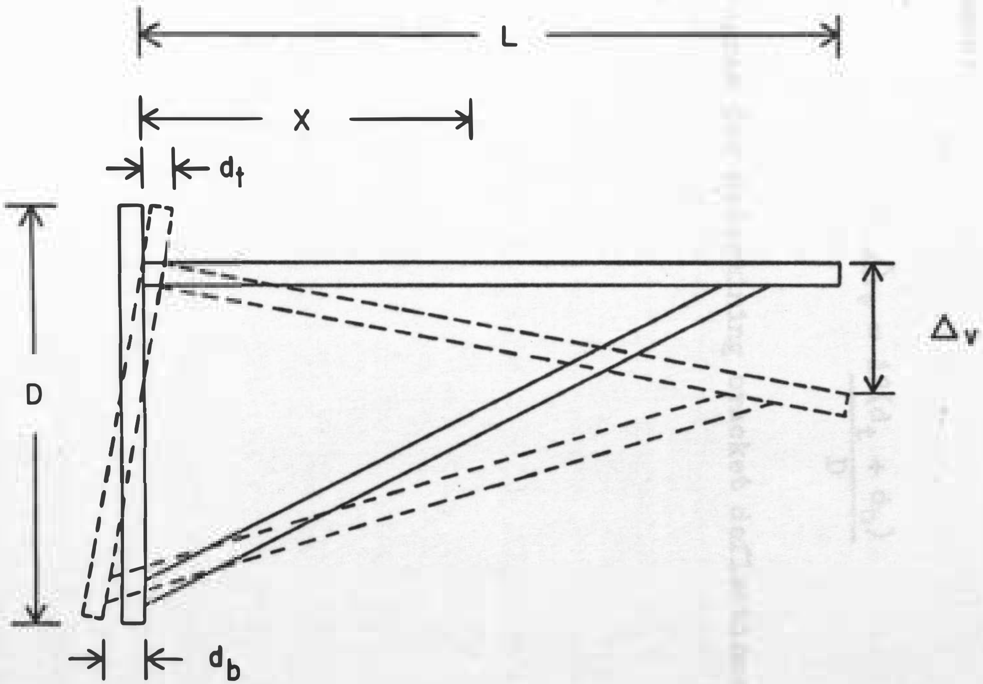

FIGURE 18. Bracket Deflections After Loading

the bracket is then given by the formula:

$$\triangle_v = \frac{x(d_t + d_b)}{D} \tag{4}$$

At one foot out from the web, x equals 12 inches, and the deflection equation becomes:

$$\triangle_v = \frac{12(d_t + d_b)}{D} \tag{5}$$

A computer program for determining bracket deflections is given in the Appendix.

CHAPTER III

TEST RESULTS

The results of the testing are presented in two parts:

A. Web Stresses

B. Bracket Deflections

A. Web Stresses

Stresses in the plate girder web due to loading of the bracket were found to vary in direct proportion to the moment applied. Consequently, sample results will be shown for only one test load, a load of two kips placed at a distance of 3-1/2 feet from the web. The moment transmitted to the web from this load is seven foot kips. Stress values for other loads would be in proportion to the moment applied. All results are presented in the form of maximum principal stress contours across a girder width of 5-1/2 feet. Stresses beyond this width were found to be quite small and were neglected.

1. Bracket Type I

Bracket Type I, when placed at panel point 24, 20 inches from the nearest stiffener, produced high web stresses under the applied load. A stress of 35.3 ksi was recorded at the point of tension loading around the bolt hole and a stress of -25.1 ksi was recorded at the point of compression loading at the bottom of the bracket. When this bracket was placed at panel point 18, 4 inches from the nearest stiffener, the maximum web stress was reduced nearly 55% to a value of 16.2 ksi at the bolt hole. The stress at the point of compression loading was reduced to 16.1 ksi. Stress values obtained with the bracket at panel point 12, 16 inches from the nearest stiffener, produced results nearly identical to the values taken with the bracket at 20 inches from the stiffener. The maximum stress at panel point 12 was 34.1 ksi. Figures 19, 20, and 21 illustrate the stress patterns produced using Bracket Type I.

2. Bracket Type II

Bracket Type II, when placed at panel point 24, 20 inches from the nearest stiffener, produced a maximum web stress of 20.4 ksi. When placed at panel point 18, 4 inches from the stiffener, the maximum web stress was reduced to 14.5 ksi. This is the equivalent of approximately a 29% reduction. At 16 inches from a stiffener, on panel point 12, this bracket produced results nearly identical to those obtained at 20 inches from the stiffener. A maximum stress of 20.2 ksi was recorded with the bracket at

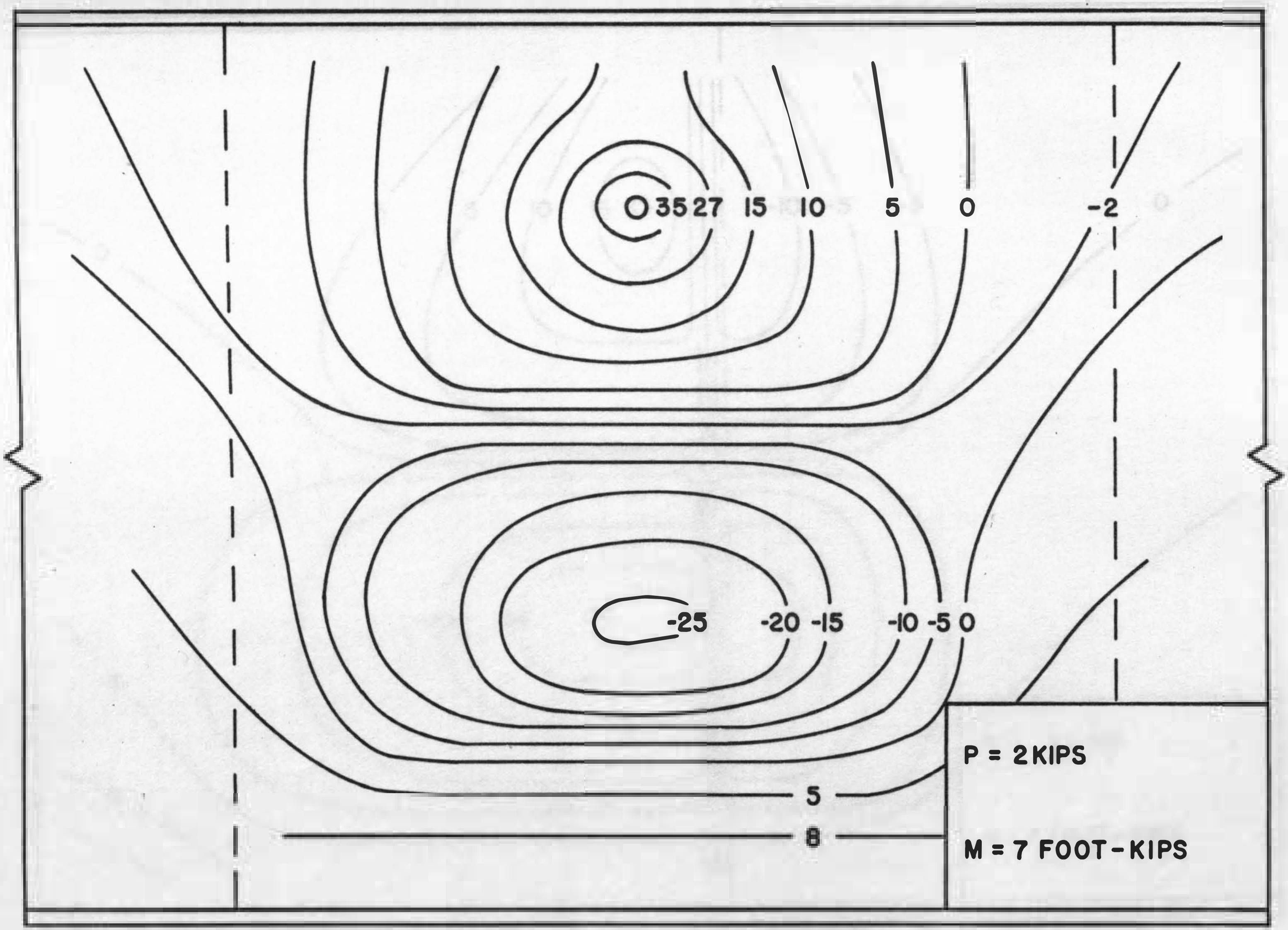

FIGURE 19. Principal Stresses (ksi) for Bracket Type I on Panel Point 24 (20 inches from nearest stiffener)

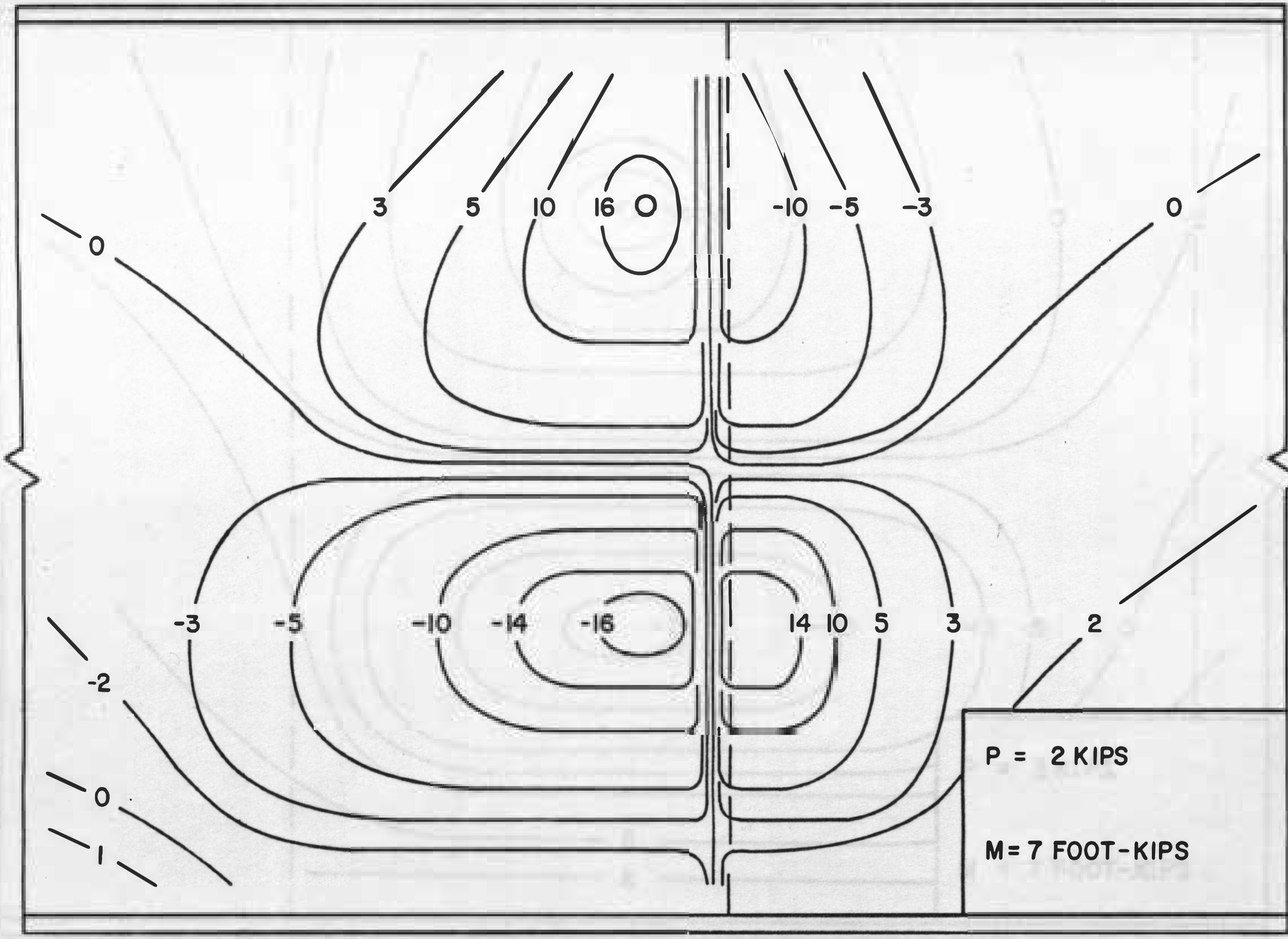

FIGURE 20. Principal Stresses (ksi) for Bracket Type I on Panel Point 18 (4 inches from nearest stiffener)

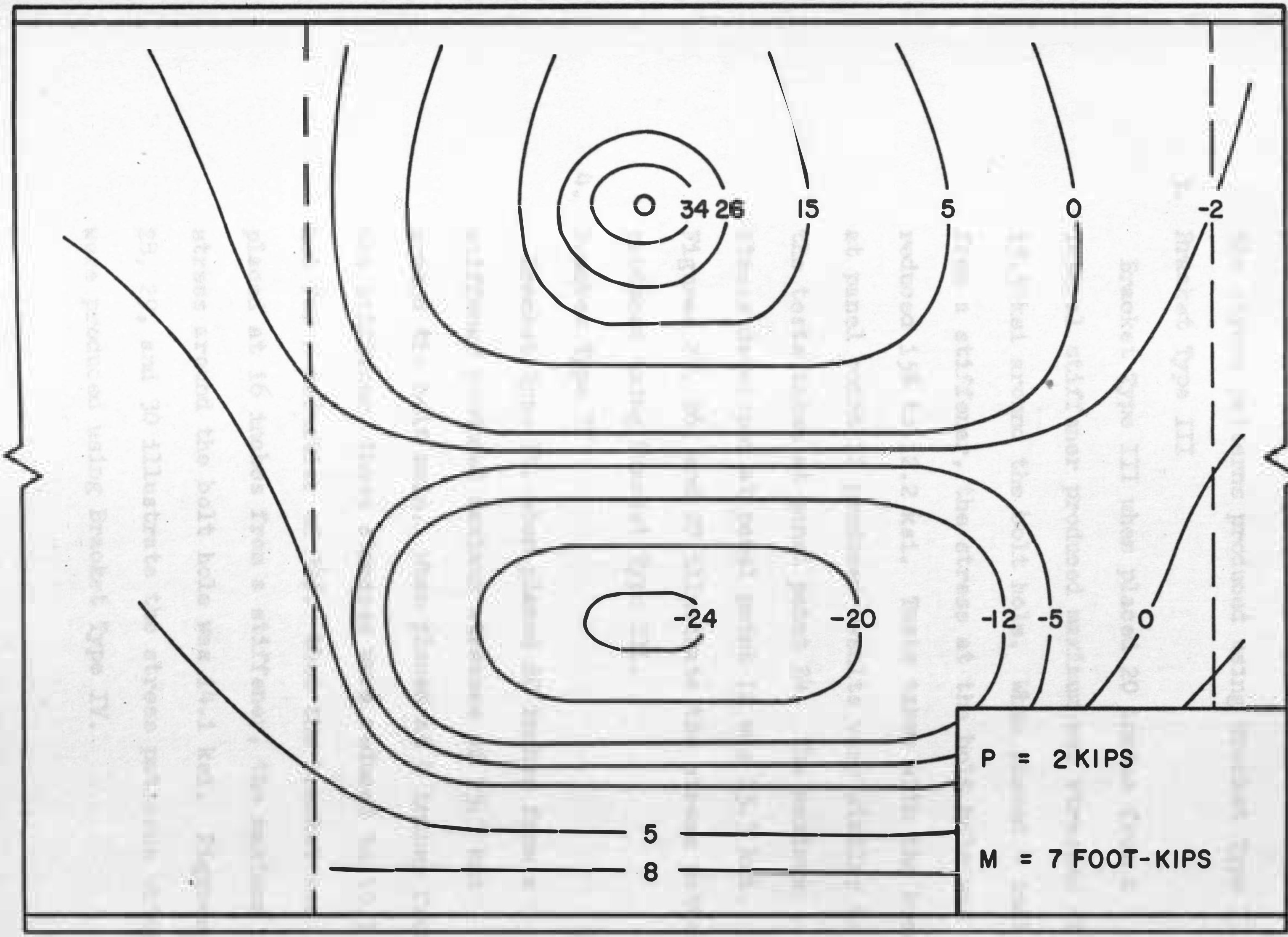

FIGURE 21. Principal Stresses (ksi) for Bracket Type I on Panel Point 12 (16 inches from nearest stiffener)

panel point 12. Figures 22, 23, and 24 illustrate the stress patterns produced using Bracket Type II.

3. Bracket Type III

Bracket Type III when placed 20 inches from a lateral stiffener produced maximum web stresses of 15.5 ksi around the bolt hole. When placed 4 inches from a stiffener, the stress at the bolt hole was reduced 15% to 12.2 ksi. Tests taken with the bracket at panel point 12 produced results very similar to the tests taken at panel point 24. The maximum stress developed at panel point 12 was 15.3 ksi. Figures 25, 26, and 27 illustrate the stress patterns produced using Bracket Type III.

4. Bracket Type IV

Bracket Type IV, when placed 20 inches from a stiffener produced maximum stresses of 15.3 ksi around the bolt hole. When placed at 4 inches from the stiffener, these stresses were reduced to 10.1 ksi for a reduction of 34%. When the bracket was placed at 16 inches from a stiffener, the maximum stress around the bolt hole was 14.1 ksi. Figures 28, 29, and 30 illustrate the stress patterns which were produced using Bracket Type IV.

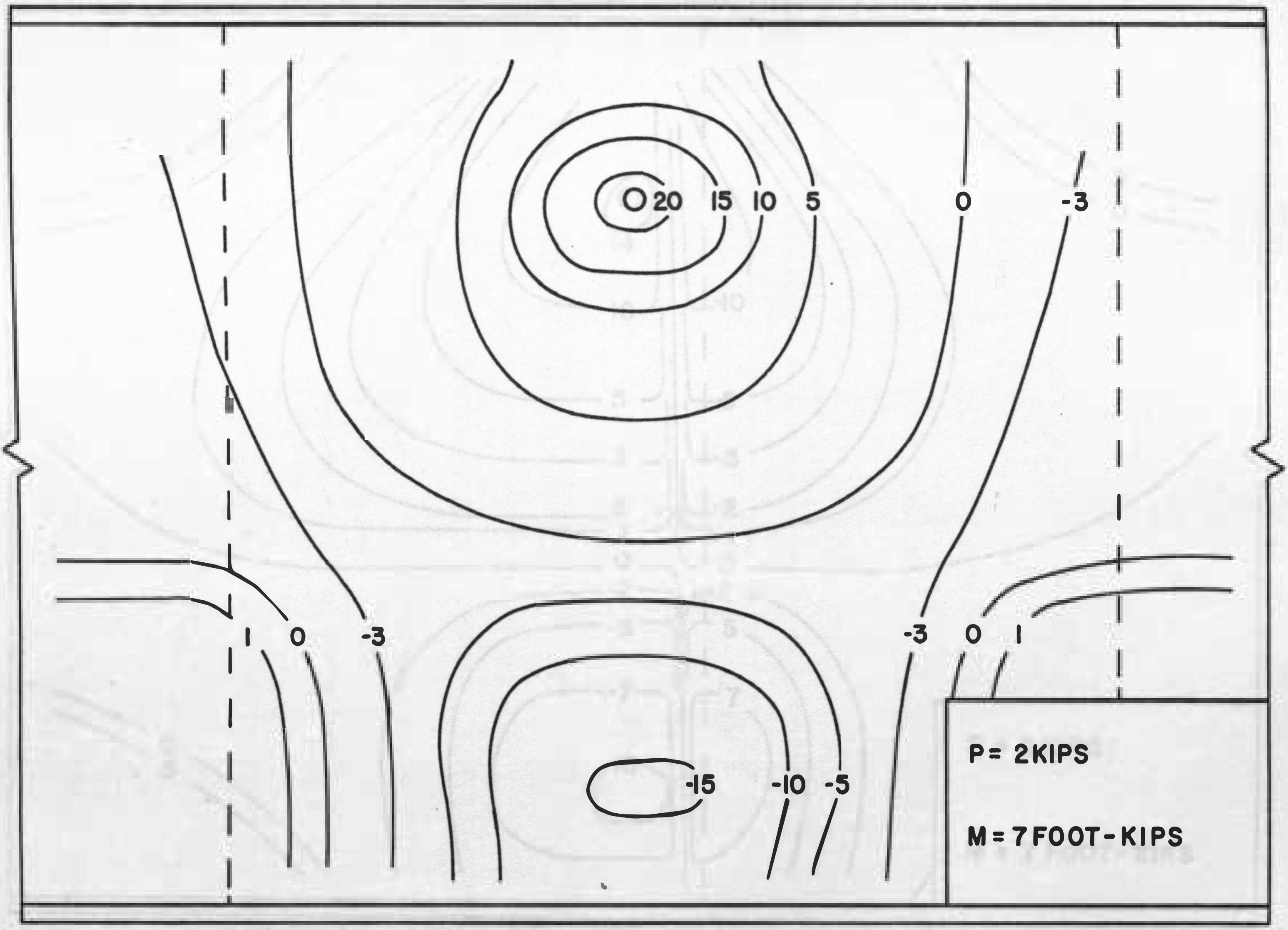

FIGURE 22. Principal Stresses (ksi) for Bracket Type II on Panel Point 24 (20 inches from nearest stiffener)

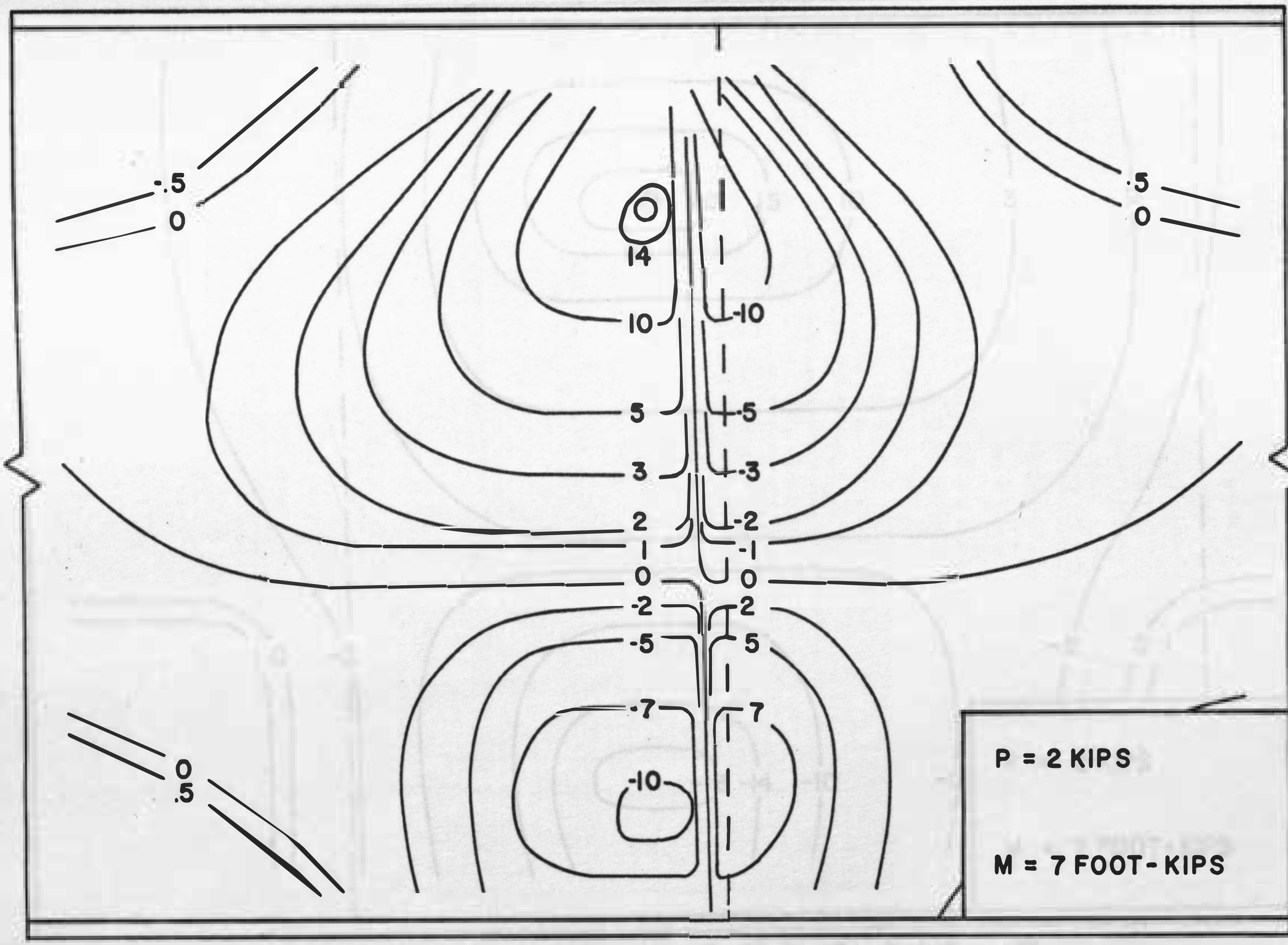

FIGURE 23. Principal Stresses (ksi) for Bracket Type II on Panel Point 18 (4 inches from nearest stiffener)

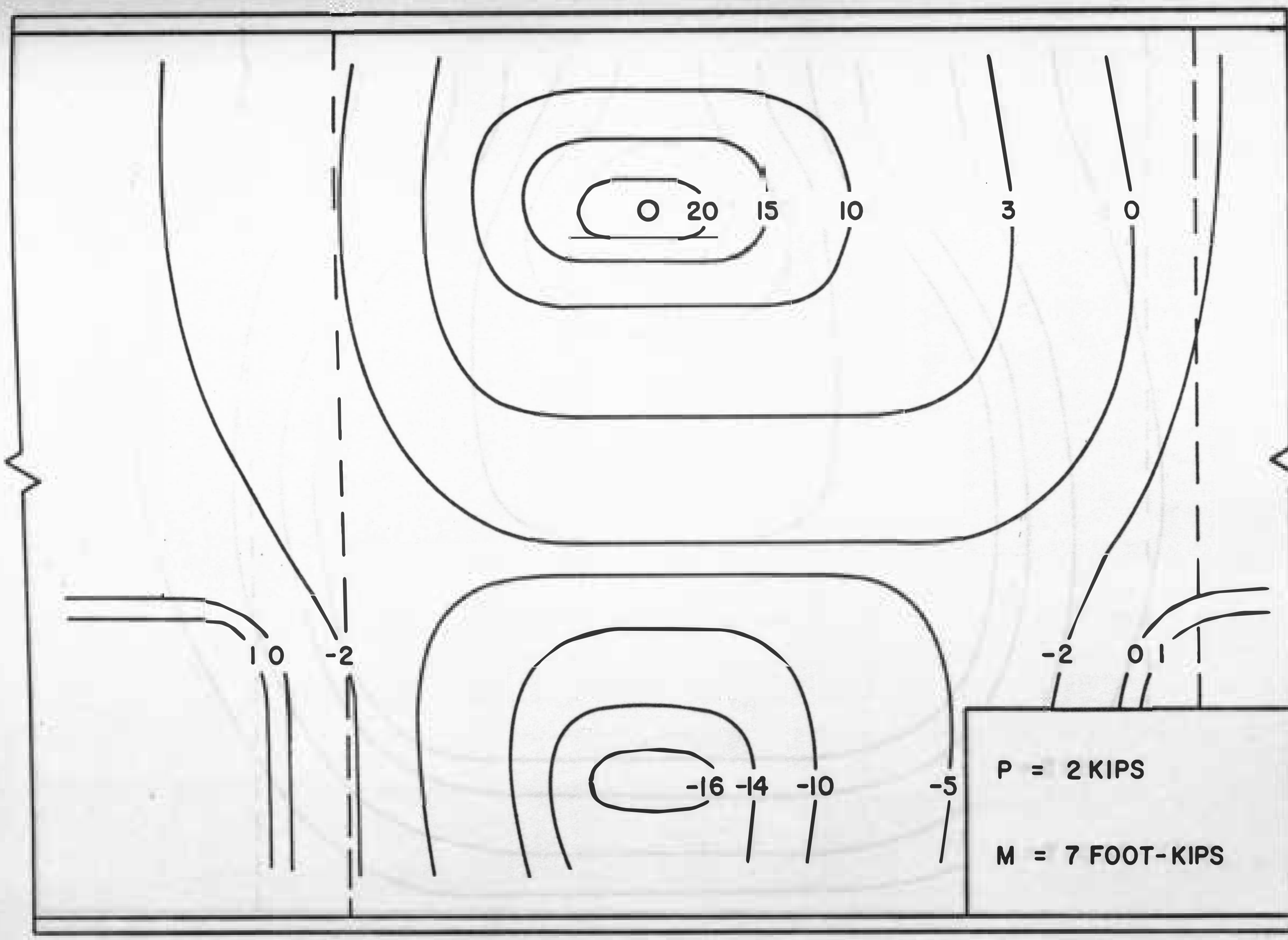

FIGURE 24. Principal Stresses (ksi) for Bracket Type II on Panel Point 12 (16 inches from nearest stiffener)

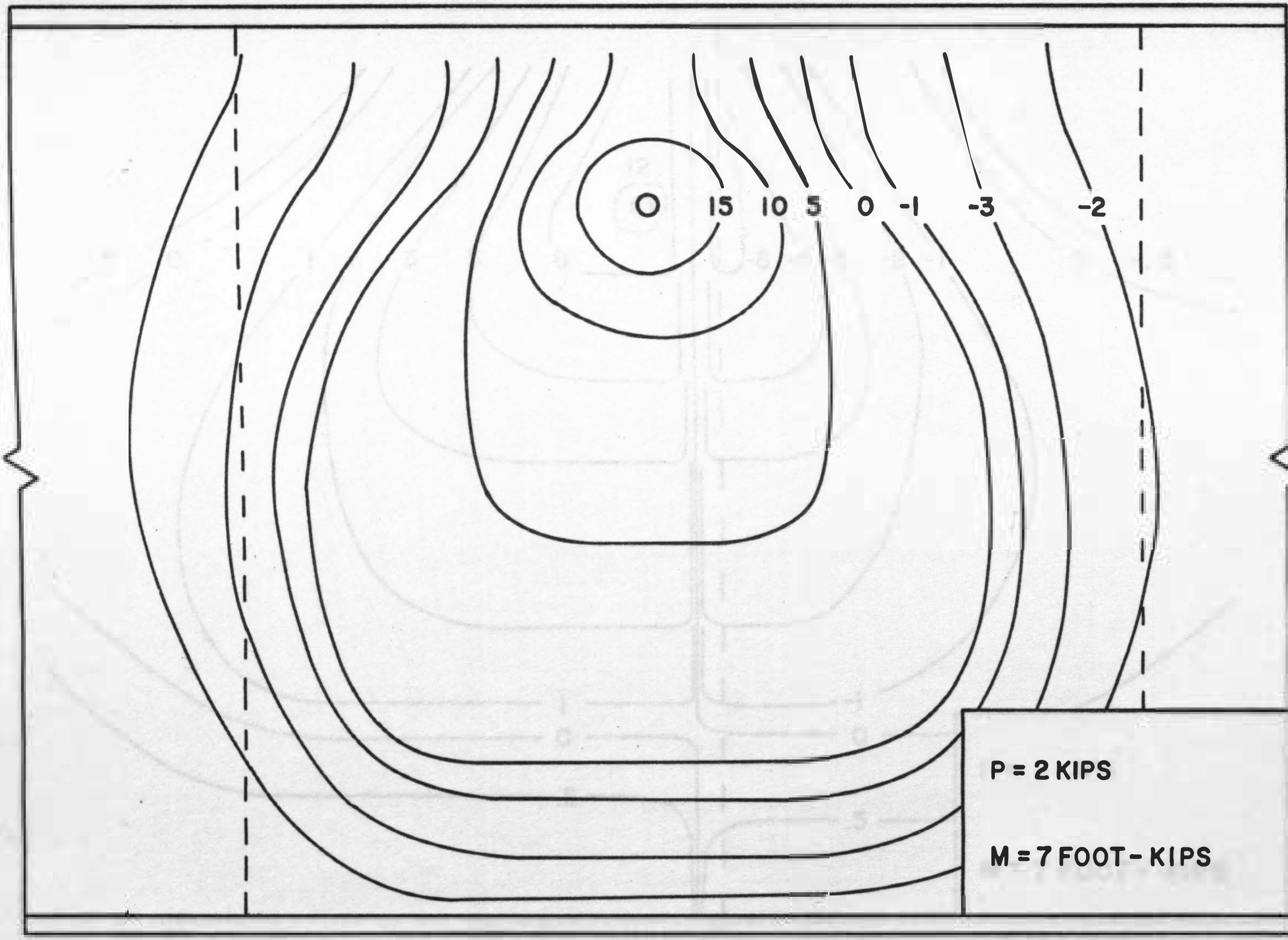

FIGURE 25. Principal Stresses (ksi) for Bracket Type III on Panel Point 24 (20 inches from nearest stiffener)

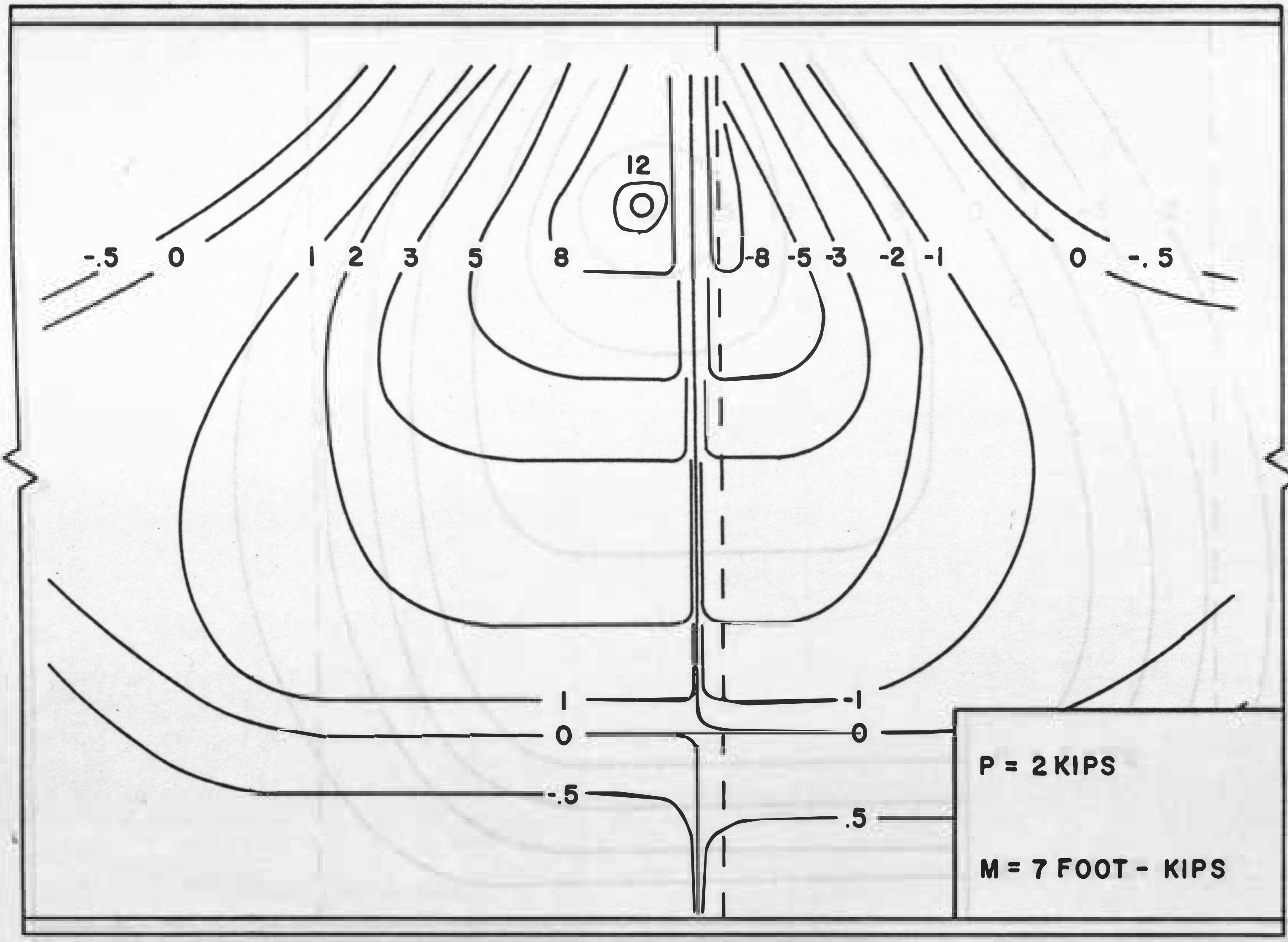

FIGURE 26. Principal Stresses (ksi) for Bracket Type III on Panel Point 18 (4 inches from nearest stiffener)

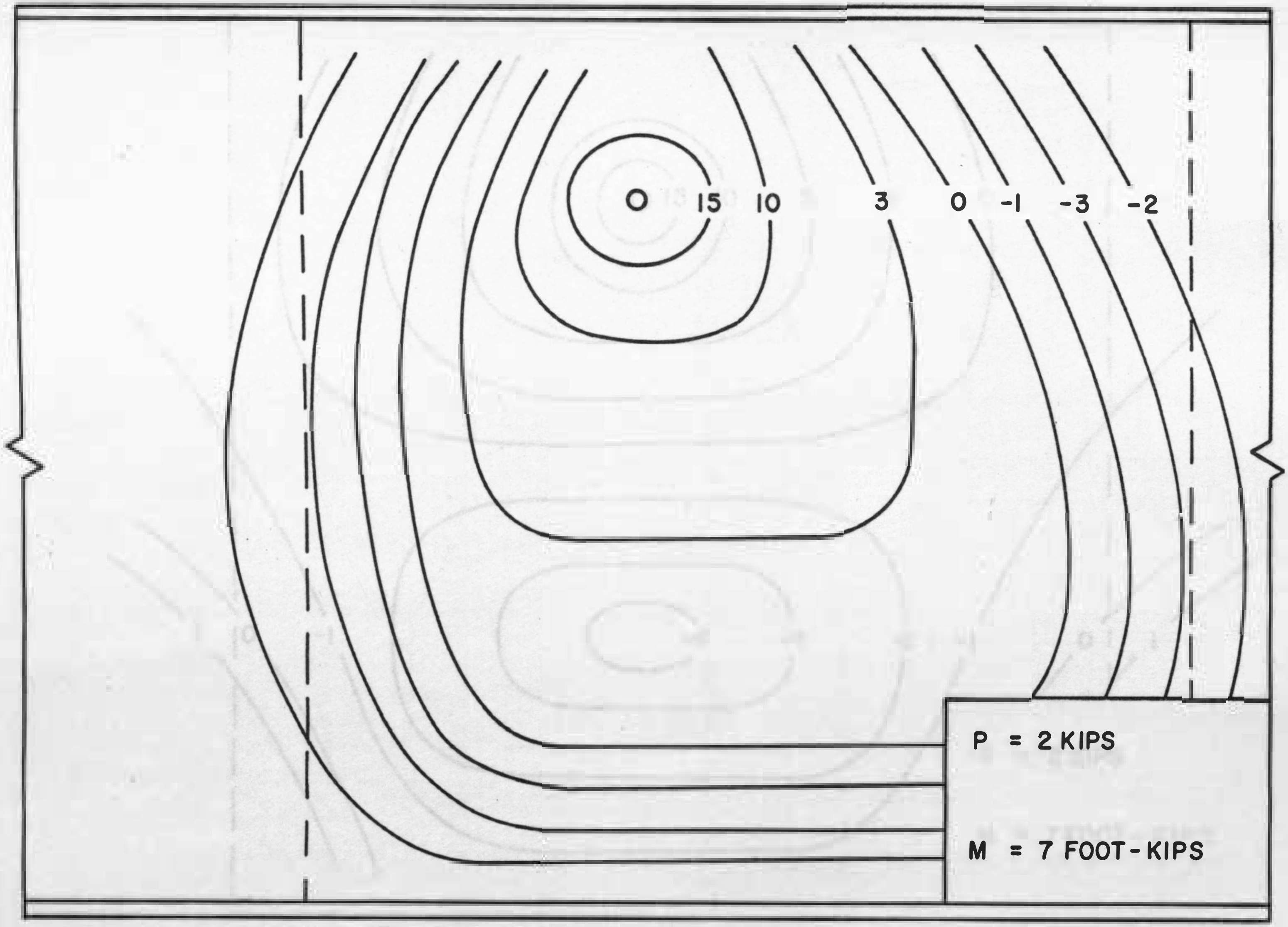

FIGURE 27. Principal Stresses (ksi) for Bracket Type III on Panel Point 12 (16 inches from nearest stiffener)

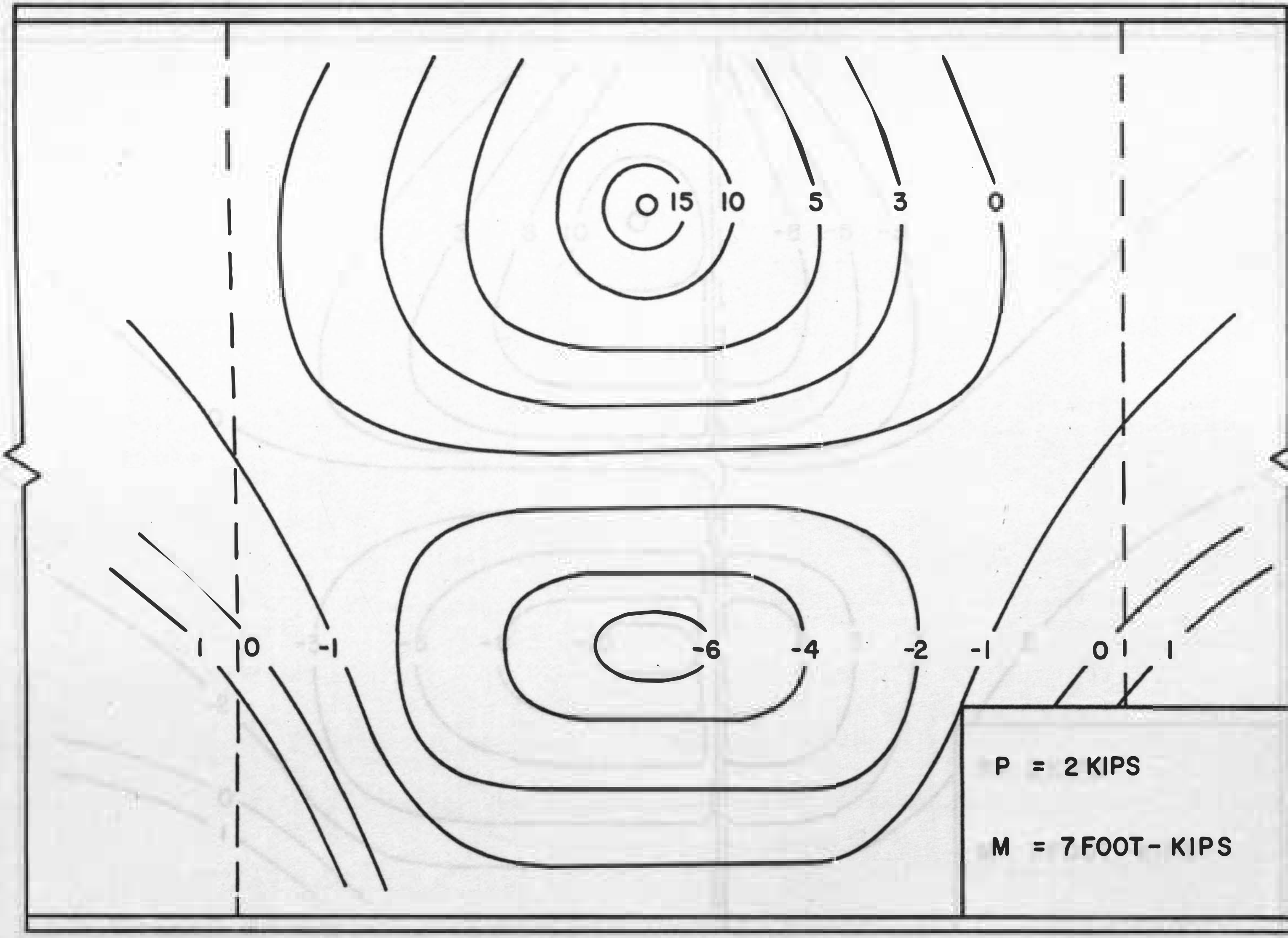

FIGURE 28. Principal Stresses (ksi) for Bracket Type IV on Panel Point 24 (20 inches from nearest stiffener)

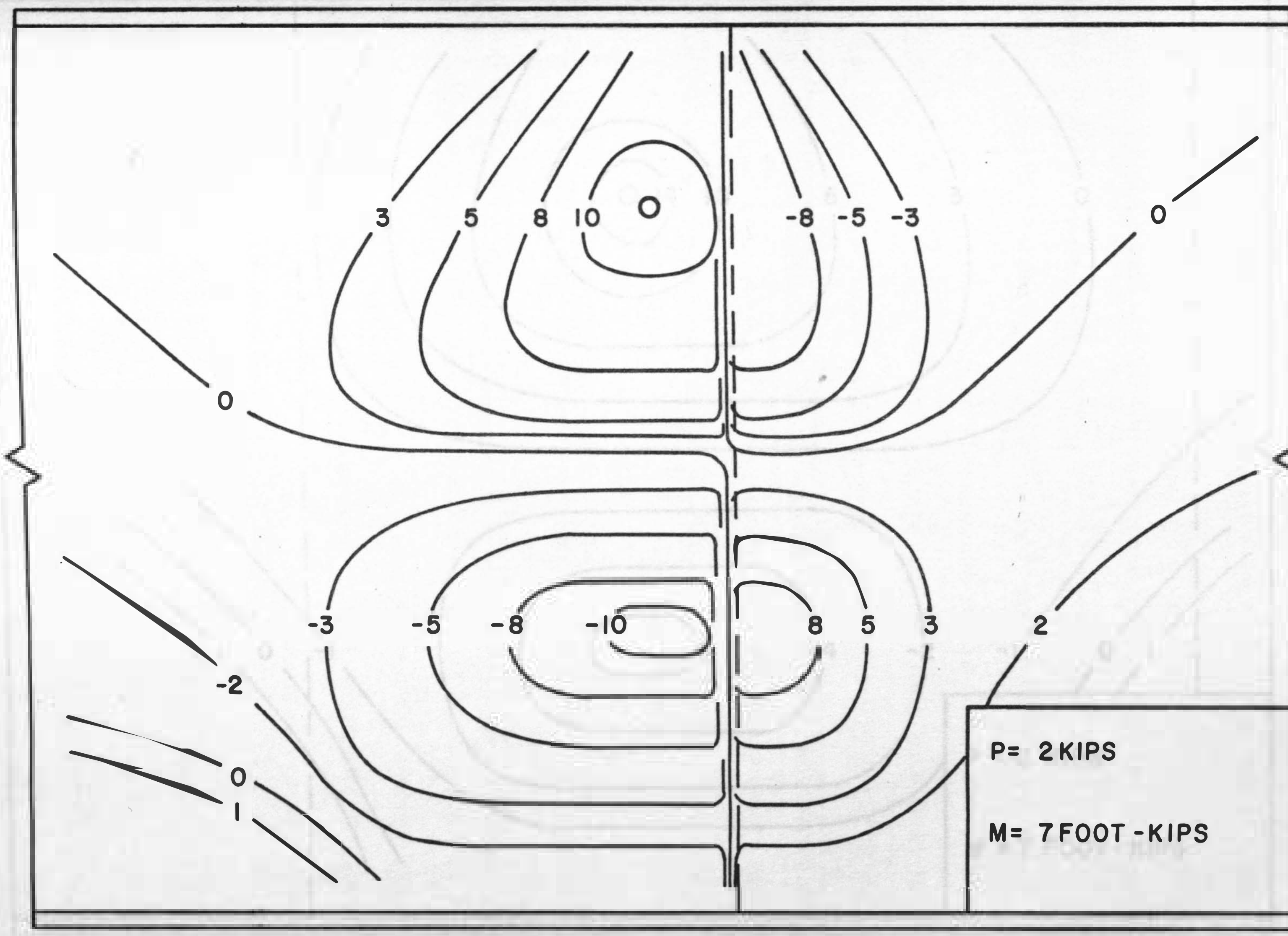

FIGURE 29. Principal Stresses (ksi) for Bracket Type IV on Panel Point 18 (4 inches from nearest stiffener)

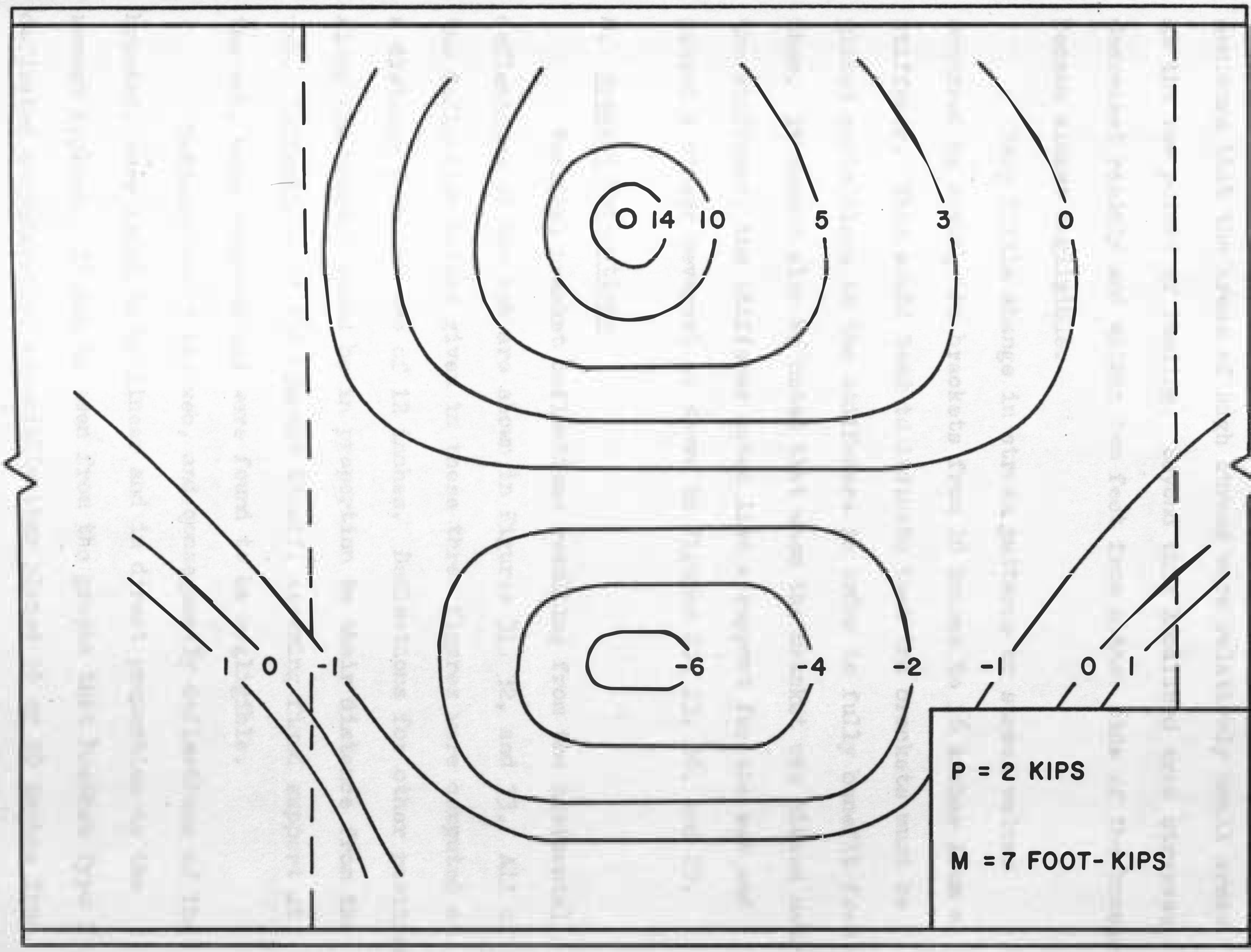

FIGURE 30. Principal Stresses (ksi) for Bracket Type IV on Panel Point 12 (16 inches from nearest stiffener)

As a general observation, it can be seen from the stress contours that the areas of high stress were relatively small areas at the two points of loading. Beyond this localized area stresses decreased rapidly and within two feet from either side of the bracket became almost negligible.

Very little change in stress patterns or stress values occurred by moving the brackets from 20 inches to 16 inches from a stiffener. This would tend to indicate that the brackets must be placed quite close to the stiffeners in order to fully benefit from them. It should also be noted that when the bracket was placed near the stiffener, the stiffener acted like a support for the web and caused a stress reversal as shown in Figures 20, 23, 26, and 29.

B. Bracket Deflections

Vertical bracket deflections resulting from the horizontal deflections of the web are shown in Figures 31, 32, and 33. All of the deflection values given in these three figures were computed at a distance from the web of 12 inches. Deflections for other positions along the bracket would be in proportion to their distance from the web. Deflections of the bracket itself, assuming fixed support at the web, were computed and were found to be negligible.

Deflections of the web, and consequently deflections of the bracket, were found to be linear and in direct proportion to the moment applied. It can be seen from the graphs that Bracket Type I deflected considerably, especially when placed 16 or 20 inches from

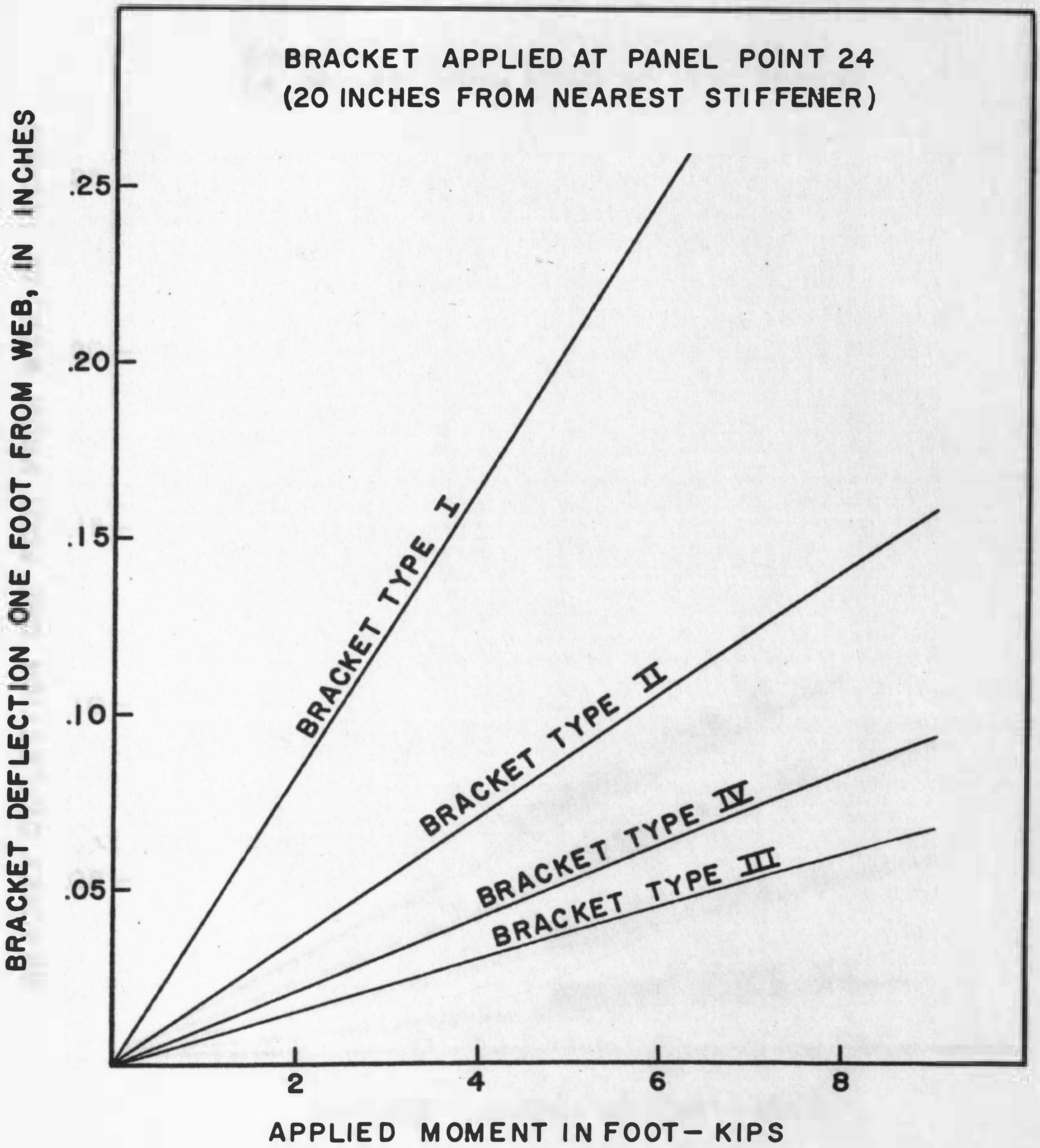

FIGURE 31. Bracket Deflections at Panel Point 24

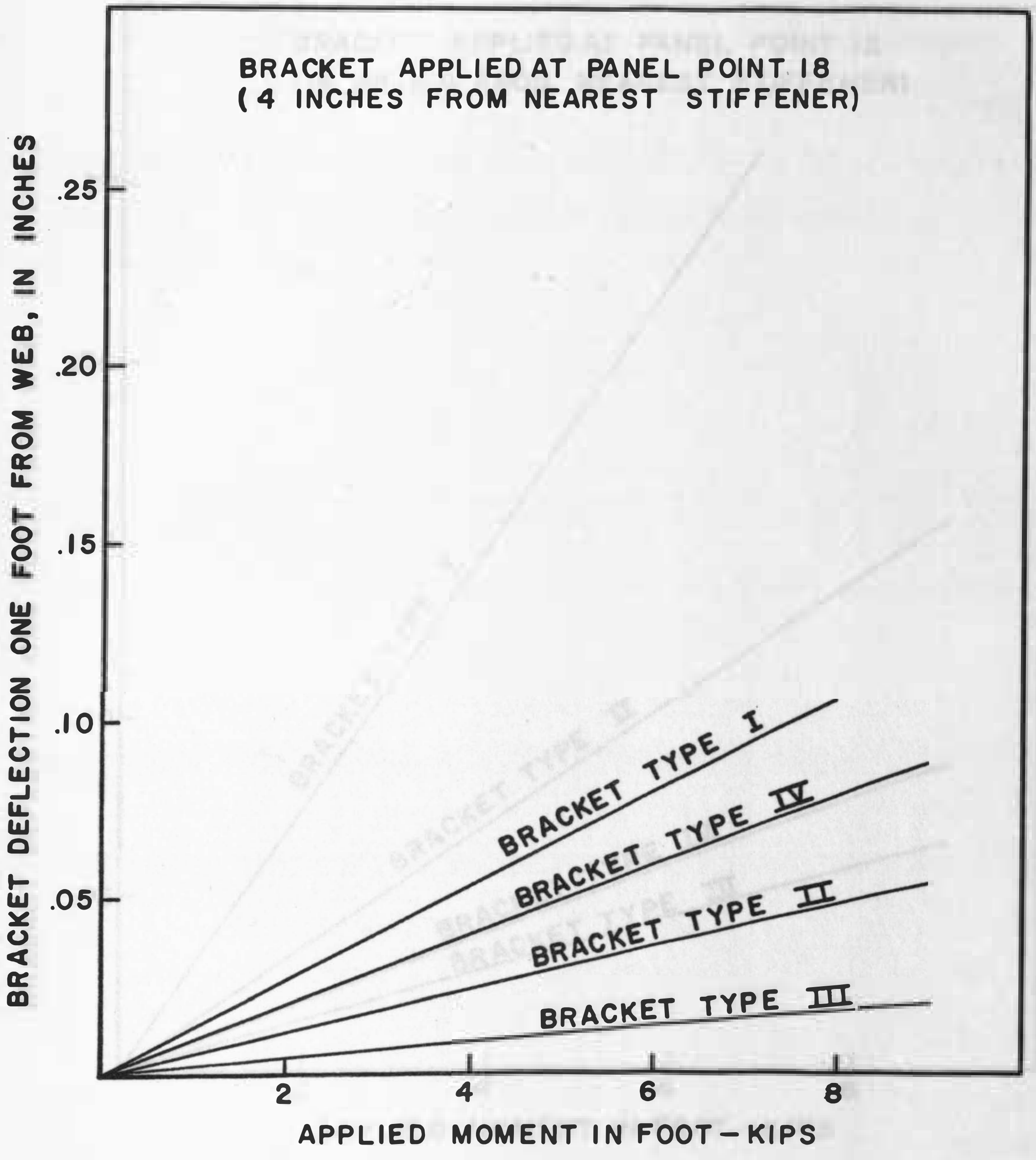

FIGURE 32. Bracket Deflections at Panel Point 18

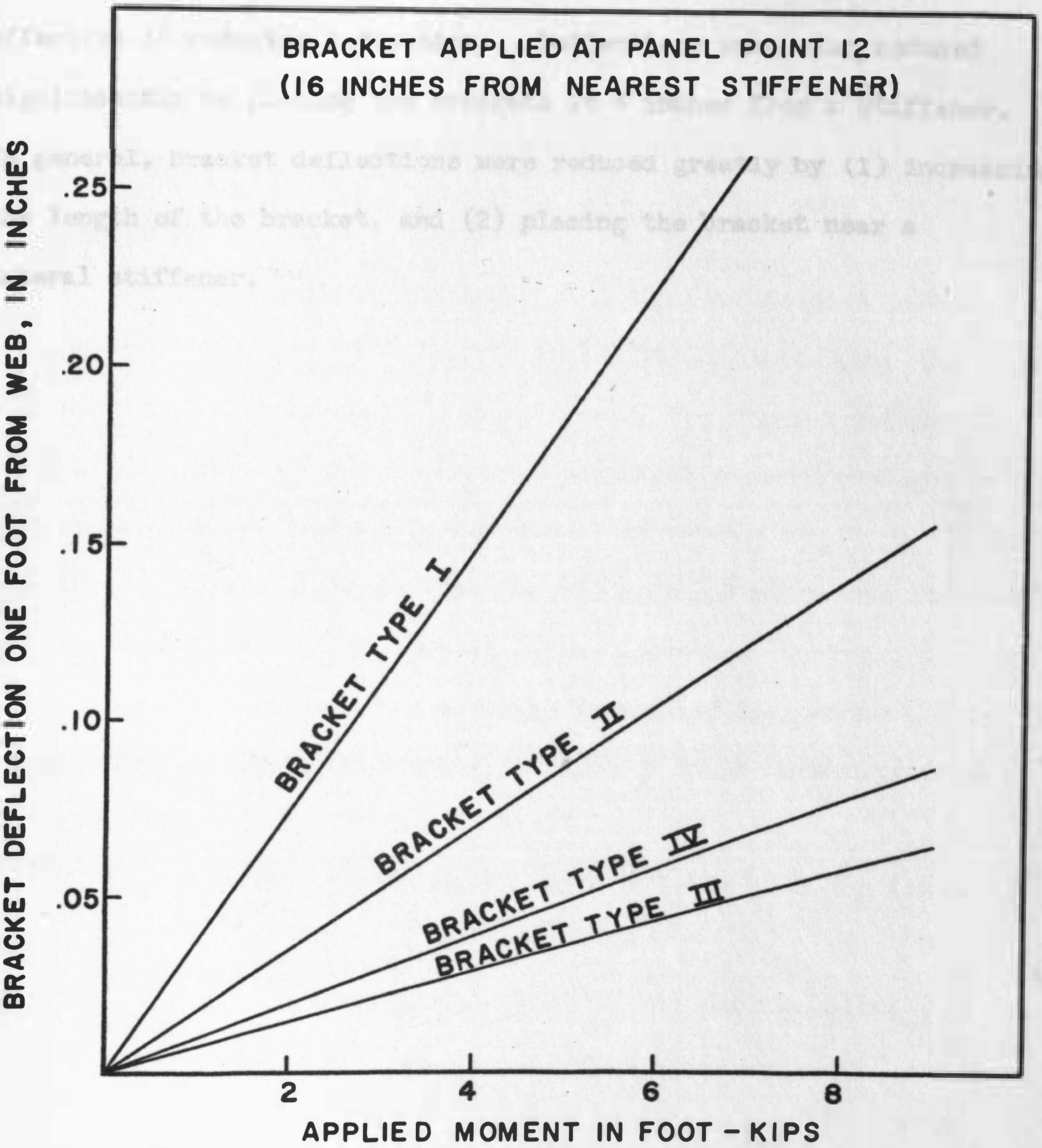

FIGURE 33. Bracket Deflections at Panel Point 12

a stiffener. Bracket Types II, III, and IV were all much more effective in reducing deflections. Deflections were also reduced significantly by placing the brackets at 4 inches from a stiffener. In general, bracket deflections were reduced greatly by (1) increasing the length of the bracket, and (2) placing the bracket near a lateral stiffener.

CHAPTER IV

SUMMARY AND CONCLUSIONS

A. Summary of Results

The results of the testing, using the four bracket types, are summarized in Table 1. In this table, the bracket type, the distance from the nearest lateral stiffener, the bracket deflection one foot from the web and the maximum principal stress developed in the web, are listed for an applied moment of seven foot-kips. Table 2 is similar to Table 1 except that the values in it have been factored to correspond to an applied moment of one foot-kip. Therefore, deflections and stresses corresponding to any applied moment can be obtained by multiplying the values in Table 2 by the moment applied in foot-kips.

The following results have been formulated from the tests in this study:

1. High web stresses occurred only over relatively small areas at the points of loading.
2. Stresses and deflections decreased significantly as the length of the bracket increased.
3. Web stresses and deflections were greatly reduced by placing the bracket within four inches of a lateral stiffener.

TABLE 1

Experimental Bracket Deflections and Web Stresses for a Moment of 7 foot-kips

Bracket Type	Distance From Nearest Stiffener (inches)	Bracket Deflection One Foot From Web (inches)	Maximum Principal Stress in Web (ksi)
(1)	(2)	(3)	(4)
I	4	0.092	16.2
I	16	0.263	34.1
I	20	0.291	35.3
II	4	0.043	14.5
II	16	0.122	20.2
II	20	0.124	20.4
III	4	0.021	12.2
III	16	0.052	15.3
III	20	0.052	15.5
IV	4	0.072	10.1
IV	16	0.073	14.1
IV	20	0.073	15.3

TABLE 2

Bracket Deflections and Web Stresses for a Unit Moment

Bracket Type (1)	Distance From Nearest Stiffener (inches) (2)	Bracket Deflection One Foot From Web (inches) (3)	Maximum Principal Stress in Web (ksi) (4)
I	4	.0131	2.31
I	16	.0376	4.87
I	20	.0416	5.04
II	4	.0061	2.07
II	16	.0174	2.89
II	20	.0177	2.91
III	4	.0030	1.74
III	16	.0074	2.19
III	20	.0074	2.21
IV	4	.0103	1.44
IV	16	.0104	2.01
IV	20	.0104	2.19

4. The backup angle, when applied to the full depth of the girder, acted similar to a lateral stiffener in reducing web stresses and deflections.

B. Conclusions

The following conclusions have been derived from the results obtained during testing:

1. It appears that the short bracket now in common use functions satisfactorily when placed within four inches of a lateral stiffener provided that the stiffener spacing is not unusually large.
2. When the stiffener spacing is large, or when for some other reason it is desired to place the brackets without regard to the stiffener spacing, web stresses can be kept within allowable limits by either of two methods:
   a. By using a bracket which occupies the full depth of the girder.
   b. By providing a backup angle for the short brackets.
3. As some bracket deflection will occur regardless of the type used, it is suggested that compensation for this deflection be achieved by adjustment of the paving machine. Table 2 can serve

as a guide to the designer for determinimg anticipated deflections.

4. Because of the advantage of using brackets which cover the full depth of the plate girder, it would be of great benefit to bridge builders to develop an adjustable bracket. Such a bracket could be made with sliding parts so as to permit its use on girders of any depth. Although the initial cost of such brackets would be higher than the cost of those in present use, it is felt that in the long run they would be cheaper because of their versatility.

C. Recommended Areas of Future Study

In this study the primary objective was to investigate the effect of bracket depth. It is suggested that an additional study be made to investigate the effect of bracket width. The effective width of a bracket could be greatly increased by welding a bearing plate or steel angle to the bottom of it. This would increase the bearing area against the web, and could very possibly reduce undesireable web stresses. The bearing area around the bolt hole could be increased by the use of a similar plate or angle. By increasing the width of a bracket, its depth could be reduced and consequently, it could be used on shallow as well as on deep girders.

BIBLIOGRAPHY

1. American Association of State Highway Officials, Standard Specifications for Highway Bridges, American Association of State Highway Officials, Washington, D. C., 1966.

2. American Institute of Steel Construction, Steel Construction Manual, American Institute of Steel Construction, New York, New York, 1963.

3. Texas Highway Department Bridge Division, Construction Inspection of Welded Steel Bridges, Texas Highway Department Bridge Division, Austin, Texas, 1956.

4. Dally, James W., and Riley, William F., Experimental Stress Analysis, McGraw-Hill, New York, New York, 1965.

5. Perry, Charles C., and Lissner, H. R., The Strain Gage Primer, McGraw-Hill, New York, New York, 1962.

APPENDIX

COMPUTER PROGRAMS

PROGRAM I

WEB STRESSES

DISK OPERATING SYSTEM/360 FORTRAN

```
   A FORTRAN IV PROGRAM FOR DETERMINING THE PRINCIPAL STRESSES
   IN WEB OF BEAM
   DIMENSIONA(500),B(500),C(500)
   DO21 I=1,500
   READ(11,1) A(I),B(I),C(I)
 1 FORMAT(F8.6,4X,F8.6,4X,F8.6)
   EL=29000.
   YJ=A(I)-C(I)
   ZK=A(I)&C(I)
   P=EL*((ZK/1.4)&(1./2.6)*(SQRT((YJ**2)&(((2.*B(I))-ZK)**2))))
   Q=EL*((ZK/1.4)-(1./2.6)*(SQRT((YJ**2)&(((2.*B(I))-ZK)**2))))
   WRITE(12,2) A(I),B(I),C(I)
 2 FORMAT(1H0,25X,2HA=,F8.6/25X,2HB=,F8.6/25X,2HC=,F8.6)
   WRITE(12,3) P,Q
 3 FORMAT(1H ,25X,2HP=,F9.2/25X,2HQ=,F9.2)
21 CONTINUE
   STOP
   END
   /*
   // EXEC LNKEDT
   // EXEC
   /*
   /+
```

PROGRAM II

BRACKET DEFLECTIONS

DISK OPERATING SYSTEM/360 FORTRAN

```
       A FORTRAN IV PROGRAM FOR DETERMINING BRACKET DEFLECTIONS
       DIMENSIONB(50),T(50),W(50)
       DO21 I=1,50
       READ(11,1) B(I),T(I),W(I)
     1 FORMAT(F5.3,4X,F5.3,4X,F5.2)
       E=12.0
       YJ=B(I)&T(I)
       ZK=1.0/W(I)
       D=E*YJ*ZK
       WRITE(12,1) B(I),T(I),W(I)
     2 FORMAT(1HO,25X,2HB=,F5.3/25X,2HT=,F5.3/25X,2HW=,F5.2)
       WRITE(12,3) D
     3 FORMAT(1H ,25X,2HD=,F5.3)
    21 CONTINUE
       STOP
       END
       /*
       // EXEC LNKEDT
       // EXEC
       /*
       /+
```