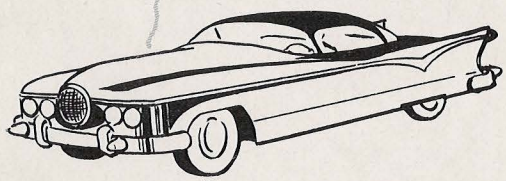
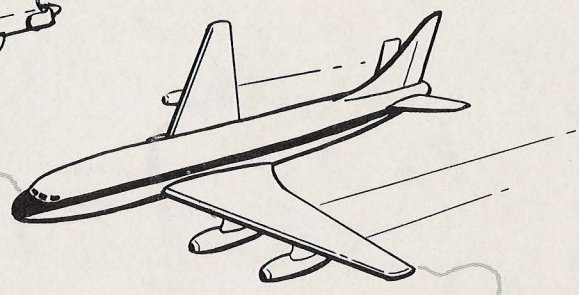
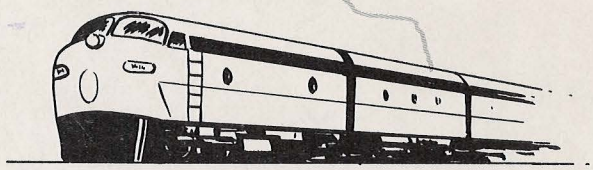
FULL SCALE FIELD TESTS OF A PRESTRESSED COMPOSITE I-BEAM BRIDGE

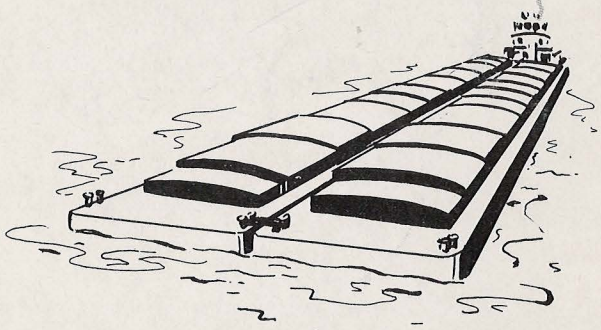
FINAL REPORT

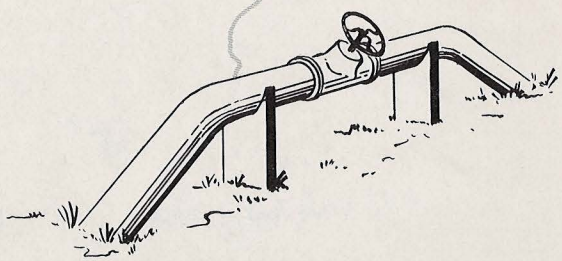
PROJECT RP-10











Texas Transportation Institute

TEXAS TRANSPORTATION INSTITUTE
TEXAS A. AND M. COLLEGE SYSTEM
College Station, Texas

A Report

on

FULL SCALE FIELD TESTS OF A PRESTRESSED COMPOSITE I-BEAM BRIDGE

By

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District Bridge Engineer (Retired)
Texas Highway Department, District 9

Final Report from Research Project RP-20
Project 52097(1), Control 513-1
Coryell County
Highway No. State 236
State of Texas

Prepared for the Research Committee

of the

TEXAS HIGHWAY DEPARTMENT

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PREFACE

These tests were proposed by John M. Graham and were approved by the Texas State Highway Department Research Advisory Committee. The actual tests were made by the field forces of District No. 9 Bridge office. The Texas Transportation Institute; Agricultural and Mechanical College of Texas; College Station, Texas, was in general charge of the tests. Henson K. Stephenson, Research Engineer; and Truman R. Jones, Jr., Associate Research Engineer, of the Institute contributed their advice and help in setting up the tests.

The actual field tests were supervised by Franklyn R. Hitz, Draftsman. Computations, compilations and checking were accomplished by William M. Wood, Senior Engineering Assistant; E. L. Hardeman, Senior Designing Engineer; and Larry G. Walker, Senior Designing Engineer.

The Texas Transportation Institute furnished the Leeds-Northrup Precision Potentiometer with copper-constantan thermocouple for temperature results on the steel beams, concrete slab and standard bar. Precision thermometers were furnished by the Texas State Highway Department Laboratory. The Department of Civil Engineering; University of Texas; Austin, Texas, furnished the 20 inch H. O. Berry strain gage and standard bar.

FULL SCALE FIELD TESTS OF A PRESTRESSED COMPOSITE I-BEAM BRIDGE

Previous Design Experience:

On Project M 1308-1-5, Control 1308-1-5, Bell County, Highway No. F.M. 1123, State of Texas, 2-40 foot simple I-Beam spans were washed out by large drift in the flood of May 1955. These 2 spans were in the center of the bridge which consisted of a series of 40 foot I-Beam spans. Investigation showed an 80 foot span was required to handle the drift. To keep adequate head room between high water and the bottom of the steel, a shallow design was required. Ten salvaged 30" wide flange beams at 108 pounds per foot were used. Five of the 39'-8" long beams were cut in half and butt welded at either end of five other 39'-8" beams. This made available five lines of 30" wide flange steel beams for the 80 foot span. The "Prestressed Composite I-Beam" design was used. The span was built in placing during July & August 1955. Actual initial dead load and final live load deflections checked the theoretical deflections. The depth of steel beam to span ratio by the use of these 30" wide flange beams was about 1 to 31. No excessive vibration was noted. Actual span length, from center line of bearing to center line of bearing, was 78.5 feet. With this experience record as a background, the Leon River Bridge on State Highway 236 in Coryell County was designed as a "Prestressed Composite I-Beam" structure and was instrumented to collect research information.

Description of the Design:

"Prestressed Composite I-Beam Design" in this case is simply jacking up the individual steel beams a predetermined amount at the center line of the spans and holding the ends of the steel beams down with anchor bolts. The jacking up of the steel beams at the center line of the span results in a tension stress in the top flange of the steel beams and a compression stress in the bottom flange of the steel beams. Shear devices are welded in place on the ground with the steel beams in an unstrained position. The steel beams are not jacked up until all forms for the concrete slab are in place and the reinforcing steel is in place in these forms. The weight of the concrete slab and forms adds more tension in the top flange and more compression in the bottom flange of the steel beams, due to this dead load weight acting on the steel beams while they are temporarily shored up at the center of the span.

After the concrete slab has attained about 85 percent of its 28 day strength, the shores are gradually released at the rate of one-half inch per hour. The composite I-Beam span then acts as a simple span. Due to the fact that the beams were jacked up at the center of the span while the concrete gained strength, the entire composite cross section is active in carrying the dead load.

Since the top flange of the steel beam is below the neutral axis of the composite section, the simple composite span stress is an increase in the tension stress present in the top flange, due to the jacking up of the beam. In the bottom flange, this simple composite span stress is also tension and is larger

than the initial compression stress due to the prestressing process. This reversal of stress in the bottom flange means an equivalent tension working stress in the bottom flange of about 30,000 psi, instead of the 18,000 psi normally used. There is a considerable reduction in the final dead load top flange tension stress, due to shrinkage and plastic flow in the concrete slab. The shrinkage and plastic flow effect on the bottom flange is the opposite, or a tension increase; but, in magnitude only about one-fourth the effect in the top flange.

Purposes of the Tests:

The purposes of these tests were, (1) to check the theoretical design calculated stresses and deflections with the actual observed stresses and deflections, and (2) to determine the effect of shrinkage and plastic flow of the concrete slab on the prestressed composite I-beam spans. During the final dead load test period, corresponding morning and afternoon readings for stresses and deflections were made to determine the effect on stresses and deflections due to heat storage in the concrete during the day.

Description of the Old Existing Bridge:

The problem which led to this test bridge and the use of the "Prestressed Composite I-Beam" span was the rebuilding of the Leon River Bridge on State Highway 236 in Coryell County, Texas. The existing structure was composed of two 50' simple spans, containing eight 21" CB 59# beams with timber flooring and two short timber approach spans. It was decided that by using a "Prestressed Composite I-Beam" design, only four of the existing 21" CB 59# beams would be required in each new 50' simple span; thereby, leaving four beams from each of the two existing 50' spans to build two new 50' "Prestressed Composite I-Beam" approach spans.

In this case, the three existing interior concrete bents could be reused and only two new concrete abutment bents would be required to complete the four 50' spans and, thereby, solve the problem of the new bridge layout.

Description of the New Test Bridge:

The new bridge layout is shown in Figure 1. The bridge was designed for H-15-44 loads (AASHO Specifications - 1953), 26' roadway, 9" curbs, and an effective span length of 48'-6". The 21" CB 59# beams were placed at 7'-1", centers as shown, in the typical half-section of Figure 2. Initial design calculations indicated a need for the 5" x 3/8" cover plate on the bottom flange. The typical shear devices, shown in Figure 2, were designed to resist bending and shear, and were placed at an angle to resist any pulling apart effect between the beam and the concrete when the shoring jacks were removed. Bending stresses in the shear device itself were within the design allowable unit stresses. The bearing of the concrete on the exposed flanges of the shear devices were well within the unit bearing stress allowed by specifications. No "slip" was anticipated.

Construction and Testing Procedure:

The construction and testing procedure parallel very closely in the early stages, namely from Stage #1 to Stage #8; therefore, for convenience, the outline of procedure to follow will be given in terms of tests procedure and the construction procedure which corresponds will be explained. All tests, except as noted, were performed between the hours of 4:00 to 7:00 a.m. These early morning hours, besides being convenient for the contractor, were chosen for the testing period because the air temperature varies the least during this period. The construction and testing of this bridge began in February 1957. The bridge was completed in October 1957, and opened for use by the public; however, testing was not concluded until September 1958.

The test stage procedure was as follows: (See Figure 3)

Stage #1. All beams were blocked level by eyesight on the ground in an unstrained position and strain gage measurements were made on test points cl, cr, dl and dr, for beams 2B and 2C only. This is the initial reference stage for beams in Span #2.

Stage #2. The cover plates were welded on all beams and strain gage measurements were made on test points cl, cr, dl and dr, for beams 2B and 2C only. Beams are still in an unstrained position.

Stage #3. The shear connectors were welded on all beams, and strain gage measurements were made on test points cl, cr, dl and dr, for all beams. This is the initial reference stage for all beams. Beams are still in an unstrained position.

Stage #4. All beams were placed and the diaphragms were welded in place, then all forms were set and the reinforcing steel was placed in these forms. With the forms and reinforcing steel in place, all beams were jacked up the desired amounts and strain gage measurements were made on test points cr, cl, dl and dr, for all beams.

Stage #5. Steel test plates for measuring strains for the bottom of the concrete slab were placed about 20" apart against the bottom forms before the slab was poured. Before the concrete slab had taken its initial set, similar test plateswere placed flush with the top of the slab. When the concrete slab had set with forms in place, Stage #5 readings for test points a, cl, dl and dr were contemplated, but were not taken.

Stage #6. After the forms were removed and just before the jacks were removed, strain gage measurements were made on test points, a, b, cl, cr, dl and dr, for all beams. The test stage is a combination of Stages #5, #6, and #7 and is referred to as Stage #6. Note that this is the initial reference stage for all concrete test points.

Stage #8. The jacks were removed from all beams and thus completing the construction of the bridge except for placement of the railing. On the morning following removal of the jacks, strain gage measurements were made on all test points on all beams.

Since no additional loads of any significance are placed on the beams beyond Stage #8, the following test stages were made for the purpose of verifying shrinkage, plastic flow and live load stresses on the beams.

Stage #9. Dead load strain gage measurements were made on all test points approximately one (1) month after shoring was removed for all beams tested in Span #2 and Span #3 only. The reason for not using this stage on Spans #1 and #4 was because these spans were under flood water for 22 days.

Stage #10. Dead load strain gage measurements were made on all test points appriximately three (3) months after shoring was removed for all beams tested.

Stage #11. Dead load strain gage measurements were made on all test points approximately six (6) months after shoring was removed for all beams tested.

Stage #12. Dead load strain gage measurements were made on all test points approximately twelve (12) months after shoring was removed for all beams tested.

Stage #13. Live Load (test truck) plus dead load strain gage measurements were made on all test points approximately twelve (12) months after shoring was removed for all beams tested.

Stage #12 and #13 readings were repeated on two separate days as follows:

Mornings (Stages #12A, #12B, #13A, #13B on all spans). Stage #12A on one morning and Stage #12B the following morning.

Afternoons (Stages #12C and #12D dead load gage readings on Spans #1 and #4 only). Note that Stages #12C and #12D readings are the afternoon readings which were taken to check the heat storage effect on stresses and deflections).

All corresponding readings checked fairly close, and readings on the opposite sides of the steel beam flange were averaged to reduce possible human error in instrumentation.

Figure 3 shows the location of all test points and gives an explanation of the type of tests used in the testing of this bridge. Reference made to any test point stress in this report will be as shown in the following examples:

1. A stress at test point "d" on top of the bottom flange of Span #2 and beam B will be referred to as 2-B-d. Notations, "dr" and "dl", will not be used in this report, as the results of these tests are based on the averages of all stresses where readings were taken on the left and right sides of the steel beam. 2-C-a would indicate the stress on top of the slab of beam C in Span #2.

2. 4-C-c would indicate the stress on the bottom of the top flange of beam C in Span #4.

Testing Equipment and Test Calculation Procedure:

All test points consisted of two very small holes, approximately 20" apart, drilled in the steel beam or in steel plates anchored in the concrete. A 20" H. O. Berry strain gage with an Ames dial reading in .001 inches, mounted on the pivot arm was used to measure the strain movement between the test holes at the different test stages, as referred to the initial unstrained readings. Gage readings on a standard bar were taken corresponding to each test stage and for each test point, for the purpose of determining the strain effect due to temperature differences measured at the initial stage reading.

Since test beams and concrete were not at the same temperature as the standard bar when gage readings were taken, it was necessary to determine this difference of temperature and make adjustments to the beam and concrete strain measurements. In each case, the beam and concrete temperatures were referred to the standard bar temperatures, that is, adjusted to read the same in temperature as the standard bar. The coefficient of expansion for steel as measured on the Ames dial was 0.6 divisions for 1° F difference in temperature. Therefore, if a beam temperature measured 2° F higher than the standard bar, the test beam strain measurement was increased 0.6 x 2 = 1.2 divisions. If the beam temperature was lower by 2° F, then the test beam strain measurement was decreased by 1.2 divisions. For these tests, the coefficient of expansion for concrete was assumed to be the same as steel, and similar adjustments in strain measurements were made for differences in concrete temperatures (at the rate of 0.6 divisions for 1° F).

Temperatures of the beam, concrete and the standard bar, were measured with a lead shielded thermocouple of copper and constantan which was clamped on the test beam or concrete or standard bar (see Figure 5). Measurements in the temperature circuit (Figure 5) were made on a Leeds-Northrup precision potentiometer. The reference temperature for the circuit was measured with a precision centigrade thermometer in water in a thermos bottle. The reference centigrade thermometer was calibrated in 0.5° C, and was the same type of thermometer used to measure air temperatures.

Temperatures were taken or measured corresponding to strain measurements for each test point and also corresponding to a standard bar measurement as closely as time permitted. It took approximately two to three minutes for the shielded thermocouple to reach the same temperature as the member being tested.

After the test point strain was adjusted for temperature difference, final strains were computed by taking the difference of the adjusted strains at any test stage and the initial unstrained stage. This difference was then referred to the corresponding standard bar strain difference over the same stage period and added or subtracted, as signs indicated. Like signs being added and unlike signs being subtracted for the final strain measurement, final stress was determined by the equation:

$$\text{Stress} = \text{strain} \times E_s \times 10^{-5} \text{ (for steel).}$$
$$\text{Stress} = \text{strain} \times E_c \times 10^{-5} = \text{strain} \times \frac{E_s}{10} \times 10^{-5} \text{ (for concrete).}$$

Note that strains were in divisions of the Ames dial in the above equation. The Ames dial, however, reads 1 ÷ 1000 inches per inch for each division and should be multiplied by: 1 ÷ pivot arm distance x gage length = 1 ÷ 5 x 20, therefore, strains in divisions would be multiplied by 1 ÷ 1000 x 1 ÷ 100 = 1 ÷ 10000 10^{-5}, to get strain in inches per inch. The resulting stresses for all stages for test points "a", "c", "d" are plotted against time in graphs A through G of this report.

E_s was taken as 29 x 10^6 psi for purposes of correlating the field test data with the theoretical data based on AASHO Specifications. However, an average E_s value determined experimentally from test specimens taken from the beam ends cut on the job was 31 x 10^6 psi. For concrete stresses a value of n equals 10 was used. At a later date, tests were made to determine the modulus of elasticity for the concrete from beams cast on the job. This E_c at one year of age was about 5.2 x 10^6. This results in an "n" factor of 6.

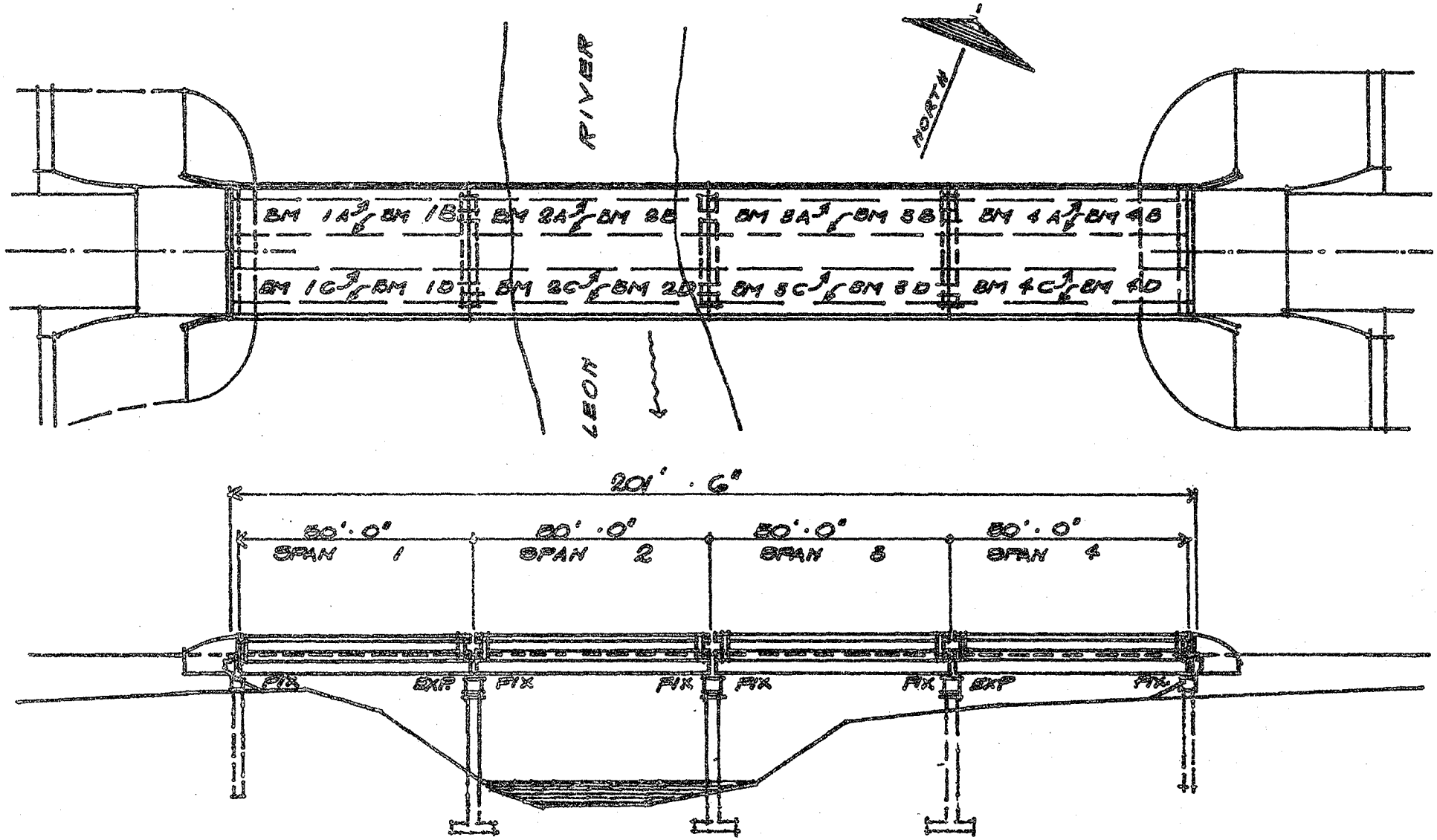
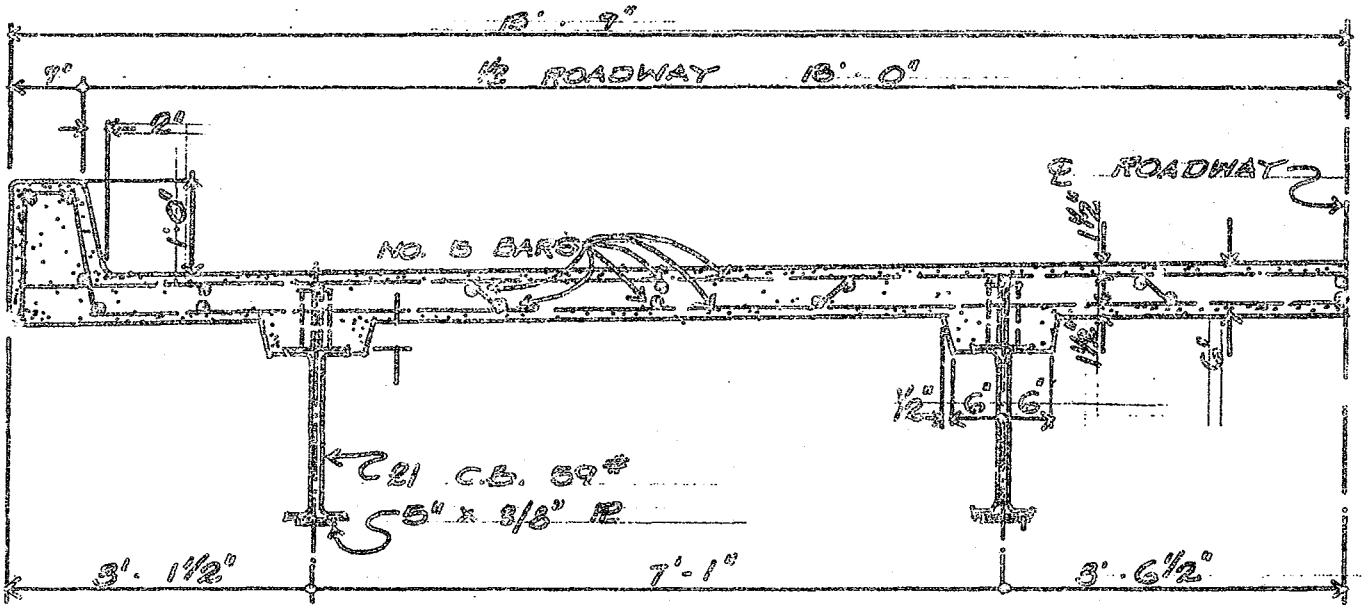
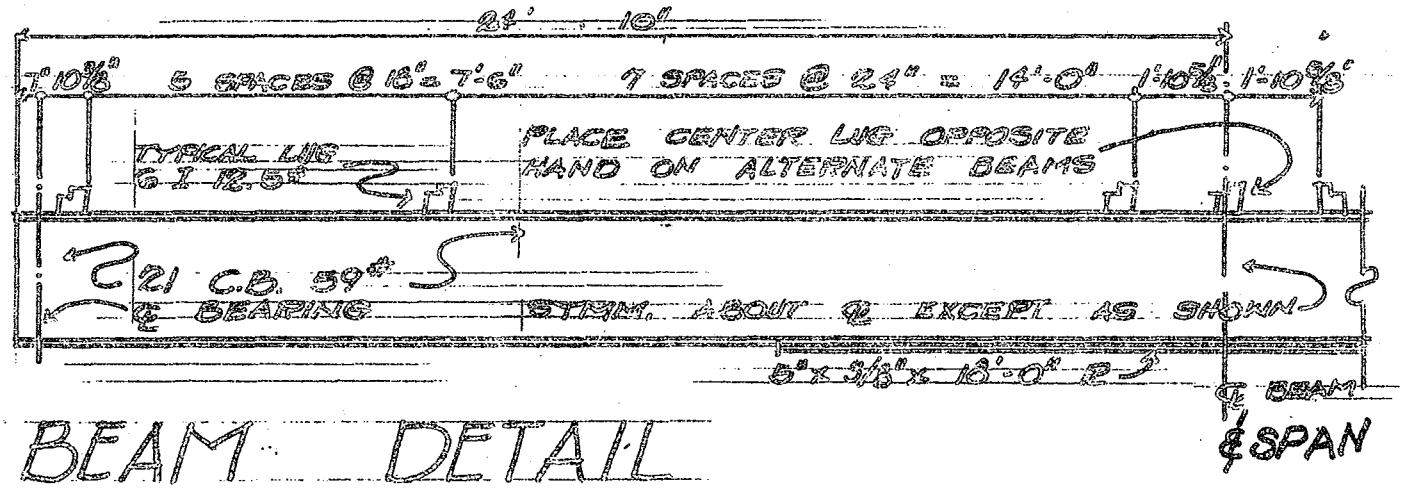


FIGURE 1 BRIDGE LAYOUT

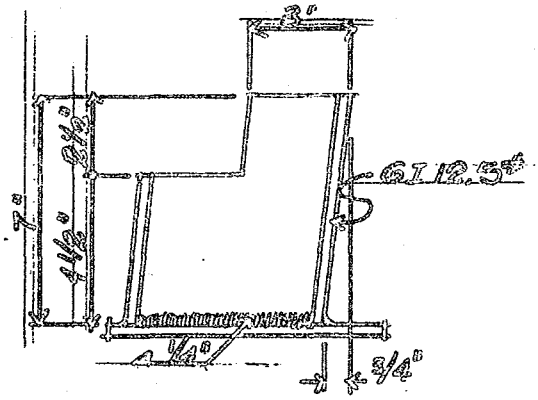
LEON RIVER BRIDGE
CORYELL COUNTY
SH 236 RP-57-10



TYPICAL HALF SECTION

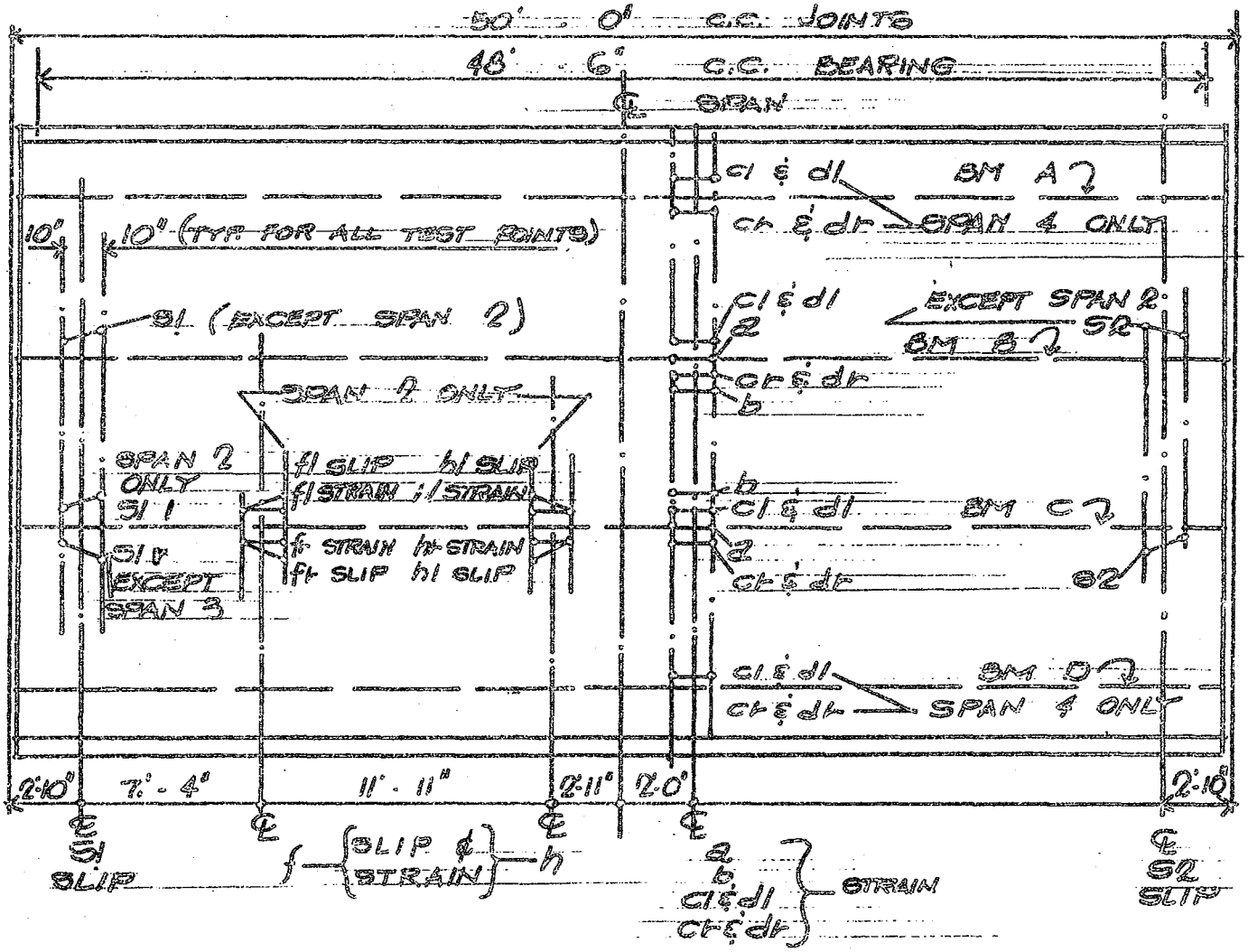


BEAM DETAIL

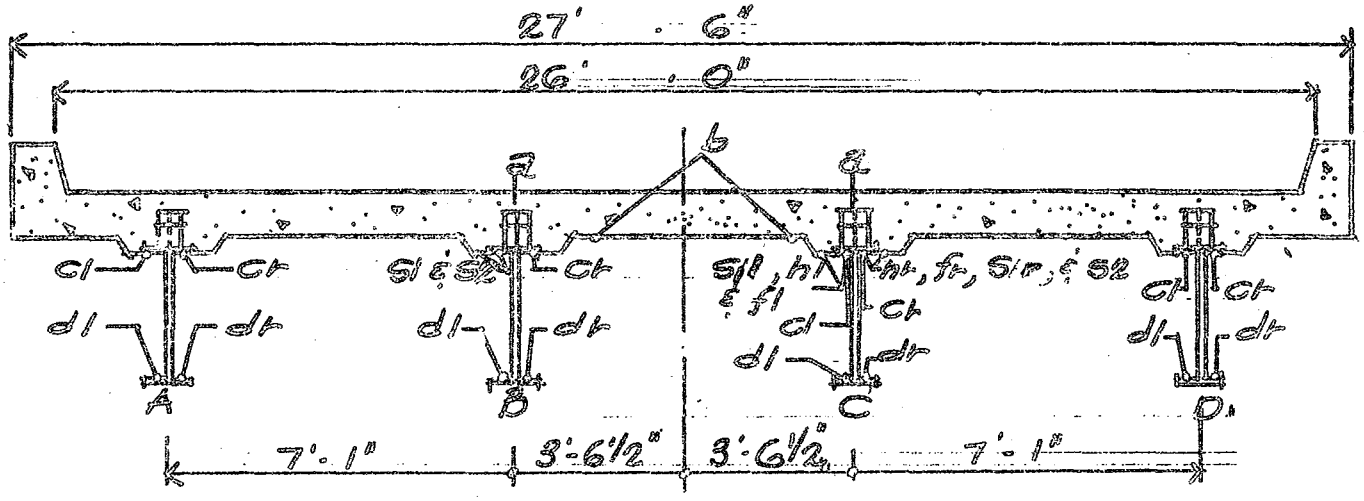


SHEAR LUG DETAIL

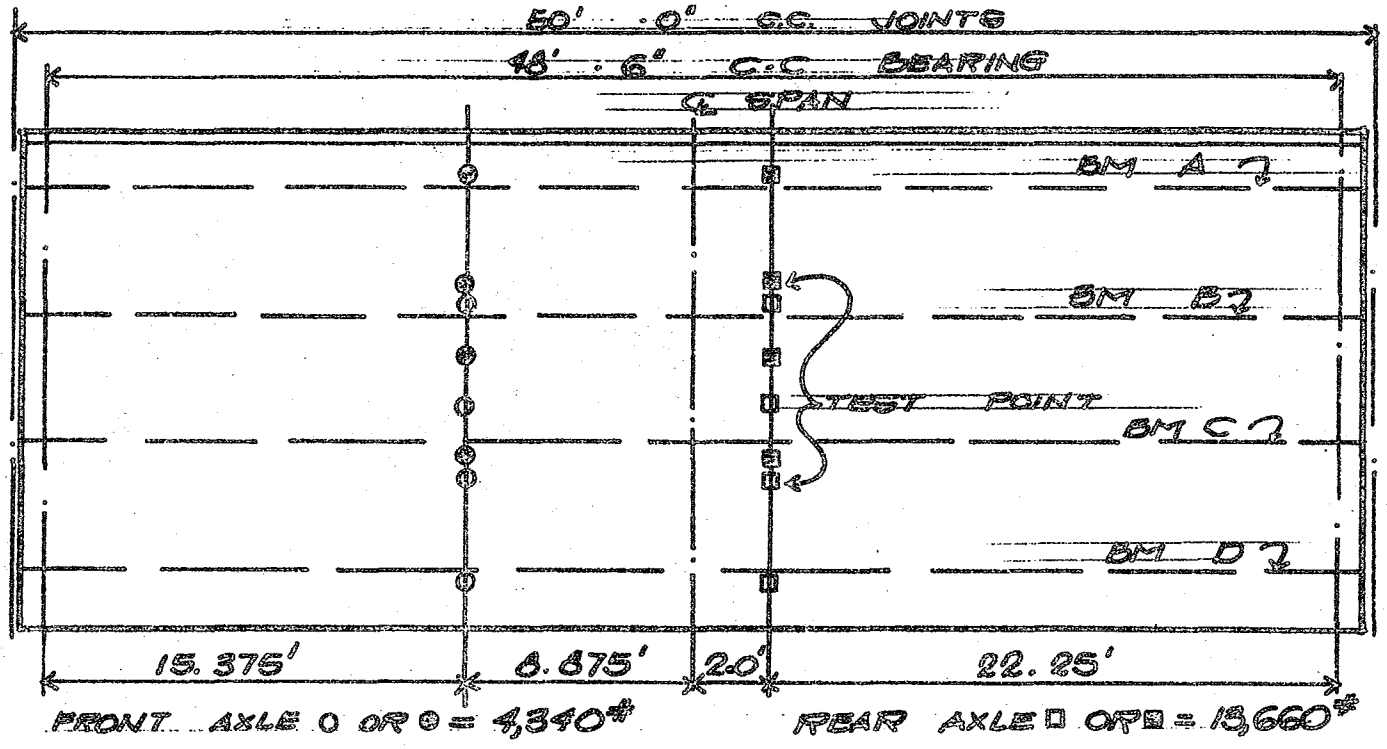
FIGURE 2



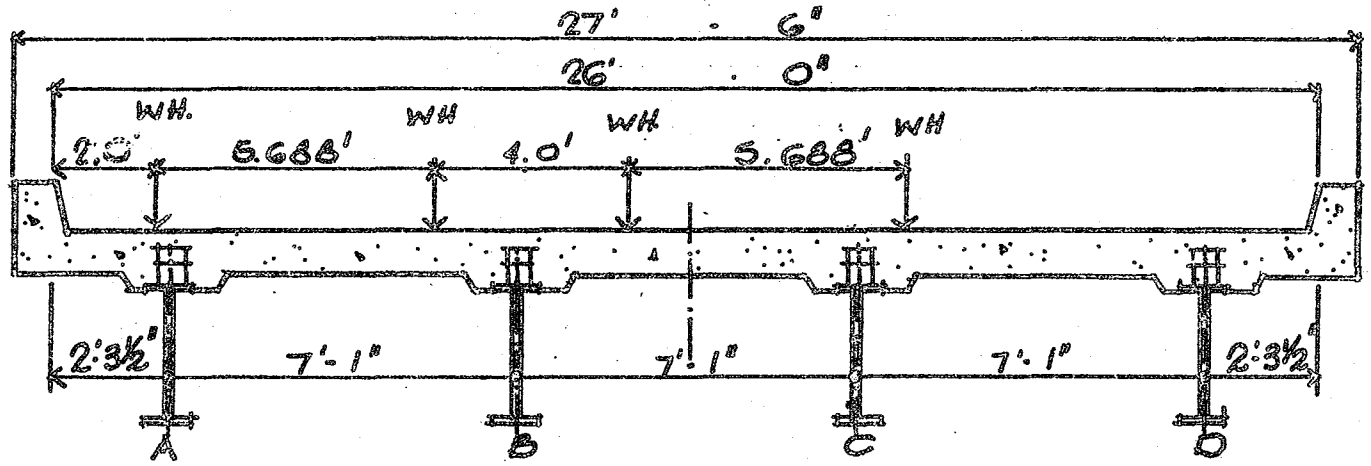
TYPICAL SCHEMATIC TEST POINT LAYOUT
(EXCEPT AS NOTED)



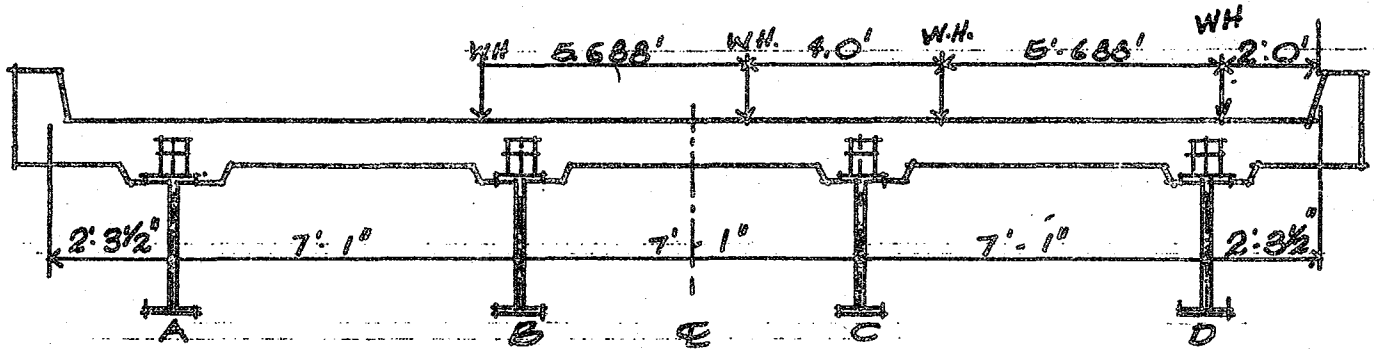
TYPICAL SCHEMATIC SECTION
(SHOWING TEST POINT LOCATION) FIGURE 3



TYPICAL TEST TRUCK LOADING PLAN

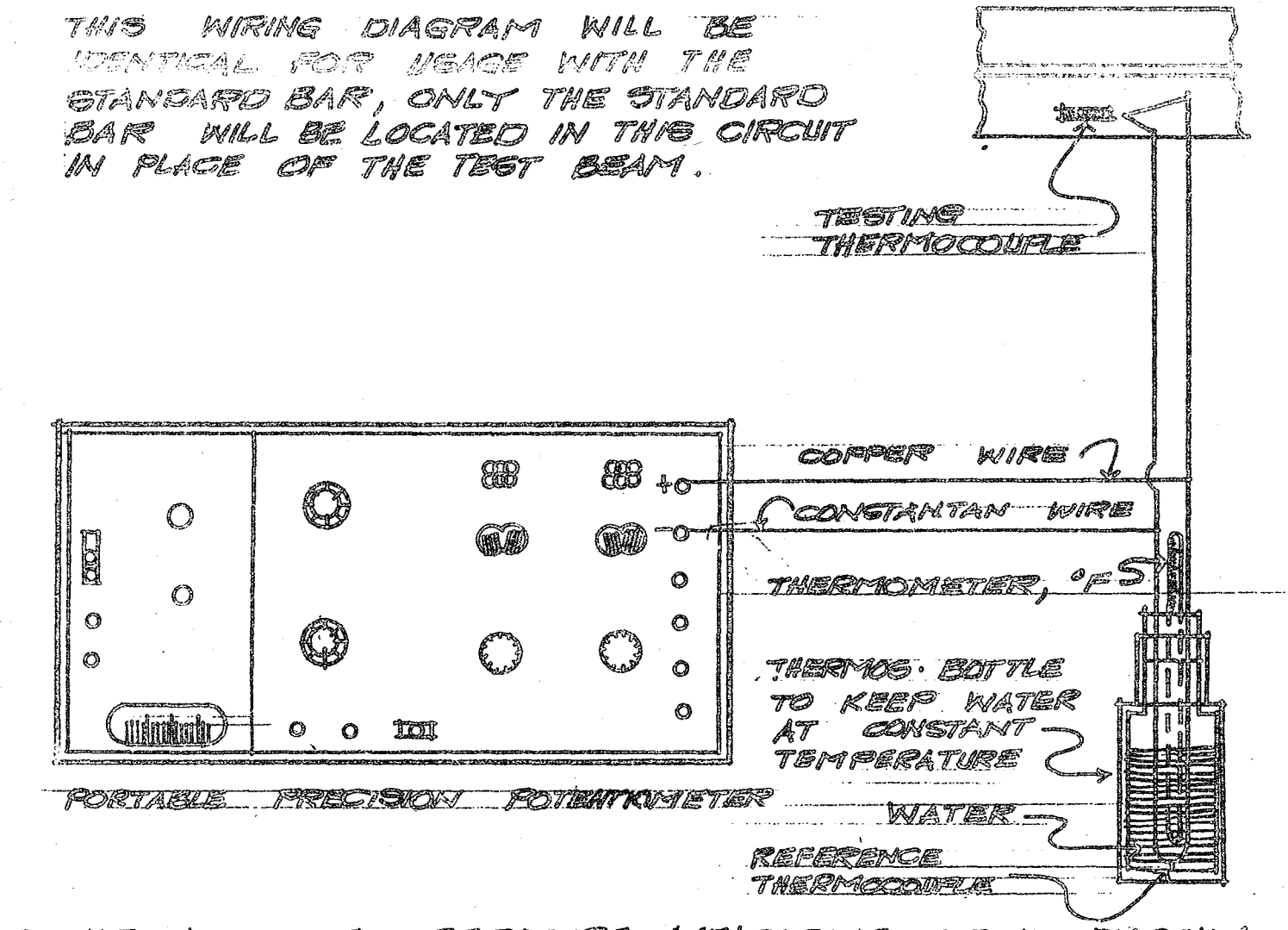


TEST TRUCK POSITION FOR BEAMS A & B

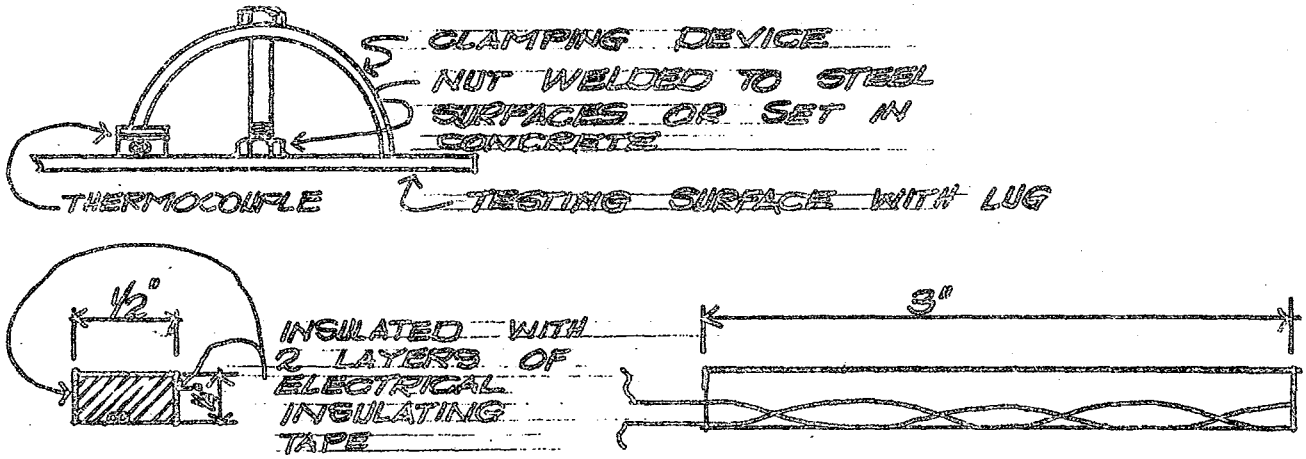


TEST TRUCK POSITION FOR BEAMS C & D

FIGURE 4

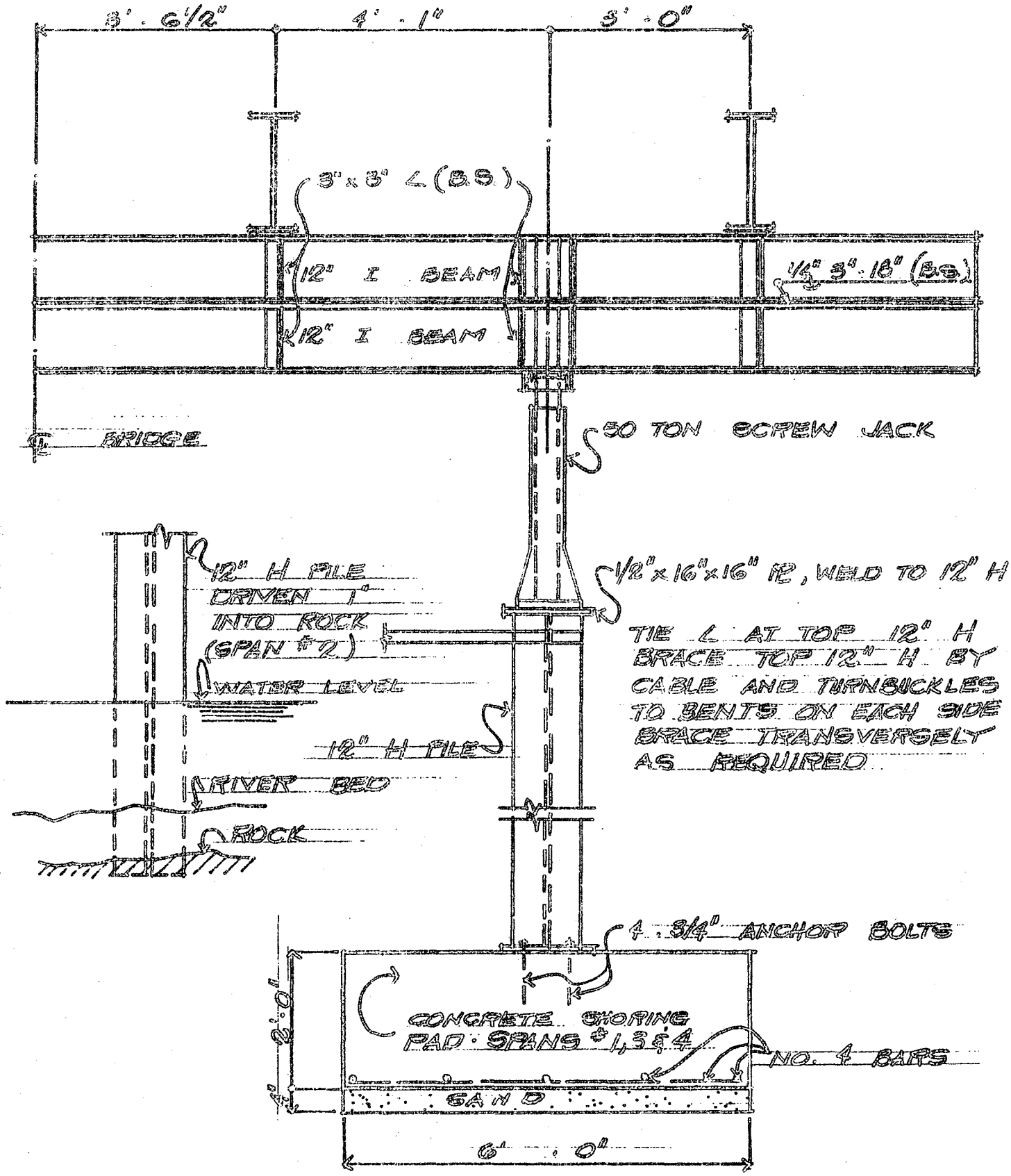


SCHEMATIC TEMPERATURE MEASURING CIRCUIT DIAGRAM



LEAD SHIELDED THERMOCOUPLE

FIGURE 5



TYPICAL SHORING DETAILS

FIGURE 6

Possible Stress Measurement Errors:

1. Concrete slab: 1 division error in reading the Berry gage equals 29 psi; 3° F temperature error equals 51 psi: Total error possible 80 psi plus or minus.

2. Steel surfaces: 1 division error in reading the Berry gage equals 290 psi; 2° F temperature error equals 348 psi error: total possible error 638 psi plus or minus.

These errors may be additive or may tend to cancel out in readings at bottom of steel beam, at top of steel beam, at bottom of concrete slab and at top of concrete slab. These errors could be additive from one stress stage to the other stress stages.

Possible Errors in Theoretical Stresses:

1. Care was used to record the right thickness of concrete slabs. A comparison of the measured stress curves and the theoretical stress curves indicates our recorded net amount of upward shoring after settlement and readjustment could be in error. Hence, our recorded thickness of slab could be in error causing our calculated stresses to be in error: for one quarter inch error in thickness of slab: top flange steel stresses would be 260 psi in error and bottom flange steel stresses would be 63 psi in error.

2. The distribution of dead load to the steel beams was unknown. We tabulated stresses due to equal distribution, due to moment distribution and due to the relative composite moment of inertia of each of the four beams. Since the theoretical stress calculations due to equal distribution seemed to check closer to measured stresses, it was decided to use this distribution for compiling our theoretical curves. The error due to distribution is unknown. Maximum minus dead load stress errors for the 2 interior beams would be as follows: top of slab 75 psi; top of I-beams 200 psi to 300 psi; bottom of I-beams 2050 psi to 2150 psi.

3. Theoretical stresses in the I-beams, due to the fact that the actual detailed measurements of the I-beams varied from the handbook values for their full length and varied from beam to beam, may cause an error in the shored position stresses as much as 2% $\pm$. This error would occur in the top and bottom stresses of the I-beams acting without composite action. The final composite stresses in the I-beams are directly affected by this error by the larger or smaller original prestressed stresses in the I-beams.

4. Errors due to welding the cover plate and welding the shear devices to the I-beams: Beams 2B and 2C were the only beams tested for stresses due to these 2 operations. Results: welding cover plate: Beam 2B: 28 psi tension in top flange and 3016 psi compression in bottom flange;

beam 2C: 174 psi tension in top flange and 2088 psi compression in bottom flange. After shear devices are welded on: Beam 2B: 6598 psi compression in top flange and 3611 psi compression in bottom flange. Beam 2C: 7772 psi compression in top flange and 2784 psi compression in bottom flange. These stresses are on the safe side and some theoretical advantages could be taken of these stresses. Our stress curves all assume zero stress after the welding is accomplished.

5. Concrete slab shrinkage plus creep has an effect on all stresses. All theoretical curves are based on "n" equals 10 and a shrinkage plus creep factor of 0.0002. From actual tests on I-beams and concrete, we know that "n" equals 6 plus or minus. See appendix item "Shrinkage plus Creep." The error due to using "n" equals 10 and shrinkage plus creep factor equals 0.0002 is analyzed in this item.

6. Errors in span 4 due to the following circumstances are unknown: placing of concrete in the outside curb caused a deflection in the outside overhang forms and a slight twist inthe outside I-beam. This was due to inadequate struts across the bottom flanges of the I-beams. Corrective measures were taken by shoring the outside overhang forms from the ground and trying to measure the effective amount of upward shoring at the jacks and at the outside shores.

7. Errors due to the fact that our recorded amounts of upward net shoring of the I-beams after settlement and the readjustment of the jacks was in error in some cases: A comparison of the theoretical and measured stress curves indicates there were some errors in recording the amount of net shoring. Composite section stresses due to an error of plus one-eighth of an inch in the amount of shoring are: top of slab 21 psi compression; top flange of beam 1300 psi tension and bottom flange of beam 420 psi compression.

8. Beam A and B in span 2 were let down with a jerk some seven-eighths of an inch when the jack on that side of the roadway moved downward without any warning. An attempt was made to bring this jack up to the same level as the jack on the other side, and lowering of both jacks was continued. All this occured during the lowering of the jacked up shoring.

9. On Figure 1 please note the fixed and expansion ends of the spans as built. On span 1 and span 4 the amount of temperature restraint to move the I-beam over the expansion bearing shoe was theoretically figured as 8690 pounds. This force would be on the bottom flange of the I-beams and would tend to increase or decrease all stresses involved according to whether or not the force was producing tension or compression in the bottom flange. According to our details of bearing shoes, the anchor bolt holes in each fixed end would allow a movement of one quarter inch without producing more than the 8690 pound force. Therefore, all spans would have the same temperature restraining force for a drop or rise in temperature. In the appendix, see the item, "Stresses Due to Temperature Changes." In this item the results of increases and decreases in temperatures is analyzed between the various stages of the tests on any one beam.

Explanation of Curves:

This family of curves shows the calculated stresses from test stage to test stage, varying in the same manner as the measured stresses. One distinct variation is the effect of wet and dry weather on the shrinkage and swelling of the concrete slab, which we will comment on later.

Curves (A), (B), (C), and (D):

Curves (A) and (C) are based on field measurement stresses, while Curves (B) and (D) are based on theoretical stress from our calculations. All of these stresses originate at stage #4, when the beams are first jacked up.

Curves (E) and (G):

All of these stresses originate from stage #8, which is the morning after the shoring was removed. Field stress measurements shown on curve (E) did not take account of the fact that the Berry gage readings included shortening, due to compressive stress plus shortening due to shrinkage plus creep. The shrinkage plus creep shortening would be the equivalent of 580 psi for a shrinkage plus creep factor of 0.0002 inches per inch. Values of theoretical stresses shown on curve (G) were increased 580 psi and placed on curve (e), so as to correlate with field measurement stresses.

Curve (H):

These curves show center line of beam deflections with respect to time in months. Zero time is at stage #8 immediately after shoring was removed.

Weather Conditions for Each Span:

On curve (E) a very definite effect is shown of the weather conditions as listed elsewhere, by the effect of alternate shrinkage and swelling of the concrete slab. On curve (A) alternate shrinkage and swelling has less effect on stresses, and on curve (C) a very definite effect of alternate shrinkage and swelling of slab is shown.

1. Span 1: from 0.5 month to 0.2 month before shoring was removed, heavy rain and floods (slab not submerged); from 0.2 month before shoring was removed to 0.1 month after shoring was removed, cloudy, cool, scattered light rain, floods receding; from 0.1 month to 1.2 months after shoring was removed, heavy rain and floods (slab submerged 22 days); from 1.2 months to 2.6 months, hot, dry and clear, very light rains; from 2.6 months to 3.6 months, hot, dry and clear; from 3.6 months to 5.5 months, fairly heavy rain fall, warm to mild; from 5.5 months to 5.8 months, heavy rain, mild, continuous cloudy; from 5.8 months to 11. 7 months, scattered heavy rains, mild to warm; from 11.7 months to 12.1 months, dry, warm, and clear.

2. Span 2: from 0.4 month to 0.1 month before shoring was removed, hot, dry, clear; from 0.1 month before shoring was removed to 0.1 month after shoring was removed, hot, dry and clear; from 0.1 month tc 0.7 month after shoring was removed, scattered heavy rains, hot to warm; from 0.7 month to 1.1 months, warm, dry and clear; from 1.1 months to 2.8 months, scattered heavy rains, warm to mild; from 2.8 months to 3.1 months, warm, dry and clear; from 3.1 months to 5.9 months, cold to mild, scattered heavy rains; from 5.9 months to 6.2 months, clear to partly cloudy, cool to mild; from 6.2 months to 11.9 months, scattered heavy rains warm to hot; from 11.9 months to 12.0 months, hot, dry, clear.

3. Span 3: from 0.4 month before shoring was removed to 0.9 month after shoring was removed, hot, dry, clear; from 0.9 month to 2.8 months after shoring was removed, hot to warm, scattered heavy rains; from 2.8 months to 3.1 months, warm, heavy rains, continuous cloudy; from 3.1 months to 6.5 months, cold to mild, scattered heavy rains; from 6.5 months to 6.8 months, clear to partly cloudy, cool to mild ; from 6.8 months to 12.2 months, scattered heavy rains, warm to hot; from 12.2 months to 12.6 months, hot, dry, clear.

4. Span 4: from 0.5 month to 0.2 month before shoring was removed, heavy rains and floods (slab not submerged); from 0.2 month before shoring was removed to 0.1 month after shoring was removed, cloudy, cool, scattered light rain, floods receding; from 0.1 month to 1.2 months after shoring was removed, heavy rain and floods (slab submerged 22 days); from 1.2 months to 2.8 months, hot, dry and clear, very light rains; from 2.8 months to 5.6 months, fairly heavy rain fall, warm to mild; from 5.6 months to 5.9 months, heavy rain, mild, continuous cloudy; from 5.9 months to 11.8 months, scattered heavy rains, mild to warm; from 11.8 months to 12.1 months, dry, warm, and clear.

STRESS VS. TIME FOR TEST PT. "d"
AVG. STRESSES FOR PTS. d1 & d1
ON TOP OF BOTTOM FLANGE

LEON RIVER BRIDGE
CORTELL CO. S.H. 236
FP-57-10

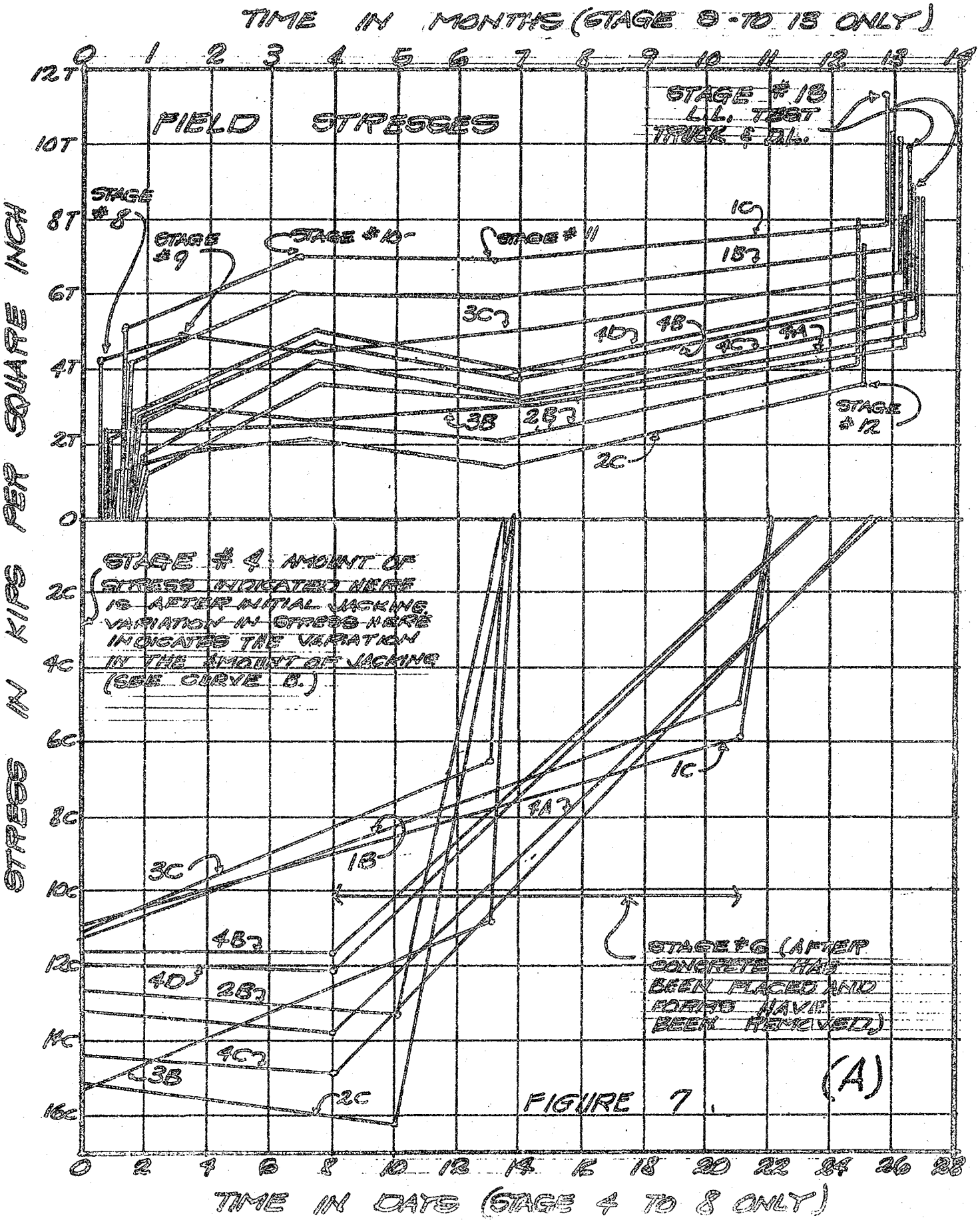


FIGURE 7 (A)

STRESS VS. TIME FOR TEST PT. "d" LEON RIVER BRIDGE
AVG. STRESSES FOR PTS. d-r & d-l CORYELL CO. SH 236
ON TOP OF BOTTOM FLANGE RP-57-10

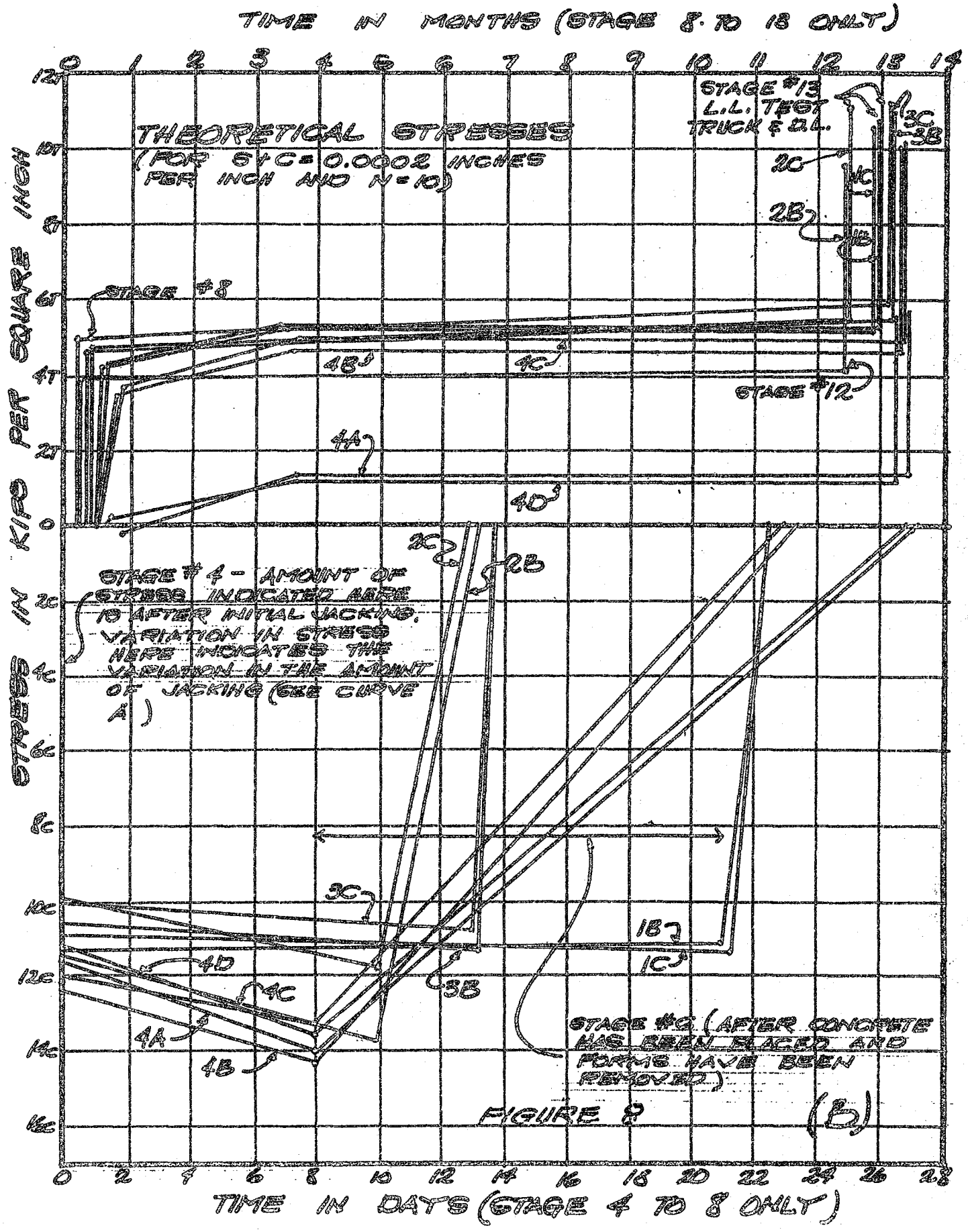


FIGURE 8 (B)

STRESS VS. TIME FOR TEST PT "C"
AVG. STRESSES FOR POINTS CL & C1
ON BOTTOM OF TOP FLANGE

LEON RIVER BRIDGE
CORYELL CO. CH. 236
RP. 57-10

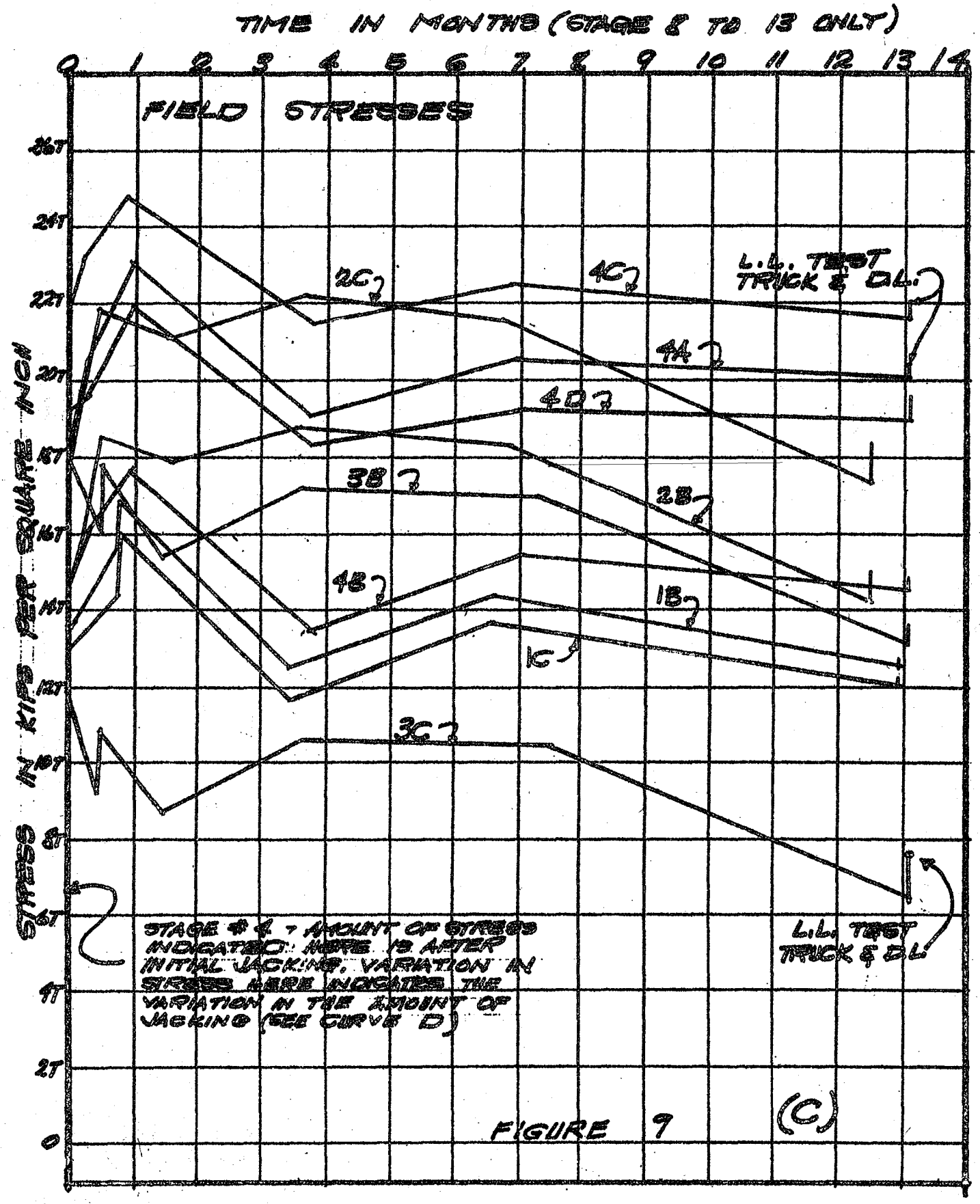


FIGURE 9

(C)

STRESS VS TIME FOR TEST PT "C"
AVG. STRESSES FOR PTS. C1 & C1
ON BOTTOM OF TOP FLANGE

LEON RIVER BRIDGE
CORYELL CO. SH. 236
RP. 57.10

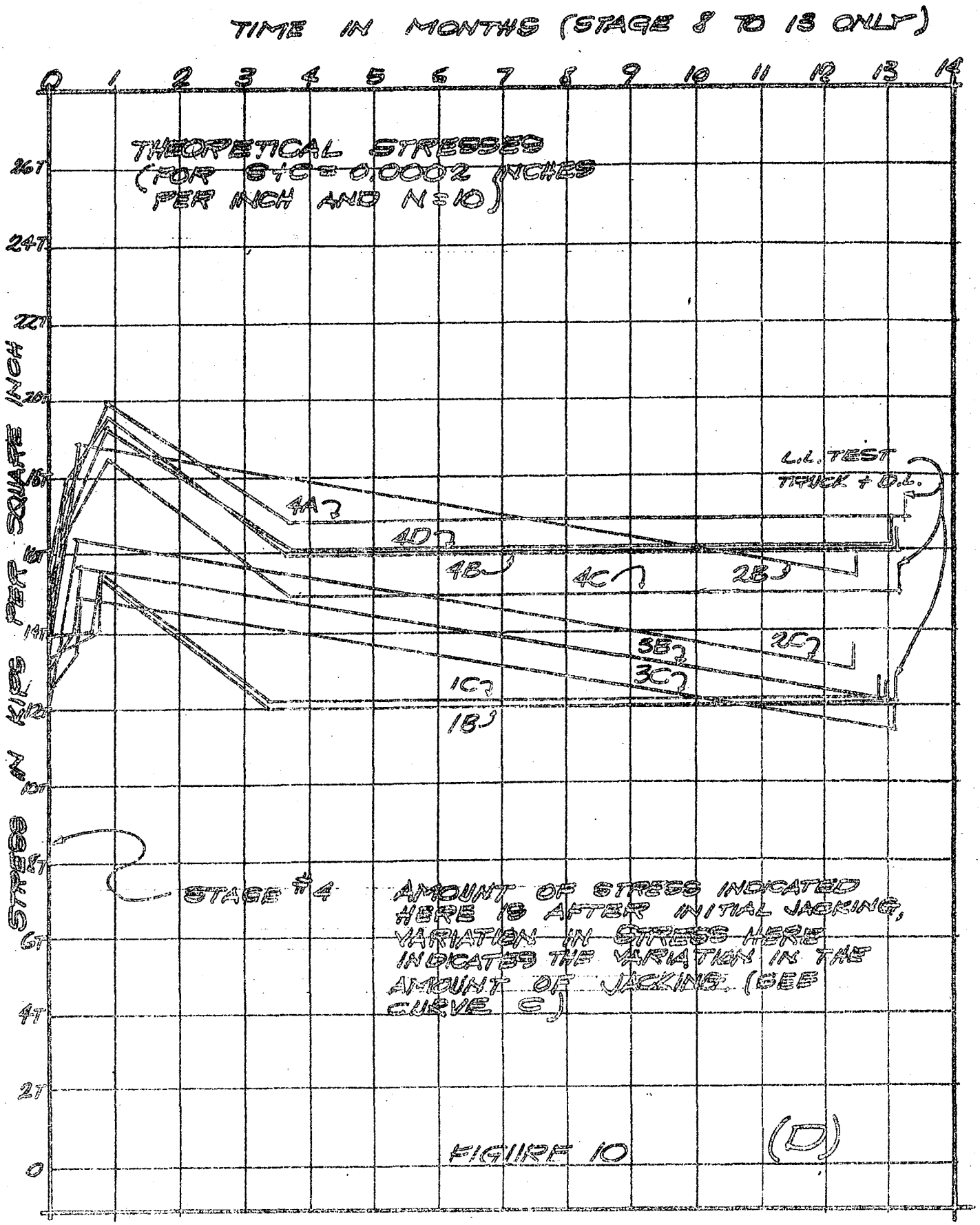


FIGURE 10

(D)

STRESS VS. TIME FOR TEST PT. "8" ON TOP OF SLAB (n=10)

LEON RIVER BRIDGE
CORYELL CO. SH236
RP-57-10

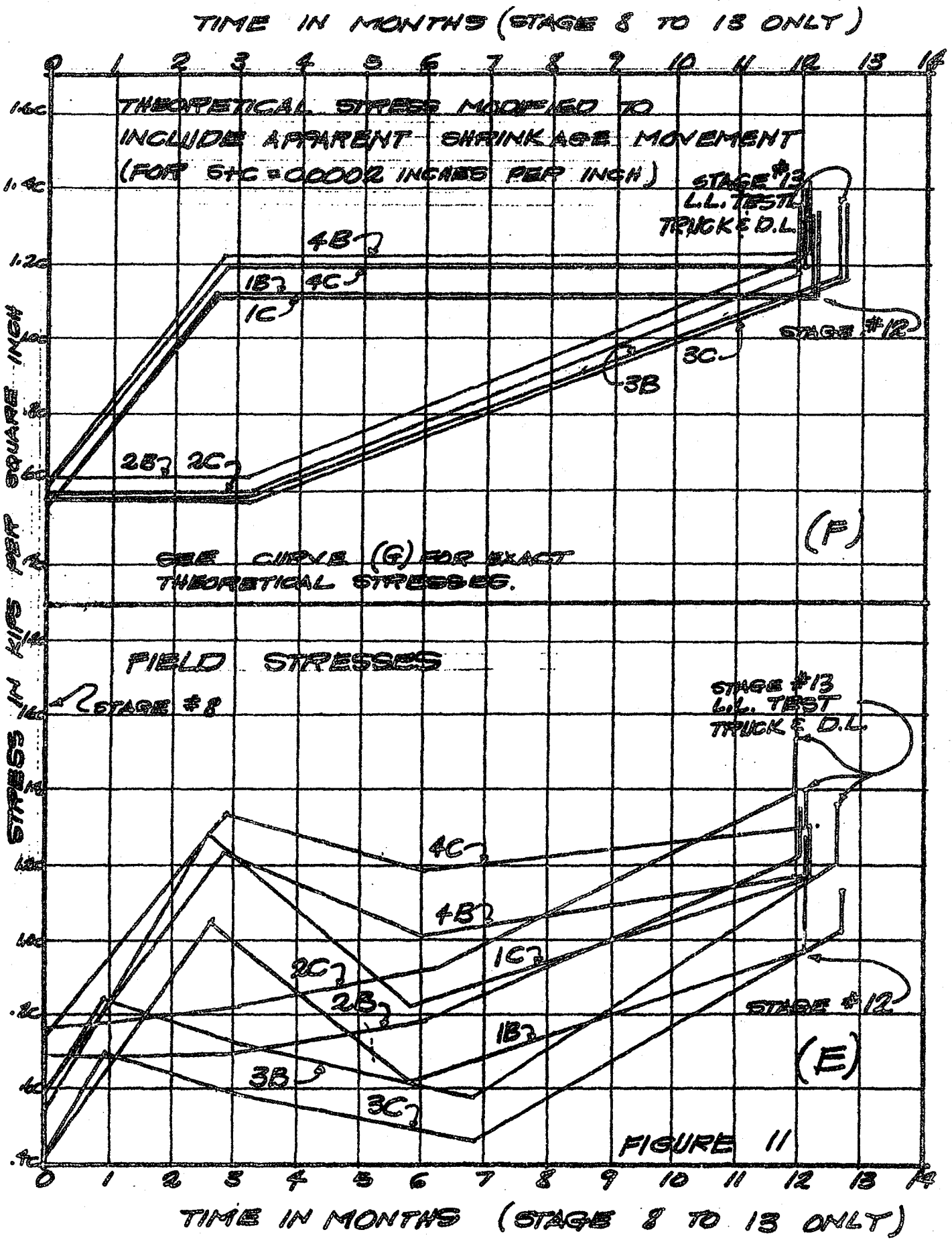


FIGURE 11

STRESS VS. TIME FOR TEST PT. "J" ON TOP OF SLAB (n=10)

LEON RIVER BRIDGE
CORYELL CO. S# 036
RR-57-10

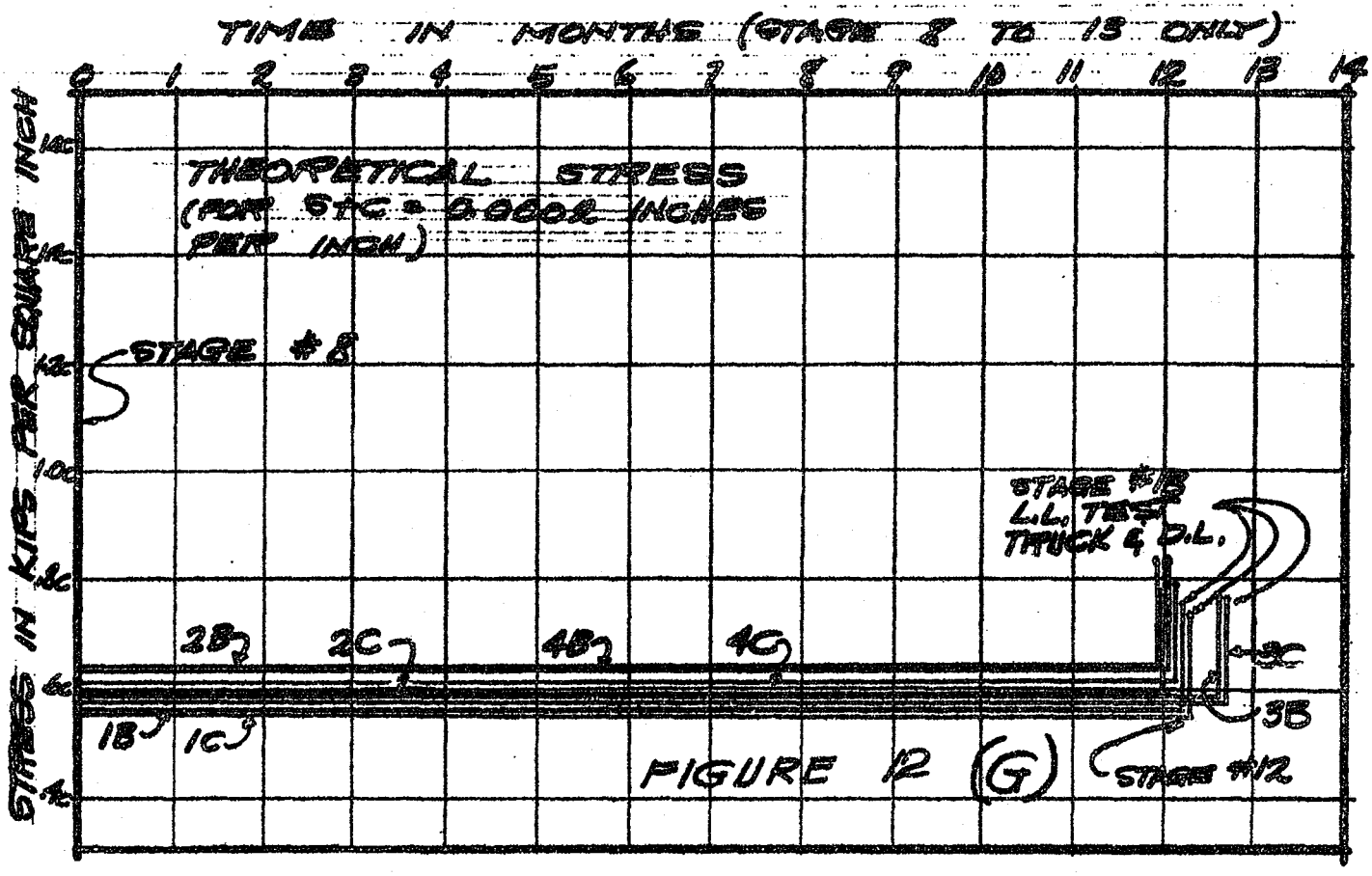
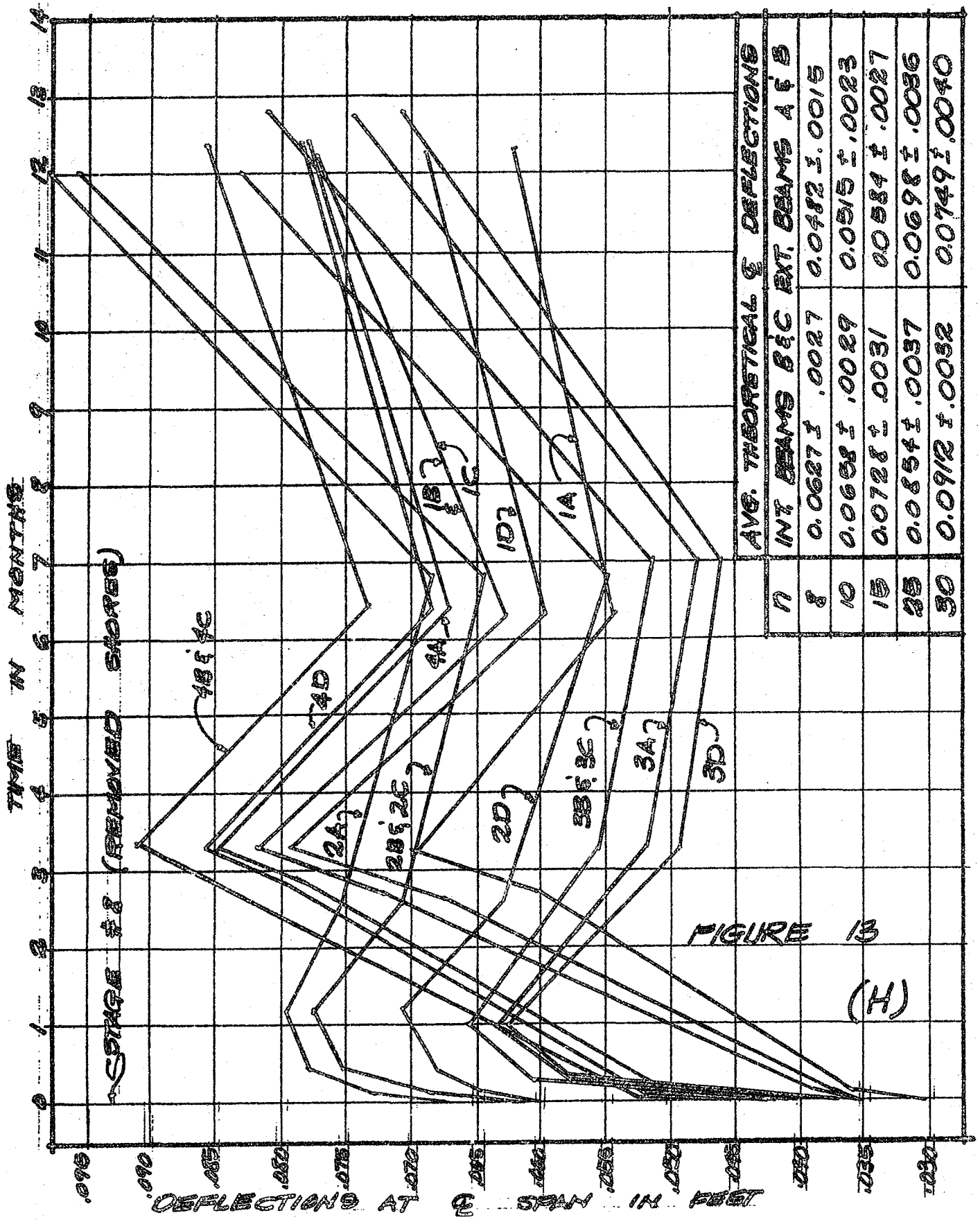


FIGURE 12 (G)

DEFLECTION AT ℄ SPAN VS. TIME

LEON RIVER BRIDGE
CORTELL CO. SH 236
RP - 57-10



n	AVG. THEORETICAL ℄ DEFLECTIONS INT. BEAMS B & C	EXT. BEAMS A & B
8	0.0627 ± .0027	0.0482 ± .0015
10	0.0658 ± .0029	0.0515 ± .0023
15	0.0728 ± .0031	0.0584 ± .0027
25	0.0854 ± .0037	0.0698 ± .0036
30	0.0912 ± .0032	0.0749 ± .0040

FIGURE 13

(H)

Heat Storage Effects on
Stresses & Deflections

Leon River Bridge
Coryell Co. - S.H. 236
R.P. - 57 - 10

Beam	Test Point	Avg. Field Stresses			Field Deflections		
		12 A&B AM	12 C&D PM	Diff.	12 A&B AM	12 C&D PM	Diff.
1B	"c" Top Flg.	12514T	123 1T	153C	.0762	.0686	.0076
	"d" Bot. Flg.	7069T	5256T	1813C	.0783	.0771	.0012
1C	"c" Top Flg.	12057T	11629T	428C	.0762	.0686	.0076
	"d" Bot. Flg.	7787T	6358T	1429C	.0783	.0771	.0012
4B	"c" Top Flg.	14368T	14399T	239C	.0862	.0799	.0063
	"d" Bot. Flg.	5786T	3922T	1864C	.0851	.0819	.0032
4C	"c" Top Flg.	21743T	21511T	232C	.0862	.0799	.0063
	"d" Bot. Flg.	5322T	3828T	1494C	.0851	.0819	.0032
1B	"a" Top Slab	967C	1201C	234C			
	"b" Bot. Slab	631C	705C	74C			
1C	"a" Top Slab	1159C	1425C	266C			
	"b" Bot. Slab	548C	563C	15C			
4B	"a" Top Slab	1163C	1491C	328C			
	"b" Bot. Slab	487C	560C	73C			
4C	"a" Top Slab	1301C	1551C	250C			
	"b" Bot. Slab.	364C	438C	74C			

Conclusions from this Test Project:

This type of design makes a very tough and rugged bridge. In spite of a relatively large ratio of span length to depth of I-beam of about 28, there was no excess vibration. By use of the 5-inch by 3/8-inch bottom cover plate, the allowable temporary prestress top flange stress was greater than the bottom flange stress. Since failure at yield point occurs in the bottom flange, our factor of safety from design stress to the yield point stress was about 2.5 instead of the usual 1.83 allowed for steel. In future designs, it would be well to use a minimum weight beam for the depth required and use a cover plate on the bottom of this beam. For instance, if a 27-inch by 102-pound wide flange beam was required, then by the use of the 27-inch by 94-pound wide flange beam with cover plate, we would raise the factor of safety well above 1.83 on the governing bottom flange for this particular design.

The theoretical and the measured stress curves show that the theory used in design produced the same kind of changes in stress from stage 8 to stage 12. All of the unknown factors such as shrinkage plus creep, exact thickness of concrete slab, distribution of dead load to individual I-beams, variation in size of I-beams, mishaps during construction on span 4 and span 2 as described under , "Possible Errors in Theoretical Calculations," the exact net amount of each beams upward jacking and the actual effect of expansion and contraction of the I-beams over the bearing shoes; all of the preceding factors affect the accuracy of the theoretical calculations. See "Stresses Due to Temperature Changes" in the appendix.

There was a very small error due to our using "n" equals 10 in our theoretical calculations. With "n" equals 6, a shrinkage plus creep factor of 0.0001 inch per inch would cause the following stresses in our design: Top of slab 29 psi compression, top of I-beam 1870 psi compression, bottom of I-beam 450 psi tension. With "n" equals 10, a shrinkage plus creep factor of 0.0001 would cause the following stresses in our design: Top of slab 10 psi compression, top of I-beam 1760 psi compression, bottom of I-beam 540 psi tension. When the designer determines the correct shrinkage plus creep factor to use by reference to the appendix item "Shrinkage Plus Creep," he would be wise to use "n" equals 6 with the proper shrinkage plus creep factor. For Texas Highway Department Class "A" concrete the factor of 6 is about right for "n". On our design the only stresses affected materially on governing stresses is the bottom flange of the beam. The bottom flange additional stress due to use of a shrinkage plus creep factor of 0.0004 inch per inch would be 1800 psi tension. For a factor of 0.0003 the additional stress would be 1350 psi tension. To be on the safe side, use a shrinkage plus creep factor of 0.0004 inch per inch and use an additional final working stress allowable in the bottom flange of 2500 psi due to the welding of the cover plate and shear devices. The 2500 psi stress in the bottom flange due to welding is compression; hence, this would reduce the working stress in tension due to bending moment by 2500 psi. See item 4 under, "Possible Errors in Theoretical Stresses."

Expansion or contraction of the span over the end bearing would cause 8690 pounds of force on the bottom flange of the beam at the bearing shoe. This is a condition that exists on all spans of steel and concrete designed in the past. This factor has been ignored and is one reason why a factor of safety of 2.5 is used in concrete design and a factor of safety of 1.83 is used in steel design.

There would have to be a rise or fall in the effective temperature of the composite structure of 0.48° F to overcome this force of 8690 pounds. This force, due to falling temperature, would cause 18 psi compression in top of the slab, 197 psi tension in the top flange and 799 psi tension in the bottom flange.

With a cover plate on the bottom of the I-beam, the prestress tension stress in the top flange of the I-beam governs the design. Initial compression stresses in the top and bottom flanges of the I-beam, due to welding, could well be taken account of in the design. This is a temporary high stress, and shrinkage plus creep will reduce this stress some 4000 psi. The above governing high stress occurs at stage 8 (after shoring is removed) when all dead load is on the span. The shrinkage plus creep reduces this high stress and the live load increases this stress very slightly, since the top flange of the I-beam is a very small distance below the neutral axis of the composite section. Because of this, the failure of the composite section occurs when the bottom of the I-beam reaches the yield point. At this stage, the slab stress and stress at the top of the I-beam is not at yield point. Before live load is considered, the dead load plus shrinkage tension stress in the bottom flange is low and the design live load will bring this bottom flange stress to a tension stress of well below the allowable of 18,000 psi tension. Remember, the bottom flange stress after prestressing is in compression. The dead load plus shrinkage plus creep plus live load has to reverse this compression stress to a tension stress. In other words, the compression stress has to be forced out by bending moment acting on the composite section before the bottom flange has a net tension stress.

The initial compression stress in the top and bottom flange of the I-beam is due to the heating of the welding. There was 7.8 inches of one-quarter inch bead weld per linear foot of top flange. There was 7 inches of one-quarter inch bead weld per linear foot on the bottom flange of our beam. See item 4, under, "Possible Errors in Theoretical Stresses." The stresses shown under this item were due to the effect of this heat on our sections as shown on Figure 2. With a known amount of welding on any I-beam section, it would be possible to approximate these compression stress values by comparing with our section and our amount of welding.

It can be readily shown that this type of design is very economical. The bid cost to the Texas Highway Department for the furnishing of shear connector steel in pounds and for welding these connectors on top of the I-beams was the equivalent of adding 8 pounds per linear foot of structural steel to the weight of I-beam sections. The contractor kept costs on shoring and said the bid price of the shoring work would be $200.00 per span. To compare costs, simply design a simple span I-beam or a continuous I-beam span and compare the net cost of the structural steel involved.

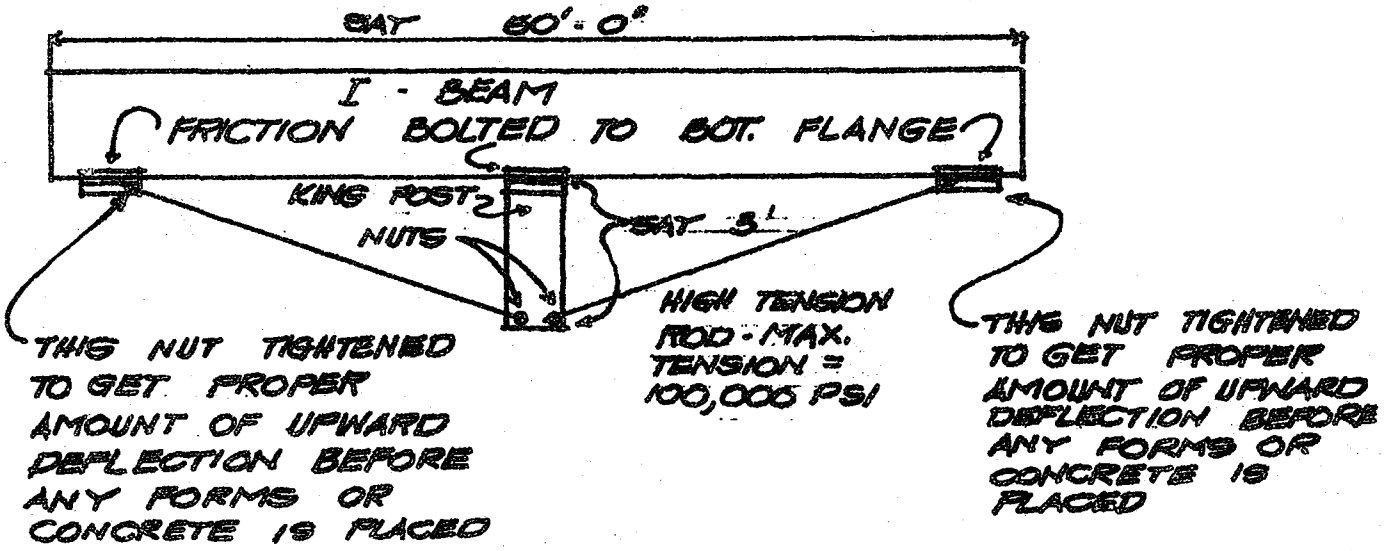
From our experience, we offer the following methods of adjusting the amount of jacking necessary due to settlement in the shoring as the concrete is placed in the forms:

It would be better to jack up each beam individually by use of a screw jack under every beam at the center line of the span.

A good method to adjust the jacking would be to jack the beams up with forms and reinforcing in place one-eighth inch higher than the theoretical. Check this jacking by means of an Engineer's level, reading to one thousandth of a foot. Take rod readings on each end on the top of the I-beam and on the top of the I-beam at the center line of the I-beam. Then, before concrete is placed, set the top of a bolt fastened to the top flange of the I-beam to the proper height to check with the center line of the strike-off board. As the concrete is placed, keep the jacked up height right by adjusting the jack until the rod reading on the top of the bolt is correct. There should be a man working on the jacks the entire time the concrete is being placed and until the concrete has been in place 2 hours.

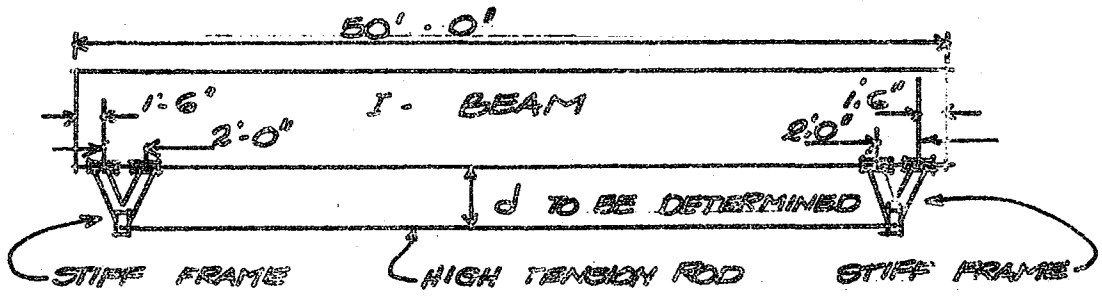
Another method would be to take levels by means of an inverted level rod on the bottom of the I-beams one foot from the center line of the beams until all adjusting of the jacks has been accomplished and until all concrete has been placed.

<u>Alternate Methods of Prestressing</u>:



An inverted king post truss, as shown, used for the purpose of prestressing the I-beam was investigated. It was found that there was no advantage gained by prestressing in this fashion over the unshored composite I-beam span.

If this scheme had worked, there would have been no adjustment of the amount of upward deflection due to weight of plastic concrete. The deflection due to the weight of the concrete could be figured and predicted to an accuracy of one-sixteenth of an inch.

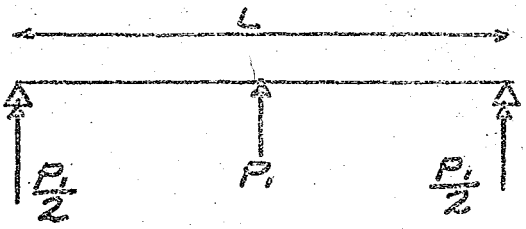
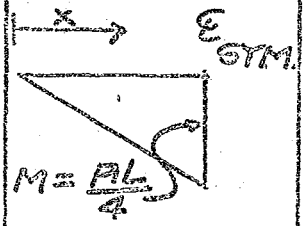
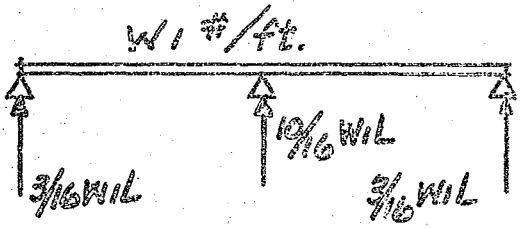
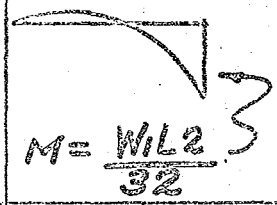
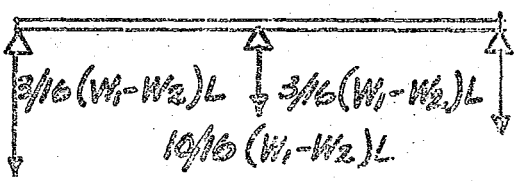
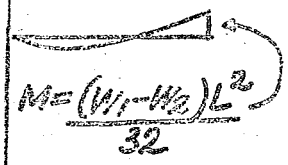
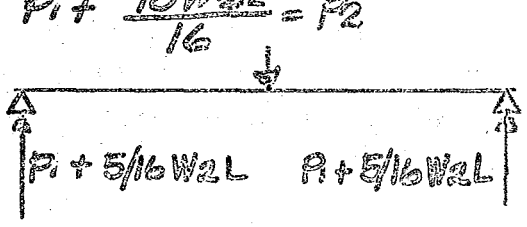
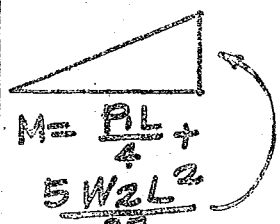
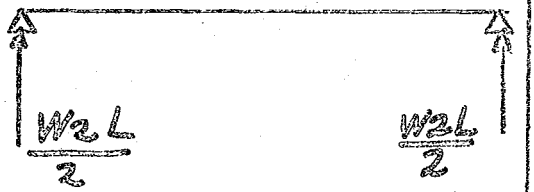
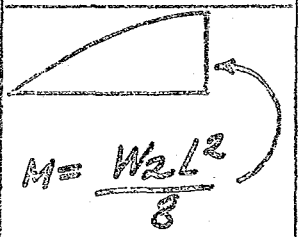


The above general scheme has been suggested as a means of prestressing the I-beam and eliminate the necessity of shoring. An analysis will show that on a 50' span about 10000 psi of the prestressing stress in the top flange of the beam is lost due to the weight of the beam from stiff frame to stiff frame. The only way to prevent this loss in prestressing stress is to deflect the center of the beam up more than would otherwise be necessary, to take account of the loss in prestressing stress. This additional prestressing stress added to the stress due to the original upward deflection would make the psi stress well above the allowable in the top flange of the I-beam even for a temporary stress. All upward deflection noted above would be forced into the I-beam before the plastic concrete and the forms were placed in order that no adjustment of upward deflection would be necessary.

APPENDIX

METHODS OF CALCULATING STRESS ON SHORED COMPOSITE BEAM (CONSTANT SECTION)

PG. 1 OF 2

	LOAD DIAG.	MOM. DIAG.	MOM. EQUATION
I			DEFLECTION UPWARD $M_x = \frac{P_1 x}{2}$ (BEAM I)
II			UNIF. DL + FORMS ETC. $M_x = 3/16\, W_1 L x + \frac{W_1 x^2}{2}$ (BEAM I)
III			REMOVAL OF FORMS ETC. $M_x = -13/16\, W_1 L x + 3/16\, W_2 L x + \frac{(W_1 - W_2)}{2} x^2$ (COMP. I)
IV			LOAD EQUIVALENT TO REMOVING SHORE — $M_x = \frac{P x}{2} + \frac{10 W_2 L x}{32}$ (COMP I)
V			NET EXTERNAL MOM. $M_x = \frac{W_2 L}{2} - \frac{W_2 x^2}{2} = ① - ② - ③ + ④$ DOES NOT GIVE TRUE STRESS DIRECTLY

M_x I REVERSED + M_x II REV. + M_x V $= x\left(\frac{P_1}{2} + \frac{W_2 L}{2} - \frac{3 W_1 L}{16}\right) + x^2\left(\frac{W_1 - W_2}{2}\right)$

M_x III + M IV $= x\left(\frac{P_1}{2} + \frac{W_2 L}{2} - \frac{3 W_1 L}{16}\right) + x^2\left(\frac{W_1 - W_2}{2}\right)$

METHOD A

Moments I and II are the moments locked into the shored beam when it becomes composite. The reverse of these moments will act with the net DL to deflect the beam downward when the shore is removed. Therefore, M_x ($I_{rev.}$ + V) is the moment, induced by removing forms and shore. The stress due to this moment is based on the composite section and should be added algebraically to the stress due to I and II which is based on the beam section.

METHOD B

Applying a load, which is equal and opposite to the shore reaction, to the beam in shored position is equivalent to removing the shore reaction. Therefore, M_x (III + IV) is the moment induced by removing forms and shore. The stress due to this moment is based on the composite section and should be added algebraically to the stress due to I and II, which is based on the beam section. As shown above;

$$M_x\ (I_{rev.} + II_{rev.} + V) = M_x\ (III + IV)$$

The two methods will yield equal results.

SHORING MOMENT (Stage #4-7)

Span 2 Beam C

LEON RIVER BRIDGE
CORYELL CO.
SH - 236
RP-10 Pg. 1

Shoring performed by jacking beam up 1.152" at ℄ span.

Beam data:
CB-21-59
$I_o = 1{,}250\ in^4$
depth = 20.91 in.
Fig. t = 0.575 in.
$A = 17.42\ in^2$

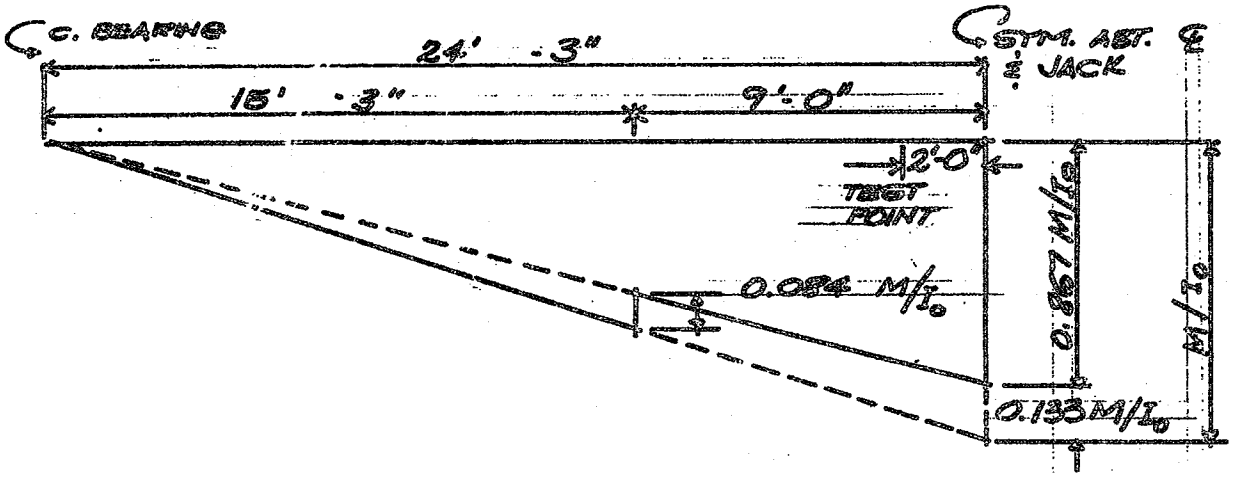
Cov. PL data:
length = 18' - 0"
t = 3/8 in.
w = 5 in.
$A = 1.88\ in^2$

Moment of Inertia Beam + Plate

Part	A	$\bar{y}$	$A\bar{y}$	d	Ad^2	I_o
Beam	17.42	0	0	-1.037	18.7	1,250.0
PL	1.88	10.643	20.009	9.61	173.6	negl.
Total	19.30	--	20.009	--	192.3	1,250.0

$$I_{BM+PL} = 192.3 + 1{,}250 = 1{,}442.3\ in^4$$

$$\frac{I_o}{I_{BM+PL}} = 0.867$$



Shoring M/I_o Diagram

Moment in terms of Δ at ℄:

$$EI_o\ \Delta/M = \tfrac{1}{2} \times 2 \times 24.25^2 - \tfrac{1}{2} \times 0.133 \times 2/3 \times 24.25^2 + \tfrac{1}{2} \times 0.084 \times 2/3 \times 15.25^2$$

$$EI_o\Delta = M\,(196.0 - 26.1 + 6.5) = 176.4M$$

for: $E = 29 \times 10^6$ psi $\quad I_o = 1{,}250\ in^4$, Δ in inches

$$M_{℄} = \frac{EI_o\Delta}{176.4} = \frac{29 \times 10^6 \times 1{,}250 \times \Delta}{176.4 \times 1{,}728} = \underline{118{,}920\,\Delta\ (ft.\text{-}lb.)}$$

$$M_{@\ test\ pt.} = M_{℄} \times \frac{22.25}{24.25} = \underline{109{,}110\,\Delta\ (ft.\text{-}lb.)}$$

$$= 125{,}695\ (ft.\text{-}lb.)$$

$$= 1{,}508{,}340\ (in.\text{-}lb.)$$

2-SPAN CONTINUOUS UNIF.
LD. MOMENT COEF. AT SHORING
SUPPORT

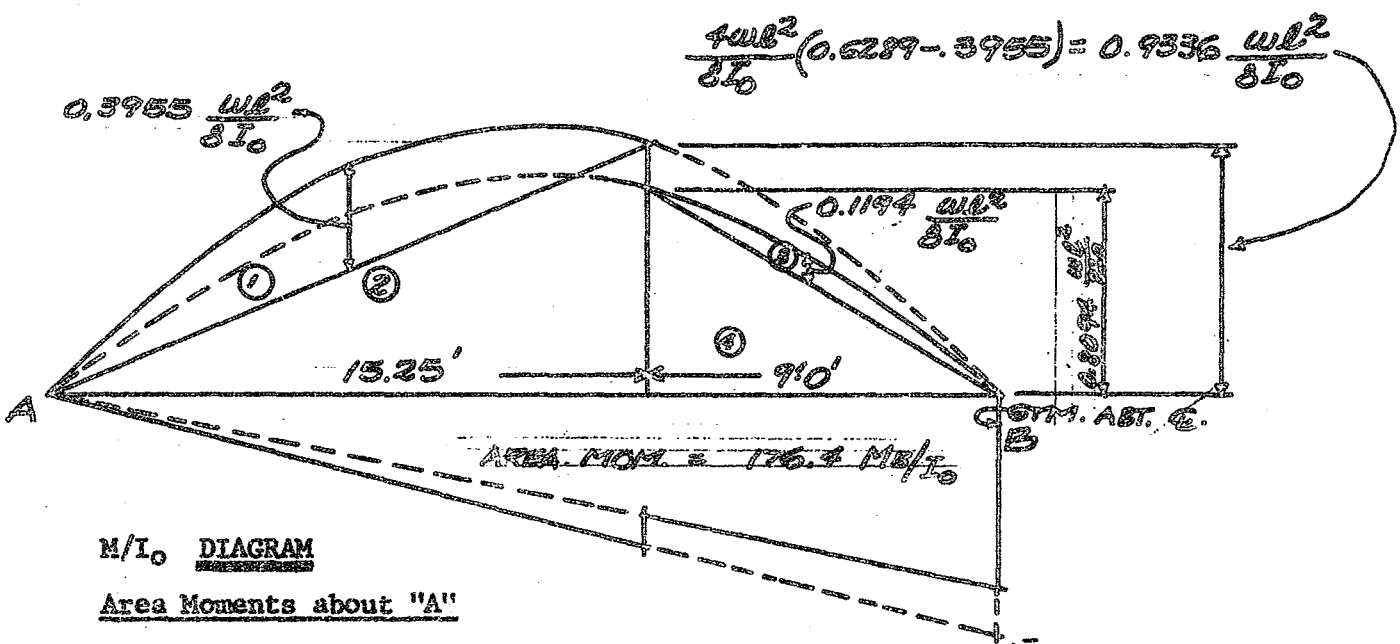
LEON RIVER BRIDGE
CORYELL CO.
SH - 236
RP-10 Pg. 2

SPAN 2 BEAM C Stage #4-7

$\left(\frac{9}{24.25}\right)^2 = 0.3711^2 = 0.1377$

$\left(\frac{15.25}{24.25}\right)^2 = 0.6289^2 = 0.3955$

$0.1377\ (.867) = 0.1194$
$0.9336\ (.867) = 0.8094$



M/I_o DIAGRAM

Area Moments about "A"

AREA	$\bar{X}$	$A\bar{X}$
① 2/3 (15.25) (0.3955) $\left(\frac{wl^2}{8I_o}\right)$	15.25/2	30.660 $\left(\frac{wl^2}{8I_o}\right)$
② 1/2 (15.25) (0.9336) (do)	2/3 (15.25)	72.272 (do)
③ 2/3 (9.0) (0.1194) (do)	15.25 + 9/2	14.149 (do)
④ 1/2 (9.0) (0.8094) (do)	15.25 + 9/3	66.472 (do)
		183.654 $\frac{wl^2}{8I_o}$

$176.4\ M_B/I_o = \Sigma A\bar{X}$

$\therefore\ M_B = (-)\ \frac{183.654}{8\ (176.4)}\ wl^2 = -0.1301\ wl^2$

At Test Point (20' from "B")

$M = \left[-0.1301\left(\frac{22.25}{24.25}\right) + 4/8\ (0.918 - 0.918^2)\right] wl^2$

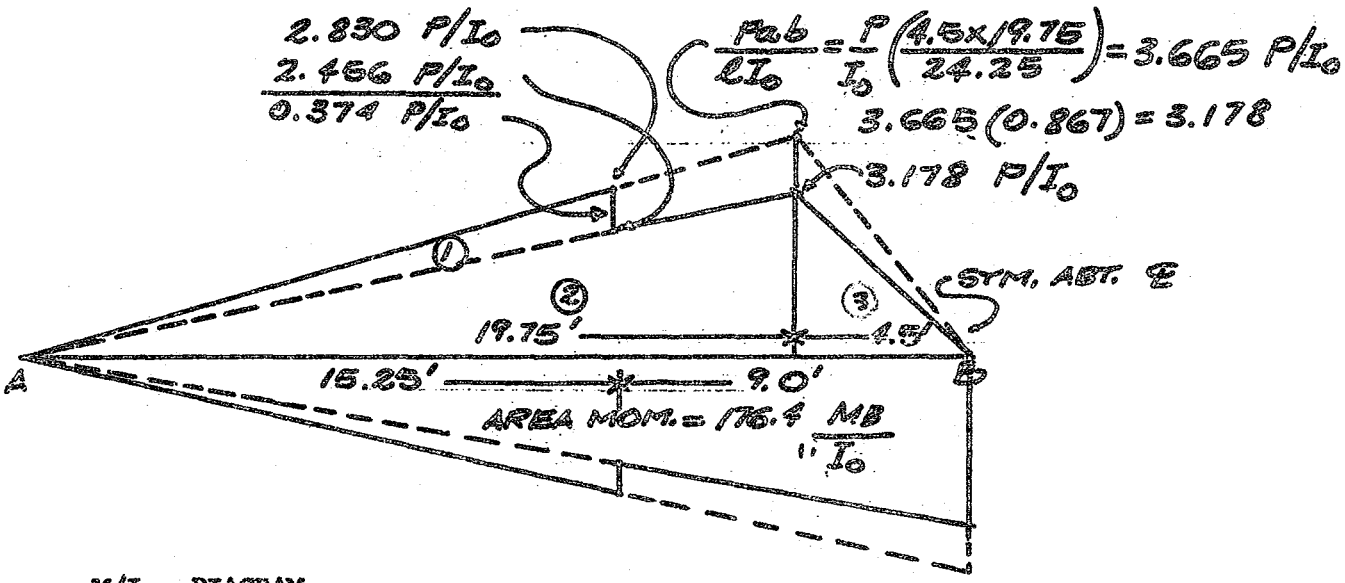
$\frac{22.25}{24.25} = 0.918$

$= \boxed{-0.0819\ wl^2}$

2-SPAN CONTINUOUS POINT
L.D. MOMENT COEF. AT
SHORING SUPPORT

LEON RIVER BRIDGE
CORYELL CO.
SH - 236
RP-10

Span 2 Beam C Stage #4-7



M/I_o DIAGRAM

Area Moments about "A"

Area	$\bar{X}$	$A\bar{X}$
1 0.374/2 (15.25) (P/I_o)	2/3 (15.25)	29.001 P/I_o
2 3.178/2 (19.75) (P/I_o)	2/3 (19.75)	413.205 P/I_o
3 3.178/2 (4.5) (P/I_o)	19.75 + 45/3	151.950 P/I_o
176.4 M/I_o = $\Sigma A\bar{X}$		594.156 P/I_o

$$\therefore M_B = -\frac{594.156}{176.4} P = -3.37 P$$

$$\therefore M_{@T.P.} = -3.37P(0.918) + 3.665 (2/4.5) = \boxed{-1.47 P}$$

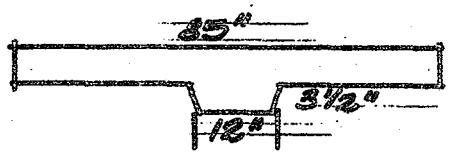
Diaphragm P (16 WF 36) = 36 (7.0') = 252 lb. (INT. BM.)
126 lb. (EXT. BM.)

UNIFORM LOADS Stage #4-7

LEON RIVER BRIDGE
CORYELL CO.
SH - 236
RP - 10 Pg. 4

SPAN 2 BEAM C

SCAFOLDS	=	30.8 p.l.f.	
FORMS	=	52.0	
BEAM	=	59.0	
COV. PL	=	2.4	Avg. over entire span
SHEAR LUGS	=	3.1	Avg. over entire span
REIN. STL.	=	32.5	Avg. over entire span
CONC. HAUNCH & SLAB	=	555.1	wet concrete*
		734.9 p.l.f.	



6.44" [.2917 + 7.083(.5367)](.15) = 0.614 K/F
= 614 p.l.f.*

* By simple beam dist. on half of bridge with wet concrete = 555.1 p.l.f.
Note reduction is due primarily to Cantilever of curb forms and concrete.

TOTAL MOMENT ON BEAM Stage #4-7 ℓ = 24.25'

Δ = 1.152 in. w = 734.9 p.l.f. P = 252 lb.

- 109,110 Δ	=	- 125,695 (ft.-lb.)
- 0.0819 $w\ell^2$	=	- 35,393
- 1.47 P	=	- 370
M @ TP	=	- 161,458 (ft.-lb.)
	=	-1,937,496 (in.-lb.)

M/I = M/1442.3 = 1,343.3 lb/in^3

STAGE # 4-7 STRESSES psi n = 10				
THEORETICAL STRESS				Average Measured Stress
Test Pt.	C in.	M/I	C x M/I	
"c" f_s top	10.917	1,343.3	$14,665_T$	$20,315_T$
"d" f_s bot.	8.843	do	$11,879_C$	$16,255_C$

SHRINKAGE STRESS ANALYSIS
COMPOSITE BEAM

LEON RIVER BRIDGE
CORYELL CO.
SH - 236
RP-10 Pg. 5

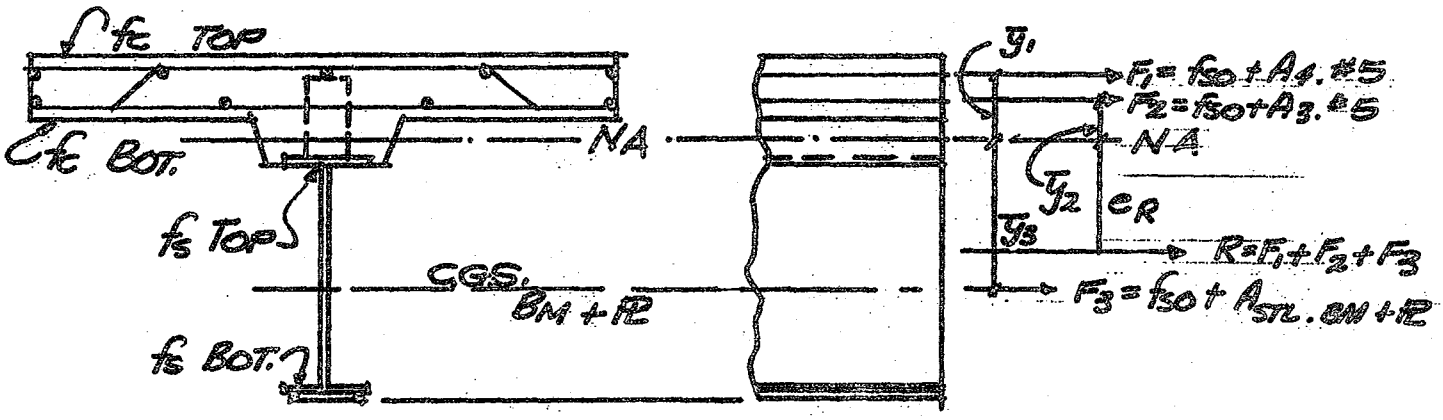
AASHO Design Specifications

Max Allowable concrete shrinkage strain (S) = 0.0002 in/in

$E_S = 29 \times 10^6$ psi $\qquad n = 10$

Max Allowable initial shrinkage stress on steel (f_{so}) :

$f_{so} = s \cdot E_s = -5{,}800$ psi Compression



(F_1) (F_2) & (F_3) are the resisting forces of the steel in the opposite direction to the compressive shrinkage stress. (R) is the total resisting force of the section, acting a distance (e_r) from the neutral axis (NA) of the composite section.

By moments of forces about the (NA) :

$$e_r = \frac{F_1\bar{y}_1 + F_2\bar{y}_2 - F_3\bar{y}_3}{R}$$

Final stresses are calculated as follows:

$$f_{c\ top} = \frac{1}{n}\left(\frac{R}{A_T} - \frac{Re_R\ c\ top\ slab}{I_T}\right)$$

$$f_{c\ bot.} = \frac{1}{n}\left(\frac{R}{A_T} - \frac{Re_R\ c_{bot.\ slab}}{I_T}\right)$$

$$f_{s\ top} = -5{,}800 + \frac{R}{A_T} + \frac{Re_R\ c_{top\ flg.}}{I_T}$$

$$f_{s\ bot.} = -5{,}800 + \frac{R}{A_T} + \frac{Re_R\ c_{bot.\ flg.}}{I_T}$$

Where:

A_T = Transformed area of section.

I_T = Transformed mom. of Inertia.

Note: A_T & I_T are in terms of steel.

PROCEDURE FOR CORRELATION OF FIELD STRESSES AND THEORETICAL STRESSES FOR CONCRETE DUE TO FULL DEAD LOAD PLUS SHRINKAGE (S) PLUS PLASTIC FLOW (C).

Shrinkage + Creep Strain (S+C) = 0.0002 in./in.

$E_s = 29 \times 10^6$ psi (AASHO)

$\therefore f_{sc} = S \cdot E_s = 0.0002 \cdot 29 \cdot 10^6 = 5{,}800$ psi Comp.

= initial steel stress due to Shrinkage + Creep.

Final Field stress on Concrete due to dead load + shrinkage = $f_{c_{fld.}}$

Final theor. stress on Concrete due to dead load = $f_{c_{d.l.}}$

Final theor. stress on Concrete due to shrinkage + plastic flow = $f_{c_{(S+C)}}$

also $$f_{c_{(S+C)}} = \frac{1}{n}\left(\frac{R}{A_T} - \frac{Re_R \cdot C}{I_T}\right)$$

however $$f_{c_{fld.}} \neq f_{c_{d.l.}} + f_{c_{(S+C)}} \quad \text{(By Correlation)}$$

$\therefore$ Since $f_{c_{fld.}}$ was measured with the H. O. Berry 20" Strain gage and probably includes actual shrinkage movement the correlation should be as follows:

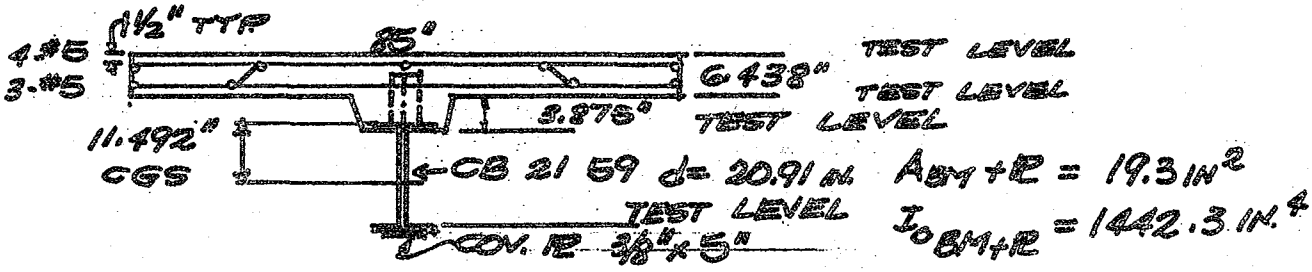
$$f_{c_{fld.}} = f_{c_{d.l.}} + \frac{1}{n}\left(\frac{R}{A_T} - \frac{Re_R \cdot C}{I_T}\right) + \frac{5{,}800}{n}$$

for n = 10

$$f_{c_{fld.}} = f_{c_{d.l.}} + \frac{1}{10}\left(\frac{R}{A_T} - \frac{Re_R \cdot C}{I_T}\right) + 580$$

MOMENT OF INERTIA
COMPOSITE BEAM 2-C
(Neglect haunch) n=10

LEON RIVER BRIDGE
CORYELL CO.
SH - 236
RP-10 Pg. 7



Member	A	$\bar{y}$	$A\bar{y}$	d	d^2	Ad^2	I_o	$\Sigma Ad^2 + \Sigma I_o$
Bm. + R	19.30	0	0	13.31	177.15	3419.0	1442.3	
Conc. Flg.	54.72	18.01	985.51	4.70	22.09	1208.8	189.0	
TOTAL	74.02		985.51			4627.8	1631.3	6259.1

$C_{top\ slab}$ = 4.70 { +3.22 = 7.92"
$C_{bot.\ slab}$ = 4.70 { -3.22 = 1.48"

$C_{top\ stl.\ tp}$ = 13.31 { -10.917 = 2.39"
$C_{bot.\ stl.\ tp}$ = 13.31 { + 8.843 = 22.15"

SHRINKAGE STRESSES Stage #9-12

S = 0.0002 in/in (AASHO) $E_s = 29 \times 10^6$ psi $f_{so} = sE_s$ = 5,800 psi

F_4-#5 = 1.24 (5,800) = 7192 lb
F_3-#5 = 0.93 (do) = 5394
F_{bm+R} = 19.30 (do) = 111940

R = ΣF = 124,526#

$$e_r = \frac{7192\ (5.795) + 5394\ (3.605) - 111{,}940\ (13.31)}{124{,}526} = -11.47''$$

R/A_T = 1,692 psi Re/I_c = 228 psi/in.

$f_{s\ top}$ = -5,800 + 1,692 + 228 (2.39) = -3,563 psi C

$f_{s\ bot.}$ = -4,108 + 228 (22.15) = + 942 psi T

$f_{c\ top}$ = 169.2 - 22.8 (7.92) = - 11 psi C

$f_{c\ bot.}$ = 169.2 - 22.8 (1.48) = + 136 psi T

48' - 6" SIMPLE COMPOSITE MOMENT COEF.
& STRESSES DUE TO REMOVAL OF SHORING

LEON RIVER BRIDGE
CORYELL CO.
SH - 236
RP-10 Pg. 8

Stage #8-12

A. Shoring deflection: $\ell = 48.5'$

$118{,}920\Delta = R_A \ell/4 \quad \therefore \quad R_A = 9{,}808\Delta$ (lb.)

B. Uniform load:

$V_s = 0.5\ \omega\ell/2\ (2) = 1.0000\ \omega\ell/2$
$V_c = 0.1301\ \omega\ell/2\ (2) = 0.2602\ \omega\ell/2$ } $V_{TOT} = 1.2602\ \omega\ell/2$

$\therefore R_\omega = 1.2602\ (48.5/2)\omega = 30.56\omega$ (lb.)

C. Point Load (DIAPHRAM)

$V_s = 19.75/24.25\ (P)\ (2) = 1.6288P$
$V_c = 3.37/24.25\ (P)\ (2) = 0.2780P$ } $V_{TOT} = 1.9068P = R_P$

$R_{TOTAL} = A + B + C = 9{,}808\Delta + 30.56\omega + 1.9068P$

$M_{@\mathbb{C}} = R_{TOT}\ \ell/4 = 48.5/4\ (R_{TOT}) = 12.125\ R_{TOT.}$

$\therefore M_{@\ T.P.} = 0.918\ (12.125)\ R_{TOT.} = \boxed{11.125\ R_{TOT.}}$

$\Delta = 1.152"$ $\quad \omega = 750.5$ p.l.f.* $\quad P = 252$ lb.

* for equal dist. of all D.L.

$M_{T.P.} = (11.125)\ (12)\ [9{,}808\ (1.152) + 30.56\ (750.5) + 1.9068\ (252)]$

$= 4{,}634{,}400$ in-lb $\qquad M/I_c = 740.4$ lb/in^3

$I_{comp} = 6{,}259.1$ in^4

THEORETICAL STRESS psi (n = 10)						Average Measured Stress psi
Test Pt.	C in	M/I	M/I x C	Shrinkage	(4-7) + S + (8-12)	
"a" f_c top	7.92	74.04	586C	11C	597C	1,381C
"b" f_c bot.	1.48	do	110C	136T	26T	541C
"c" f_s top	-2.39	740.4	1,770T	3,563C	12,872T	17,357T
"d" f_s bot.	-22.15	do	16,400T	942T	5,463T	3,589T

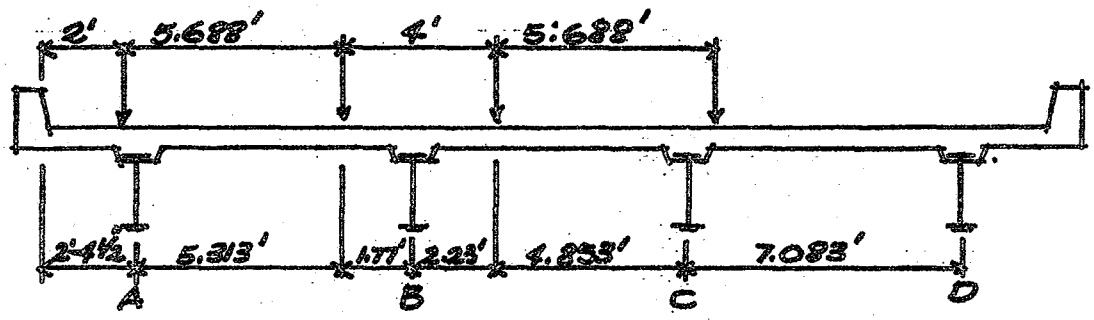
LIVE LOAD MOMENT & STRESSES
FOR TEST TRUCK & H-15 TRUCK
COMPOSITE BEAM 2-C
AASHO Spec. Stage #13

LEON RIVER BRIDGE
CORYELL CO.
SH - 236
RP-10 Pg. 9

TEST TRUCK DATA: (Dump Truck w/side bds., filled w/asph.)

Wheel spread	=	5'-8½"	=	5.688'
Wheel base	=	10'-10½"	=	10.875'
Rear Axle	=	13,660 lb.	=	6,830 lb./wh.
Front axle	=	4,340 lb.	=	2,170 lb./wh.

TEST TRUCK POSITION ON BRIDGE: (2 trucks used)

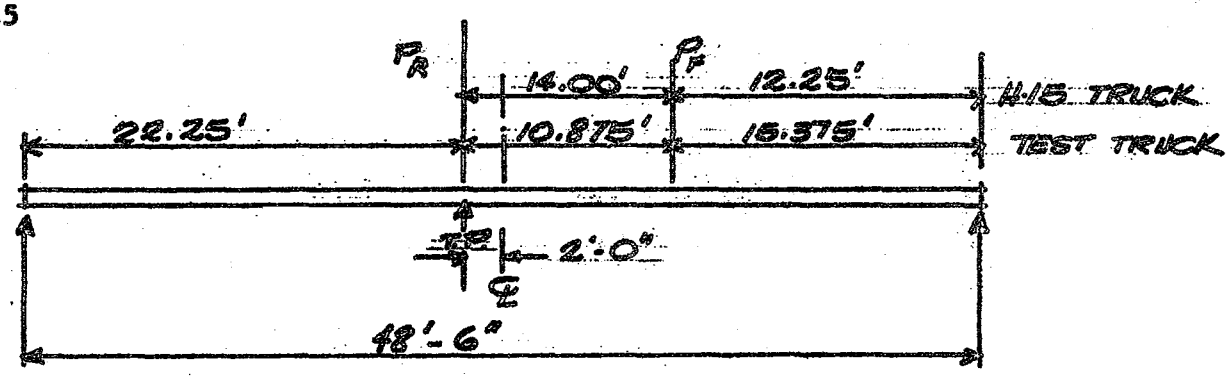


Above position for max. moment on beam A & B reverse position on bridge for max on beam C & D.

FOR INTERIOR BEAMS: $S/5.5 = \frac{7.083}{5.5} = 1.288 \frac{wh}{G_{1R}} = 0.644 \frac{lanes}{G_{1R}}$

EXTERIOR BEAMS: $\frac{S}{4.0+0.25(S)} = \frac{7.083}{4.0+(7.083)(0.25)} = 1.227 \frac{wh}{G_{1R}} = 0.614 \frac{lanes}{G_{1R}}$

$I = \frac{50}{125+48.5} = 0.286$



LIVE LOAD MOMENT & STRESSES
FOR TEST TRUCK & H-15 TRUCK
COMPOSITE BEAM 2-C
AASHO S ec. Stage #13

LEON RIVER BRIDGE
CORYELL CO.
SH - 236
RP-10 Pg. 10

Test Truck	Int. Beams	H-15 Truck	
P_R = 6,830 (1.288)	= 8.8^k	P_R = 12 (1.286) (1.288)	= 19.9^k
P_F = 2,170 (1.288)	= 2.79^k	P_F = 3 (1.286) (1.288)	= 4.97^k

for Ext. Beams multiply above by $\frac{1.227}{1.288}$ = 0.953

H-15 Lane Int. P = 13.5 (.644) (1.286) = 11.2^k

Does Not Control ω = 0.48 (.644) (1.286) = $0.40^{k/f}$

for Ext. Bm. multiply above by 0.953

Test Truck (int. beams)

$$L.L.M. = 2.79 \left(\frac{22.25 \times 15.375}{48.5}\right) + 8.8 \left(\frac{22.25 \times 26.25}{48.5}\right)^{12.043}$$

= 125.7 Kip-feet x 12,000 = 1,508,400 in-lb

I_c = 6,259.1 in^4 for ext. beams = 1,437,505 in-lb

Int. M/I_c = 241 psi/in.

H-15 Truck (int. beams)

$$L.L.M. = 4.97 \left(\frac{22.25 \times 12.25}{48.5}\right) + 19.9 \ (12043) =$$

= 27.9 + 239.7 = 267.6^{kf} = 3,211,200 in-lb

for ext. beams = 3,060,274 in-lb

Int. M/I_c = 513 psi/in.

LIVE LOAD MOMENT & STRESSES
FOR TEST TRUCK & H-15 TRUCK
COMPOSITE BEAM 2-C
AASHO Spec. Stage #13

LEON RIVER BRIDGE
CORYELL CO.
SH - 236
RP-10 Pg. 11

Test Truck L.L. Stresses & Dead Load Stresses n = 10

THEORETICAL STRESSES psi					Average Measured Stress psi
Test. Pt.	C	M/I_c	M/I_c x C	#8-12 + #13	
"a" f_c top	7.92	24.1	191C	788C	1,541C
"b" f_c bot.	1.48	do	36C	10C	451C
"c" f_s top	2.39	241	576T	13,448T	18,256T
"d" f_s bot.	22.15	do	5338T	10,801T	7,285T

H-15 Truck L.L. Stresses & Dead Load Stresses n=10

THEORETICAL STRESSES psi					Average Measured Stress psi
Test Pt.	C in.	M/I_c	M/I_c x C	#8-12 + #13	
"a" f_c top	7.92	51.3	406C	1,003C	
"b" f_c bot.	1.48	do	76C	50C	
"c" f_s top	2.39	513	1,226T	14,098T	
"d" f_s bot.	22.15	513	11,363T	16,826T	