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Field Testing of Bridge Girder Webs Subjected to Horizontal Loads

Duane Darrrel Boice

FIELD TESTING OF BRIDGE

GIRDER WEBS SUBJECTED

TO HORIZONTAL LOADS

BY

DUANE DARREL BOICE

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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

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DDB

DEDICATION

This thesis is dedicated to the author's wife, Lois, whose loyal encouragement and sacrifice has made this project possible. To her the author is truly indebted.

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CHAPTER I

INTRODUCTION

A. General

In construction of plate girder bridges, when no ground supported falsework is used, metal brackets bolted to the web of the exterior girders are used to support construction loads. The loads include the weight of the falsework, the weight of the freshly poured concrete of the overhanging portion of the bridge deck, and the weight of the finishing machine. Figure 1 shows a cross section of the bridge with the bracket mounted on the exterior girder and formwork in place.

The brackets transmit to the plate girder web a vertical shear force and a couple. The couple applied to the girder web causes both significantly high stresses and deflections which, in most cases, have not been considered in designing the girder. Since the deflection allows rotation of the bracket, the overhanging portion of the deck is lowered causing a corresponding lowering of the finishing machine. The result is an undesirable decrease in deck thickness over the girders. Figure 2 shows construction brackets mounted on a bridge girder before formwork was in place.

At the present time in South Dakota, it is common practice to place the brackets at a distance of six inches or less from the

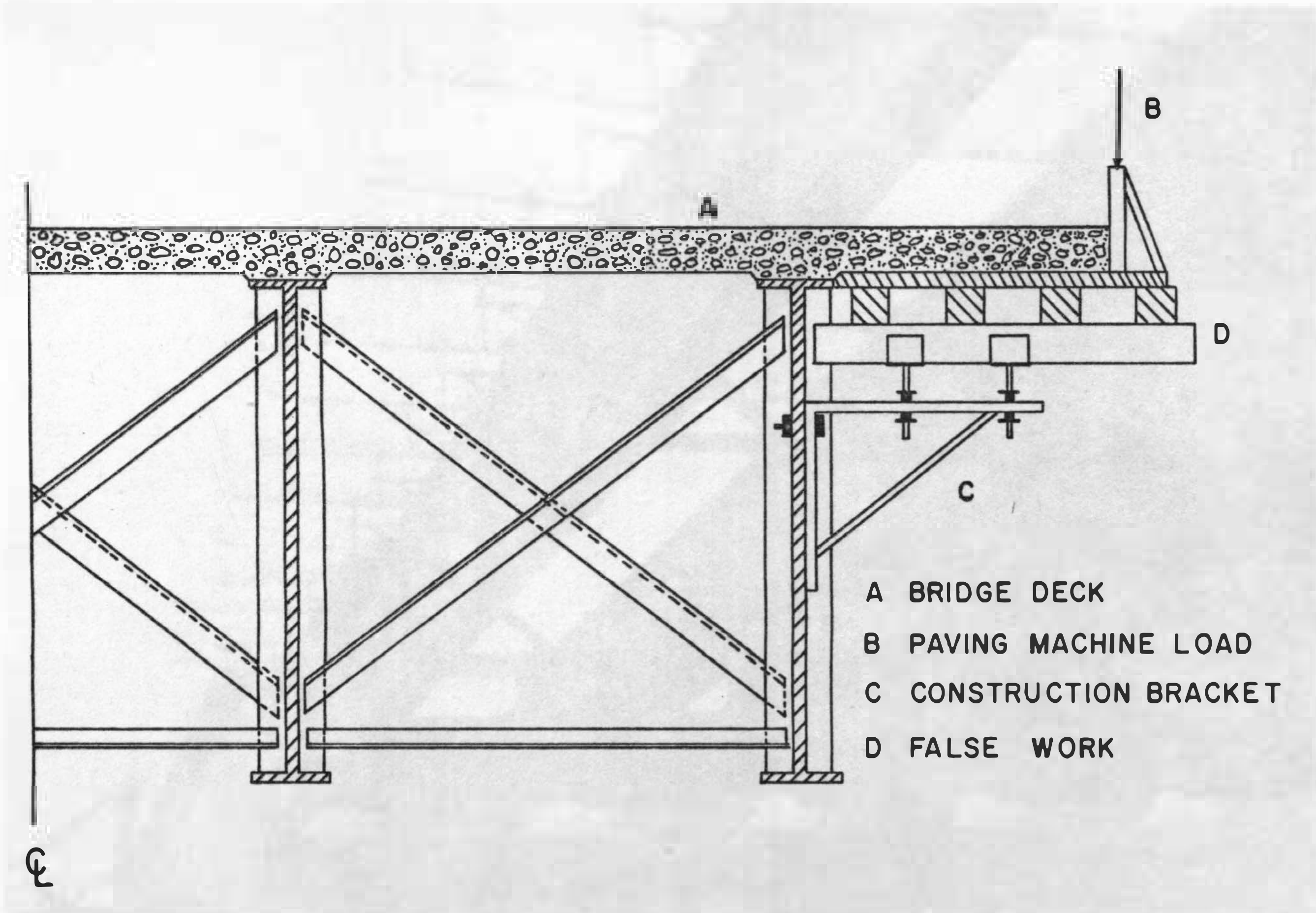


FIGURE 1. Cross Section View of Bridge with Construction Bracket Mounted on External Girder

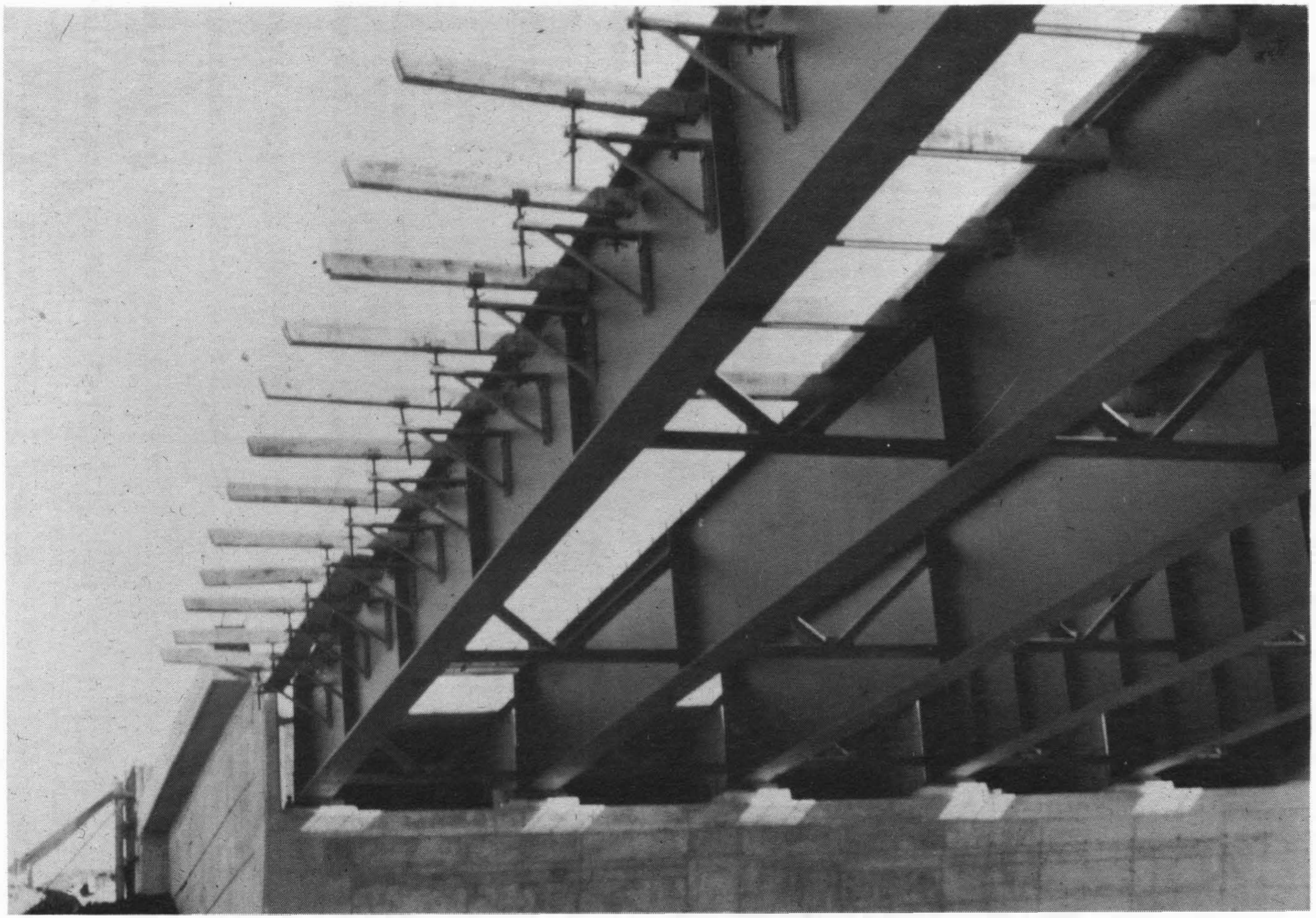


FIGURE 2. Construction Brackets Mounted on Test Bridge Prior to Forming

nearest stiffener. Since stiffener spacing cannot be standardized economically across the entire span, a bracket to be placed without regard to stiffener spacing is needed. If brackets could be placed at standard intervals, the necessary formwork could be standardized and used on different bridges. Such standardization would contribute a great deal to economy in bridge construction. The newly recommended Load Factor Method of design which would eliminate the lateral stiffeners in areas of low shear dictates a need for a bracket that can be used without regard to stiffener spacing. (1)*

Therefore stresses and deflections relative to bracket depth and distance from the nearest stiffener are being studied to determine if certain brackets could be placed without restriction to distance from the nearest stiffener.

B. Historical Background

Metal brackets have been used in plate girder bridge construction for many years with little known of their effect on the girder web. The first use involved brackets built specifically for a certain depth girder with the fastening bolt near the top flange and the bracket extending to the bottom flange. Since the girders were generally wide flange sections of standard depth, any bridge with a certain section could be built using one size bracket.

With the introduction and acceptance of plate girders in bridge design, economy dictated a need for a variable web.

*Numbers in parentheses refer to entries in the Bibliography.

It was impractical to use a special bracket for each web depth at different positions along the length of the bridge. Therefore it became common practice to use one standard bracket which was short enough to fit the girder at its shallowest point. However such brackets were not designed to extend to the bottom flange in places where the web was very deep. Such shallow brackets, when placed on a deep web, cause excessive stresses and deflections. A typical example of shallow brackets mounted on a deep web is shown in Figure 3.

This problem has been approached differently by different states and contractors. Some Highway Departments do not allow the use of these brackets, relying instead on needle beam or ground supported falsework. (2) A bracket using two bolts to fasten it to the girder web is used by some contractors. Others use a bracket which is adjusted by means of a bolt in the bottom of the bracket which bears directly on the girder web. This is probably the most critical since it applies a point load at the bottom of the bracket, causing very high stresses. The type of bracket used has been left pretty much to the contractor's discretion and many different types are in use at the present time.

The problem of analysing stresses and deflections in plate girder webs due to horizontal loads has been approached many different ways. All methods of solution to this date are both tedious and subject to error because of the large number of variables and assumptions involved.

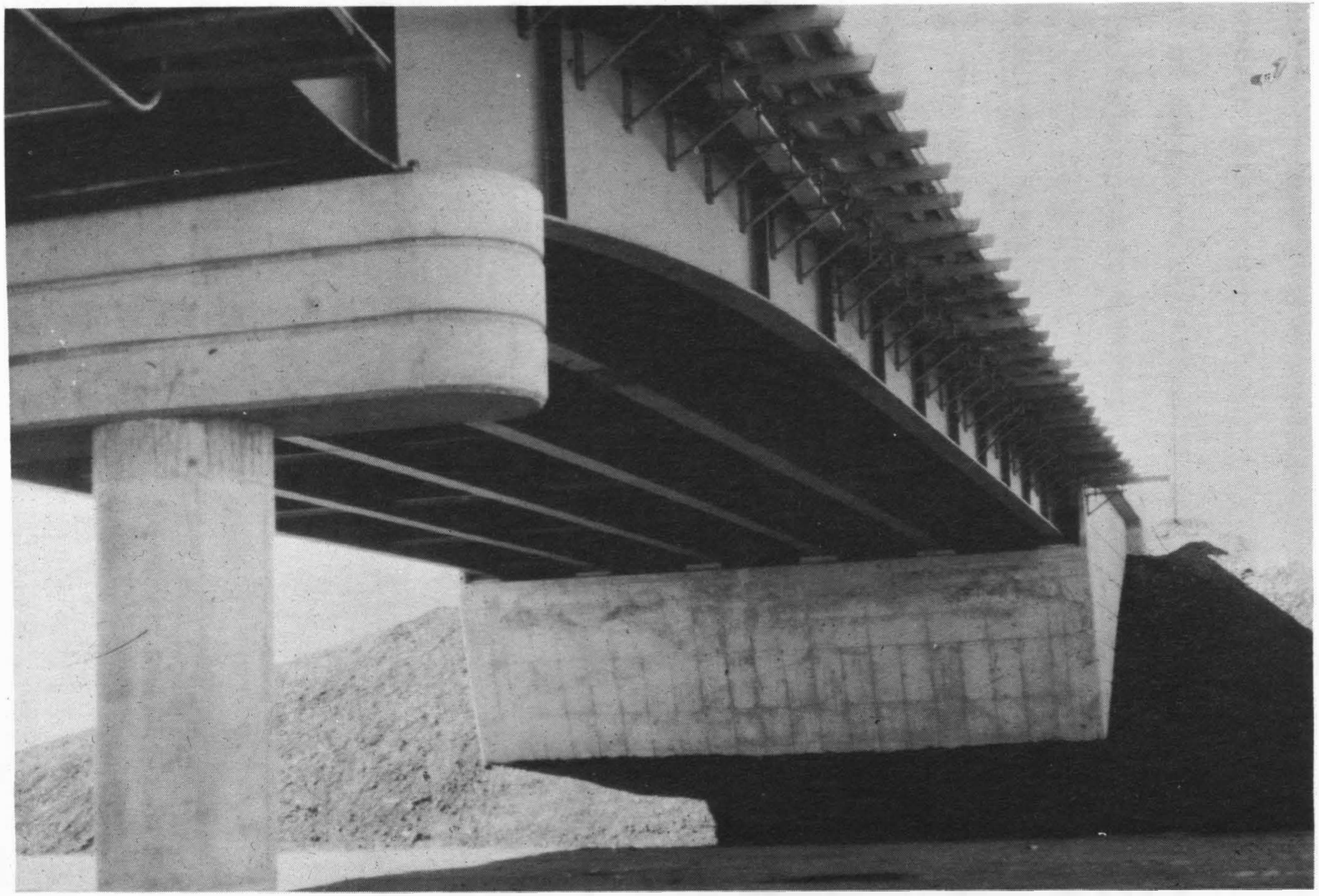


FIGURE 3. Shallow Construction Bracket Mounted on Deep Web

Since the high stresses caused by the brackets are local, the South Dakota Department of Highways has restricted placement of the bracket with respect to the nearest vertical stiffener. The amount of damage to the web of the girder is not known and has never been evaluated experimentally.

C. Object and Scope of Investigation

The objective of this investigation was to determine experimentally the web stresses and deflections which occur as a result of construction loads applied by the temporary brackets. The stresses and deflections obtained will be compared with laboratory tests to aid in accurately determining the behavior of a girder web when subjected to horizontal loading.

The study includes an investigation of four brackets; two which are now in common use and two that may find future application. Both bracket depth and position relative to the nearest stiffener were varied in this study.

CHAPTER II

TESTING PROGRAM

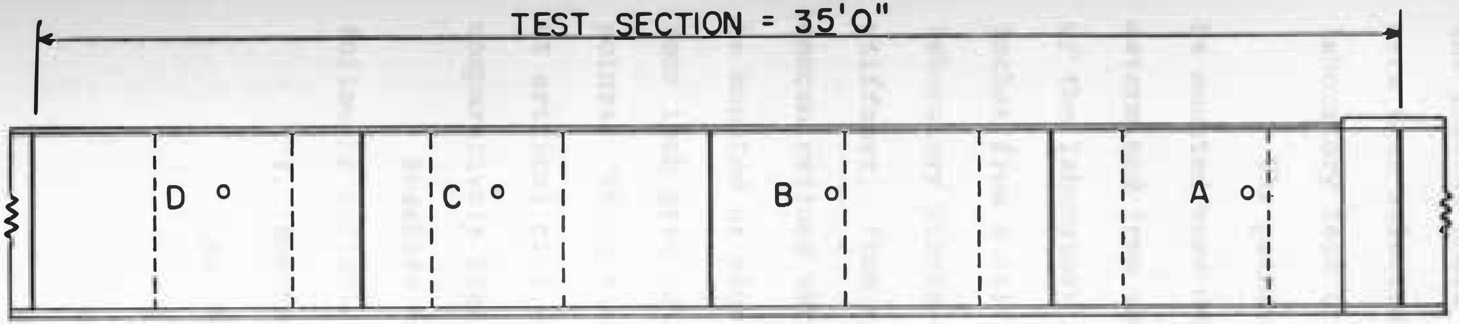
A. Description of Test Bridge

The bridge tested was I29-5(10)134 which served as an overpass for a two lane county road crossing Interstate Highway I29 two miles north of the city limits of Brookings, South Dakota. It was a two span 210 foot continuous composite girder bridge with a 32 foot road way having a fixed support at the center bent with rocker supports at both abutments. The girders were fabricated in three sections and field spliced using high strength steel bolts. The center haunched section had a variable web depth ranging from 51 inches at the field splice to 90 inches at the center bent. The end sections had a constant web depth of 51 inches. The girder dimensions are shown in Figure 4. The four girders were spaced nine foot two inches on centers. The diaphragms were spaced 17 foot six inches on centers with stiffeners spaced three foot six inches on centers.

The bridge deck was 7 3/4 inch reinforced concrete with a three foot seven inch overhang over both exterior girders. The deck was continuous with expansion joints at both abutments.

B. Preparation for Testing

It was determined that testing should be done on a section of positive moment to allow comparison with laboratory testing done in



TOP FLANGE = 12X5/8 inch PL

BOTTOM FLANGE = 14X1 1/4 inch PL

WEB = 51X5/16 inch PL

STIFFENERS = 5X3/16 inch PL

FIGURE 4. Plate Girder Details for Test Section

the initial phase of the project. Reference positions on the girder were then selected to locate the test brackets much the same as the laboratory test brackets relative to the nearest diaphragm. (3)

The points around each test bracket where strain gages were to be mounted were determined from a careful study of stress contours determined from the laboratory tests. Figure 5 is a typical example of the laboratory stress contours when a bracket is mounted four inches from a stiffener, while Figure 6 is a typical example of the laboratory stress contours when a bracket is mounted 20 inches from a stiffener. From these stress contours it was noted that stress concentrations were localized and therefore the strain gages should be mounted as closely to the brackets as possible. A four inch by four inch grid pattern was chosen with bolt holes as the reference points. Three element rectangular rosette strain gages were mounted at critical grid points where stresses and deflections are comparatively high with respect to other points.

Brackets to be tested were selected according to the following criteria.

1. Brackets being used at the present time.

   a. Bracket A was a standard metal construction bracket in common use by contractors at the present time. It was placed six inches from the nearest stiffener which corresponds with the present recommendations of the South Dakota Department of Highways for

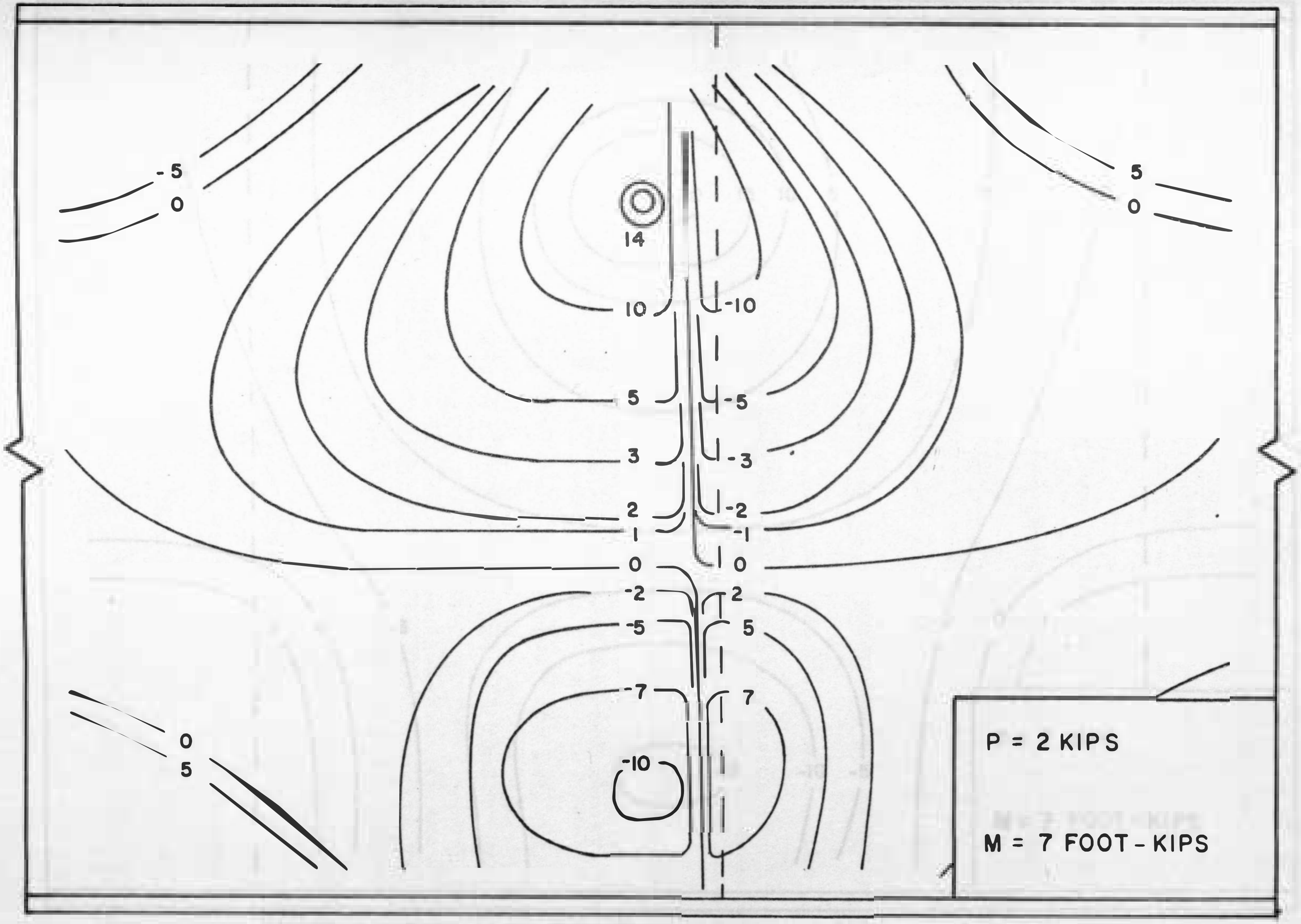


FIGURE 5. Laboratory Test Stress Contours for a Bracket Mounted 4 inches from a Stiffener

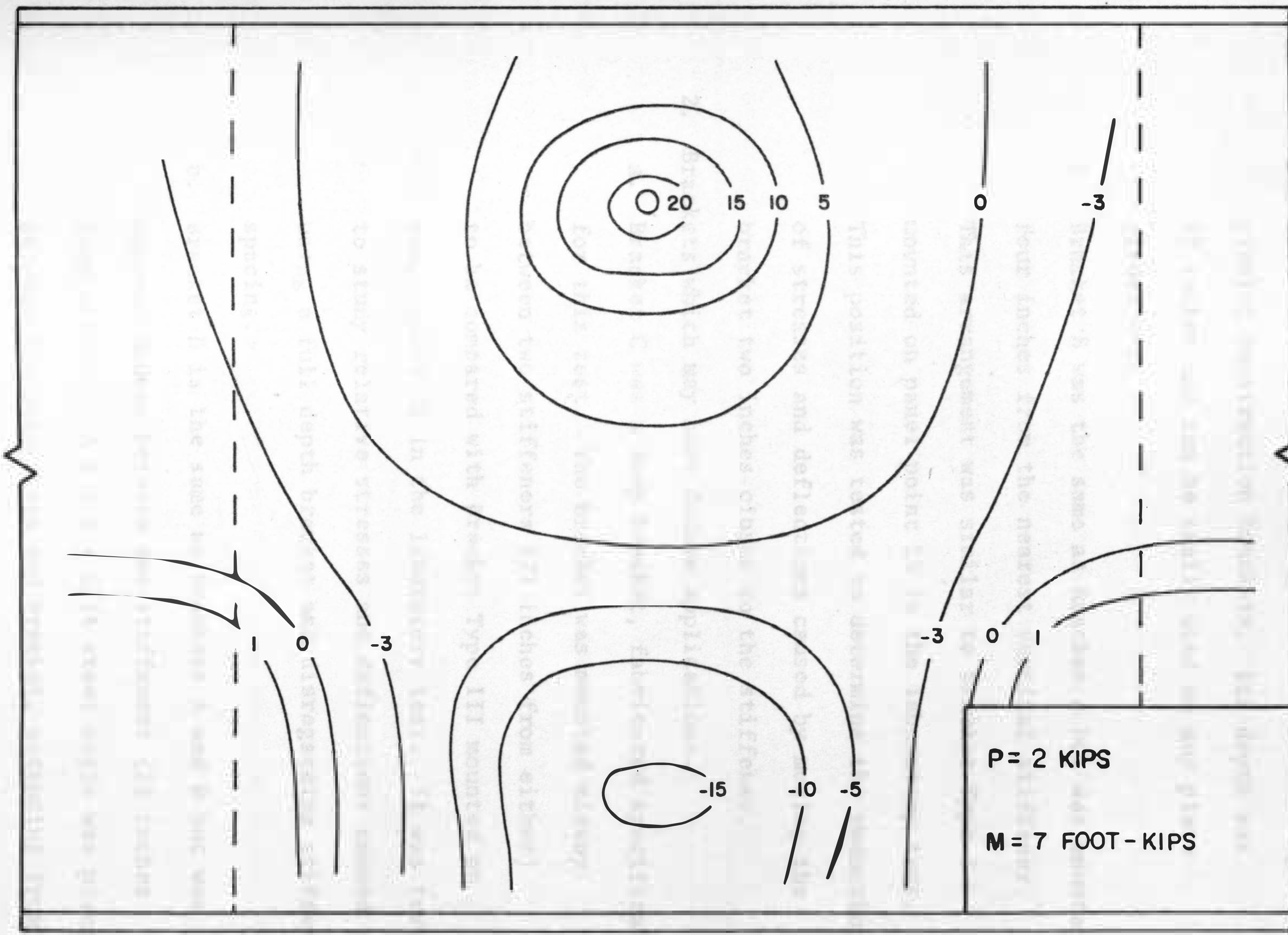


FIGURE 6. Laboratory Test Stress Contours for a Bracket Mounted 20 inches from a Stiffener

placing construction brackets. Its depth was 17 inches and can be easily used on any plate girder web.

b. Bracket B was the same as Bracket A but was mounted four inches from the nearest vertical stiffener. This arrangement was similar to Bracket Type I mounted on panel point 18 in the laboratory test. This position was tested to determine the reduction of stresses and deflections caused by moving the bracket two inches closer to the stiffener.

2. Brackets which may have future applications.

a. Bracket C was a deep bracket, fabricated specifically for this test. The bracket was mounted midway between two stiffeners (21 inches from either) to be compared with Bracket Type III mounted on panel point 24 in the laboratory test. It was tested to study relative stresses and deflections caused by using a full depth bracket and disregarding stiffener spacing.

b. Bracket D is the same as Brackets A and B but was mounted midway between two stiffeners (21 inches from either). A 5 x 5 x 5/16 steel angle was placed between the girder web and bracket, extending from the bolt hole to the bottom flange. A similar steel angle was placed behind the web extending from the bolt hole to the top flange. Laboratory tests showed that these backup angles reduced stresses and deflections much the same as a conventional stiffener. It was felt that such backup angles would function as temporary stiffeners during construction of the bridge. Bracket D may find application in bridge construction where no stiffeners are to be used or where brackets are to be placed with no regard to stiffeners.

Details of Brackets A, B, C, and D are shown in Figures 7, 8, and 9. The preselected test points were marked into grids and the relative position of each rosette was determined. The surfaces were prepared using an industrial sander to remove the mill scale and pits. Final preparation included hand sanding the surfaces with emery cloth to remove grinding marks and a thorough cleaning with carbon tetrachloride to remove any dust and grease.

SR-4 strain gage rosettes were mounted at the established grid points using quick drying cement. Figure 10 shows that gage points at each bracket position. The gages were tested for continuity and inspected for possible air bubbles trapped beneath the gage after 24 hours of curing.

A double lead wiring system was used to compensate for temperature changes in the wires with a dummy gage used to compensate for temperature changes of the gages themselves during the testing. Since each rosette required six wires and there were four rosettes on

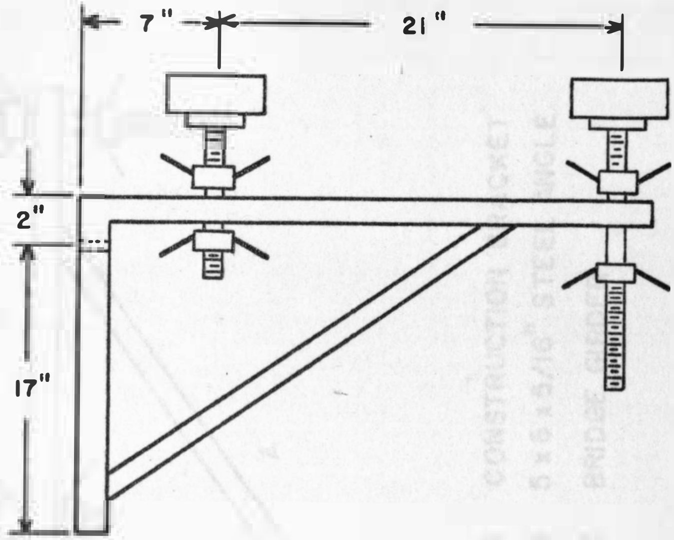


FIGURE 7. Dimensions of Brackets A and B

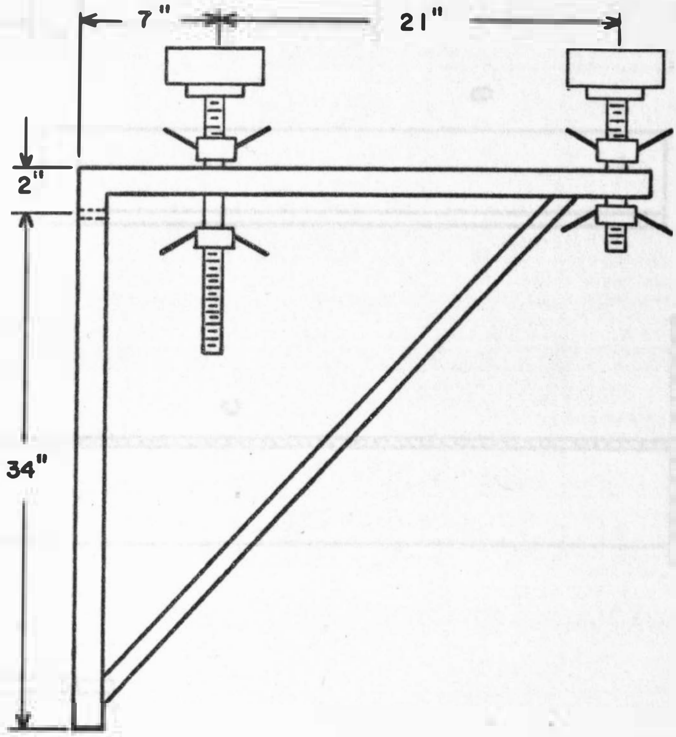


FIGURE 8. Dimensions of Bracket C

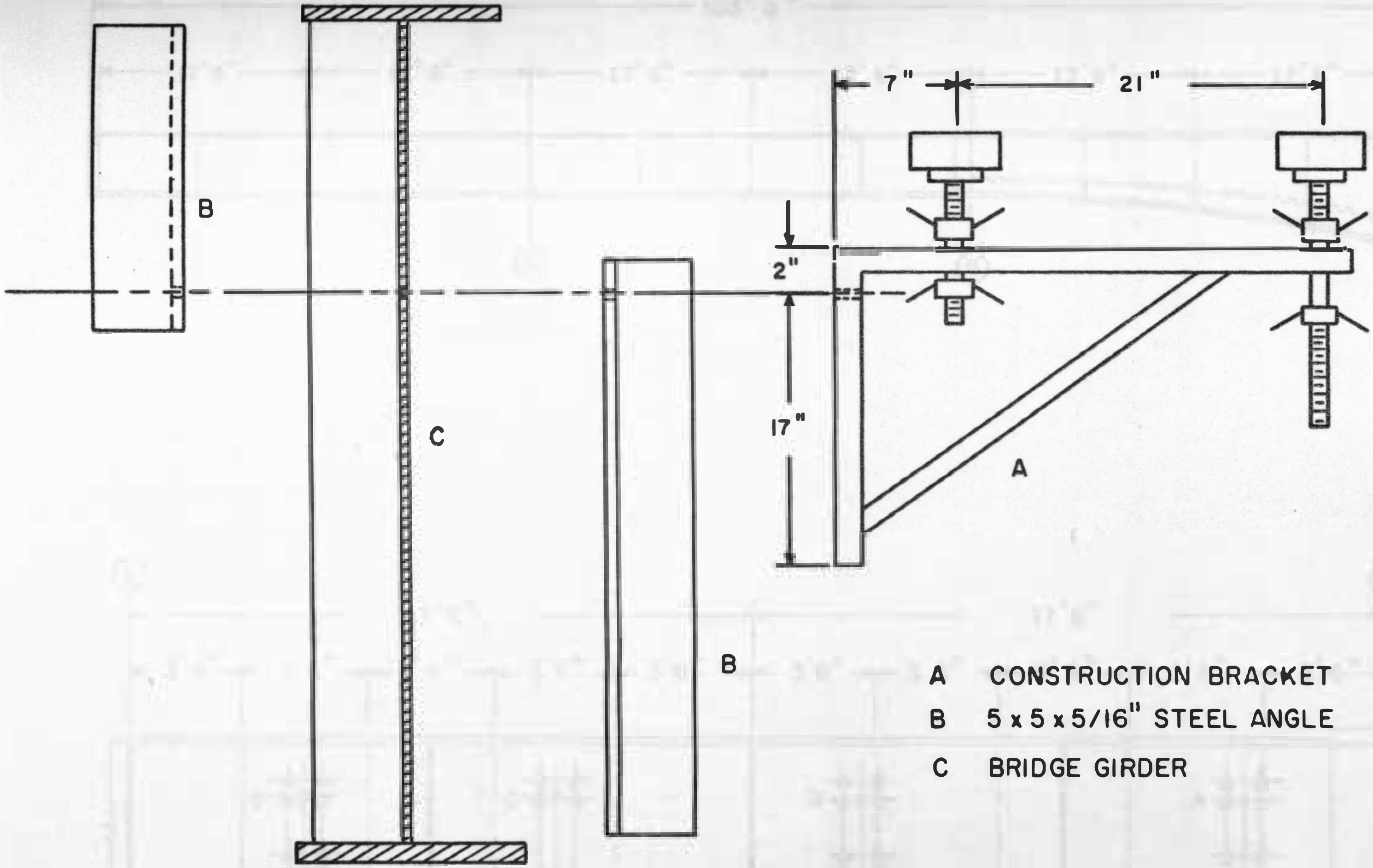


FIGURE 9. Dimensions of Bracket D

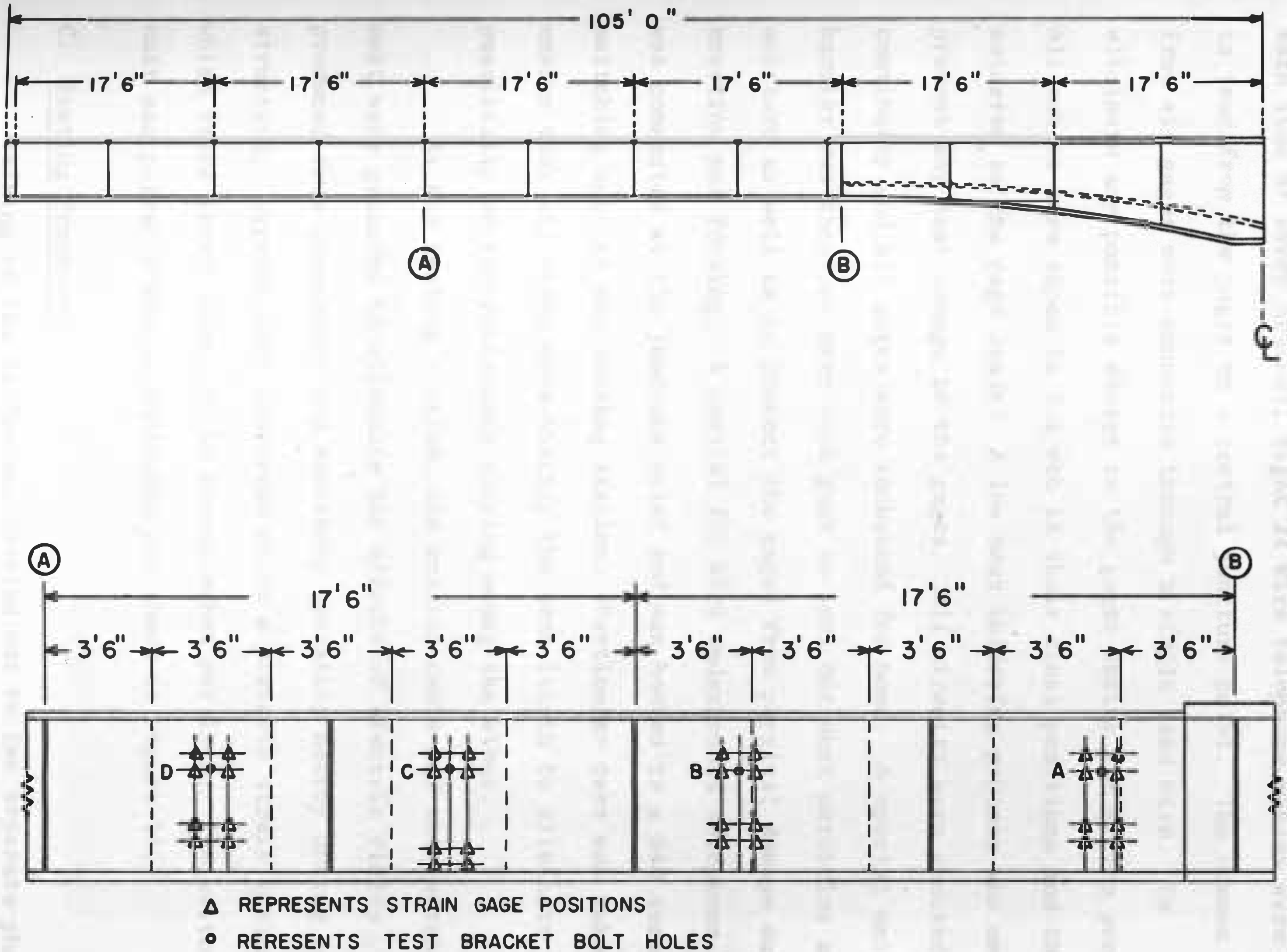


FIGURE 10. Strain Gage Rosette Positions on Test Bridge

each side of every bracket, eight 24 wire telephone cables were used to lead from the gages to a central juncture point. The common leads from all gages were connected through a single lead wire. To eliminate any possible damage to the gages during the wiring process, all cables were taped to the web in their final positions and then soldered to the gage leads. A low heat soldering process was used to prevent any heat damage to the gages. All circuits were checked for continuity and all gages were rechecked for bond. A special moisture barrier was then put over each gage to seal out dust particles and moisture as well as to protect the gages from physical damage during erection and forming. A coaxial 200 wire underground telephone cable was connected at the juncture point and was hooked to a 540 terminal switching unit at the testing station. Particular care was taken to ensure that all wires were exactly the same length to eliminate the possibility of the resistance varying among the wires.

At the testing station, the coaxial cable and the switching unit were grounded to eliminate the effects of electric fields produced from generators and machinery operating nearby during construction. Strains were monitored using a portable strain indicator which reads strain directly in micro-inches per inch. The switching unit and portable strain indicator are shown in Figure 11.

C. Testing Procedure

Testing of the bridge was carried out in two separate phases. Phase one was a preliminary static test, while phase two was a test as the bridge deck was being poured.

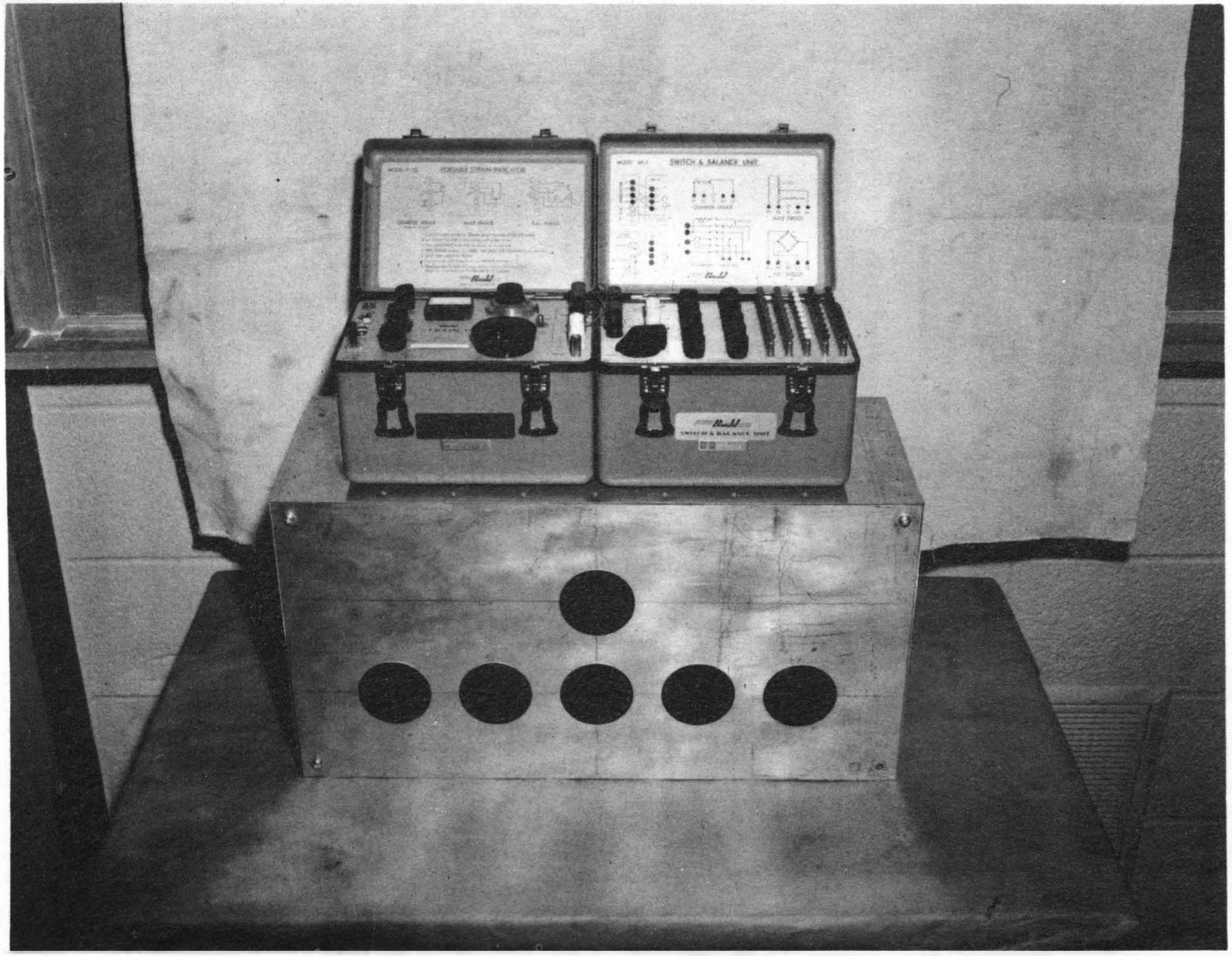


FIGURE 11. Switching Unit and Strain Measuring Equipment

1. Before actual testing of the bridge under construction loads, preliminary tests were performed by applying concentrated static loads on the brackets. The preconstruction tests were performed for three reasons:

   a. To allow comparisons with the static laboratory tests.

   b. To compare with construction tests for the purpose of estimating actual construction loads from stresses obtained during construction loading.

   c. To help in establishing a means of estimating stress caused by construction loads on actual bridges.

The preconstruction testing was done by loading each bracket in increments to produce a 3.5 kip-foot and a 7.0 kip-foot moment at the girder web and monitoring the resulting strains in the girder web.

Prior to testing each bracket, 7.0 kip-foot moments were applied and released several times to relieve stress concentrations in the bracket and the girder web. The brackets were loaded using a chain hoist in series with a 10,000 pound capacity load cell. The calibrated load cell, was used to indicate the applied load on the bracket. Figure 12 shows the bracket mounted on the girder with the loading system attached.

Bracket deflections relative to the plate girder flanges were taken at each loading increment using dial

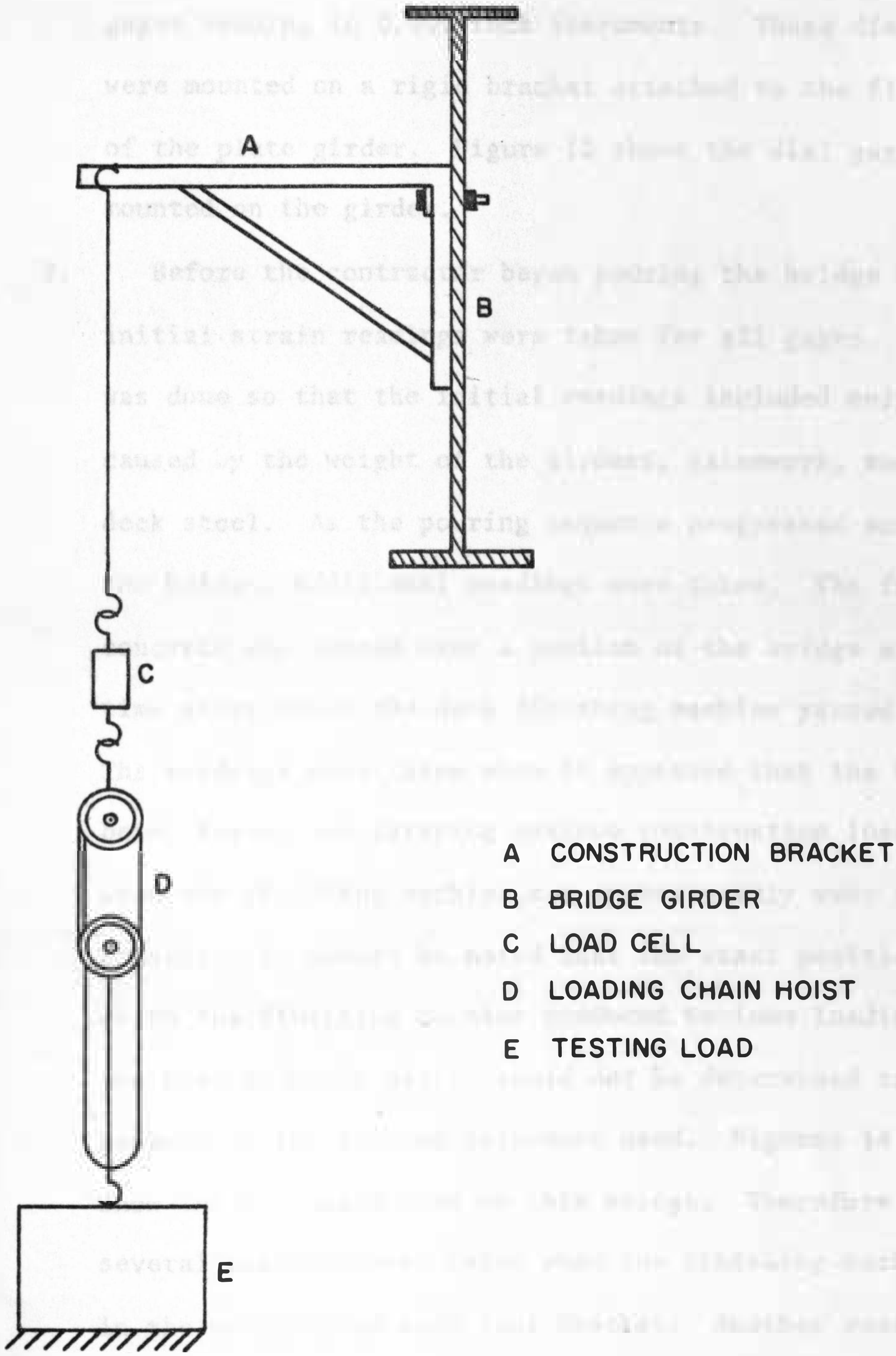


FIGURE 12. Static Test Loading System

gages reading in 0.001 inch increments. These dials were mounted on a rigid bracket attached to the flanges of the plate girder. Figure 13 shows the dial gages mounted on the girder.

2. Before the contractor began pouring the bridge deck, initial strain readings were taken for all gages. This was done so that the initial readings included only strain caused by the weight of the girders, falsework, and the deck steel. As the pouring sequence progressed across the bridge, additional readings were taken. The fresh concrete was spread over a portion of the bridge at a time after which the deck finishing machine passed over. The readings were taken when it appeared that the bracket being tested was carrying maximum construction loads; when the finishing machine was approximately over the test bracket. It should be noted that the exact position at which the finishing machine produced maximum loading on the bracket being tested could not be determined exactly because of the type of falsework used. Figures 14 and 15 show the falsework used on this bridge. Therefore several readings were taken when the finishing machine was in the vicinity of each test bracket. Another reason for taking several readings was that the finishing machine was in constant motion. No single set of readings for any

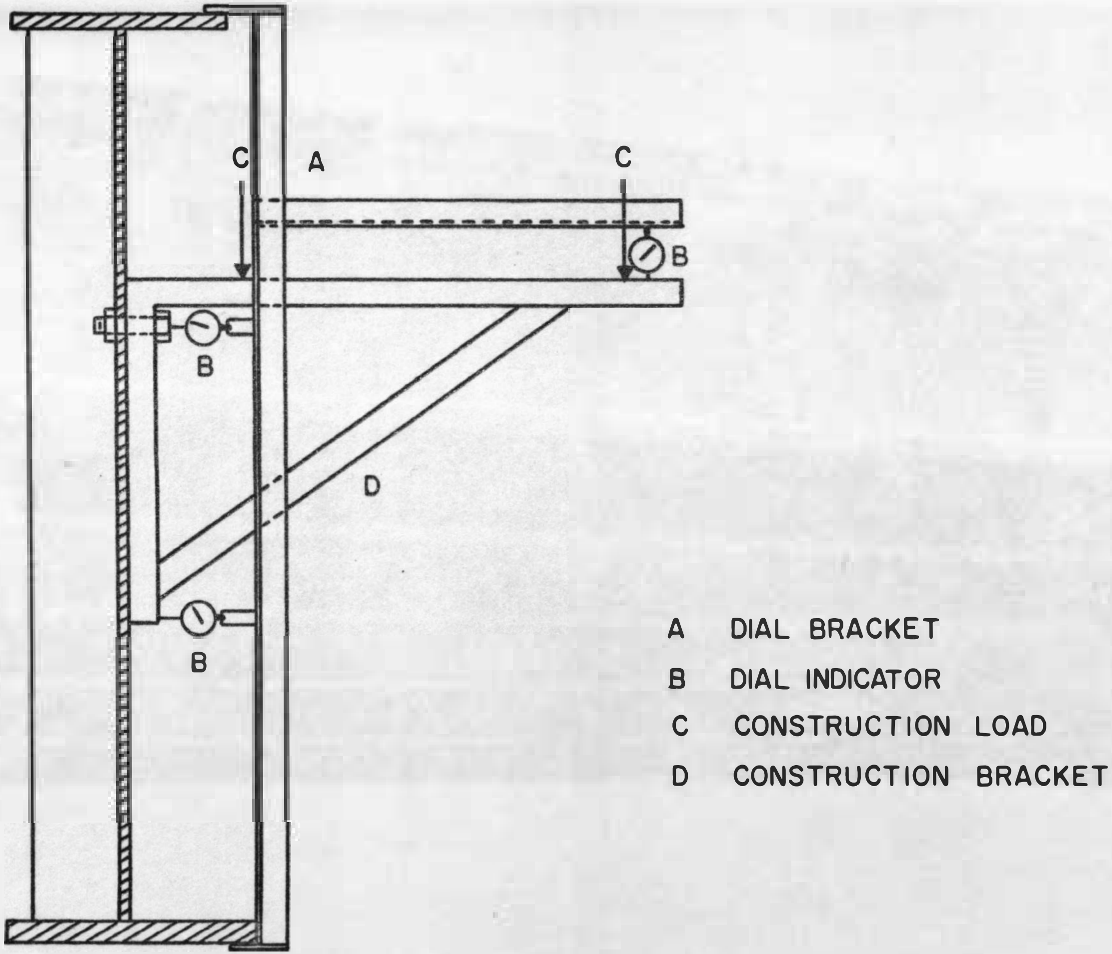


FIGURE 13. Instrumentation for Measuring Bracket Deflections

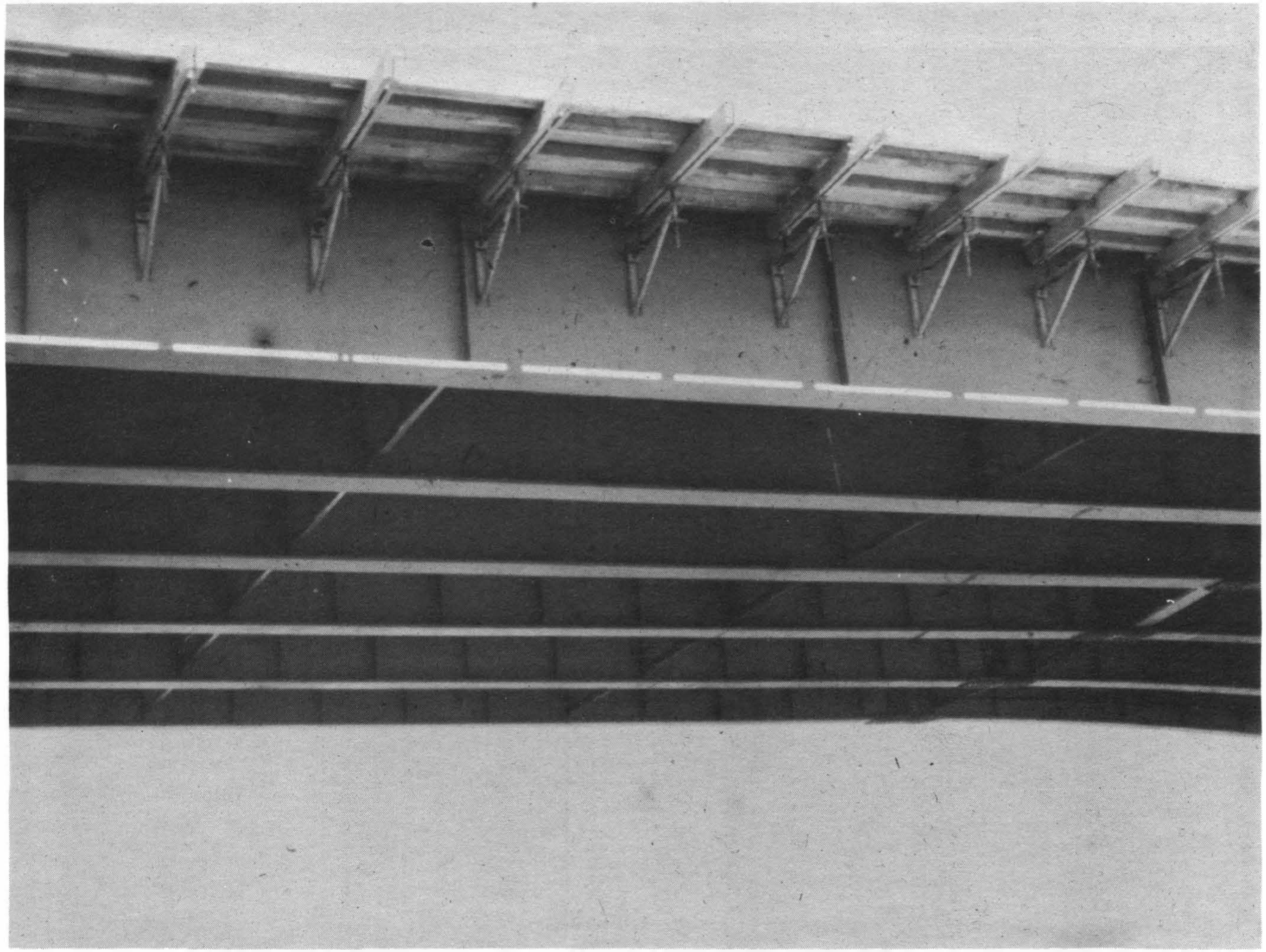


FIGURE 14. General View of Falsework

FIGURE 15. General View of Falsework

bracket could be taken and have each gage record strains produced by the same load. Therefore readings were taken as the finishing machine approached and passed over the test bracket in both forward and backward directions. These steps were repeated for each of the four test brackets. The paving machine directly over a test bracket is shown in Figure 16.

D. Reduction of Test Data

The web stresses developed in the girder web were determined by means of three element rectangular rosette strain gages. The three element rectangular rosette employs strain gages mounted at zero, 45, and 90 degree positions as indicated in Figure 17. By measuring the strains in these three directions, the principal stresses can be calculated using the equation:

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

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

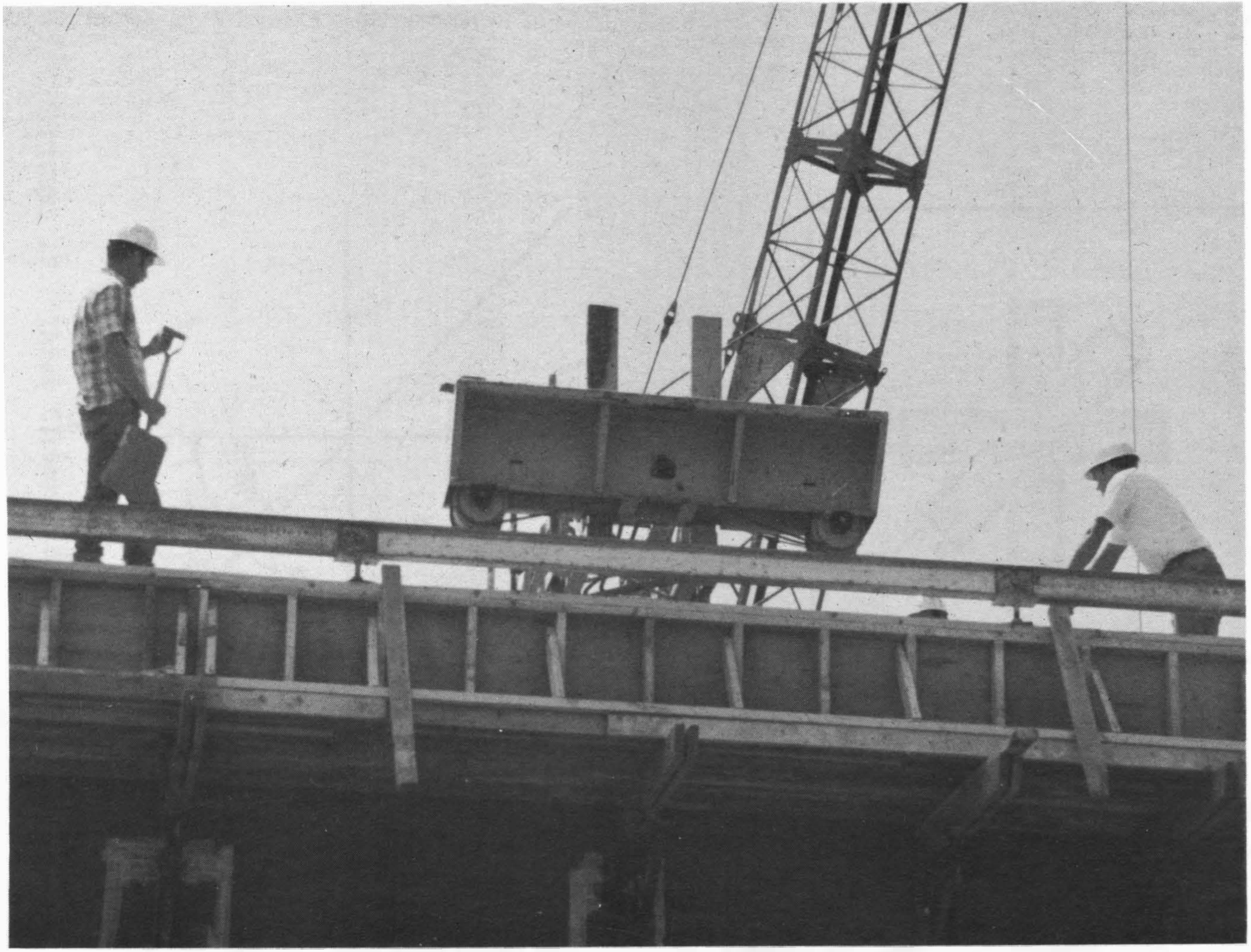


FIGURE 16. View of Finishing Machine Directly Over a Test Bracket

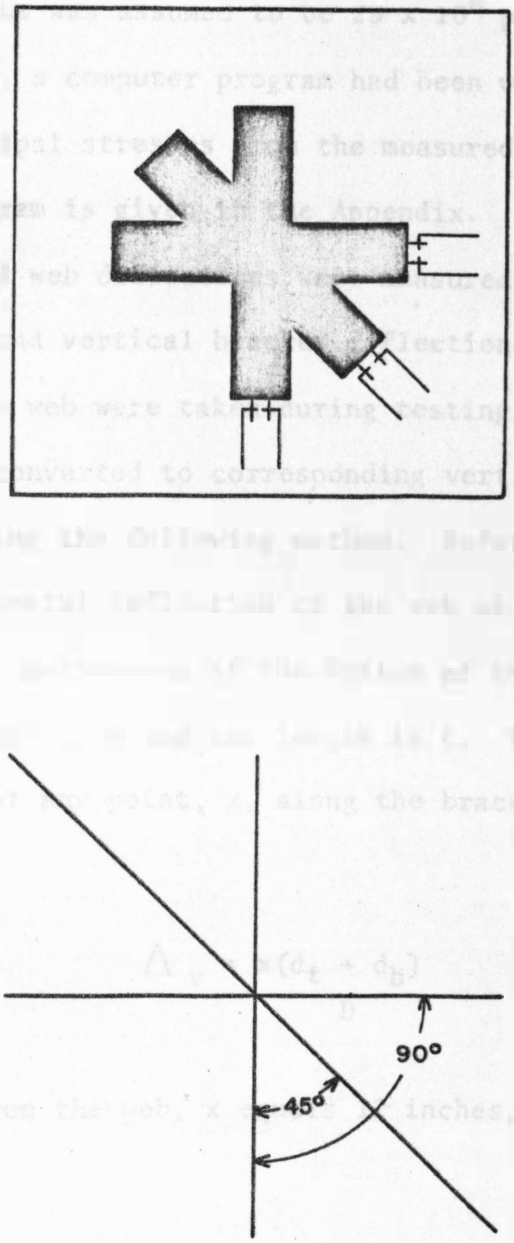


FIGURE 17. Three Element Rectangular Rosette

For the bridge girder studied, Poisson's Ratio was assumed to be 0.3, and Young's Modulus was assumed to be 29×10^6 psi. To simplify the reduction of data, a computer program had been written which calculated the principal stresses from the measured values of strain. The computer program is given in the Appendix.

Horizontal web deflections were measured at the top and bottom of each bracket, and vertical bracket deflections at a distance of 30 inches from the web were taken during testing. The horizontal web deflections were converted to corresponding vertical bracket deflections by using the following method. Referring to Figure 18, the measured horizontal deflection of the web at the top of the bracket is d_t, the deflection of the bottom of the bracket is d_b. The depth of the bracket is D and its length is L. The resulting vertical deflection, Δ_v, at any point, x, along the bracket is then given by the formula:

$$\Delta_v = \frac{x(d_t + d_b)}{D} \tag{2}$$

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

$$\Delta_v = \frac{12(d_t + d_b)}{D} \tag{3}$$

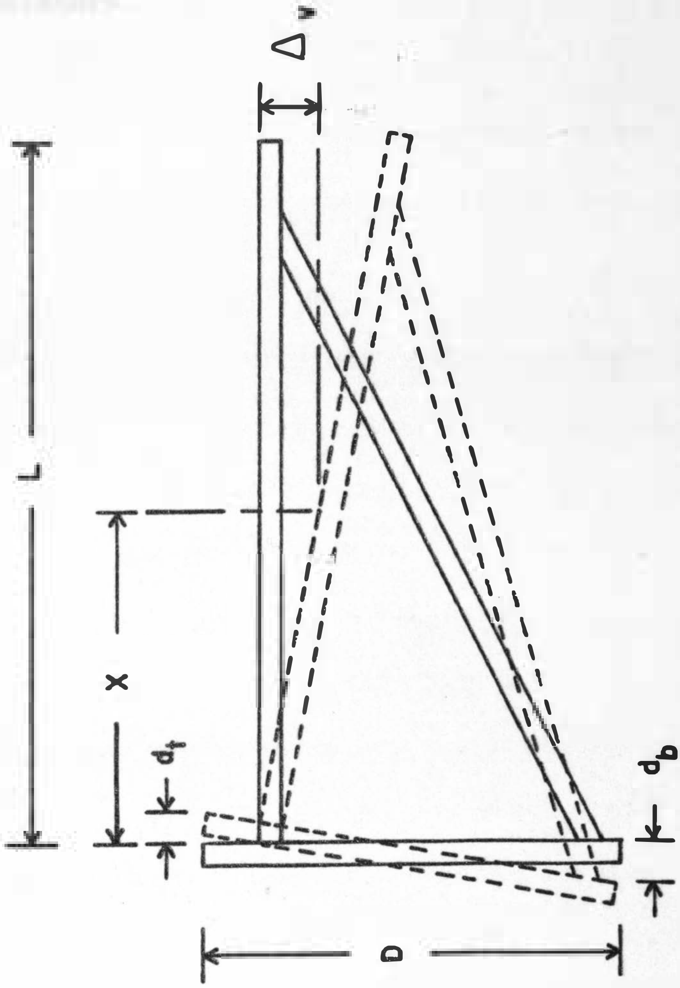


FIGURE 18. Bracket in Deflected Position

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

These computed deflections were compared to actual measured deflections as a means of checking the readings taken from each of the dial indicators.

CHAPTER III

TEST RESULTS

The results from the two phases of testing are presented separately as follows:

A. Static Preconstruction Tests

B. Bridge Deck Pour Tests

A. Static Preconstruction Tests

The static tests consisted of determining the maximum web stresses and bracket deflections produced by concentrated loads applied on the construction brackets. The results from this phase of testing are presented in two parts:

1. Maximum Web Stresses
2. Bracket Deflections

1. Maximum Web Stresses

For each bracket tested, eight individual strain gage rosettes were monitored and the strains recorded were reduced by means of a computer. The values of stress obtained were then studied to determine if any irregular or random stresses occurred. Since for each bracket tested, the stresses produced in the bridge girder web at strain gage locations near the bolt hole differed in magnitude from the stresses produced near the bottom of the bracket, the values of stress from both areas are presented. However, for each area, only the maximum values of stress are discussed because the variation in the magnitude of stresses within the areas was small.

For the static testing, loads were applied to produce moments at the girder web of 3.5 and 7.0 kip-foot. All results are presented as maximum principal stresses produced in the bridge girder web. It was noted that the stresses produced in the girder web were proportional to the load applied and therefore only the stresses produced by the 7.0 kip-foot moment are discussed in the text.

a. Bracket A

Bracket A, a standard construction bracket used by the contractor on the bridge tested, mounted six inches from the nearest stiffener, produced the largest stresses in the girder web of any bracket tested. The maximum stress of 22.1 ksi occurred at a point near the bolt hole under a 7.0 kip-foot moment. The maximum stress developed near the bottom of the bracket was 18.6 ksi. The horizontal loads transmitted to the bridge girder web by the bracket, due to the 7.0 kip-foot moment, were 4.95 kips. These concentrated loads are applied perpendicular to the girder web at the bolt hole and at the bottom of the bracket.

It was noted that stresses produced at gage points to the left of Bracket A were of opposite sign from those at gage points to the right of the bracket. This was referred to as a stress reversal, in the laboratory test, where stress contours drawn from many data points showed that the area where the stresses reversed signs was located approximately midway between the bolt hole and the stiffener. Therefore, for the case of Bracket A, the strain gage points to the right of the bracket were located slightly past the point of contraflexure.

b. Bracket B

Bracket B, identical to Bracket A but mounted four inches from the nearest stiffener, produced a maximum stress of 19.4 ksi near the bolt hole and 17.8 ksi near the bottom of the bracket due to a 7.0 kip-foot moment. The reduction of stress from Bracket A near the bolt hole was approximately 14 per cent and the reduction of stress near the bottom of the bracket was 4.3 per cent. The concentrated horizontal loads applied to the girder web were 4.95 kips, the same as Bracket A. It was noted that as in the case of Bracket A, the stresses at strain gage points left of the bracket were of opposite sign from those determined at gage points to the right of the bracket. In the case of Bracket B, mounted four inches from the stiffener, with the strain gages mounted four inches on either side of the bracket, the strain gages on the right side of the bracket were directly opposite the stiffener and were well beyond the point of contraflecture discussed for Bracket A.

c. Bracket C

Bracket C, a deep bracket, mounted 21 inches from the nearest stiffener, produced a maximum stress of 16.1 ksi near the bolt hole and 10.0 ksi near the bottom of the bracket under a moment of 7.0 kip-foot applied at the girder web. The reduction of stress from Bracket A near the bolt hole was 27 per cent and 17 per cent from Bracket B. The reduction of stress near the bottom of the bracket was 43 per cent from Bracket A and 42 per cent from Bracket B. The concentrated horizontal loads transmitted to the girder web were 2.4 kips for Bracket C. This represents approximately a 50 per cent reduction in loads which allows this deep bracket to be mounted far from a stiffener with no resulting damage to the web. Had this bracket been mounted in the same position as Bracket A or B, a 50 per cent reduction of stress would have resulted. The reduction of stress near the bolt hole of only 27 per cent as opposed to the 50 per cent reduction of load, demonstrates the increase in stress effected by increasing the distance from the nearest stiffener. No stress reversals were noted for Bracket C since it was located far from a stiffener.

d. Bracket D

Bracket D, was identical to Brackets A and B but was mounted midway between two stiffeners (21 inches from either) with steel backup angles applied to the full depth of the web. The maximum stress produced near the bolt hole was 9.2 ksi and 7.6 ksi near the bottom of the bracket. The reductions of stress near the bolt hole from Brackets A, B, and C are 58 per cent, 53 per cent, and 43 per cent respectively. The reductions of stress near the bottom of the bracket from Brackets A, B, and C were 59 per cent, 55 per cent, and 25 per cent respectively. The concentrated horizontal loads applied to the girder web were 4.95 kips as in the case of Brackets A and B. By comparing Bracket D with Bracket C, it was noted that while the horizontal loads applied to the bridge girder web were doubled, the stresses produced in the web were reduced nearly 50 per cent. No stress reversals were observed at Bracket D.

It was noted that for each bracket tested, the absolute values of stress produced near the bolt hole were larger than the absolute values of stress near the bottom of the bracket. The maximum positive value of stress near the bolt hole decreased to a maximum negative value near the bottom of the bracket. Figure 19 is a bar graph showing the absolute values of stress produced near the bolt hole and bottom of the bracket with 7.0 kip-foot moments applied to the girder web.

Figure 20 is a bar graph which shows the absolute values of stress produced in the bridge girder web near the bolt holes by 3.5 kip-foot and 7.0 kip-foot moments applied at the girder web.

2. Bracket Deflections

The experimental laboratory test results showed that vertical deflection of a bracket, although caused by both lateral deformations of the supporting web and deformation of the bracket itself, were caused primarily by the lateral deformations of the web. These deformations allow the rigid bracket to rotate, resulting in vertical deflections as illustrated in Figure 18. For each bracket tested, the horizontal deformations of the web at the bolt

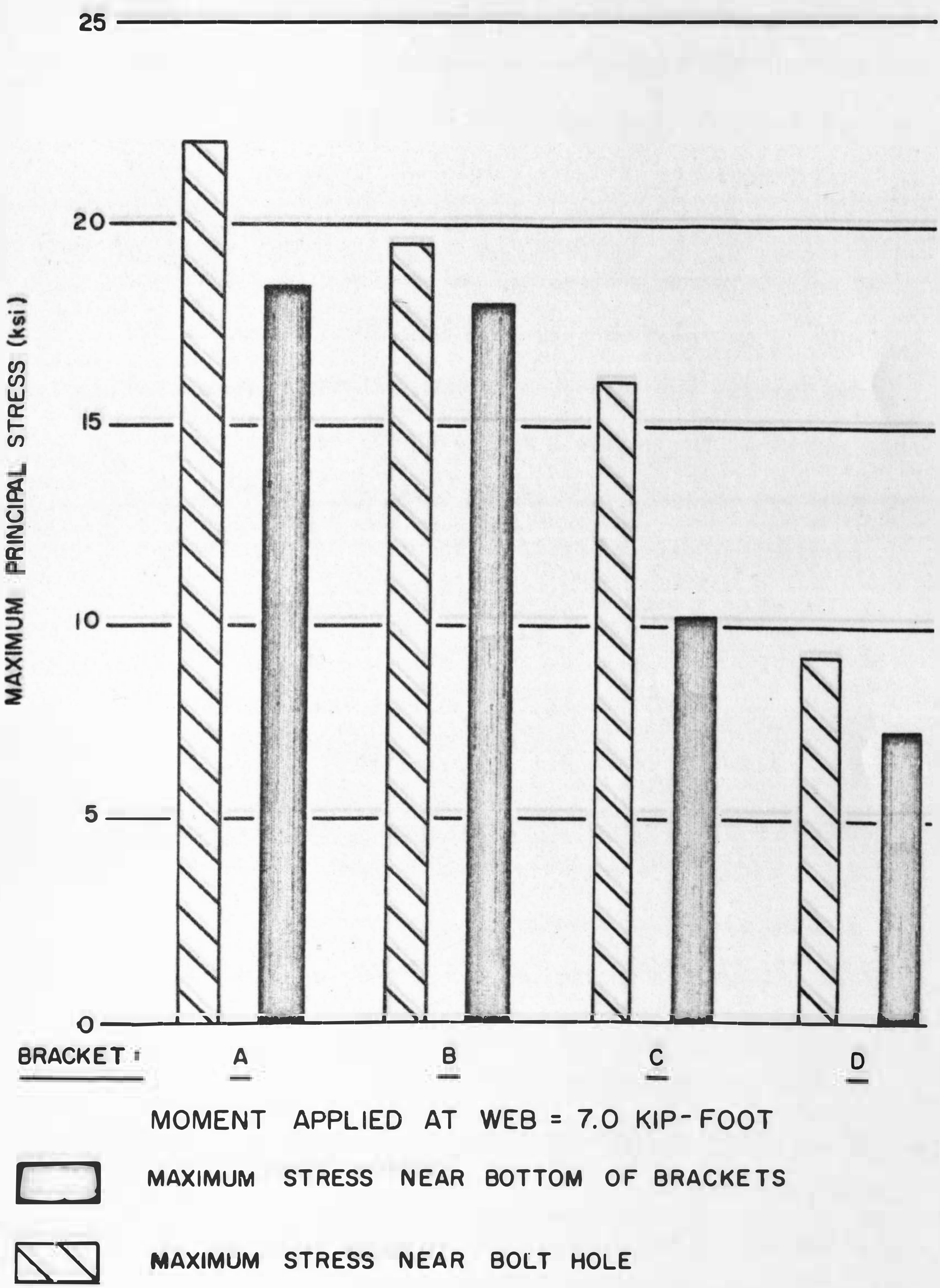


FIGURE 19. Maximum Principal Stresses Produced Near Test Brackets

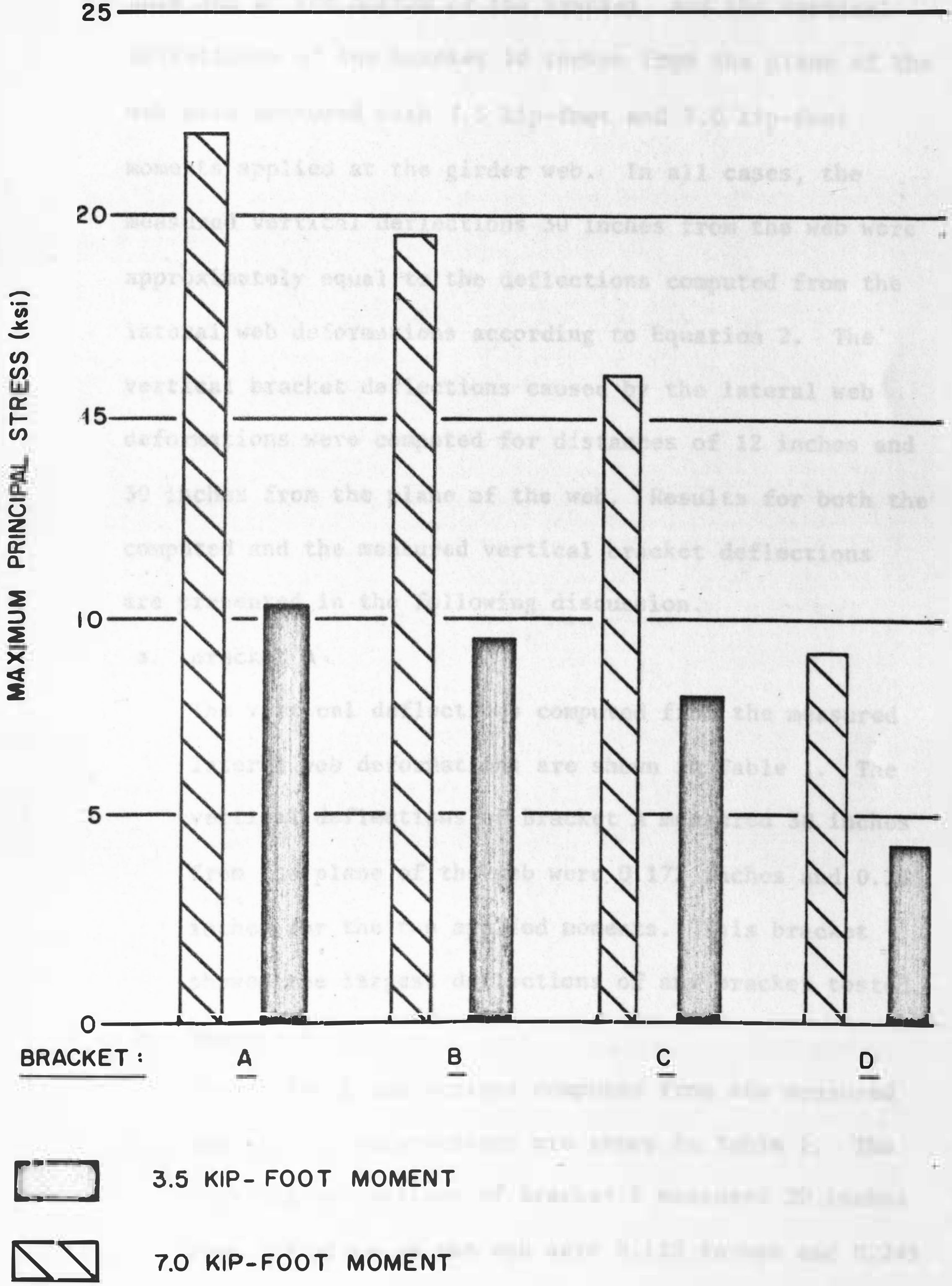


FIGURE 20. Maximum Principal Stresses Produced Near Bolt Hole

hole and at the bottom of the bracket, and the vertical deflections of the bracket 30 inches from the plane of the web were measured with 3.5 kip-foot and 7.0 kip-foot moments applied at the girder web. In all cases, the measured vertical deflections 30 inches from the web were approximately equal to the deflections computed from the lateral web deformations according to Equation 2. The vertical bracket deflections caused by the lateral web deformations were computed for distances of 12 inches and 30 inches from the plane of the web. Results for both the computed and the measured vertical bracket deflections are presented in the following discussion.

a. Bracket A

The vertical deflections computed from the measured lateral web deformations are shown in Table 1. The vertical deflections of Bracket A measured 30 inches from the plane of the web were 0.172 inches and 0.343 inches for the two applied moments. This bracket showed the largest deflections of any bracket tested.

b. Bracket B

The vertical deflections computed from the measured lateral web deformations are shown in Table 1. The vertical deflections of Bracket B measured 30 inches from the plane of the web were 0.122 inches and 0.245

Table 1

Bracket Deflections from Static Test

Bracket (1)	Applied Moment (kip-feet) (2)	Distance from Web (inches) (3)	Vertical Deflection (inches) (4)
A	3.5	12	0.068
A	7.0	12	0.136
A	3.5	30	0.170
A	7.0	30	0.340
B	3.5	12	0.048
B	7.0	12	0.095
B	3.5	30	0.119
B	7.0	30	0.238
C	3.5	12	0.023
C	7.0	12	0.046
C	3.5	30	0.058
C	7.0	30	0.116
D	3.5	12	0.030
D	7.0	12	0.060
D	3.5	30	0.075
D	7.0	30	0.150

inches for the two applied moments. If Bracket B is compared with Bracket A, a reduction in the resulting deflections of approximately 30 per cent is noted. Such a reduction indicates that the stiffener provides lateral restraint for the bridge girder web in sustaining horizontal loads.

c. Bracket C

The vertical deflections computed from the measured lateral web deformations are shown in Table 1. The vertical deflections of Bracket C measured 30 inches from the plane of the web were 0.060 inches and 0.120 inches for the two applied moments. If Bracket C is compared with Brackets A and B, deflection reductions of approximately 66 per cent and 50 per cent respectively are noted. Such reductions indicate that placing the bracket close to the bottom flange is very effective in reducing deflections.

d. Bracket D

The vertical deflections computed from the measured lateral web deformations are shown in Table 1. The vertical deflections of Bracket D measured 30 inches from the plane of the web are .077 inches and .154 inches for the two applied moments. If Bracket D is compared with Brackets A and B, reductions in vertical deflections of 56 per cent and 37 per cent are noted. Bracket D showed a 23 per cent increase in deflection from Bracket C.

Figure 21 shows bracket deflection one foot from the web versus applied moment for Brackets A, B, C, and D. Linear relationships between vertical deflection and applied moment were observed for all brackets tested. From Figure 21 the ratios of bracket deflection to applied moment for each bracket was computed and are shown below.

Bracket	Vertical Deflection in inches / Applied Moment in kip-feet
A	.048
B	.034
C	.017
D	.025

B. Bridge Deck Pour Tests

The deck pour test involved monitoring the maximum values of stress which occurred in the bridge girder web during the pouring sequence. The pouring sequence consisted of the wet concrete being spread and the finishing machine passing over it. The stresses determined are not necessarily the maximum values that occurred, however the large number of readings taken when the finishing machine was over the test brackets should give a clear indication of the stresses actually produced in the girder web by construction loads

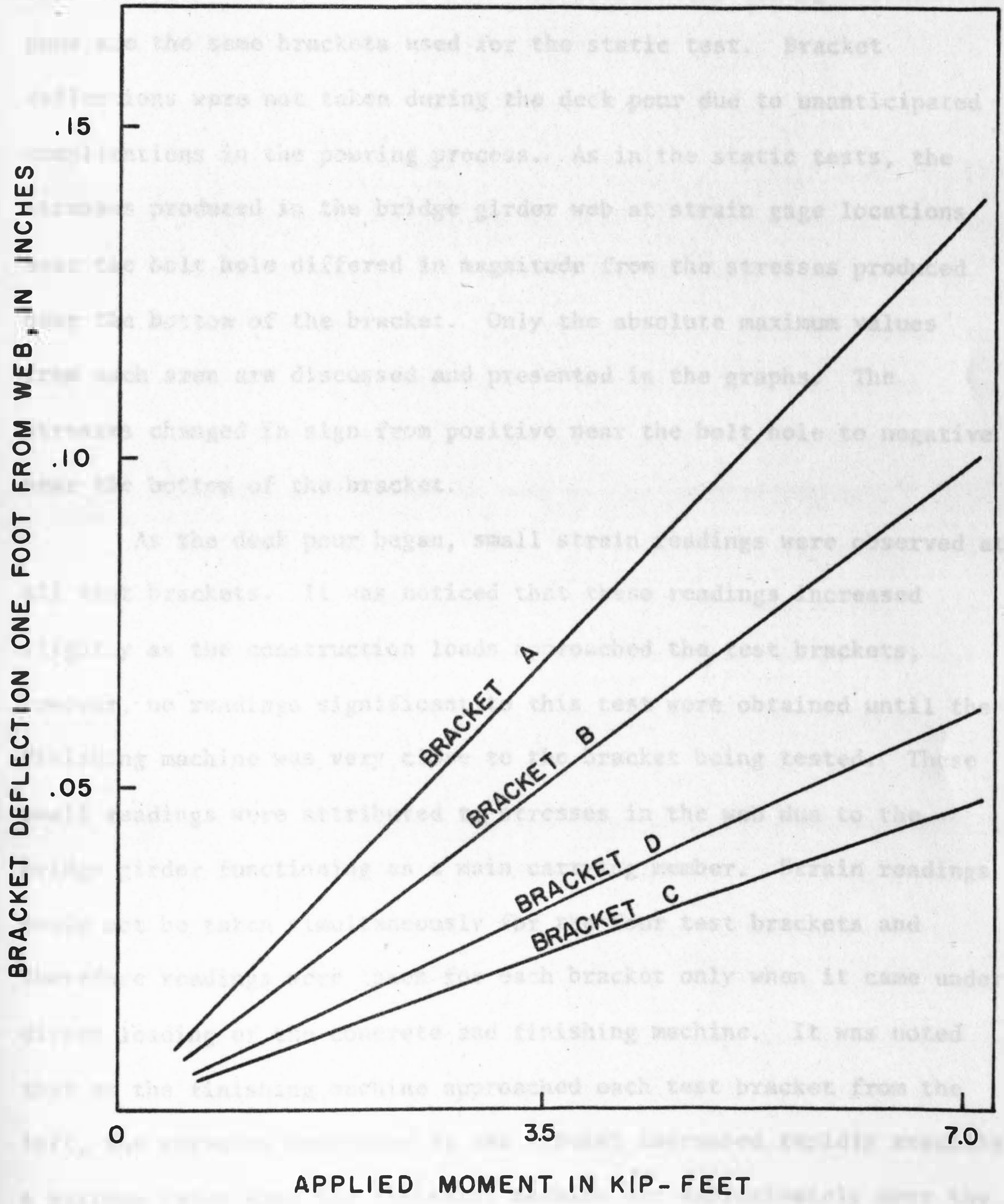


FIGURE 21. Bracket Deflections for Static Test

applied to the brackets. The four brackets tested during the deck pour are the same brackets used for the static test. Bracket deflections were not taken during the deck pour due to unanticipated complications in the pouring process. As in the static tests, the stresses produced in the bridge girder web at strain gage locations near the bolt hole differed in magnitude from the stresses produced near the bottom of the bracket. Only the absolute maximum values from each area are discussed and presented in the graphs. The stresses changed in sign from positive near the bolt hole to negative near the bottom of the bracket.

As the deck pour began, small strain readings were observed at all test brackets. It was noticed that these readings increased slightly as the construction loads approached the test brackets, however, no readings significant to this test were obtained until the finishing machine was very close to the bracket being tested. These small readings were attributed to stresses in the web due to the bridge girder functioning as a main carrying member. Strain readings could not be taken simultaneously for the four test brackets and therefore readings were taken for each bracket only when it came under direct loading of the concrete and finishing machine. It was noted that as the finishing machine approached each test bracket from the left, the stresses monitored at the bracket increased rapidly reaching a maximum value when the finishing machine was approximately over the test bracket and decreased to constant values when the finishing machine was beyond the next bracket.

1. Bracket A

The maximum stresses monitored in the bridge girder web at Bracket A were 23.0 ksi near the bolt hole and 19.0 ksi near the bottom of the bracket. The variation of stress near the bolt hole and near the bottom of the bracket as the pouring sequence moved left to right is shown in Figure 22. A rapid increase of stress for both areas near Bracket A can be seen as the pouring sequence approached from the left. As the pouring sequence moved to the right of Bracket A, the stresses monitored decreased rapidly to near constant values of 4.5 ksi and 3.5 ksi near the bolt hole and bottom of the bracket respectively. The absolute maximum stresses monitored near the bolt hole indicate that the finishing machine caused approximately 80 per cent of the stress in the girder web while the wet concrete and flexural stresses caused approximately 20 per cent of the total stress. The absolute maximum stress of 23.0 ksi near the bolt hole, when correlated with static test results, indicated a moment of approximately 7.3 kip-foot applied at the girder web by Bracket A. Stress reversals similar to those in the static test were observed at Bracket A in the pour test. Since strain gages located on the right side of Bracket A

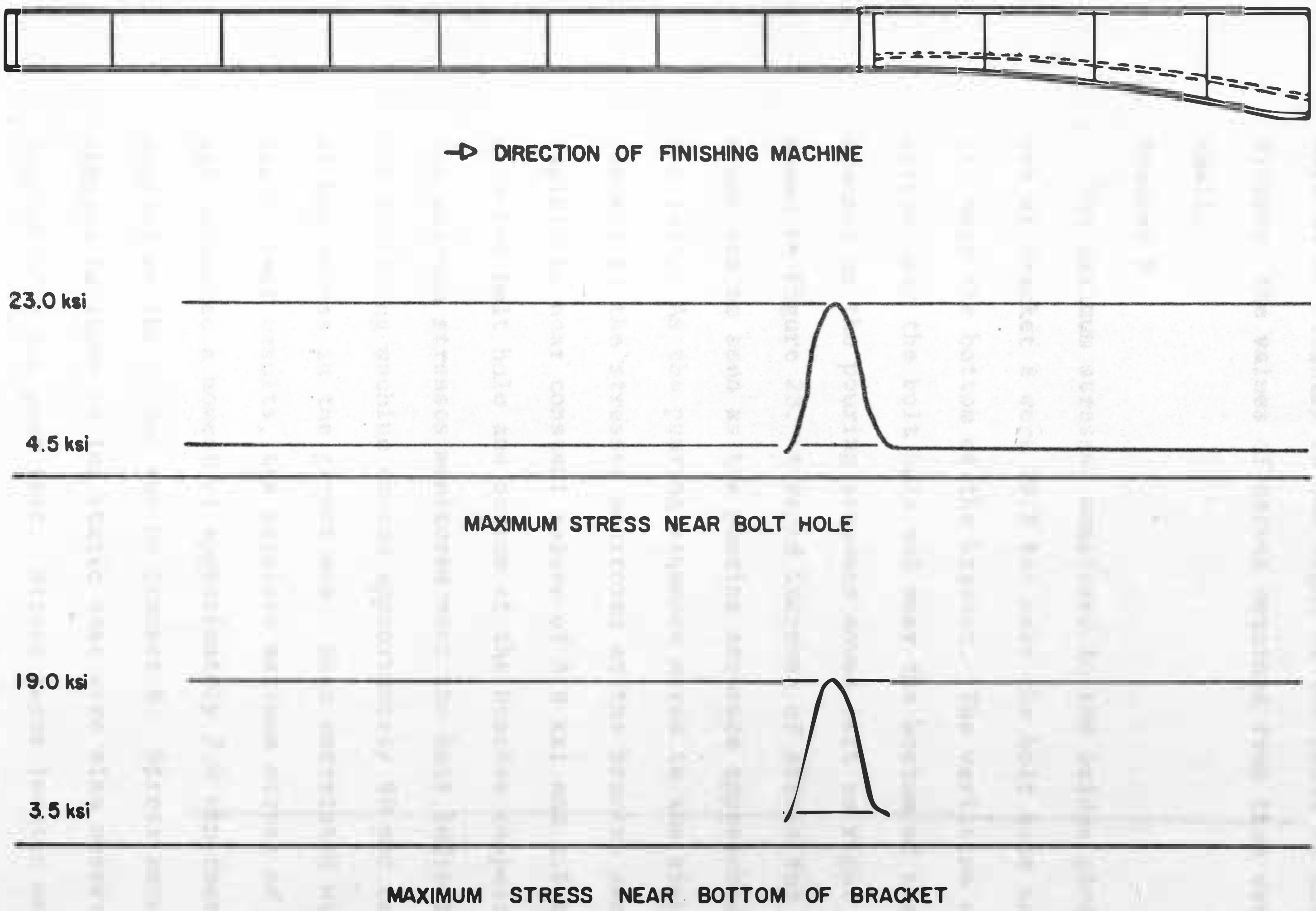


FIGURE 22. Maximum Principal Stresses for Bracket A; Pour Test

were at or slightly to the right of the point of contraflexure, the values of stress obtained from them were small.

2. Bracket B

The maximum stresses monitored in the bridge girder web at Bracket B were 19.5 ksi near the bolt hole and 18.4 ksi near the bottom of the bracket. The variation of stress near the bolt hole and near the bottom of the bracket as the pouring sequence moved left to right is shown in Figure 23. A rapid increase of stress for both areas can be seen as the pouring sequence approached from the left. As the pouring sequence moved to the right of Bracket B, the stresses monitored at the bracket decreased rapidly to near constant values of 3.8 ksi and 3.5 ksi near the bolt hole and bottom of the bracket respectively. The maximum stresses monitored near the bolt indicate that the finishing machine caused approximately 80 per cent of the stress in the girder web. When correlated with the static test results, the absolute maximum stress of 19.5 ksi indicated a moment of approximately 7.0 kip-foot applied at the girder web by Bracket B. Stress reversals similar to those in the static test were also observed at Bracket B in the pour test. Strain gages located on the right side of Bracket B were beyond the point of contraflexure and the values obtained near the bolt hole were of

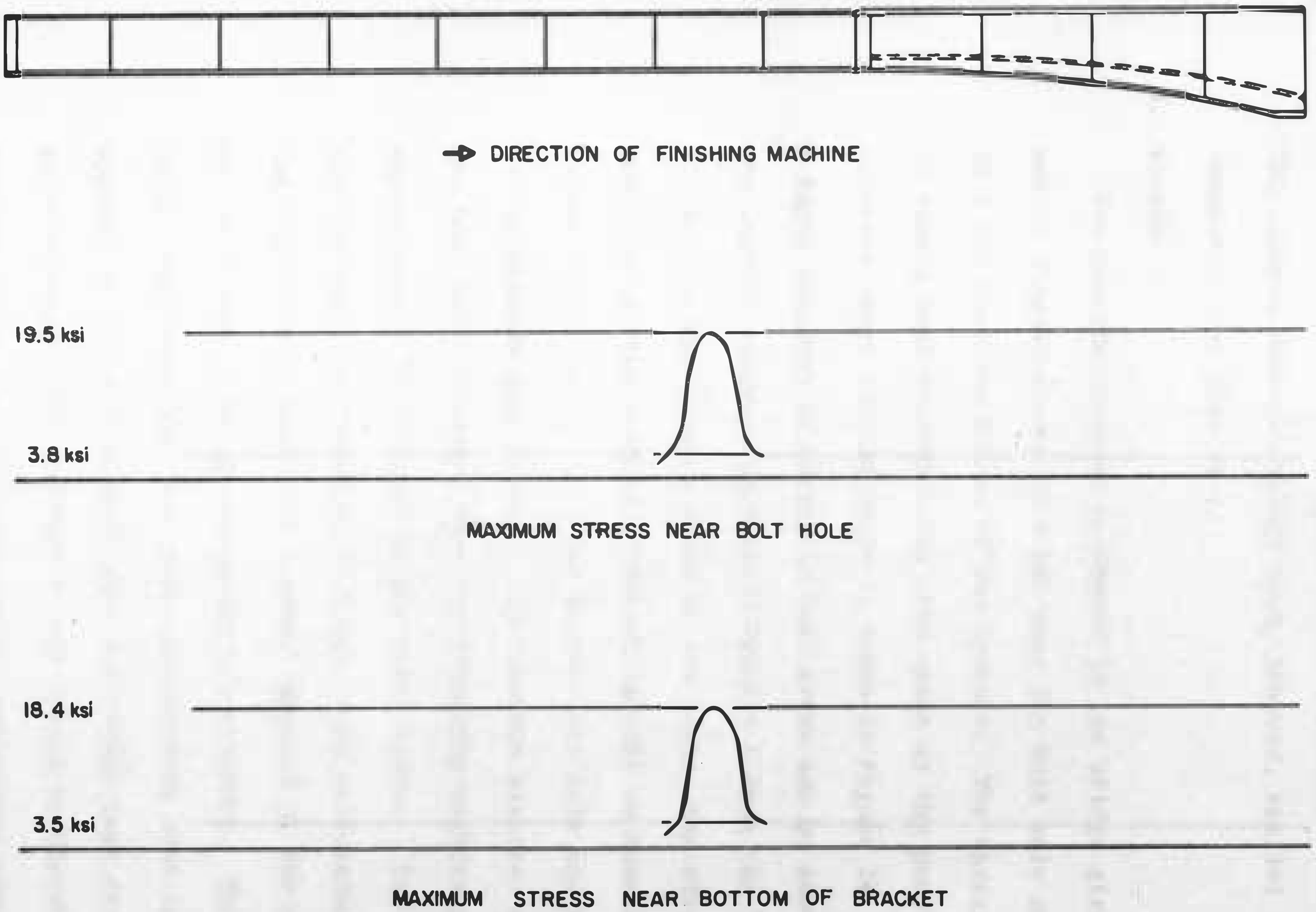


FIGURE 23. Maximum Principal Stresses for Bracket B; Pour Test

larger negative values than those obtained at Bracket A. The maximum positive values were, however, smaller for Bracket B than Bracket A.

3. Bracket C

The maximum stresses monitored in the bridge girder web at Bracket C were 13.9 ksi near the bolt hole and 10.1 ksi near the bottom of the bracket. The variation of stress near Bracket C for both areas as the pouring sequence moved left to right is shown in Figure 24. A rapid increase of stress in both areas can be seen as the pouring sequence approached Bracket C from the left. As the pouring sequence moved to the right, the stresses monitored at the bracket decreased rapidly to near constant values of 3.2 ksi and 2.0 ksi at the bolt hole and bottom of the bracket respectively. The maximum stresses near the bolt hole indicated that the finishing machine produced approximately 77 per cent of the total stress. The absolute maximum stress of 13.9 ksi, when correlated with the static test, indicated a moment applied at the girder web by Bracket C of approximately 6.1 kip-foot. This result indicates that the total construction load was not applied to the test bracket when the strain gage readings were recorded. The constant stress caused by the wet concrete after the finishing machine had passed Bracket C, if assumed to be 20 per cent of the total stress, would

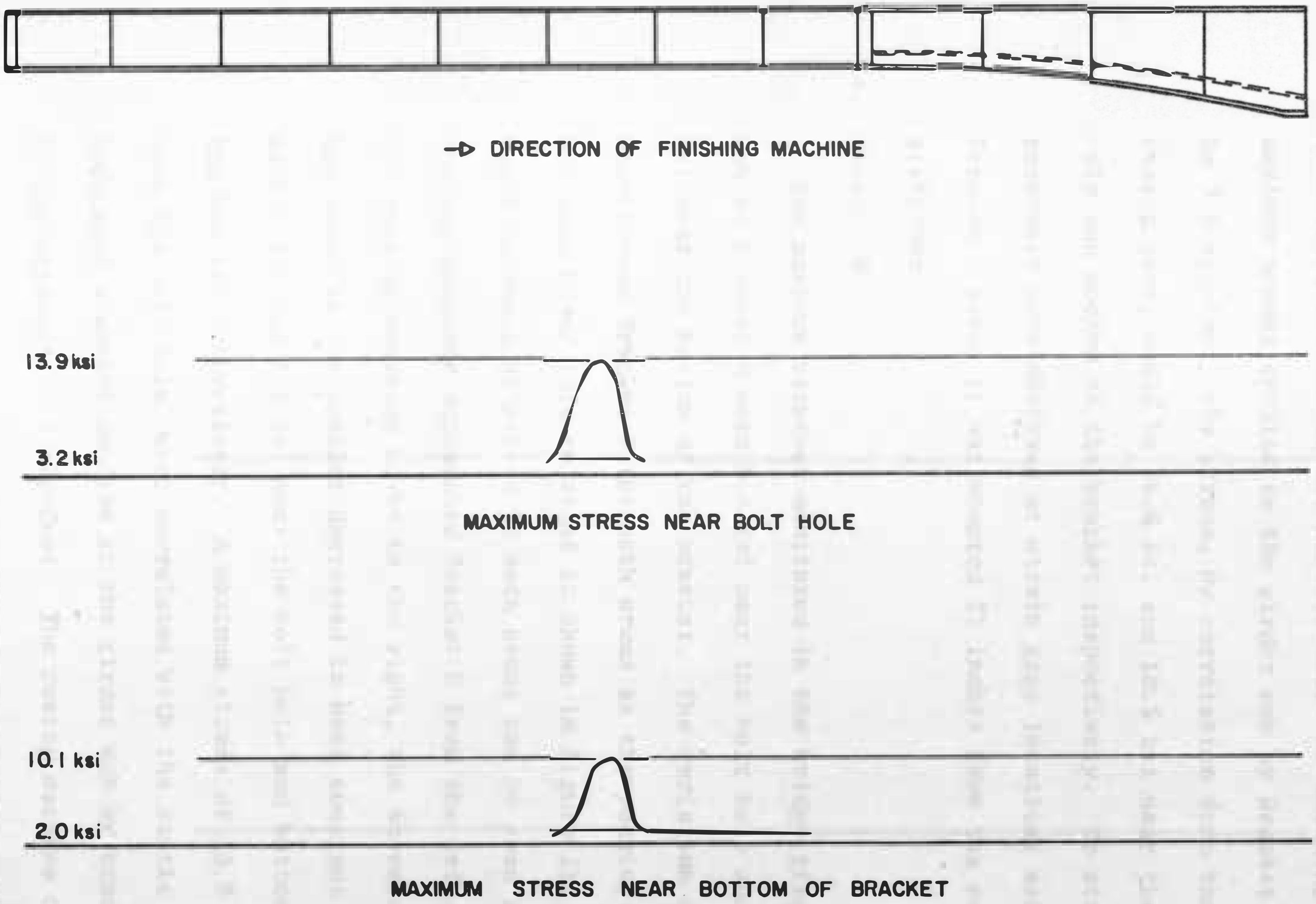


FIGURE 24. Maximum Principal Stresses for Bracket C; Pour Test

indicate a maximum stress of 16.0 ksi. By assuming the maximum moment applied to the girder web by Bracket C to be 7.3 kip-foot, the stress, by correlation from the static test, would be 16.6 ksi and 10.5 ksi near the bolt hole and bottom of the bracket respectively. No stress reversals were observed at strain gage locations around Bracket C since it was mounted 21 inches from the nearest stiffener.

4. Bracket D

The maximum stresses monitored in the bridge girder web at Bracket D were 9.4 ksi near the bolt hole and 7.0 ksi near the bottom of the bracket. The variation of stress near Bracket D for both areas as the pouring sequence moved left to right is shown in Figure 25. A rapid increase of stress in both areas can be seen as the pouring sequence approached Bracket D from the left. As the pouring sequence moved to the right, the stresses monitored at the bracket decreased to near constant values of 1.9 ksi and 1.3 ksi near the bolt hole and bottom of the bracket respectively. A maximum stress of 13.9 ksi near the bolt hole, when correlated with the static test, indicated a moment applied at the girder web by Bracket D of approximately 7.2 kip-foot. The paving machine caused approximately 80 per cent of the total stresses monitored at Bracket D. No stress reversals were observed at strain

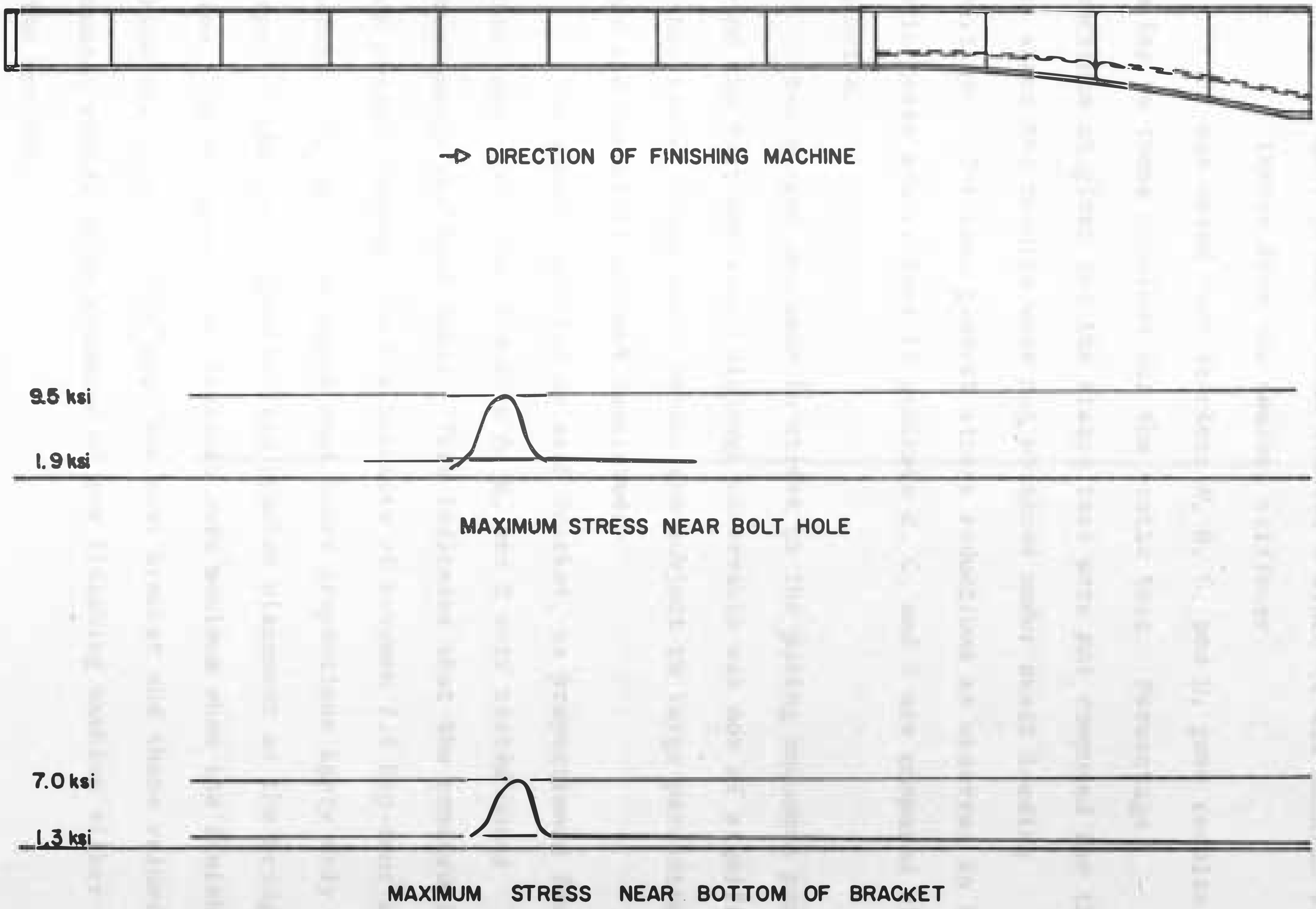


FIGURE 25. Maximum Principal Stresses for Bracket D; Pour Test

gage points around Bracket D since it was mounted 21 inches from the nearest stiffener.

It was noted that Brackets A, B, C, and D, gave results similar to those obtained for the static test. Percentage reductions as given for the static test were not computed for the pour test since the results were not obtained under exact loading conditions. The same general stress reductions as observed in the static tests are evident if Brackets B, C, and D are compared with Bracket A.

The slight decrease in stress as the paving sequence proceeded beyond the test brackets, although observable was not of significance to this study. Such small values are subject to large percentages of error and therefore are not tabulated.

The moments applied by each bracket, as proportioned from the static test, show that Brackets A, B, and D were tested under approximately the same loads. This indicates that the construction loads produced moments on the brackets of between 7.0 kip-foot and 7.3 kip-foot. It should be noted that these proportions apply only to bridges of the same dimension and bracket placement as the bridge tested. In all cases, the stresses were maximum when the finishing machine was approximately over the test bracket and these values decreased rapidly with movement of the finishing machine either side of the bracket.

CHAPTER IV

SUMMARY AND CONCLUSIONS

A. Summary of Results

Results of the static preconstruction and bridge deck pour tests are summarized in Tables 2 and 3 respectively. Table 2 shows the test bracket, its depth, and location relative to the nearest stiffener. It also shows the applied moment and the corresponding horizontal loads applied by the bracket to the web. The maximum stresses and the corresponding bracket deflections two inches from the web are also listed. Table 3 shows the test bracket, its depth, location relative to the nearest stiffener, and maximum stresses near the bolt hole and bottom of the bracket. Applied moment, horizontal load components, maximum stresses and deflections as correlated from the static test results, are also given.

The following results have been formulated from this study:

1. Effect of Stiffeners

   Static and deck pour tests showed that stiffeners restrain lateral web deformations and reduce web stresses when a bracket is placed nearby. This compared favorably with the laboratory findings where the distance from the nearest stiffener was varied.

2. Effect of Bracket Depth

   Web stresses and deflections are proportional to the

TABLE 2

STATIC TEST

Bracket	Distance from Nearest Stiffener (inches)	Bracket Depth (inches)	Applied Moment (kip-feet)	Horizontal Loads Transmitted to the Web (kips)	Maximum Stress Near Bolt Hole (ksi)	Maximum Stress Near Bottom of Bracket (ksi)	Bracket Deflection 12 inches from the Web (inches)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
A	6	17	7.0	4.94	22.1	18.6	.136
B	4	17	7.0	4.94	19.4	17.8	.095
C	21	35	7.0	2.40	16.1	10.1	.046
D	21	17	7.0	4.94	9.2	7.6	.061

TABLE 3

DECK POUR TEST

Bracket	Distance from Nearest Stiffener (inches)	Bracket Depth (inches)	Maximum Stress Near Bolt Hole (ksi)	Maximum Stress Near Bottom of Bracket (ksi)	Values Correlated from Static Test: Applied Moment (kip-feet)	Values Correlated from Static Test: Horizontal Loads Transmitted to the Web (kips)	Values Correlated from Static Test Assuming 7.3 kip-foot Moment: Maximum Stress Near Bolt Hole (ksi)	Values Correlated from Static Test Assuming 7.3 kip-foot Moment: Maximum Stress Near Bottom of the Bracket (ksi)	Values Correlated from Static Test Assuming 7.3 kip-foot Moment: Bracket Deflection 12 inches from the Web (inches)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
A	6	17	23.0	19.0	7.3	5.12	23.0	19.0	0.142
B	4	17	19.5	18.4	7.0	4.94	20.1	18.6	0.099
C	21	35	13.9	10.1	6.1	2.1	16.8	10.5	0.048
D	21	17	9.4	7.0	7.2	5.08	9.6	7.95	0.064

horizontal loads applied. Since the horizontal loads are inversely proportional to the bracket depth, a deep bracket reduces both stresses and deflections. If a deep bracket is extended to the bottom flange, the horizontal load at the bottom of the bracket is transmitted directly to the flange resulting in further reductions of stresses and deflections. Laboratory test results showed similar reductions of stresses and deflections when a shallow bracket was replaced by a deep bracket.

3. Effect of Backup Angles

The steel backup angles, when applied to the full depth of the bridge girder web, greatly reduced both stresses and deflections. The angles distribute the horizontal loads over larger areas and provide lateral support for the girder web. Similar results were noted in the laboratory test.

4. Effect of Finishing Machine

For all brackets tested, the finishing machine produced approximately 80 per cent of the total construction stresses. The loads produced by the slow moving finishing machine can be considered as static loads.

B. Conclusions

The following conclusions were derived from the test results:

1. For the bridge tested and the construction loads applied, the shallow bracket mounted six inches from the nearest

stiffener, as recommended by the South Dakota Department of Highways, produced web stresses in excess of those allowable for A-36 steel. (4) Had this bracket been mounted more than six inches from a stiffener, permanent web deformations could have resulted. The shallow bracket mounted four inches from a stiffener reduced stresses to within allowable limits. However, such spacing does not allow standardization of formwork because stiffener spacing is not the same for all bridges.

Therefore it is recommended that one of the following brackets be adopted:

a. A bracket having an adjustable depth which could be used on any depth web.

b. A shallow bracket, as used on the test bridge, with steel backup angles applied to the full depth of the girder web.

Either of these brackets could be mounted on any depth web at intervals desireable for forming and erection.

2. Because stresses and deflections are proportional to the applied loads, by using a lighter finishing machine, the shallow brackets would adequately carry the reduced loads.

3. As some deflections occur regardless of the bracket used, it is recommended that the corresponding decrease in deck thickness over the girders be compensated for by either adjusting the paving machine height or adjusting the

bracket to maintain the required deck thickness. Table 2 could be used as a guide to the designer for determining anticipated deflections.

4. Reductions of web stresses and deflections can be realized by placing any depth bracket such that it is bearing against the bottom flange of the girder.

C. Recommended Areas of Future Study

It is recommended that a study be conducted on brackets mounted on webs having no stiffeners. Such a study would aid bridge designers in determining bracket placement for bridges designed using the newly recommended Load Factor Method of analysis.

BIBLIOGRAPHY

1. American Association of State Highway Officials, Tentative Criteria For Load Factor Design of Steel Highway Bridges, American Association of State Highway Officials, Washington, D. C., 1969.

2. State Highway Commission of Kansas, Standard Specifications for State Road and Bridge Construction, State Highway Commission of Kansas, Topeka, Kansas, 1966.

3. Preheim, R. F., "Bridge Girder Webs Subjected to Horizontal Loads," M. S. Thesis in Civil Engineering, South Dakota State University, Brookings, South Dakota, 1970. (Unpublished)

4. American Association of State Highway Officials, Standard Specifications for Highway Bridges, American Association of State Highway Officials, Washington, D. C., 1965.

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(1HO,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
   /*
   /+
```