LOAD TESTS OF A PRETENSIONED GIRDER BRIDGE NEAR HAPPY, TEXAS

by

R.W. Poston
T.E. Bradberry
and
J.E. Breen

Report No. 921-1F

Research Project 3-40-85-921

Conducted for

Texas State Department of Highways and Public Transportation

by

Center for Transportation Research
Bureau of Engineering Research
The University of Texas at Austin

April 1985

PREFACE

At the request of the Bridge Division of the Texas State Department of Highways and Public Transportation (TSDHPT), the Ferguson Structural Engineering Laboratory at The University of Texas at Austin conducted a field load test of two long spans of a skewed slab-girder bridge near Happy, Texas. This report briefly summarizes the results from this load test and presents an evaluation of the bridge's structural performance. Since the bridge near Happy, Texas, had relatively long spans for a conventional slab-girder bridge, determination of the load-deflection behavior was consistent with the objectives of Research Study 3-5-84-381.

TABLE OF CONTENTS

INTRODUCTION

At the request of the Bridge Division of the Texas State Department of Highways and Public Transportatio (TSDHPT), a research study was conducted to investigate the load-deflection behavior of the middle two interior 135-ft spans of a bridge near Happy, Texas. Figure 1 shows this four-span bridge which was designed as a unidirectional single-lane ramp for access to U.S. 87. The structural capacity and serviceablity of the middle two spans were in question because of excessive dead load sag.

The objectives of this study were:

1. To conduct a load test of the two interior 135-ft spans using vehicles which simulated service plus impact load allowance magnitudes for a single lane bridge.
2. To monitor the load-deflection behavior of typical exterior and interior girders of the loaded spans.
3. To visually inspect the instrumented girders for cracks before and after the load test.
4. To calculate probable deflections from material properties and stressing histories reported by TSDHPT.
5. To report the test results and to assess the structural performance of the bridge based on these test results.

This report describes the load test of the bridge near Happy, Texas, and summarizes the results.

LOADING

Two dump trucks filled with earth were used, as shown in Figs. 2 and 3, to simulate the design level service load plus impact load magnitude on the 135-ft spans. Since the bridge is planned to be used only as a single-lane bridge for access to U.S. 87, although it has a width which could allow two-lane traffic, all loading and evaluation were done for single-lane traffic only. The gross weight of each truck was measured by the trucking company and the individual wheel loads were measured by TSDHPT at the bridge site. The recorded weights are shown in Fig. 4. There was a slight (less than 2%) discrepancy in the weights of Truck 1 as reported by the trucking company and the TSDHPT. Comparison of the total maximum moment produced by these trucks with the maximum live load plus impact moment produced by an AASHTO HS20-44 loading on the spans tested is given in Appendix A. As is shown in Appendix A, the load test performed on the bridge at Happy, Texas, subjected the loaded spans to the maximum moments that would be

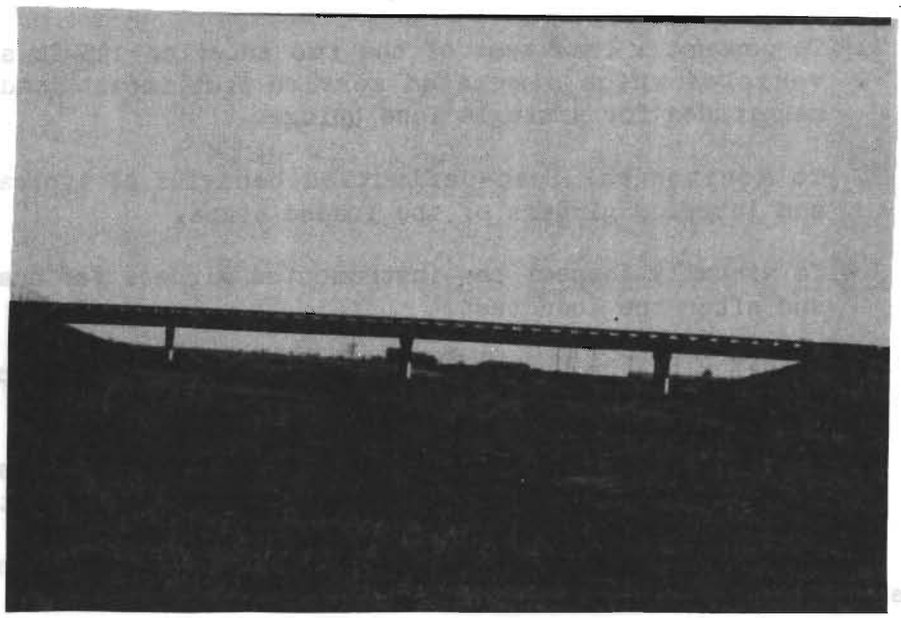


Fig. 1 Photograph of sagging bridge near Happy, Texas

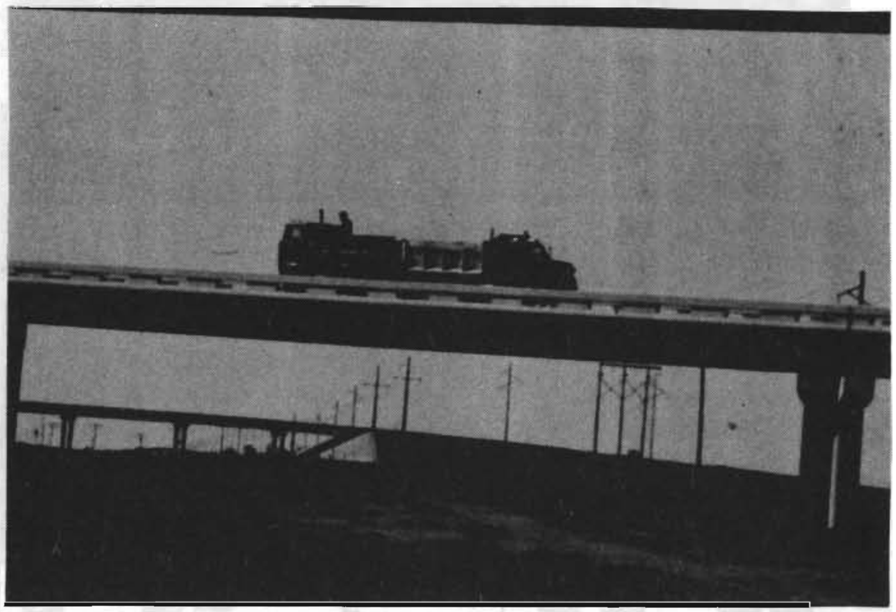
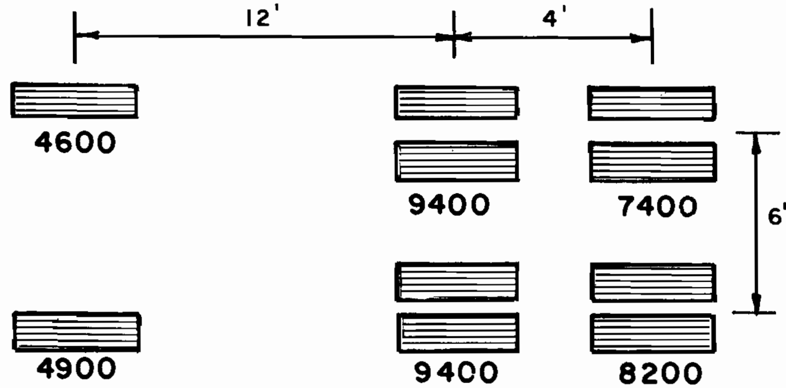


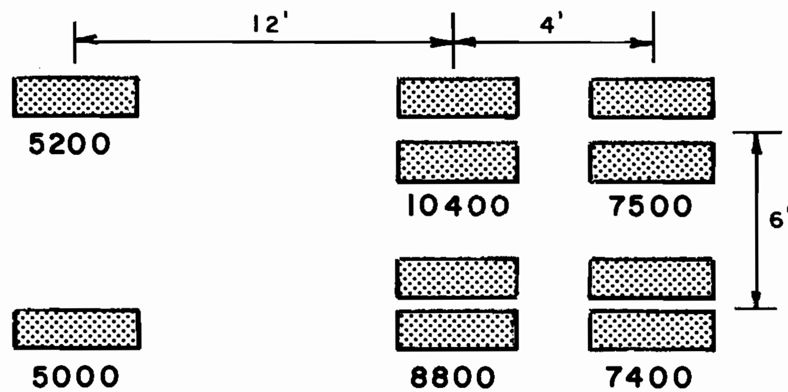
Fig. 2 Dump trucks used for simulating service
plus impact load magnitudes

Fig. 3. Positioning of dump truck for simulation of HS20-44 loading



a) SDHPT Measurement For Truck 1 (GMC Truck)

	REPORTED TOTAL WEIGHTS	
TRUCK	SDHPT	TRUCKING CO.
1	44000	44720
2	44300	44320



b) SDHPT Measurement For Truck 2 (Ford Truck)

Fig. 4 Test truck weights in pounds

produced by an HS20-44 truck loading. In all, six different tests were conducted utilizing the various truck positions shown in Fig. 5.

INSTRUMENTATION

The six girders shown in Figs. 6 and 7 were instrumented for deflection measurements. The girder deflections were monitored with respect to reference datum lines furnished by constant tension 0.016 in. dia. high-strength wires anchored to and spanning above the deck slab between bents as shown in Figs. 8 through 10. Using the initial deflected shapes of the wires and the bridge dead load condition as references, slab live load deflections over the instrumented girders were measured at the quarterpoints and centerlines. The supports were assumed not to deflect throughout the testing. Measurements of the gap between reference wire and reference points attached to the slab were made with a ruler accurate to 1/64 in. The reference points were stainless steel seats attached to tin caps epoxied to the base surface of the deck slab as shown in Figs. 11 and 12. When the tension in the piano wire is held constant or adjusted to a constant value prior to taking readings, the sagging shape of the piano wire is fixed. Therefore, the difference between an initial set of readings and readings taken with trucks on the bridge represented girder deflections under load.

Tension in the high-strength (piano) wire could not be measured directly. The indirect method being used in Research Project 3-5-84-381 was employed to maintain constant tension and thus establish the desired reference sagging shape of the piano wire. Figure 13 illustrates the essential steps of this "standard weight deflection" method of maintaining the initial shape of the reference and thus the initial wire tension throughout the testing. The wire sag due to a calibrating weight is checked and the tension adjusted to produce a constant sag. Each wire was "calibrated" with this procedure prior to taking deflection readings.

Correcting for Thermal Effects

The temperature gradient produced by solar energy induced upward girder deflections over the four-hour testing period which were of the same order as the deflection due to truck loading. Thus, thermal effects could not be ignored. Figure 14 indicates the approximate time the deflection readings were taken for a given load case. Assuming a linear change with time of girder uplift due to rising temperature differential, thermal change corrections were determined to allow calculation of net girder deflections due to the truck live load. The net girder deflections were established by adding a percentage of the deflection due to temperature effects to the measured live load deflections taken at a given time.

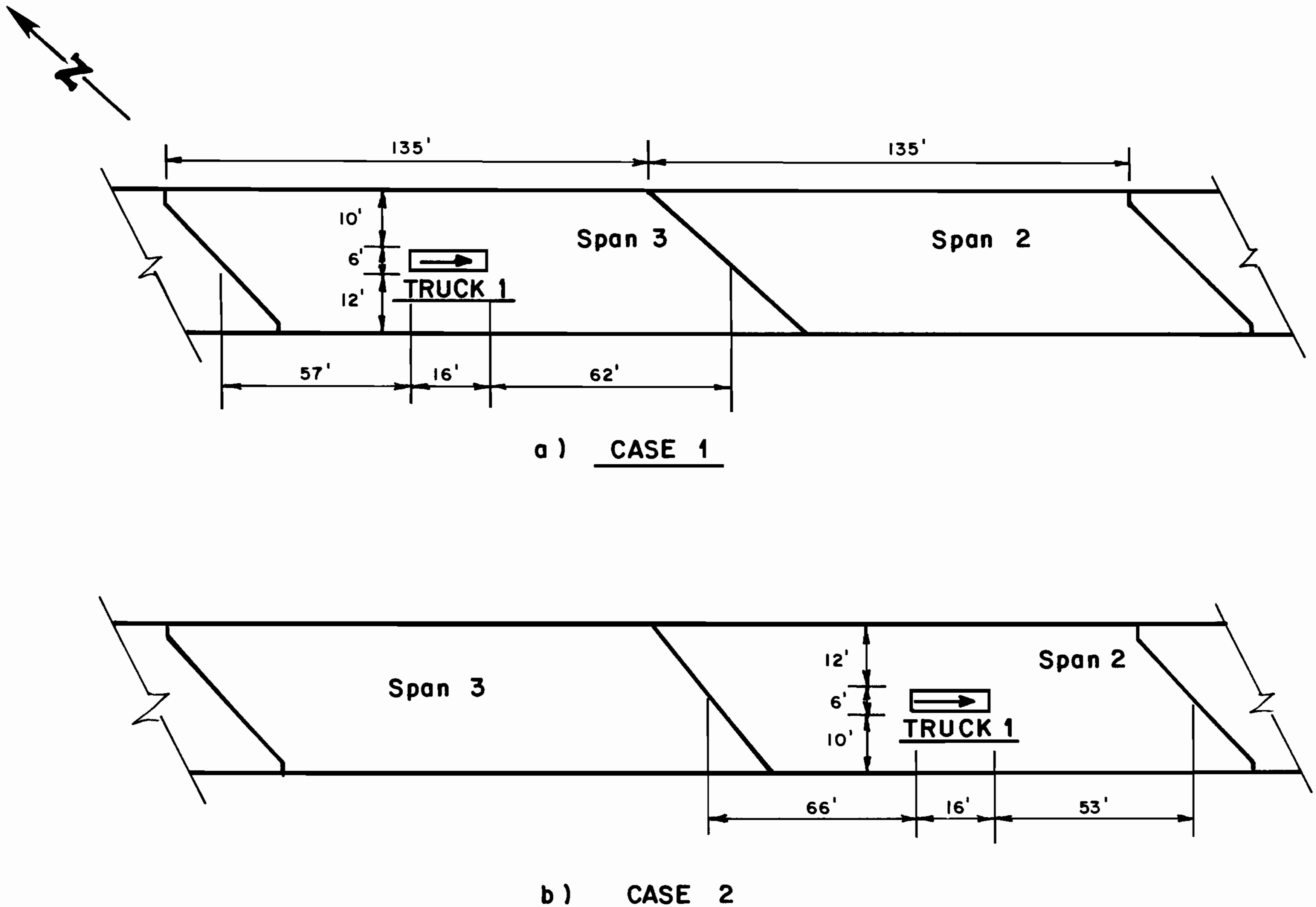
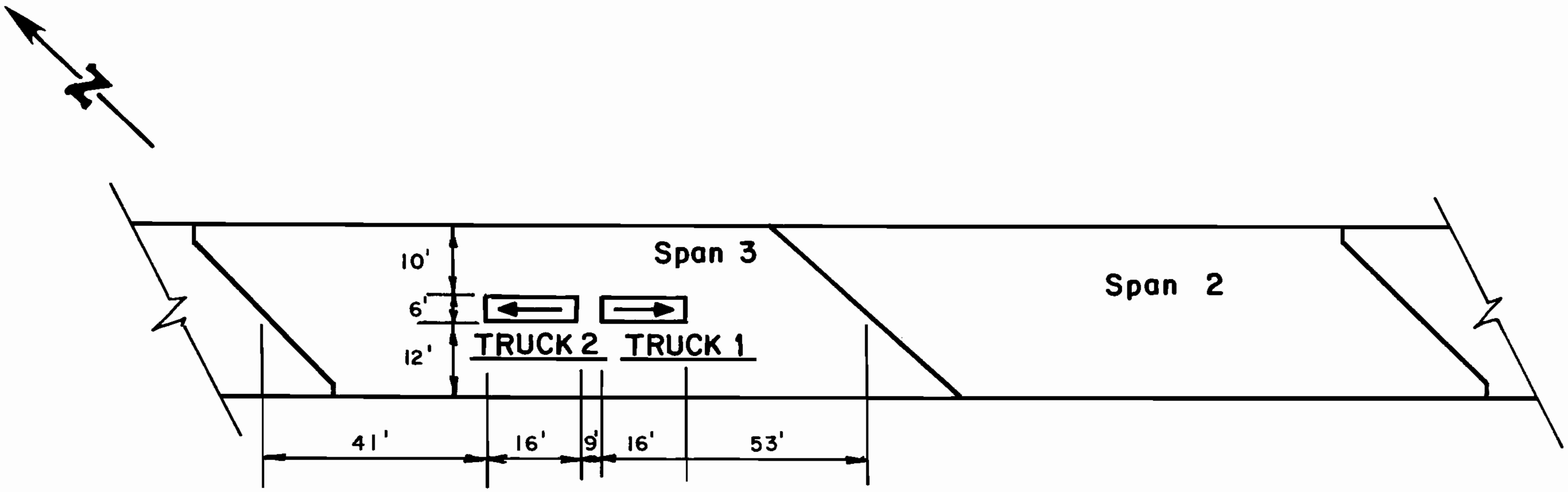
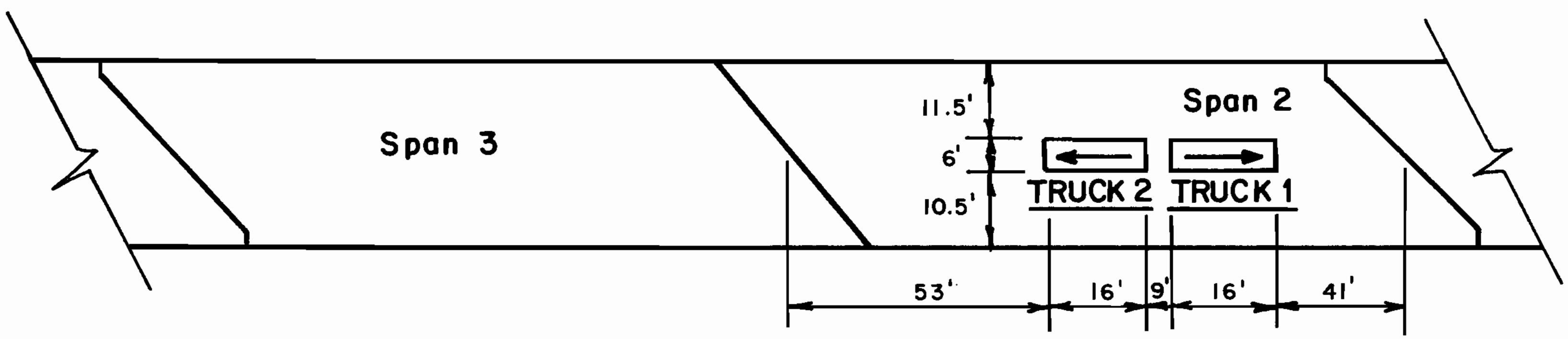


Fig. 5 Truck positions for various tests

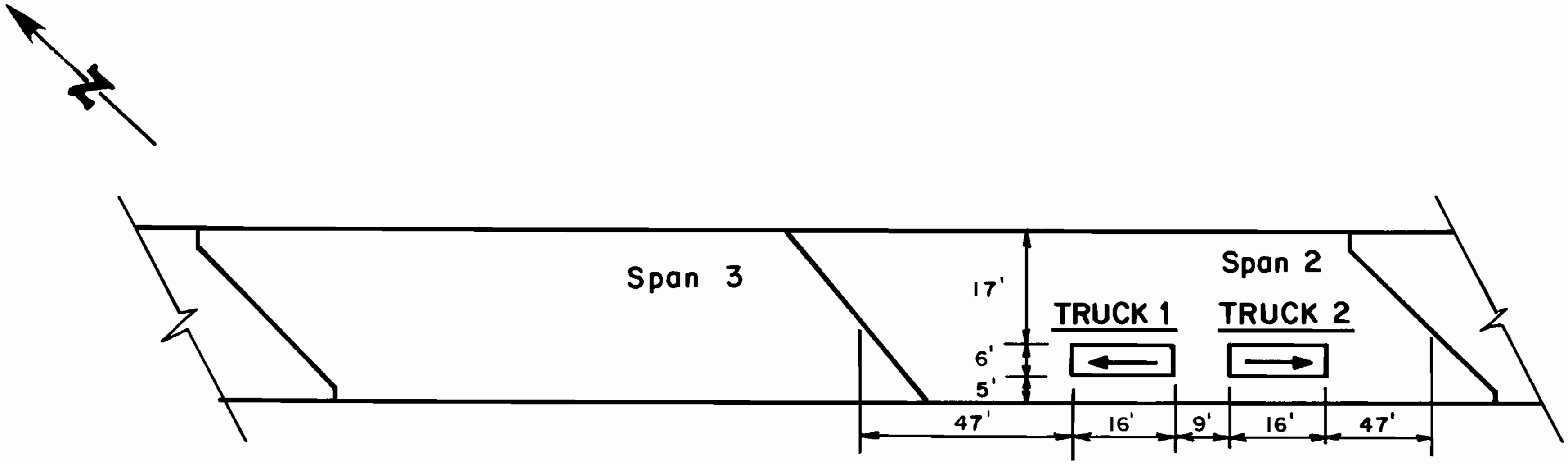


c) CASE 3

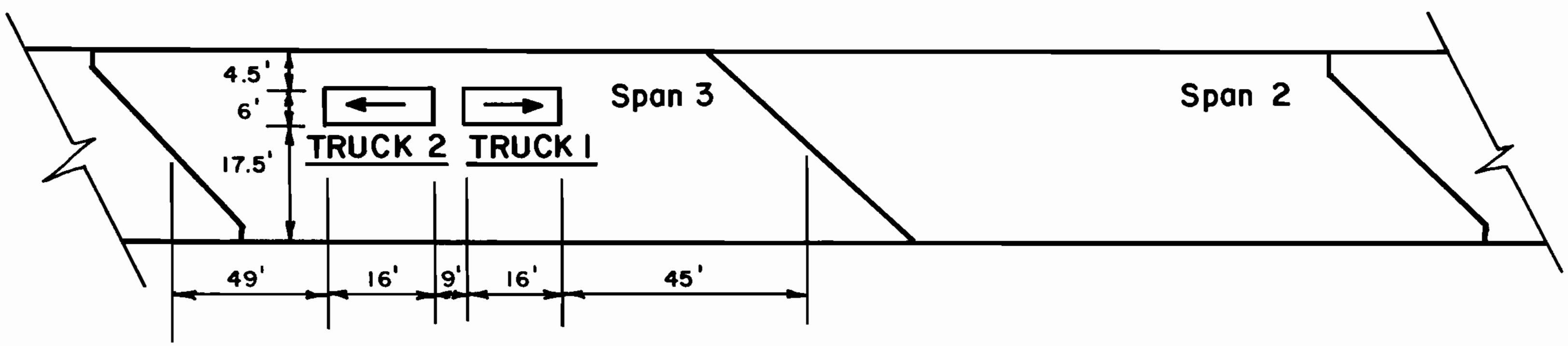


d) CASE 4

Fig. 5 Truck positions for various tests (cont'd)



e) CASE 5



f) CASE 6

Fig. 5 Truck positions for various tests (cont'd)

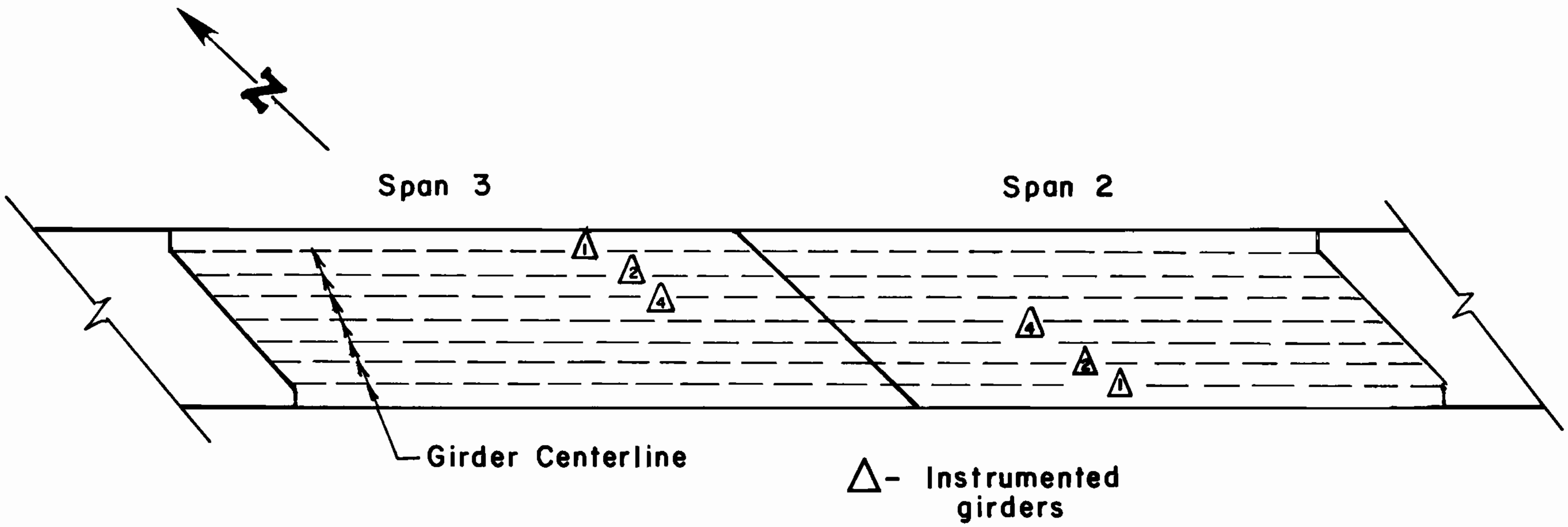
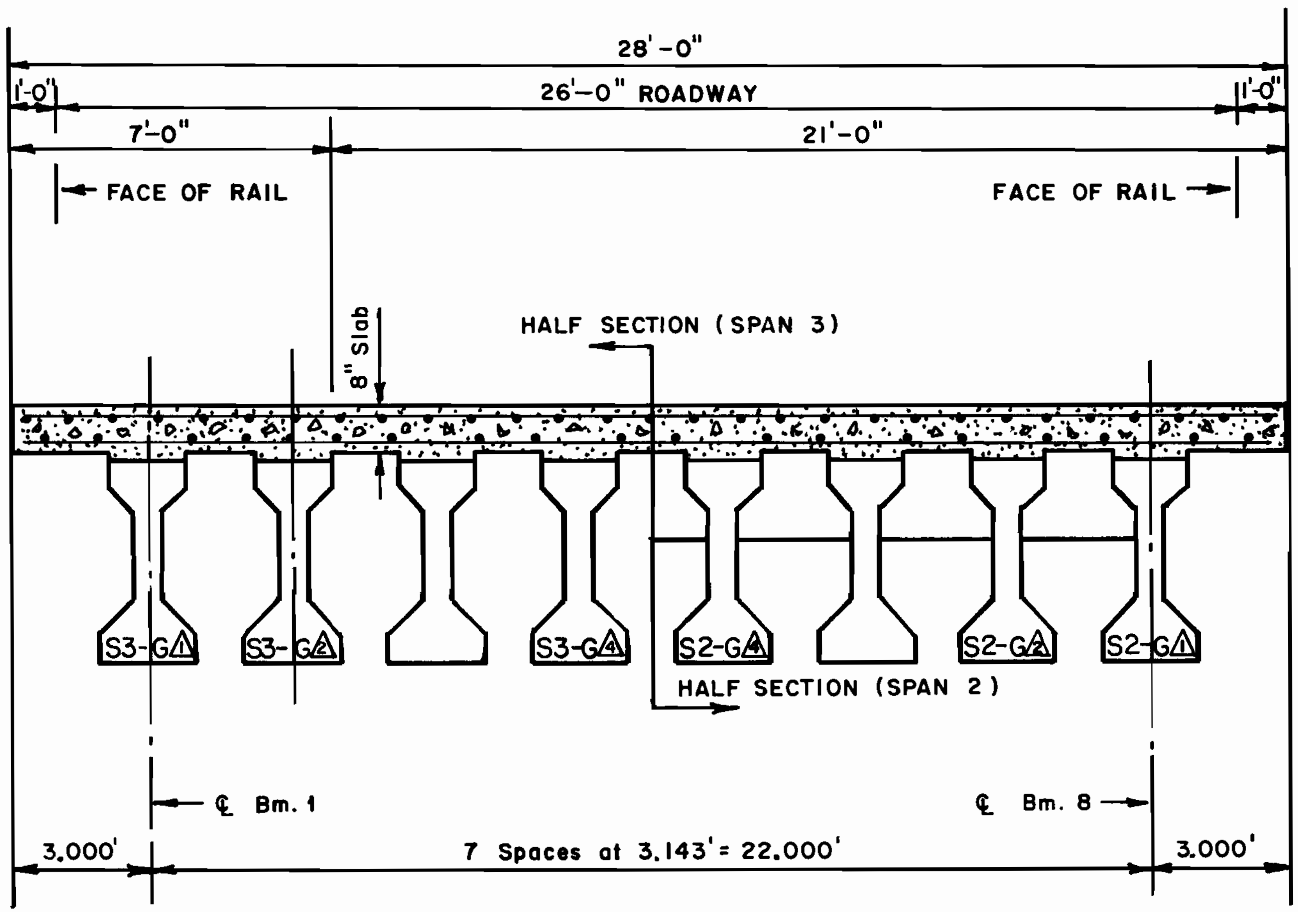


Fig. 6 Six girders instrumented for deflection measurements



NOTES: 1. DIAPHRAGM AND OVERLAY NOT SHOWN.
2. S # = SPAN
G# = INSTRUMENTED GIRDER

Fig. 7 Transverse section of bridge showing instrumented girders (looking SE)

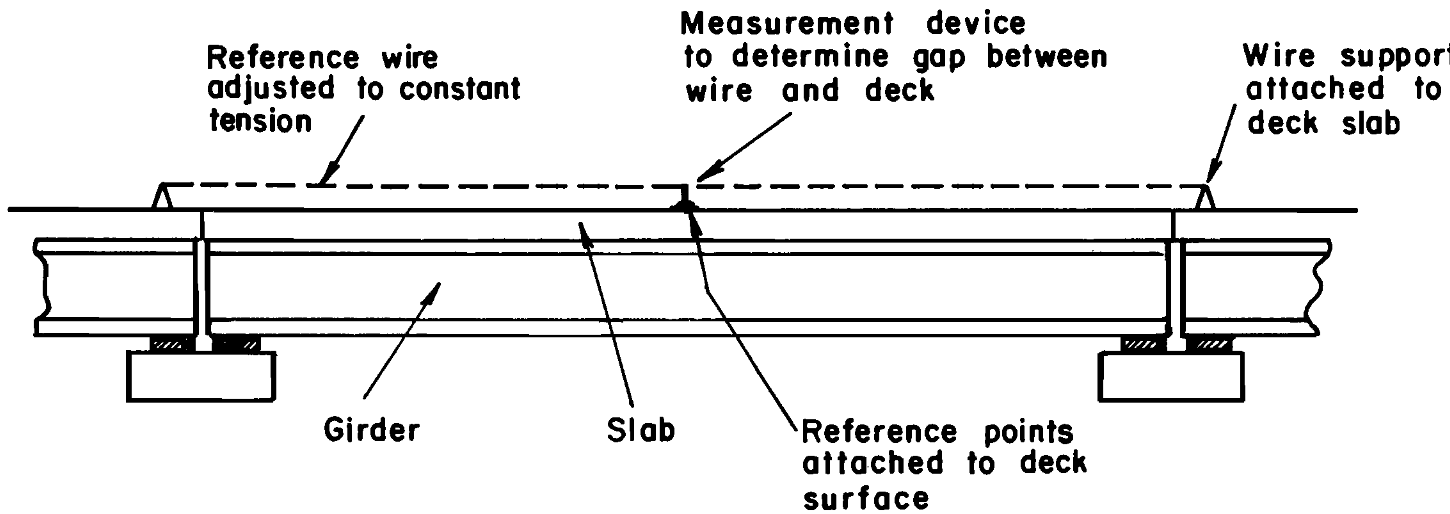


Fig. 8 Schematic of deflection measurement system

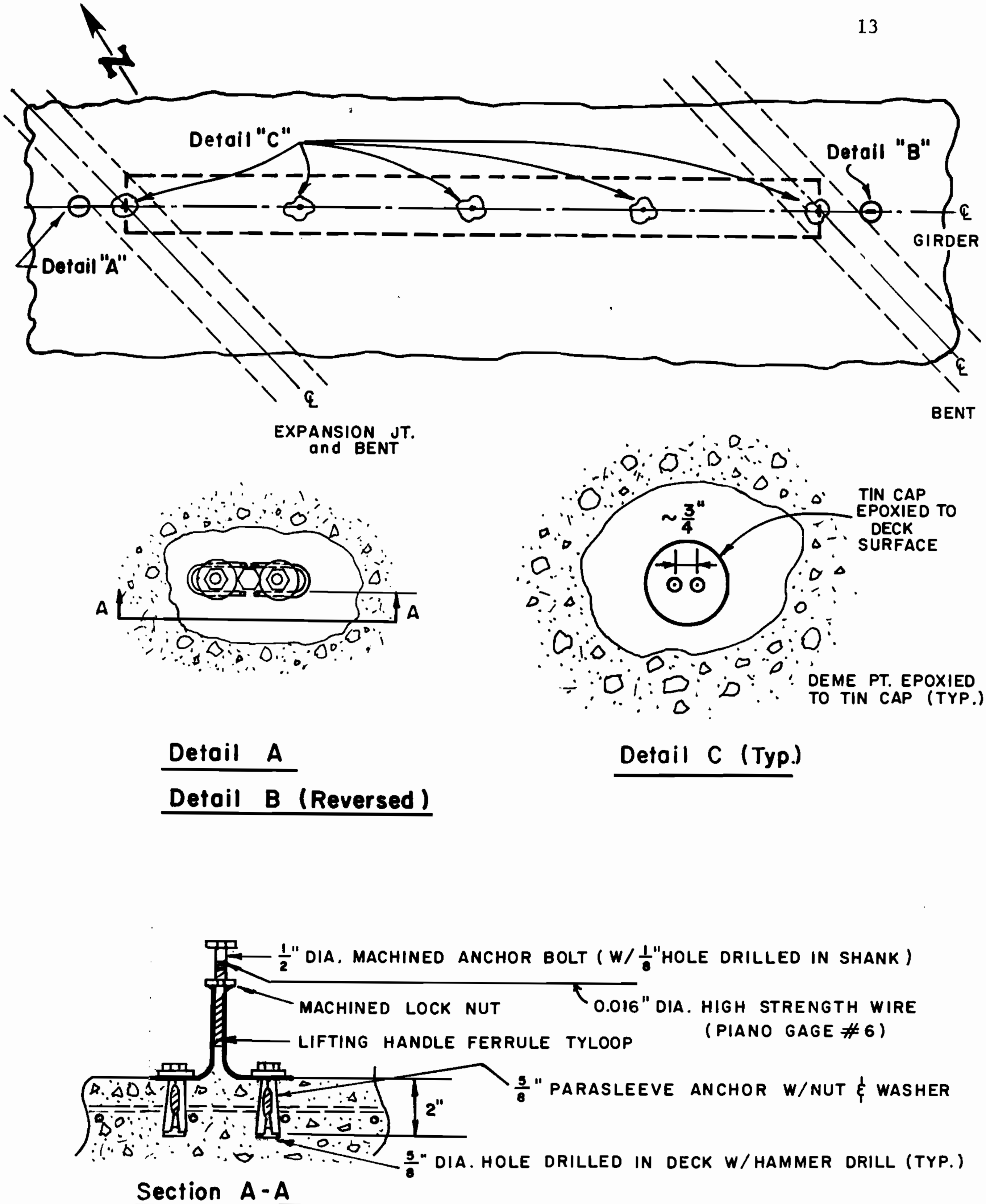


Fig. 9 Layout of deflection measuring instrumentation for a typical girder (Span 3 as shown; Span 2 reversed) (no scale)

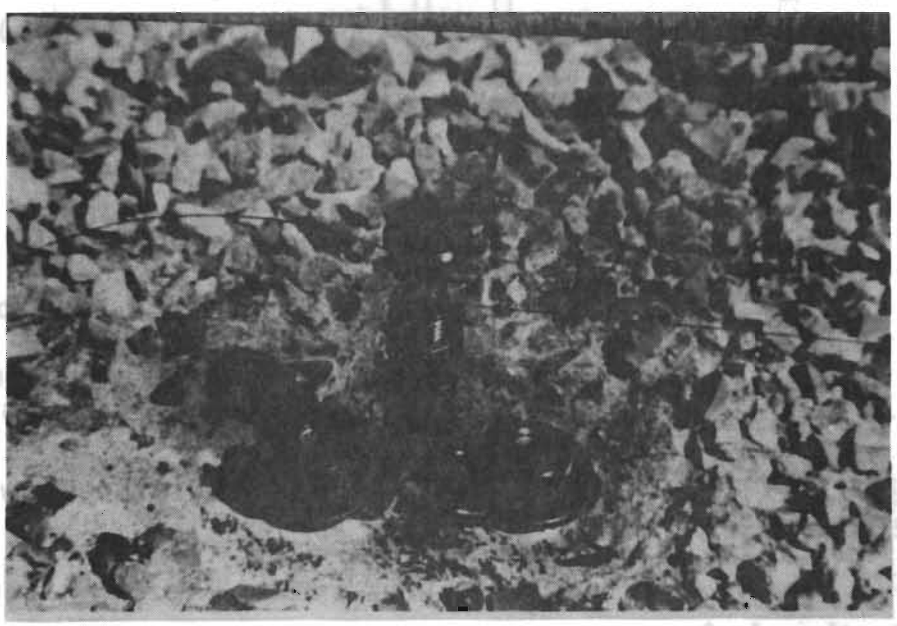


Fig. 10 Reference wires anchored in deck slab above girders of interest

Fig. 11 Taking deflection readings with stainless steel ruler
(Note: ruler seated on stainless steel reference)

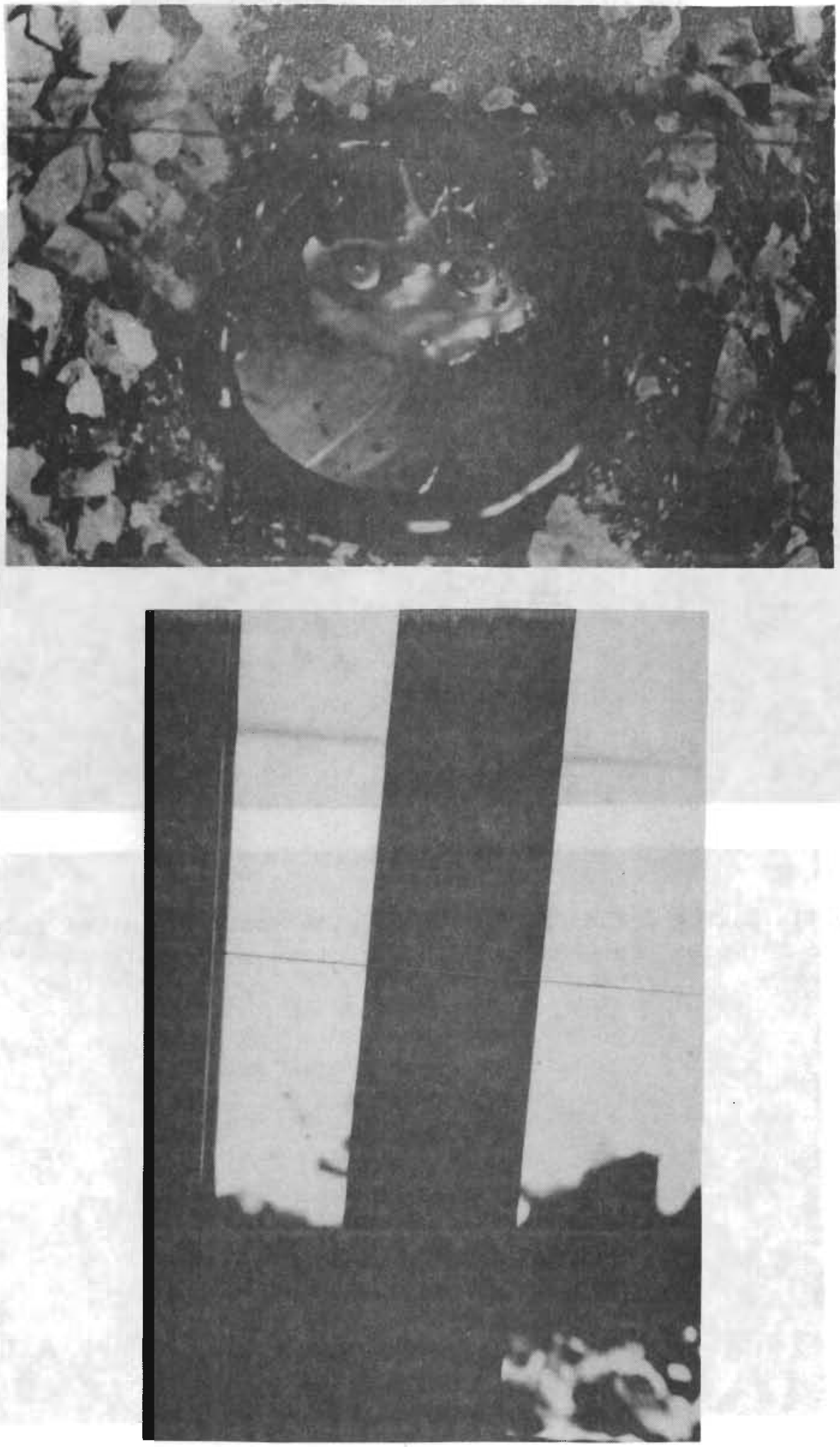
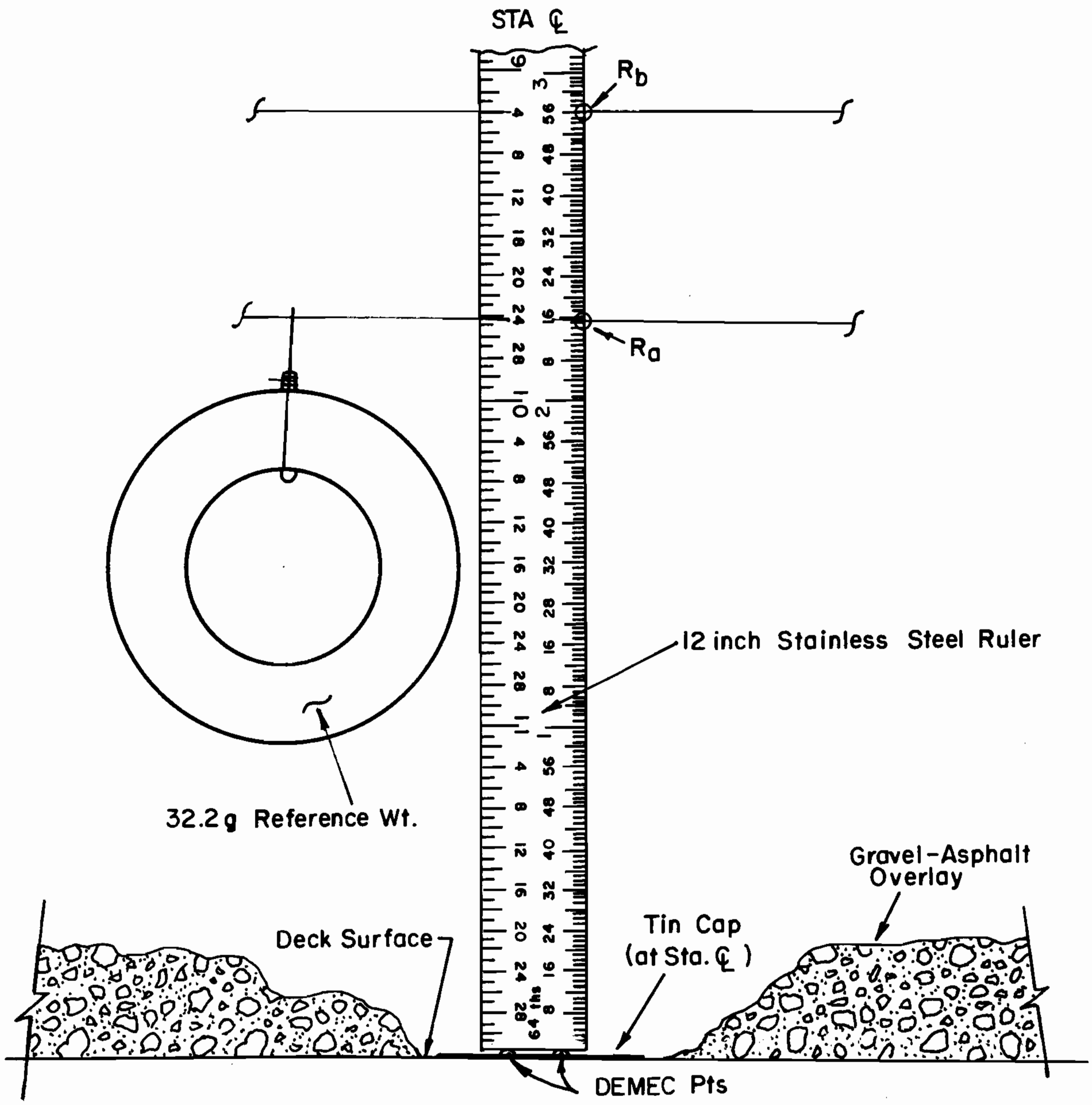
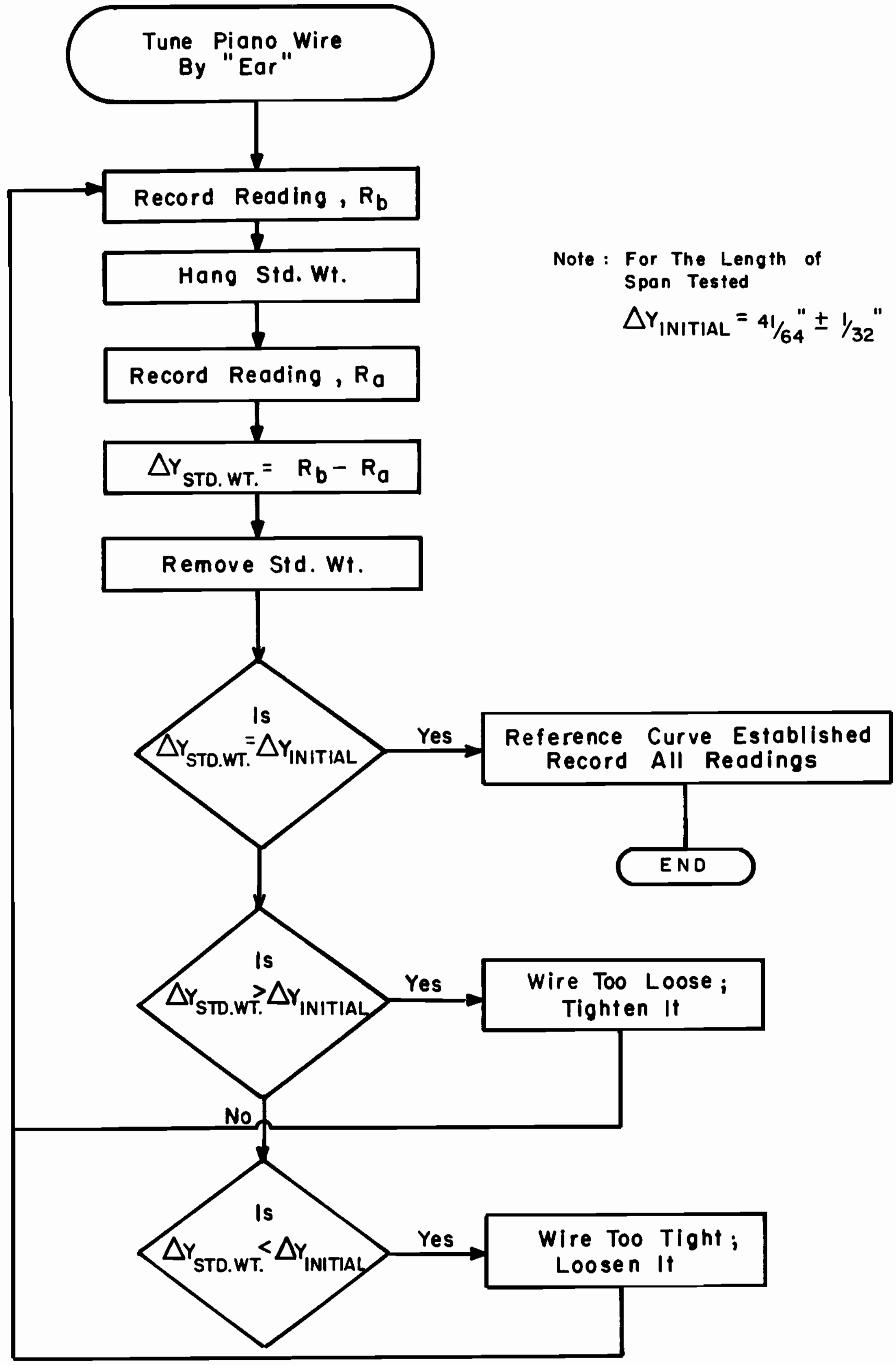


Fig. 12 Stainless steel ruler, piano wire, and reference points



a) calibration procedure

Fig. 13 Standard weight deflection procedure for establishing and maintaining a fixed reference curve with high strength wire



b) calibration operations

Fig. 13 Standard weight deflection procedure for establishing and maintaining a fixed reference curve with high strength wire (cont'd)

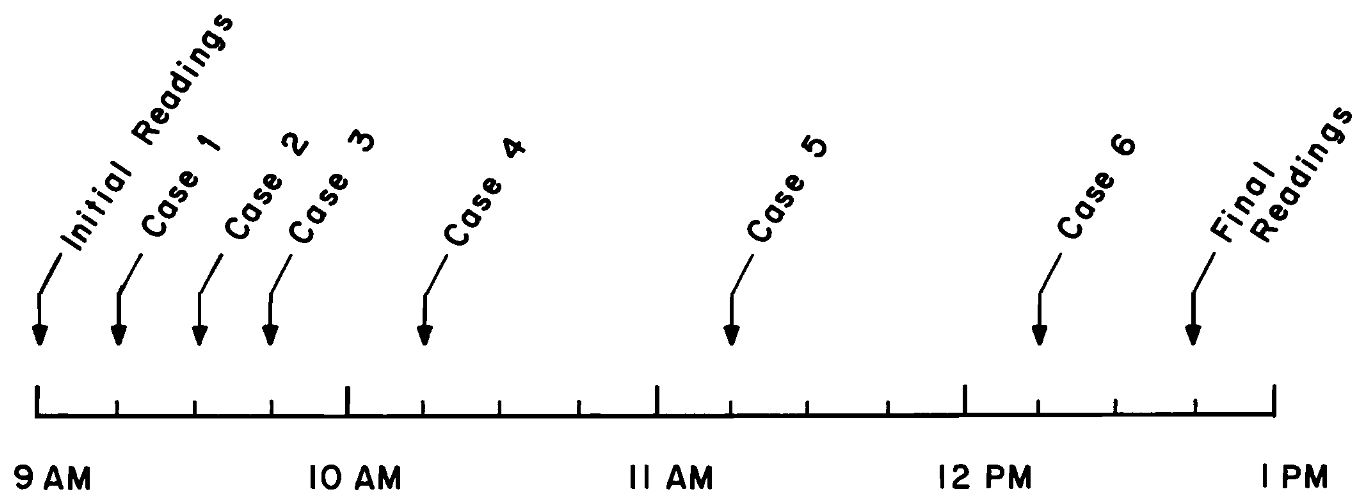


Fig. 14 Chronological sequence of testing

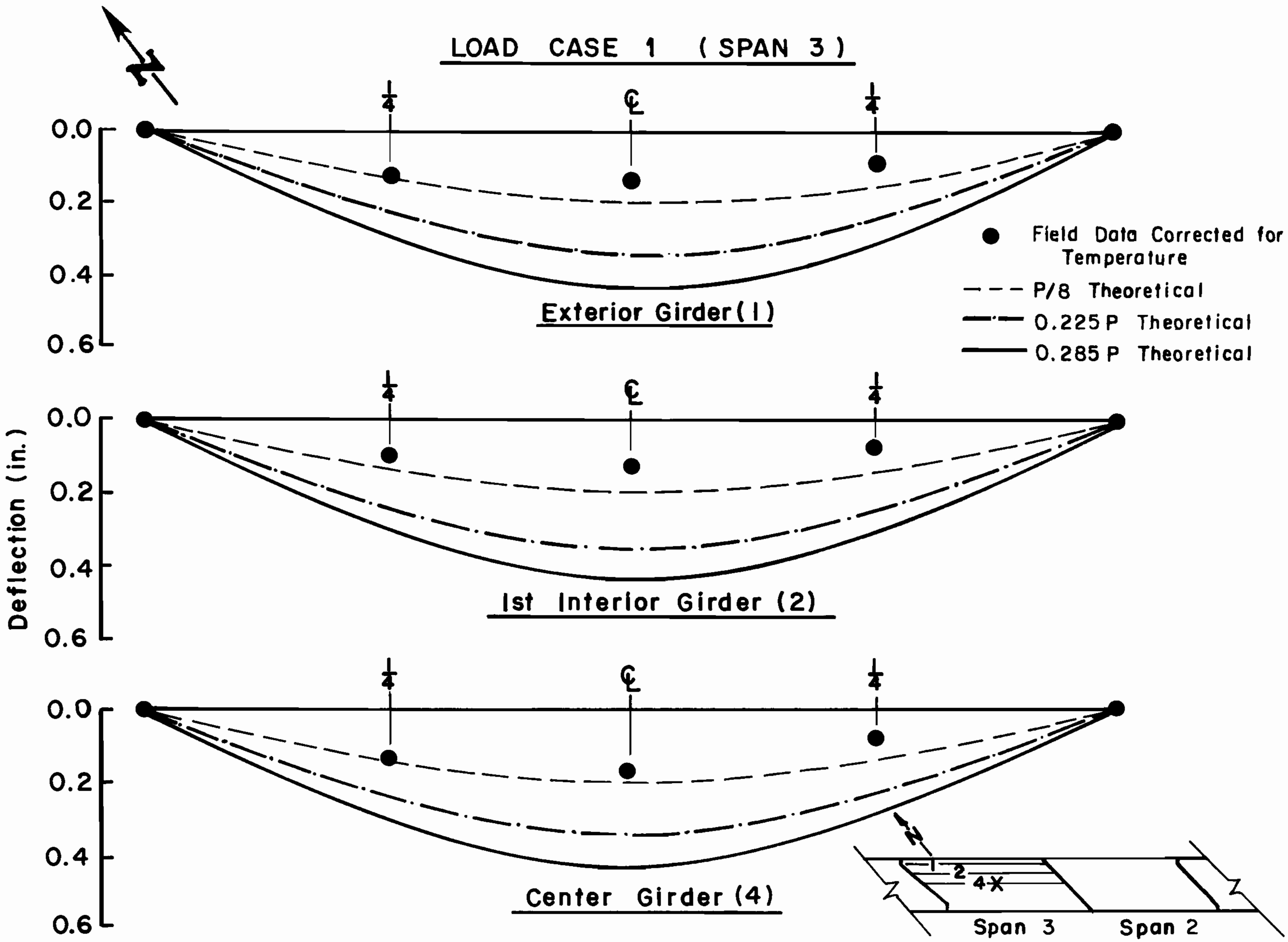
TEST RESULTS

Initial deflection readings, deflection readings for the various load cases shown in Fig. 4, and final deflection readings are summarized in Appendix B. The difference between the initial readings and the final readings represents the influence of temperature changes on girder dead load deflections. Corrections to the apparent live load deflections were made as discussed in the instrumentation section of this report in order to establish the net live load deflection due to the truck loadings.

The net girder deflections for each load case are summarized in Figs. 15 through 20. Also included on these figures are theoretical girder deflections computed assuming equal distribution of load to each of eight girders (P/8 theoretical), and a computed assuming AASHTO wheel load distribution of S/5.5 (0.285P theoretical) and S/7.0 (0.224 theoretical). These correspond to AASHTO two-lane and one-lane bridges respectively. S is the girder spacing which in this case is 3.14 ft. The load locations which were used for determining the theoretical girder deflections are summarized in Appendix C. The theoretical deflection curves are based on reported deck slab concrete strength of 4000 psi and girder concrete strength of 8750 psi. The effective slab width was taken as the girder spacing, and the slab thickness was assumed to be 8 in. which was the average thickness for the in-place concrete as previously determined from cores. The girder properties assumed for the analysis are summarized in Appendix D. Deflection calculations ignore any stiffening effect of the curb and rail systems. Such effects are discussed later.

Figures 15 through 20 indicate that the actual girder deflections are generally equal or somewhat less than the theoretical computed deflections based on an assumed equal distribution of load to each of the eight girders (P/8 theoretical). For Load Cases 5 and 6 in which both trucks were placed near the barrier railing, the actual deflections of the exterior girders are nearly identical to the P/8 distribution theoretical deflections. The results also show that the center and first interior girder deflections were very similar to Spans 2 and 3 for corresponding truck loadings. This indicates fairly symmetrical load-deflection behavior between spans. However, the quarterpoint observations of the exterior girders do not show as good agreement. In all cases, the observed deflections are substantially less than the computed values using the AASHTO distribution factors.

The observed deflections are probably less than the P/8 distribution theoretical deflections for several reasons. The simplified elastic analysis employed to compute deflections uses stiffness parameters based on assumed material properties. The actual concrete strength in this 2-1/2 year-old bridge and thus the concrete elastic modulus are somewhat greater than that assumed for the analysis. In addition, the stiffening effect of the barrier railings


N
LOAD CASE 1 (SPAN 3)
1/4
℄
1/4
0.0
0.2
0.4
0.6
Exterior Girder (1)
Field Data Corrected for Temperature
P/8 Theoretical
0.225 P Theoretical
0.285 P Theoretical
Deflection (in.)
1st Interior Girder (2)
Center Girder (4)
N
1
2
4 X
Span 3
Span 2

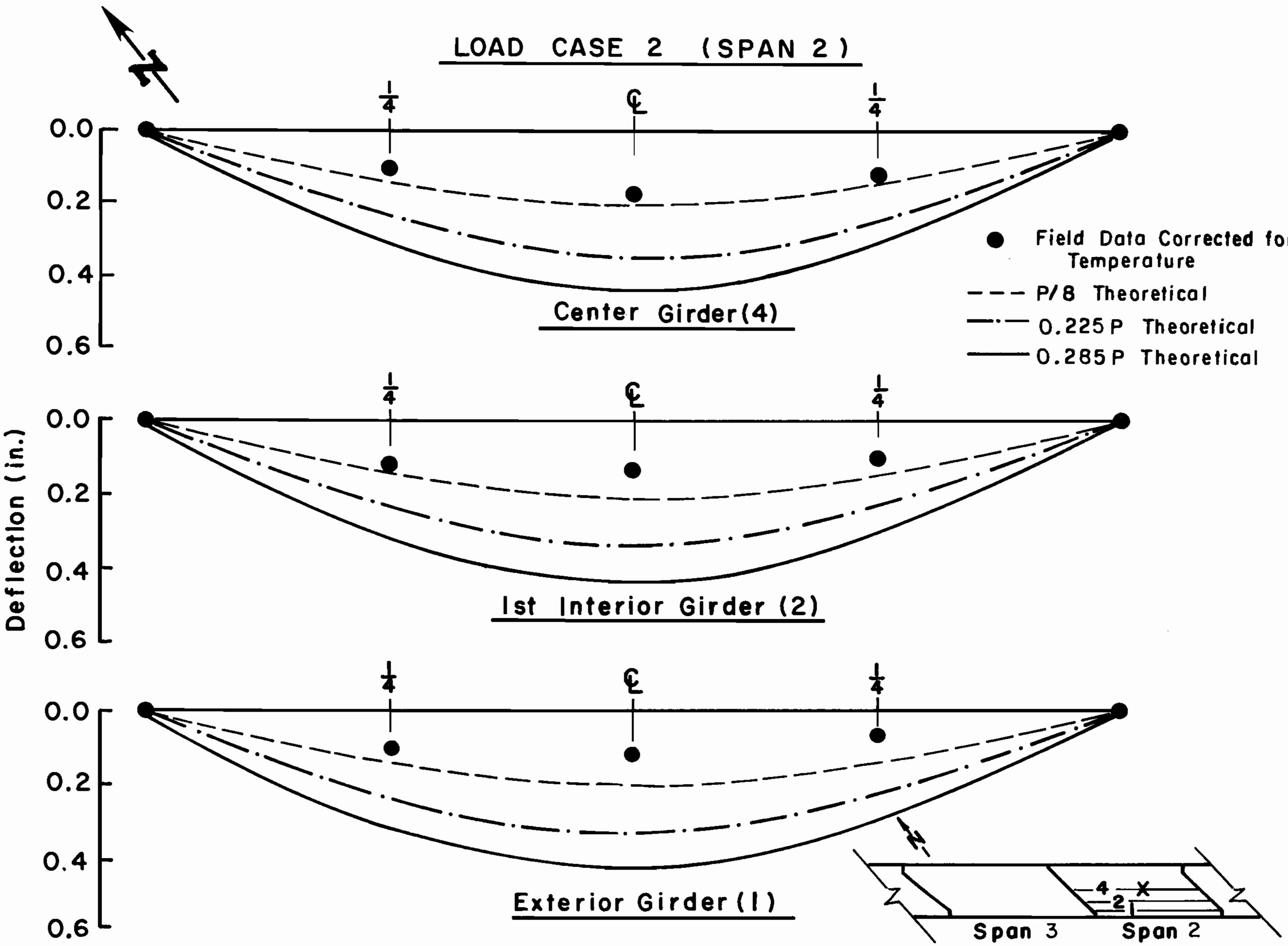


Fig. 16 Girder deflections for Load Case 2

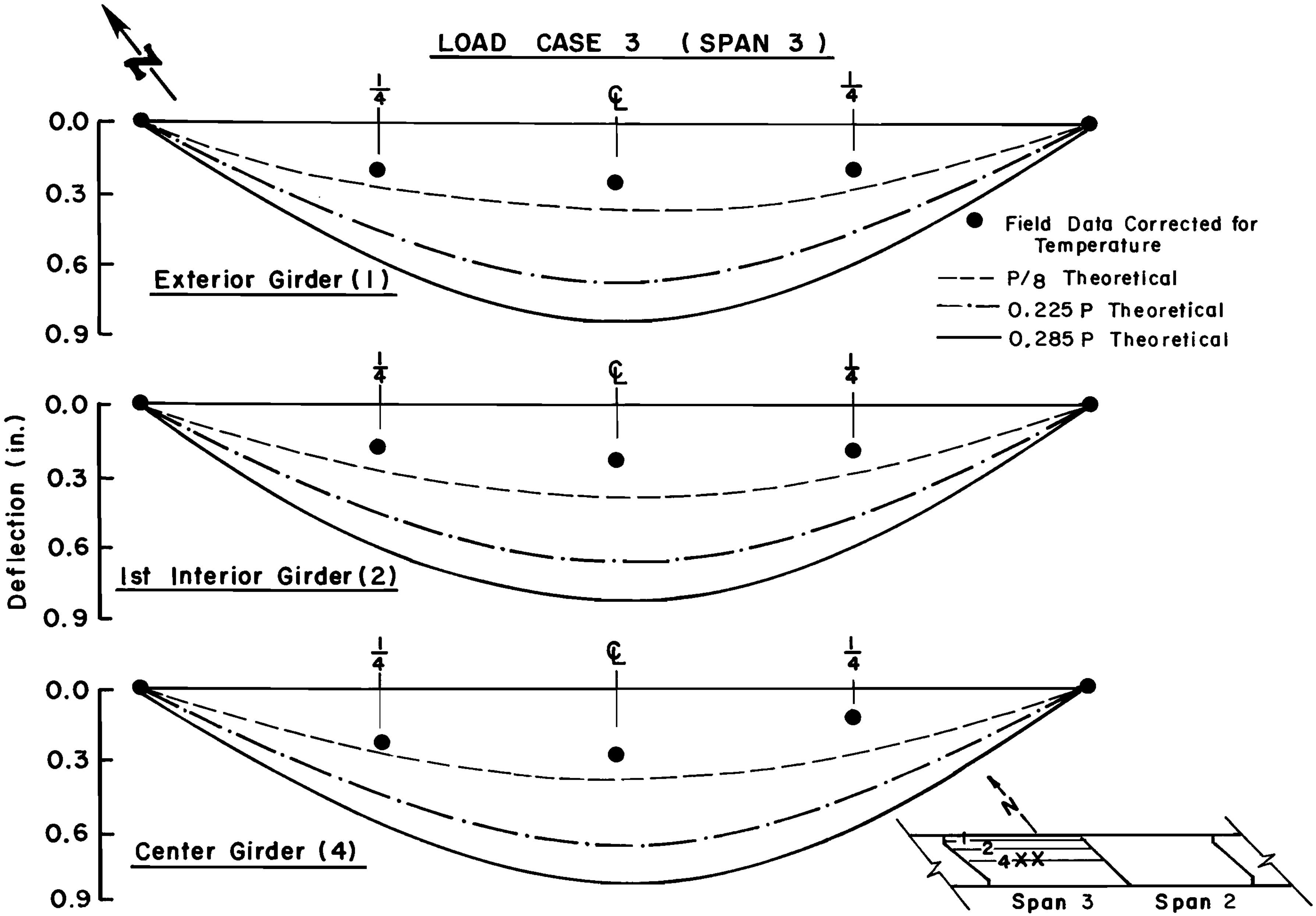


Fig. 17 Girder deflections for Load Case 3

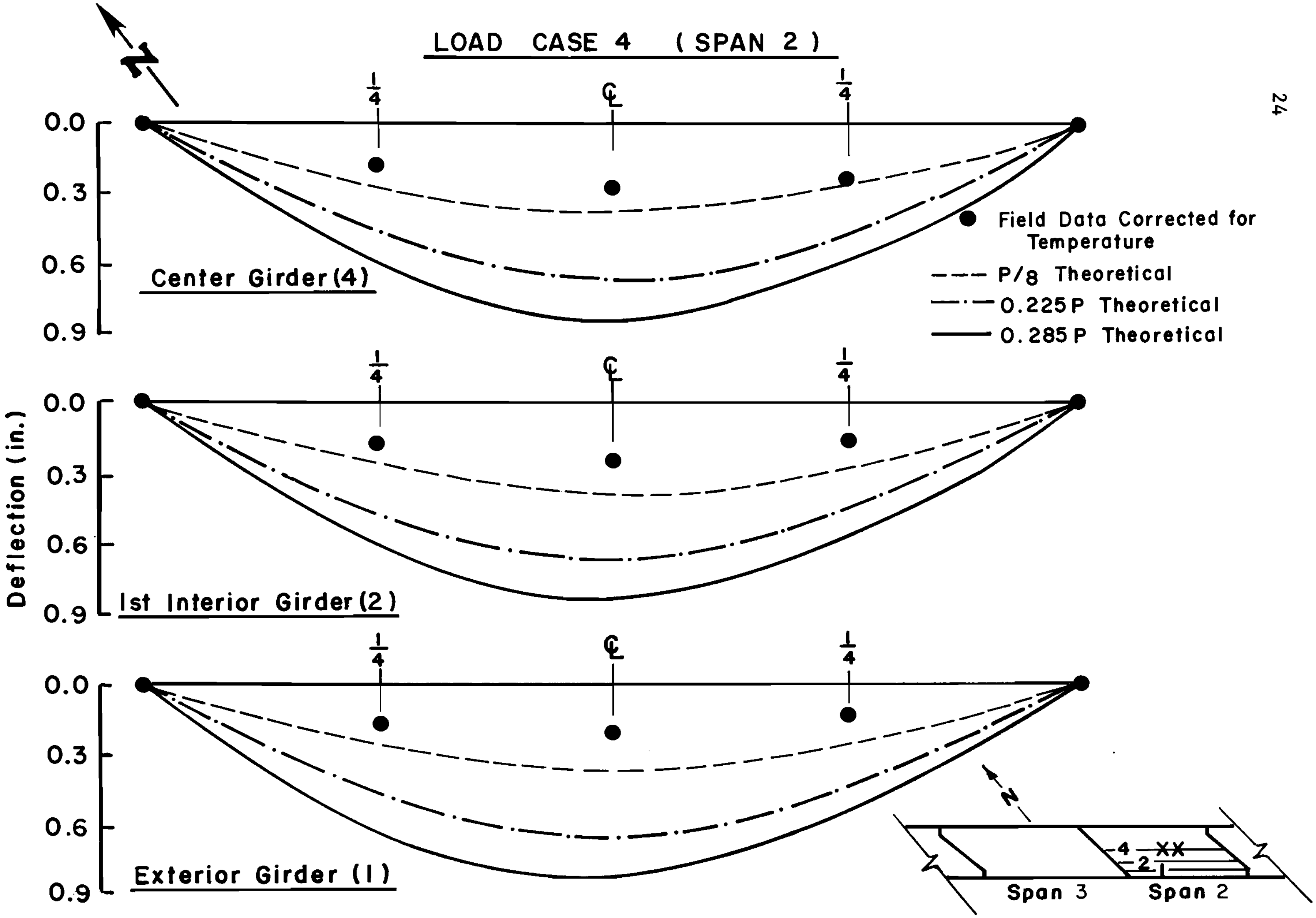


Fig. 18 Girder deflections for Load Case 4

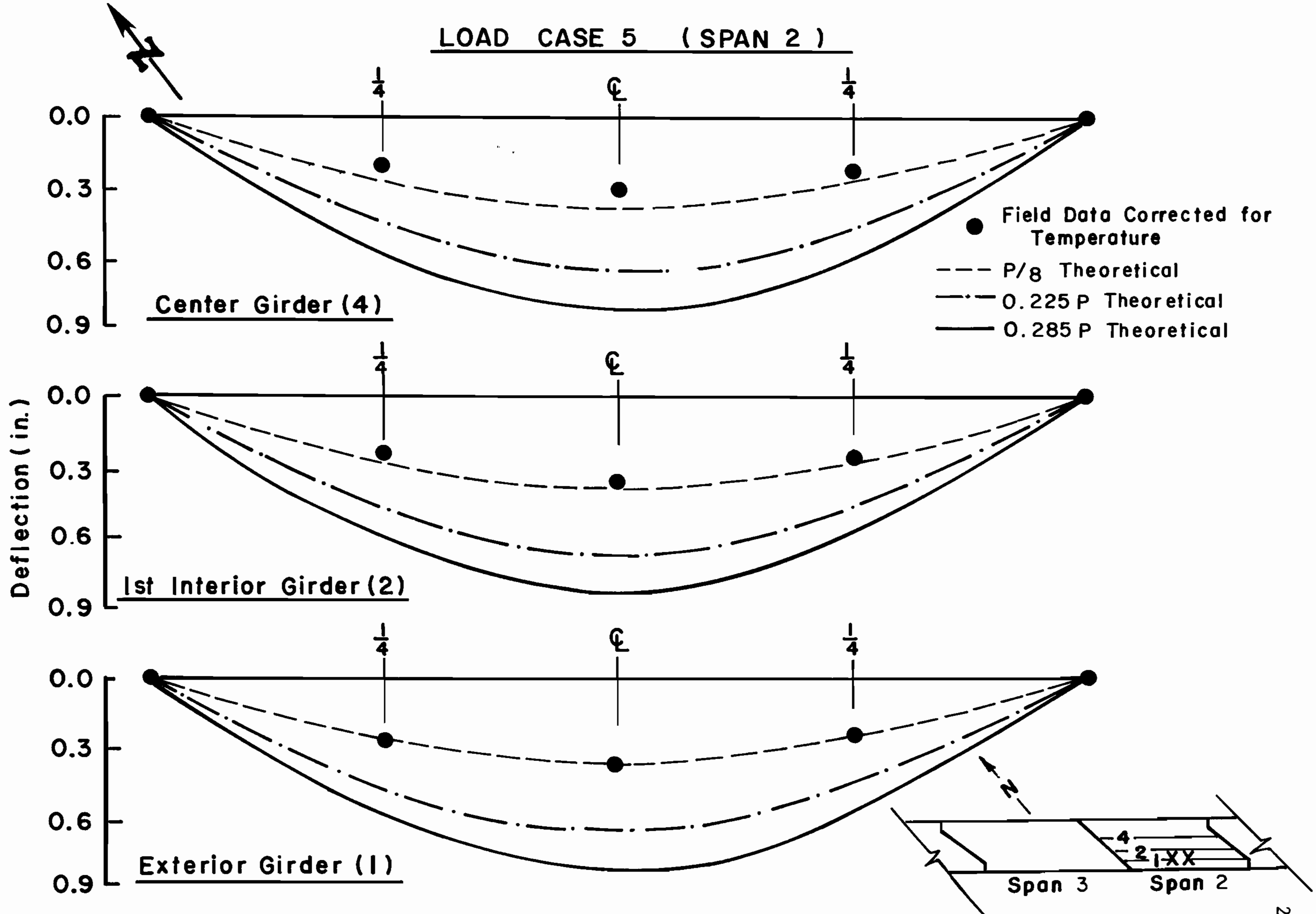


Fig. 19 Girder deflections for Load Case 5

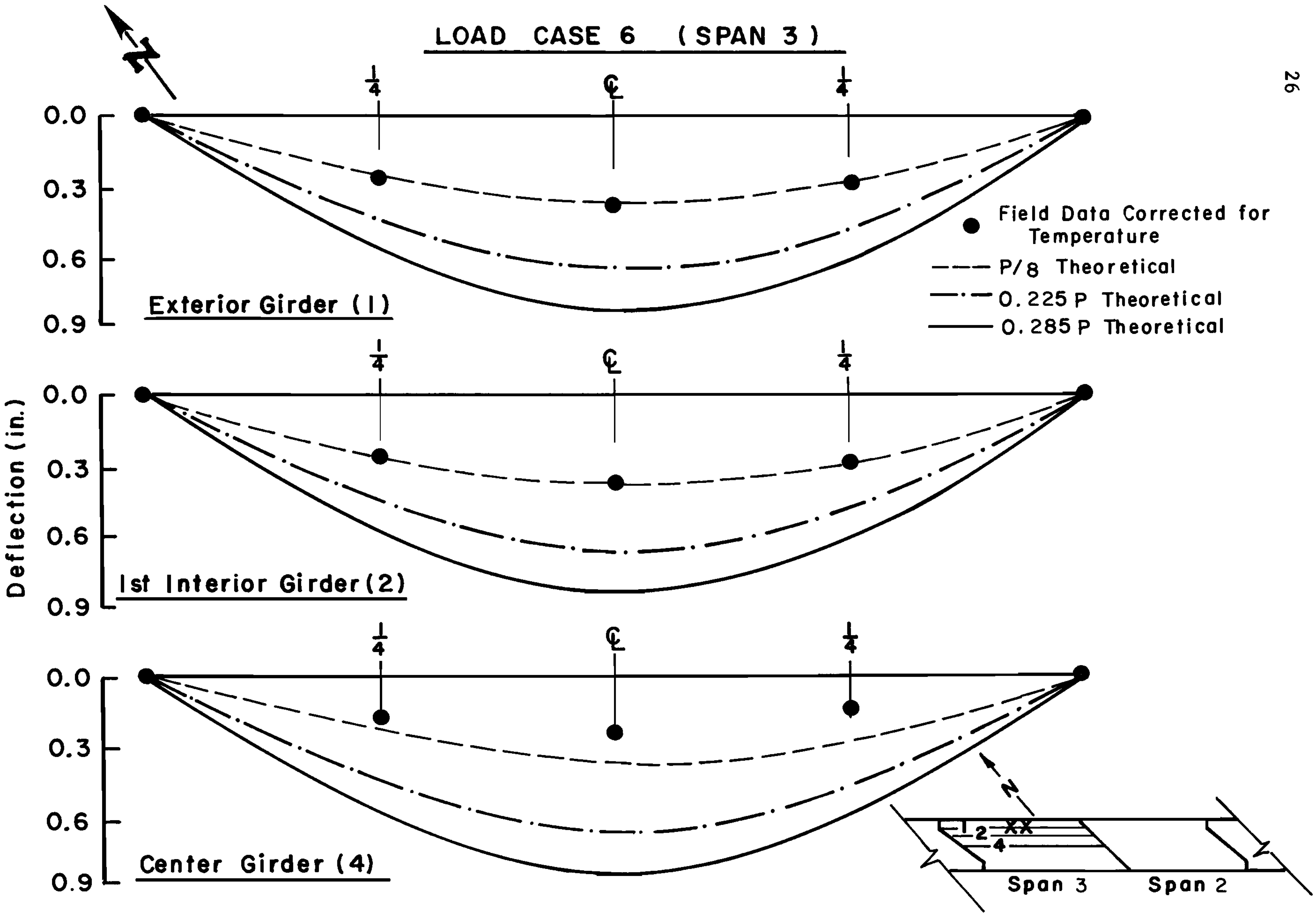


Fig. 20 Girder deflections for Load Case 6

was neglected in the simplified analysis. As shown in Appendix D, including the stiffening effect of the barrier railings and concrete strengths greater than those assumed would reduce the theoretical deflections by more than 18% (12% for rails and 6% for increased concrete strength).

While extensive cracking was apparent in the barrier railings and top surface, as shown in Fig. 21, a detailed visual inspection of the instrumented girders (see Fig. 22) before and during the load test indicated no cracking of the girders. This result is substantiated by the observed load-deflection behavior shown in Fig. 23, for the centerpoint of the middle girders of each span. Cracking in the girders would have resulted in a marked decrease in girder stiffness with increased load and would be evidenced by a decrease in the slope of the load-deflection curve. The results in Fig. 23 clearly indicate no decrease in the stiffness up to service plus impact load levels for a single-lane bridge. In fact, the observed data indicate that, if anything, the bridge seemed stiffer under the second truck application. Such apparent stiffness may reflect changed load distribution or error in compensating for temperature effects.

For the length of span of the bridge tested, the AASHTO preferred maximum live load deflection, L/800, would be more than 5-1/4 times the maximum measured deflection during load testing. Even though the girders showed marked sag under dead load, the small deflections indicate that with respect to live plus impact loadings, the Happy, Texas, bridge is within the serviceability requirements of AASHTO and, in fact, is very stiff. Bridge girders of this type have a substantial reserve between cracking and ultimate. Since there was no apparent cracking under full dead load plus live load plus impact load level as determined either directly through visual inspection or indirectly through load deflection behavior, the bridge investigated should have a substantial reserve in strength above single-traffic lane service load levels.

The bridge does have a substantial amount of girder sag (on the order of 4 in. for at least one of the girders). This sag, although aesthetically undesirable, does not appear to cause the bridge to be unsafe at service load conditions. The amount of sag will probably increase slightly when the deck overlay is placed. However, further time dependent sag should be negligible in this 2-1/2-year-old bridge, as most of the concrete creep and shrinkage should have already occurred. Since the clearance between the bottom of the girders and the subgrade of the underlying interstate may be some 4 in. less than was originally designed, adjustments to the grade of the highway may be necessary. Should the bridge ever be designated for two-lane use, further investigation and a higher level of load testing is recommended.

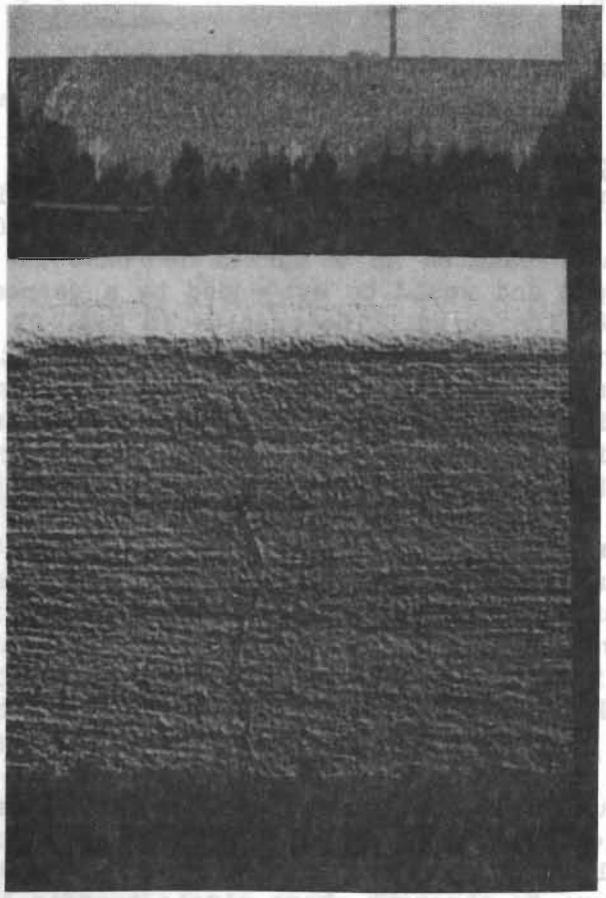


Fig. 21 Apparent shrinkage cracks in concrete guard rail

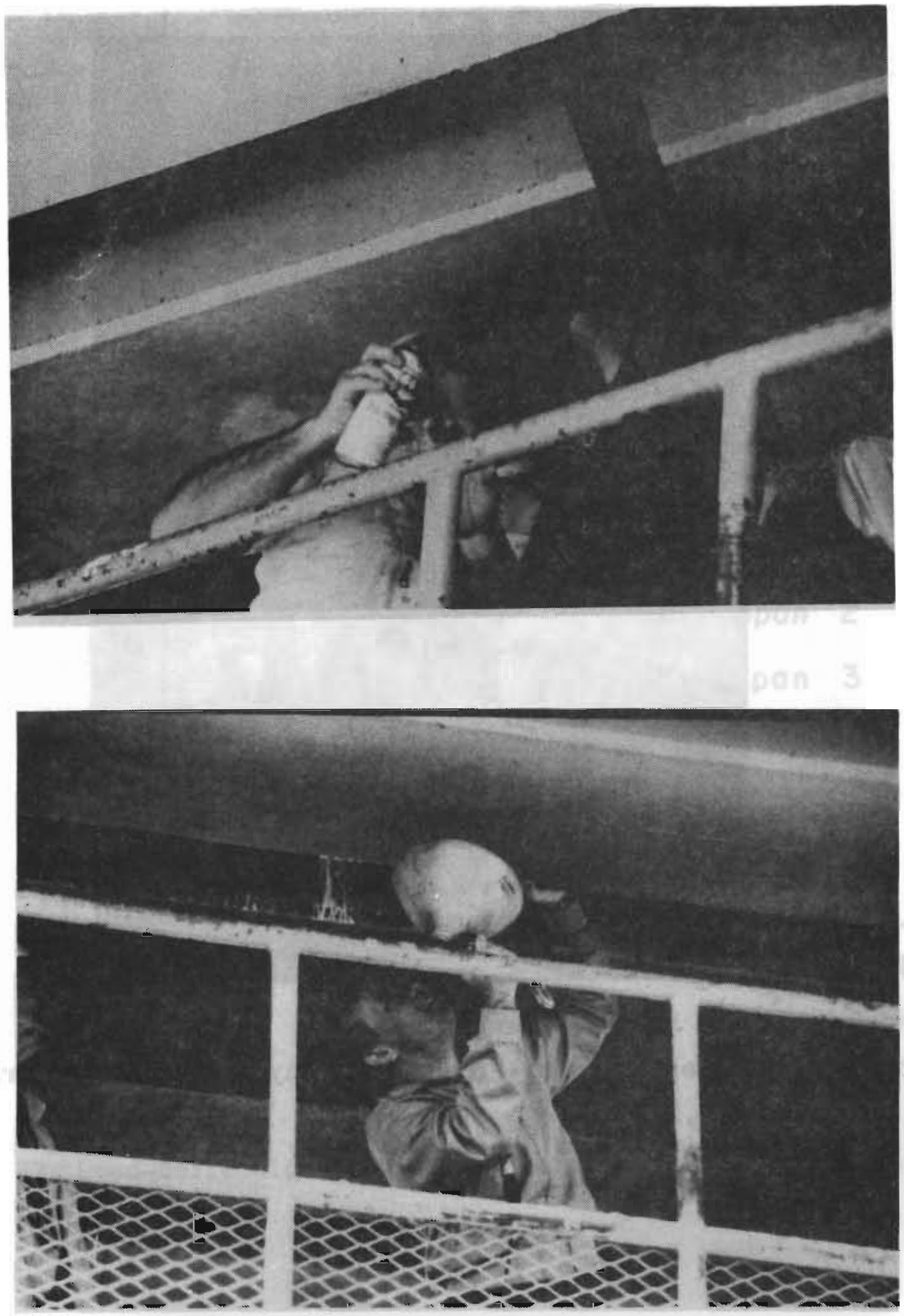


Fig. 22 Detailed inspection of girders for any indication of cracking

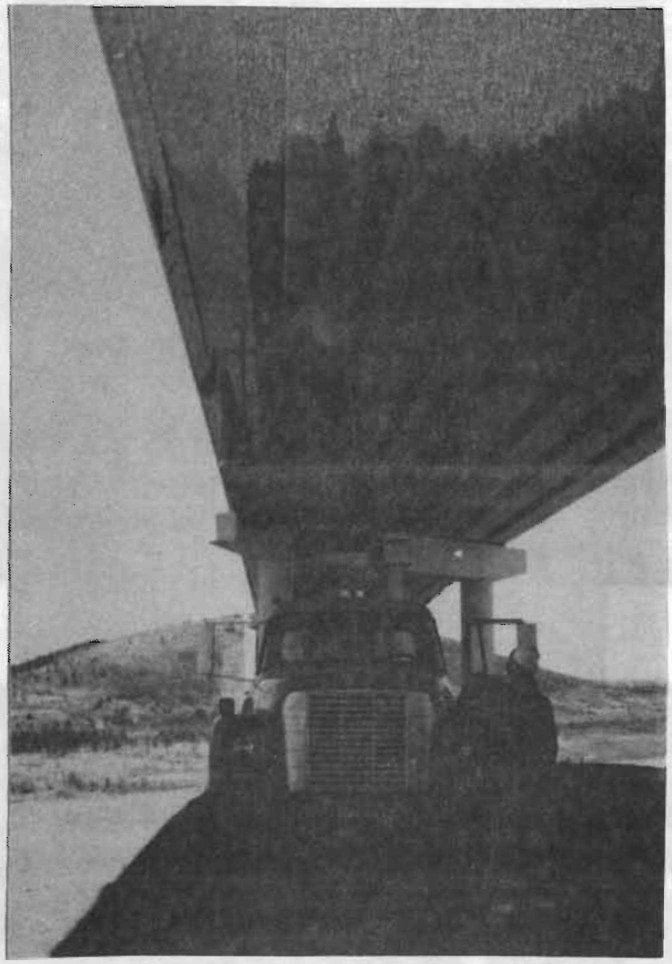


Fig. 22 Detailed inspection of girders for any indication of cracking (continued)

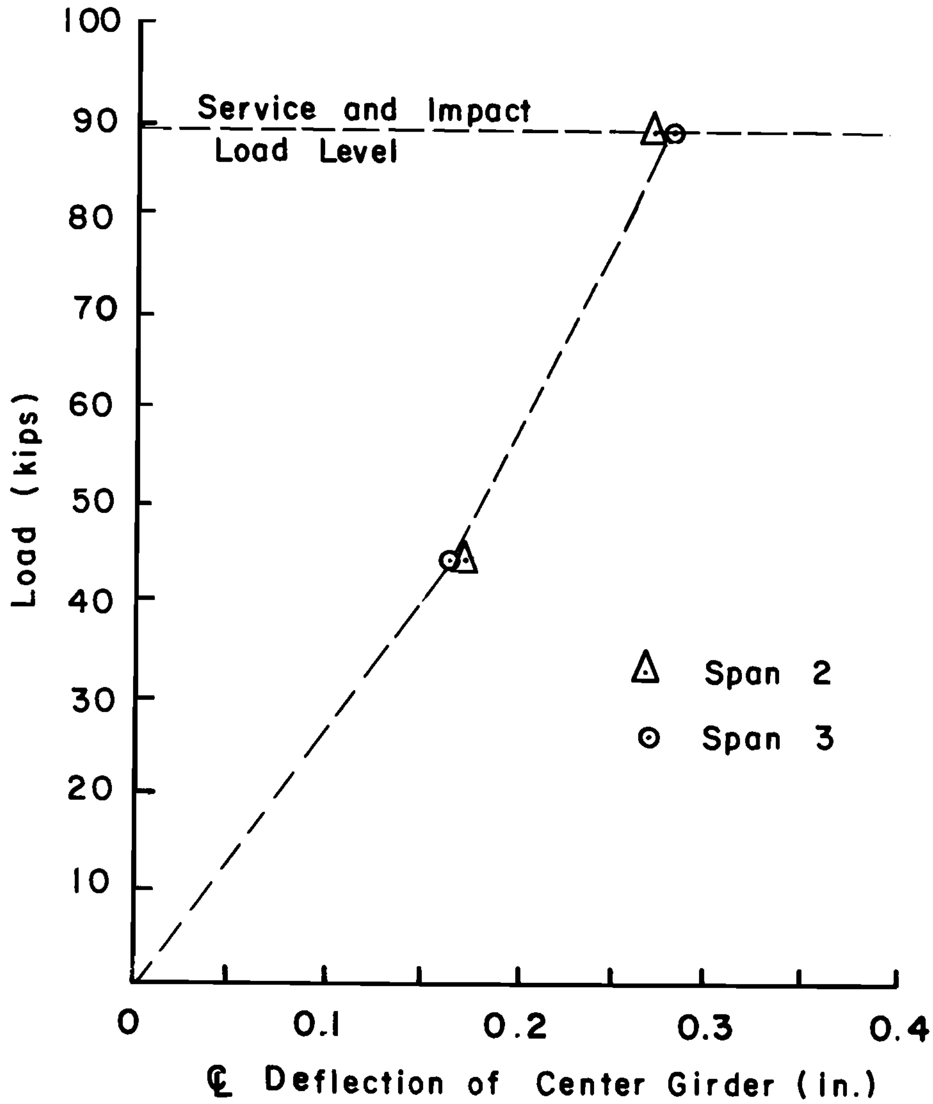


Fig. 23 Load-deflection behavior of center girders of each span

SUMMARY

The results from the single-lane load test indicate that the load-deflection behavior of the sagging bridge near Happy, Texas, is stiffer than that predicted by a simplified elastic analysis which was based on an assumed concrete modulus of elasticity and an equal distribution of load to each girder. The service live load plus impact factor deflections measured substantially less than those predicted when employing AASHTO distribution factors and are only 20% of the allowable maximum live load deflection of L/800. The overall load-deflection behavior of the girders is essentially linear up to service plus impact load levels. This supports visual evidence that no cracking of girders occurred. If the stiffening effects of the barrier rail and increases in concrete modulus due to aging are taken into consideration, the computed deflections would be reduced by more than 18%. This would result in most measured deflections being closer to the computed (P/8) theoretical values and some greater than the P/8 values. The dead load sag which exists in the long spans of the bridge apparently has no detrimental effect on the overall structural performance of the bridge at service load conditions for the contemplated single-lane use. In fact, since no cracking under full dead load plus impact load occurred in any of the girders, the bridge should have a substantial reserve in strength. Therefore, the only action which <u>may be</u> necessary to ensure the serviceability of the bridge is to regrade the underlying highway to meet any necessary clearance requirement. Should the bridge be used for two-lane traffic in the future, further study is recommended.

APPENDIX A

INDEPENDENT CHECK OF TEXAS STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION SAMPLE CALCULATIONS

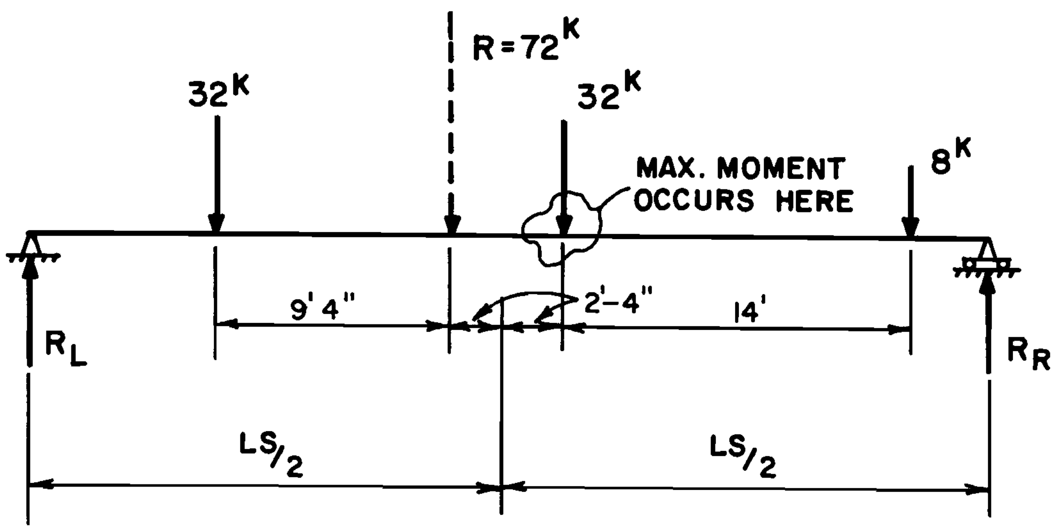
A. Calculated Live Plus Impact Load Moments

Impact factor, I, $= 50 / (L + 125)$

$= 50 / (133.58 + 125)$

$= 0.193$

HS20-44 Loading (for LS > 33 ft):



$M_{max} = R_R\ (LS - (LS/2 - 2.33) - 4.67) - 8(14)$

$= ((R(LS/2) - 2.33)/LS)\ (LS/2 - 2.33) - 112$

$= (72/2 - (72(2.33)/LS)\ (LS/2 - 2.33) - 112$

$= (36 - 168/LS)\ (LS/2 - 2.33) - 112$

$= 18LS - 84 - 84 + 392/LS - 112$

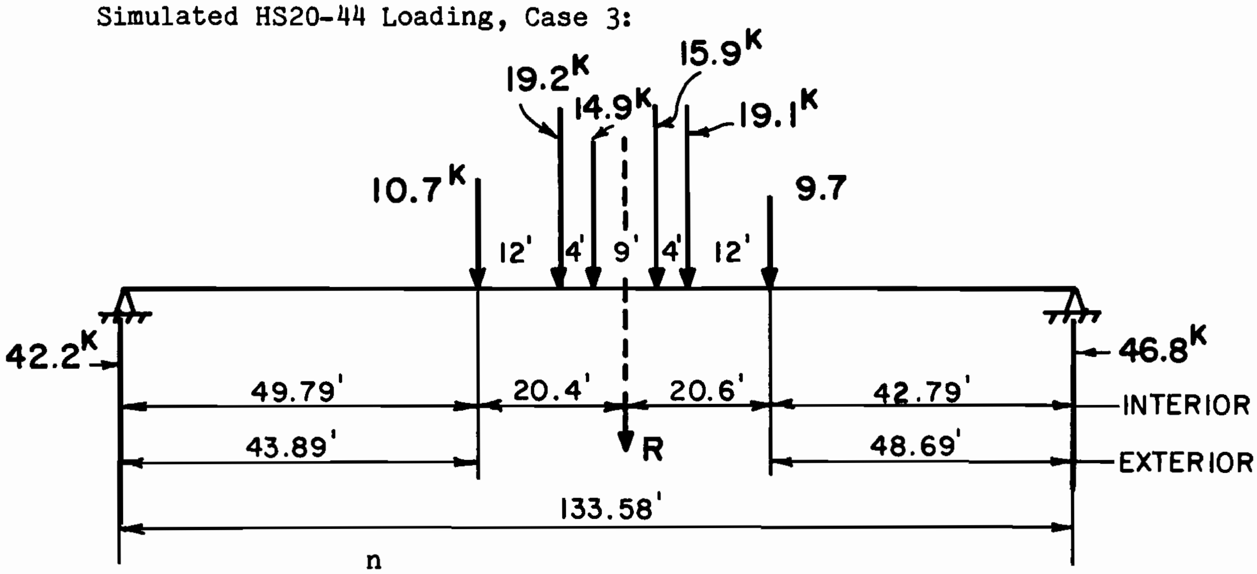
$= 18LS + 392/LS - 280$ per lane

where LS = length of span in ft

Therefore, $M_{max_{HS20-44_{LL}}} = 2127.4$ ft-kips

$M_{max_{HS20-44_{LL}}} = (1.193)\ (2127.4) = 2538$ ft-kips

Simulated HS20-44 Loading, Case 3:



$$M_{max_{applied}} \quad \sum_{i=1}^{n} = P_i b_i x/L$$

where x = the distance, which is less than a, between the support and the point of maximum applied moment for a particular axle load

a = the segment of span between the support and a particular axle load between which the point of maximum applied moment lies

$b = L - a$ (where $L = 133.58$)

n = number of applied axle loads

Therefore, for interior girder load distribution,

$$\begin{aligned} M_{max_{applied}} &= 1/133.58 \text{ ft } [(10.2k)(49.79 \text{ ft})(67.79 \text{ ft}) \\ &+ (19.2k)(61.79 \text{ ft})(67.69 \text{ ft}) \\ &+ (14.9k)(65.79 \text{ ft})(67.79 \text{ ft}) \\ &+ (15.9k)(58.79 \text{ ft})(65.79 \text{ ft}) \\ &+ (19.1k)(54.79 \text{ ft})(65.79 \text{ ft}) \\ &+ (9.7k)(42.79 \text{ ft})(65.79 \text{ ft})] \\ &= 2537.5 \text{ ft-kips} \approx 2538 \text{ ft-kips} \end{aligned}$$

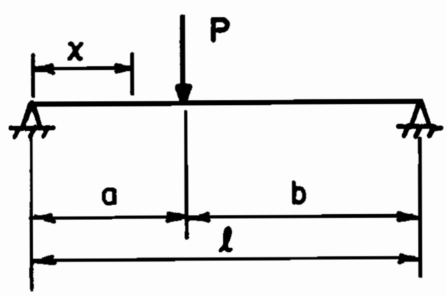
Therefore, for exterior girder load distribution,

$$
\begin{aligned}
M_{max_{applied}} &= 1/133.58 \text{ ft } [(10.2k)(43.89 \text{ ft})(64.69 \text{ ft}) \\
&+ (19.2k)(55.89 \text{ ft})(64.69 \text{ ft}) \\
&+ (14.9k)(59.89 \text{ ft})(64.69 \text{ ft}) \\
&+ (15.9k)(64.89 \text{ ft})(68.69 \text{ ft}) \\
&+ (19.1k)(60.89 \text{ ft})(68.69 \text{ ft}) \\
&+ (9.7k)(48.89 \text{ ft})(68.69 \text{ ft})] \\
&= 2541 \text{ ft-kips} > 2538 \text{ ft-kips}
\end{aligned}
$$

B. <u>Deflection Calculations</u>

$I_{comp.sec._{assumed}} = 497{,}340 \text{ in.}^4 = 23.9844 \text{ ft}^4$ say 24 ft^4

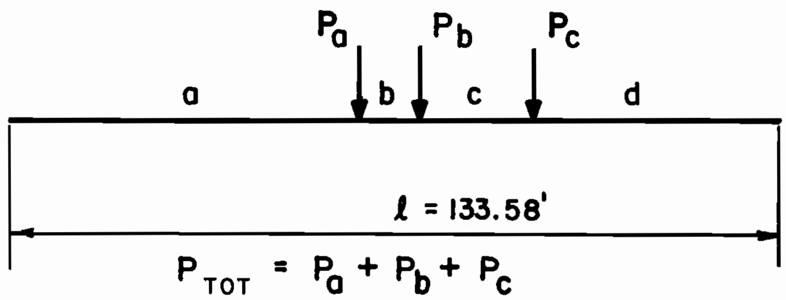
Deflection formula (simplified):



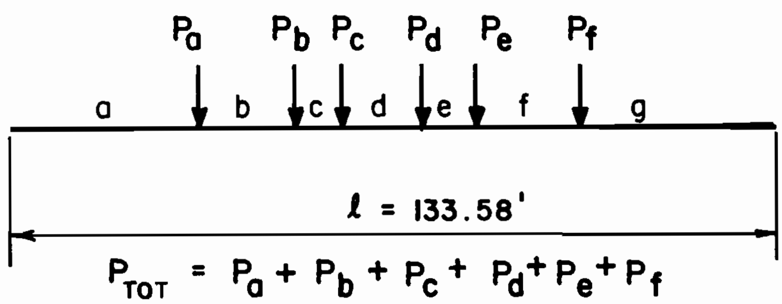
$$\Delta_X (\text{WHEN } X < a) = \frac{Pbx}{6EI} (l^2 - b^2 - X^2)$$

Δ_x (when $x < a$) = Pbx/6EI L $(L^2 - b^2 - x^2)$

Longitudinal load distribution:



CASES 1 and 2
(See Appendix C)



CASES 3 thru 6
(See Appendix C)

Deflections--Load Case No. 1 (Exterior Girder, Span 3):

$P_{TOT} = 15.9 + 19.1 + 9.7 = 44.7$

$P_a/P_{TOT} = 15.9/44.7 = 0.356$, $P_b/P_{TOT} = 19.1/44.7 = 0.427$,

$P_c/P_{TOT} = 9.7/44.7 = 0.217$

$$\Delta_{CL} = (P_{DIST}(12))/((6)(5000)(24)(133.58)(144))\ \{[(133.58)^2$$
$$- (64.29)^2 - (66.79)^2](64.29)(66.79)(0.356)$$
$$+ [(133.58)^2 - (60.29)^2 - (66.79)^2]\ (60.29)(66.79)(0.479)$$
$$+ [(133.58)^2 - (48.29)^2 - (66.79)^2]$$
$$(48.29)(66.79)(0.217)\} = (0.03347)P_{DIST}$$

Load Case No. 3 (Exterior Girder, Span 3):

$P_{TOT} = 10.2 + 19.2 + 14.9 + 15.9 + 19.1 + 9.7 = 84$

$P_a/P_{TOT} = 10.2/89 = 0.015$, $P_b/P_{TOT} = 19.2/89 = 0.216$,

$P_c/P_{TOT} = 14.9/89 = 0.167$, $P_d/P_{TOT} = 15.9/89 = 0.179$,

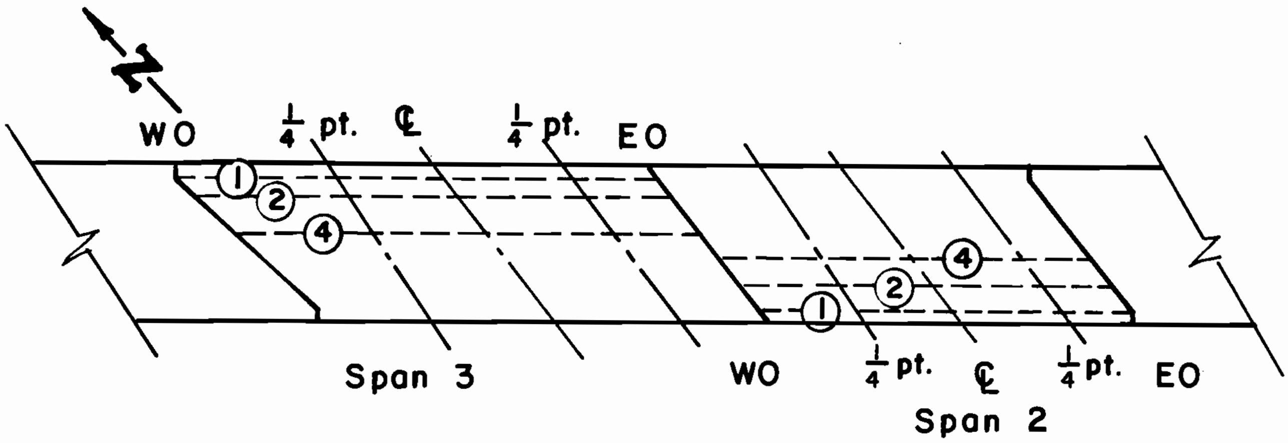
$P_e/P_{TOT} = 19.1/89 = 0.215$, $P_f/P_{TOT} = 9.7/89 = 0.108$

$$\begin{aligned}\Delta_{\mathrm{CL}} &= (P_{DIST})/((6)(5000)(24)(133.58)(144))\ \{[(133.58)^2 \\ &- (66.79)^2]\ (53.29)(66.79)(0.115) + [(113.58)^2 - (65.29)^2 \\ &- (66.79)^2]\ (65.29)(66.79)(0.216) + [(133.58)^2 - (64.29)^2 \\ &- [(133.58)^2 - (51.29)^2 - (66.79)^2]\ (51.29)(66.79)(0.215) \\ &+ [(133.58)^2 - (39.29)^2 - (66.79)^2]\ (39.29)(66.79)(0.108)\} \\ &= (0.03261)P_{DIST}\end{aligned}$$

These compare very favorably with the TSDHPT values of $(0.03372)P_{DIST}$ and $(0.03299)P_{DIST}$ respectively which were based on the output of the TSDHPT's computer program BMCOL51. That program's output was used in developing the deflection curves shown in Figs. 15 through 20 in the report text.

A P P E N D I X B

DEFLECTION DATA SUMMARY



INITIAL READINGS: 4/2/85, 9 a.m.

SPAN 3

		Girder 1	Girder 2	Girder 4
W	0	3-00/64	3-13/64	3-17/64
	1/4	2-49/64	3-03/64	3-08/64
	CL	2-56/64	3-62/64	2-37/64
	1/4	2-59/64	2-56/64	2-15/64
E	0	3-17/64	3-28/64	3-03/64

SPAN 2

		Girder 1	Girder 2	Girder 4
W	0	3-21/64	3-16/64	3-34/64
	1/4	2-34/64	2-39/64	3-11/64
	CL	2-07/64	2-29/64	3-03/64
	1/4	2-41/64	2-42/64	3-04/64
E	0	2-57/64	3-08/64	3-34/64

CASE 1: 4/2/85, 9:15 a.m.

SPAN 3

		Girder 1	Girder 2	Girder 4
W	0	2-63/64	3-12/64	3-16/64
	1/4	2-56/64	3-08/64	3-15/64
	CL	3-00/64	3-04/64	2-46/64
	1/4	3-00/64	2-59/64	2-18/64
E	0	3-17/64	3-27/64	3-02/64

CASE 2: 4/2/85, 9:30 a.m.

SPAN 2

		Girder 1	Girder 2	Girder 4
W	0	3-21/64	3-16/64	3-34/64
	1/4	2-40/64	2-45/64	3-16/64
	CL	2-14/64	2-36/64	3-12/64
	1/4	2-45/64	2-47/64	3-10/64
E	0	2-58/64	3-08/64	3-34/64

CASE 3: 4/2/85, 9:45 a.m.

SPAN 3

		Girder 1	Girder 2	Girder 4
W	0	2-63/64	3-13/64	3-16/64
	1/4	2-60/64	3-12/64	3-20/64
	CL	3-06/64	3-11/64	2-52/64
	1/4	3-05/64	3-01/64	2-19/64
E	0	3-17/64	3-27/64	3-02/64

CASE 4: 4/2/84, 10:15 a.m.

SPAN 2

		Girder 1	Girder 2	Girder 4
W	0	3-21/64	3-16/64	3-34/64
	1/4	2-43/64	2-47/64	3-19/64
	CL	2-18/64	2-40/64	3-16/64
	1/4	2-48/64	2-48/64	3-12/64
E	0	2-58/64	3-08/64	3-35/64

CASE 5: 4/2/85, 11:15 a.m.

SPAN 2

		Girder 1	Girder 2	Girder 4
W	0	3-21/64	3-16/64	3-34/64
	1/4	2-48/64	2-50/64	3-17/64
	CL	2-26/64	2-45/64	3-14/64
	1/4	2-53/64	2-52/64	3-11/64
E	0	2-57/64	3-08/64	3-34/64

CASE 6: 4/2/85, 12:15 p.m.

SPAN 3

		Girder 1	Girder 2	Girder 4*
W	0	2-63/64	3-13/64	3-25/64
	1/4	2-61/64	3-12/64	3-23/64
	CL	3-10/64	3-11/64	2-54/64
	1/4	3-07/64	3-00/64	2-24/64
E	0	3-17/64	3-26/64	3-18/64

* New reference wire

FINAL READINGS: No Load, 4/2/85, 12:45 p.m.

SPAN 3

		Girder 1	Girder 2	Girder 4
W	0	3-00/64	3-13/64	3-21/64+
	1/4	2-44/64	2-59/64	2-08/64
	CL	2-49/64	2-51/64	2-35/64
	1/4	2-52/64	2-46/64	2-12/64
E	0	3-17/64	3-28/64	3-18/64

+ Retensioned

SPAN 2

		Girder 1	Girder 2	Girder 4
W	0	3-21/64	3-16/64	3-37/64+
	1/4	3-00/64	2-31/64	2-28/64
	CL	1-62/64	2-18/64	2-54/64
	1/4	2-34/64	2-32/64	2-57/64
E	0	2-57/64	3-08/64	3-34/64

+ Retensioned

	A	B	C	D	E	F	G
1	Load Test Data from Happy, TX Bridge						
2	Span 3 -- Girder 1						
3			R	E A	D I	N G	S
4			Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
5	INITIAL		3	2.765625	2.875	2.921875	3.265625
6	CASE 1		2.984375	2.875	3	3	3.265625
7	CASE 3		2.984375	2.9375	3.09375	3.078125	3.265625
8	CASE 6		2.984375	2.953125	3.15625	3.109375	3.265625
9	FINAL		3	2.6875	2.765625	2.8125	3.265625
10							
11			0	.25	.5	.75	1
12			APPARENT	DEFLECTIONS		FROM	INITIAL
13			Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
14	INITIAL		0	0	0	0	0
15	CASE 1		-.015625	.109375	.125	.078125	0
16	CASE 3		-.015625	.171875	.21875	.15625	0
17	CASE 6		-.015625	.1875	.28125	.1875	0
18	FINAL		0	-.078125	-.109375	-.109375	0
19							
20			REAL	DEFLECTIONS		FROM	INITIAL
21			Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
22	INITIAL		0	0	0	0	0
23	CASE 1		0	.12109375	.1328125	.08203125	0
24	CASE 3		0	.18359375	.2265625	.16015625	0
25	CASE 6		0	.19921875	.2890625	.19140625	0
26	FINAL		0	-.078125	-.109375	-.109375	0
27							
28							
29				NET GIRDER	DEFLECTIONS		
30			Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
31	INITIAL	0	0	0	0	0	0
32	CASE 1	.0666667	0	.126302083333	.140104166667	.089322916667	0
33	CASE 3	.2	0	.19921875	.2484375	.18203125	0
34	CASE 6	.8666667	0	.266927083333	.383854166667	.286197916667	0
35	FINAL	1	0	0	0	0	0

	A	B	C	D	E	F	G
1	Load Test Data from Happy, TX Bridge						
2	Span 3 -- Girder 2						
3			R	E A	D I	N G	S
4			Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
5	INITIAL		3.203125	3.046875	2.96875	2.875	3.4375
6	CASE 1		3.1875	3.125	3.0625	2.921875	3.421875
7	CASE 3		3.203125	3.1875	3.171875	3.015625	3.421875
8	CASE 6		3.203125	3.1875	3.171875	3	3.40625
9	FINAL		3.203125	2.921875	2.796875	2.71875	3.4375
10							
11			0	.25	.5	.75	1
12			APPARENT	DEFLECTIONS		FROM	INITIAL
13			Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
14	INITIAL		0	0	0	0	0
15	CASE 1		-.015625	.078125	.09375	.046875	-.015625
16	CASE 3		0	.140625	.203125	.140625	-.015625
17	CASE 6		0	.140625	.203125	.125	-.03125
18	FINAL		0	-.125	-.171875	-.15625	0
19							
20			REAL	DEFLECTIONS		FROM	INITIAL
21			a. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
22	INITIAL		0	0	0	0	0
23	CASE 1		0	.09375	.109375	.0625	0
24	CASE 3		0	.14453125	.2109375	.15234375	0
25	CASE 6		0	.1484375	.21875	.1484375	0
26	FINAL		0	-.125	-.171875	-.15625	0
27							
28							
29				NET GIRDER	DEFLECTIONS		
30			Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
31	INITIAL	0	0	0	0	0	0
32	CASE 1	.0666667	0	.102083333333	.120833333333	.072916666667	0
33	CASE 3	.2	0	.16953125	.2453125	.18359375	0
34	CASE 6	.8666667	0	.256770833333	.367708333333	.283854166667	0
35	FINAL	1	0	0	0	0	0

A	B	C	D	E	F	G
Load Test Data from Happy, TX Bridge						
Span 3 -- Girder 4						
		R E	A D	I N	G S	
		Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL		3.265625	3.125	2.578125	2.234375	3.046875
CASE 1		3.25	3.234375	2.71875	2.28125	3.03125
CASE 3		3.25	3.3125	2.8125	2.296875	3.03125
CASE 6 ‡		3.390625	3.359375	2.84375	2.375	3.28125
FINAL +		3.328125	3.125	2.546875	2.1875	3.28125
		0	.25	.5	.75	1
		A P P A R E N T	D E F L E	C T I O N S	F R O M	I N I T I A L
		Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL		0	0	0	0	0
CASE 1		-.015625	.109375	.140625	.046875	-.015625
CASE 3		-.015625	.1875	.234375	.0625	-.015625
CASE 6 ‡		.125	.234375	.265625	.140625	.234375
FINAL +		.0625	0	-.03125	-.046875	.234375
		R E A L	D E F L E	C T I O N S	F R O M	I N I T I A L
		Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL		0	0	0	0	0
CASE 1		0	.125	.15625	.0625	0
CASE 3		0	.203125	.25	.078125	0
CASE 6 ‡		0	.08203125	.0859375	-.06640625	0
FINAL +		0	-.10546875	-.1796875	-.23828125	0
		N E T	G I R D E R	D E F L E	C T I O N S	
		Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL	0	0	0	0	0	0
CASE 1	.0666667	0	.13203125	.168229166667	.078385416667	0
CASE 3	.2	0	.22421875	.2859375	.12578125	0
CASE 6 ‡	.8666667	0	.1734375	.241666666667	.140104166667	0
FINAL +	1	0	0	0	0	0

‡ NEW REFERENCE WIRE
+ RETENSIONED WIRE

Load Test Data from Happy, TX Bridge
Span 2 -- Girder 1

R E A D I N G S

	Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL	3.328125	2.53125	2.109375	2.640625	2.890625
CASE 2	3.328125	2.625	2.21875	2.703125	2.90625
CASE 4	3.328125	2.671875	2.28125	2.75	2.90625
CASE 5	3.328125	2.75	2.40625	2.828125	2.890625
FINAL	3.328125	NA *	1.96875	2.53125	2.890625
FRACTION OF SPAN =	0	.25	.5	.75	1

A P P A R E N T   D E F L E C T I O N S   F R O M   I N I T I A L

	Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL	0	0	0	0	0
CASE 2	0	.09375	.109375	.0625	.015625
CASE 4	0	.140625	.171875	.109375	.015625
CASE 5	0	.21875	.296875	.1875	0
FINAL	0		-.140625	-.109375	0

R E A L   D E F L E C T I O N S   F R O M   I N I T I A L

	Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL	0	0	0	0	0
CASE 2	0	.08984375	.1015625	.05078125	0
CASE 4	0	.13671875	.1640625	.09765625	0
CASE 5	0	.21875	.296875	.1875	0
FINAL	0		-.140625	-.109375	0

N E T   G I R D E R   D E F L E C T I O N S

	TEMP. CORRECT FACTOR	Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL	0	0	0	0	0	0
CASE 2	.1333333	0	.104427083333	.1203125	.065364583333	0
CASE 4	.3333333	0	.173177083333	.2109375	.134114583333	0
CASE 5	.6	0	.284375	.38125	.253125	0
FINAL	1	0	0	0	0	0

* FINAL READING, 3+0/64, AT THIS LOCATION, IS APPARENTLY IN ERROR. THUS, ASSUMED SYMMETRIC TEMPERATURE EFFECTS TO GET NET DEFLECTIONS AT THIS QUARTER POINT.

Load Test Data from Happy, TX Bridge
Span 2 -- Girder 2

R E A D I N G S

	Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL	3.25	2.609375	2.453125	2.65625	3.125
CASE 2	3.25	2.703125	2.5625	2.734375	3.125
CASE 4	3.25	2.734375	2.625	2.75	3.125
CASE 5	3.25	2.78125	2.703125	2.8125	3.125
FINAL	3.25	2.484375	2.28125	2.5	3.125
FRACTION OF SPAN =	0	.25	.5	.75	1

A P P A R E N T   D E F L E C T I O N S   F R O M   I N I T I A L

	Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL	0	0	0	0	0
CASE 2	0	.09375	.109375	.078125	0
CASE 4	0	.125	.171875	.09375	0
CASE 5	0	.171875	.25	.15625	0
FINAL	0	-.125	-.171875	-.15625	0

R E A L   D E F L E C T I O N S   F R O M   I N I T I A L

	Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL	0	0	0	0	0
CASE 2	0	.09375	.109375	.078125	0
CASE 4	0	.125	.171875	.09375	0
CASE 5	0	.171875	.25	.15625	0
FINAL	0	-.125	-.171875	-.15625	0

N E T   G I R D E R   D E F L E C T I O N S

	TEMP. CORRECT FACTOR	Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL	0	0	0	0	0	0
CASE 2	.1333333	0	.110416666667	.132291666667	.098958333333	0
CASE 4	.3333333	0	.166666666667	.229166666667	.145833333333	0
CASE 5	.6	0	.246875	.353125	.25	0
FINAL	1	0	0	0	0	0

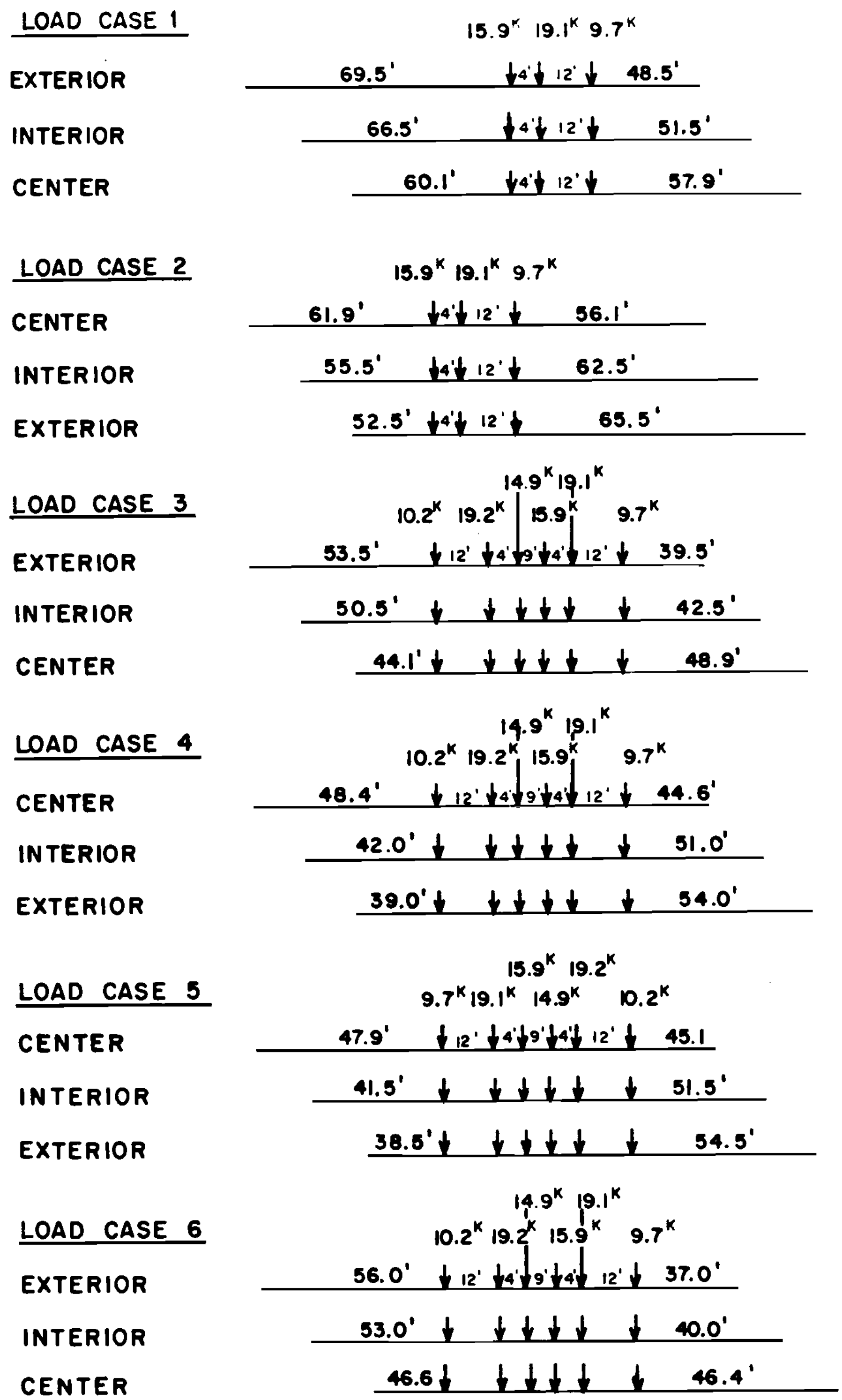
Load Test Data from Happy, TX Bridge
Span 2 -- Girder 4

A	B	C	D	E	F	G
		R	E A	D I	N G	S
		Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL		3.53125	3.171875	3.046875	3.0625	3.53125
CASE 2		3.53125	3.25	3.1875	3.15625	3.53125
CASE 4		3.53125	3.296875	3.25	3.1875	3.546875
CASE 5		3.53125	3.265625	3.21875	3.171875	3.53125
FINAL		3.578125	NA *	2.84375	2.890625	3.53125
FRACTION OF SPAN =		0	.25	.5	.75	1
		A P P A R E N T	D E F L	E C T I O N S	F R O M	I N I T I A L
		Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL		0	0	0	0	0
CASE 2		0	.078125	.140625	.09375	0
CASE 4		0	.125	.203125	.125	.015625
CASE 5		0	.09375	.171875	.109375	0
FINAL		.046875		-.203125	-.171875	0
		R E A L	D E F L E	C T I O N S	F R O M	I N I T I A L
		Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL		0	0	0	0	0
CASE 2		0	.078125	.140625	.09375	0
CASE 4		0	.12109375	.1953125	.11328125	0
CASE 5		0	.09375	.171875	.109375	0
FINAL		0		-.2265625	-.18359375	0
	TEMP.					
	CORRECT		N E T G	I R D E R D	E F L E C T I O N S	
	FACTOR	Sta. 0.00	Sta. 33.50	Sta. 67.00	Sta. 100.50	Sta. 134.0
INITIAL	0	0	0	0	0	0
CASE 2	.1333333	0	.102604166667	.170833333333	.118229166667	0
CASE 4	.3333333	0	.182291666667	.270833333333	.174479166667	0
CASE 5	.6	0	.20390625	.3078125	.21953125	0
FINAL	1	0		0	0	0

* FINAL READING, 2+28/64, AT THIS LOCATION IS APPARENTLY IN ERROR.
THUS, SYMMETRIC TEMPERATURE EFFECTS ASSUMED TO GET NET DEFLECTIONS AT THIS QUARTER POINT.

A P P E N D I X   C

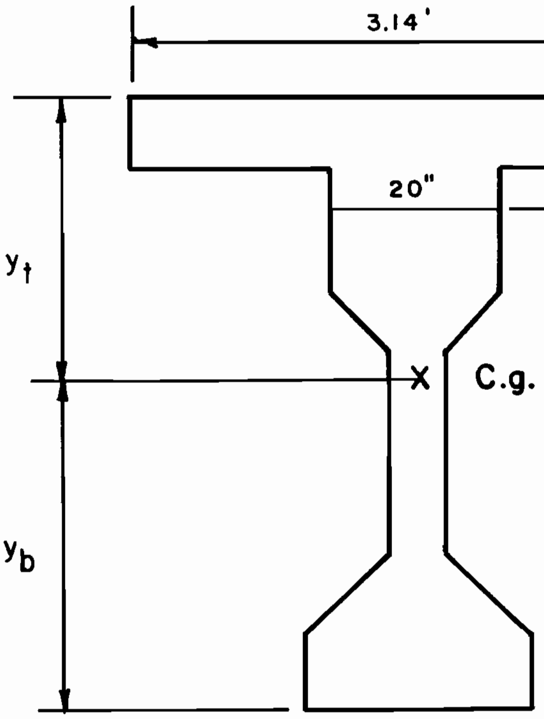
SUMMARY OF LOAD LOCATION FOR DETERMINING THEORETICAL GIRDER DEFLECTIONS


LOAD CASE 1
15.9K 19.1K 9.7K
EXTERIOR 69.5' 4' 12' 48.5'
INTERIOR 66.5' 4' 12' 51.5'
CENTER 60.1' 4' 12' 57.9'
LOAD CASE 2
15.9K 19.1K 9.7K
CENTER 61.9' 4' 12' 56.1'
INTERIOR 55.5' 4' 12' 62.5'
EXTERIOR 52.5' 4' 12' 65.5'
LOAD CASE 3
14.9K 19.1K
10.2K 19.2K 15.9K 9.7K
EXTERIOR 53.5' 12' 4' 9' 4' 12' 39.5'
INTERIOR 50.5' 42.5'
CENTER 44.1' 48.9'
LOAD CASE 4
14.9K 19.1K
10.2K 19.2K 15.9K 9.7K
CENTER 48.4' 12' 4' 9' 4' 12' 44.6'
INTERIOR 42.0' 51.0'
EXTERIOR 39.0' 54.0'
LOAD CASE 5
15.9K 19.2K
9.7K 19.1K 14.9K 10.2K
CENTER 47.9' 12' 4' 9' 4' 12' 45.1
INTERIOR 41.5' 51.5'
EXTERIOR 38.5' 54.5'
LOAD CASE 6
14.9K 19.1K
10.2K 19.2K 15.9K 9.7K
EXTERIOR 56.0' 12' 4' 9' 4' 12' 37.0'
INTERIOR 53.0' 40.0'
CENTER 46.6 46.4'

A P P E N D I X   D

GIRDER PROPERTIES ASSUMED FOR THEORETICAL DEFLECTION CALCULATIONS

a) Composite Section Properties



$I_{Girder} = 260{,}403 \text{ in.}^4$

$A_{Girder} = 788.4 \text{ in.}^2$

$$n = \frac{E_{slab}}{{}^*E_{beam}} = \frac{\sqrt{4000}}{\sqrt{8750}} = 0.68$$

$A_{slab} = 8(37.72)(0.68) = 205.2 \text{ in.}^2$

$A_{haunch} = 3(20)(0.68) = 40.8 \text{ in.}^2$

$I_{slab} = (0.68)\ (37.72(8))^3/12 = 1094 \text{ in.}^4$

$I_{haunch} = (0.68)\ (20(3))^3/12 = 30.6 \text{ in.}^4$

$Y_t = ((205.2)(4) + 40.8(9.5) + 788.4\ (40.25))/(205.2 + 40.8 + 788.4)) = 31.85 \text{ in.}$

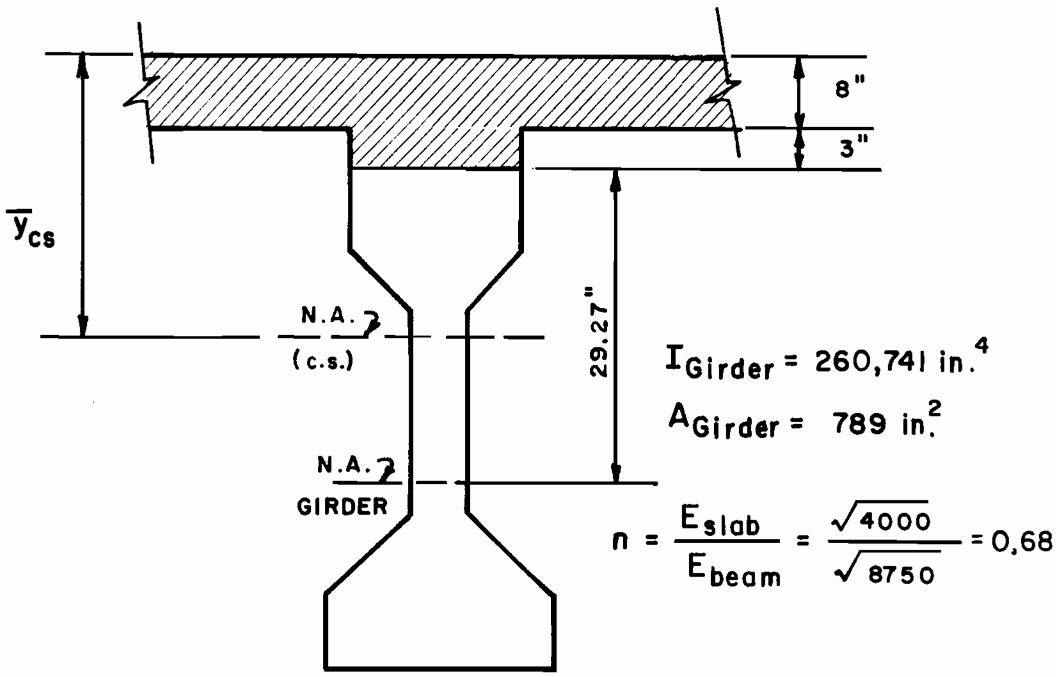
$\Rightarrow Y_b = 33.15 \text{ in.}$

$I_{comp.sec.} = 260403 + 788.4(8.4)^2 + 1094 + 205.2(27.85)^2 + 30.6 + 40.8(22.35)^2$

$I_{cs} = 496700 \text{ in.}^4$

* For deflection calculations, E_{BEAM} = 5,000,000 psi was used.

b) Independent Check of Composite Section Properties



$A_{slab} = (8)(3.143)(12)(0.68) = 205.2\text{ in.}^2$

$A_{haunch} = (3)(20)(0.68) = 40.8\text{ in.}^2$

$I_{slab} = ((0.68)(37.72)(8)^2)/12 = 1094\text{ in.}^4$

$I_{haunch} = ((0.68)(20)(3)^3)/12 = 30.6\text{ in.}^4$

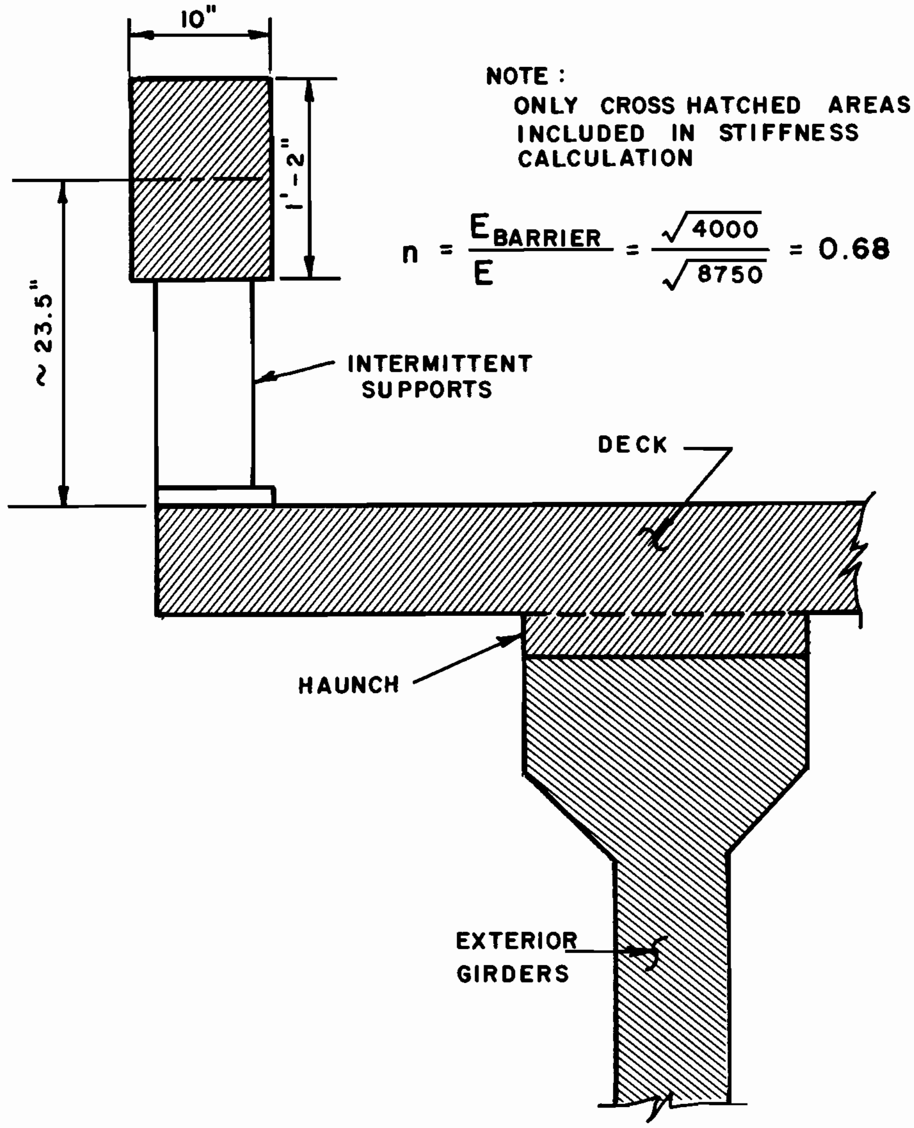
$A_{c.s.} = 789 + 205.2 + 40.8 = 1035\text{ in.}^2$

$Y_{c.s.} = [(789)(40.27) + (205.2)(4) + (40.8)(9.5)]/1035 = 31.87\text{ in.}$

$I_{c.s.} = 260{,}741 + 789(40.27 - 31.87)^2 + (205.2)(31.87 - 4)^2$

$\quad + 1094 + (40.8)(31.87 - 9.5)^2 + 30.6 = 497{,}340\text{ in.}^4$

c) Effect of Inclusion of Barrier Rails in Stiffness Calculations (Simplified)



$$n = \frac{E_{BARRIER}}{E} = \frac{\sqrt{4000}}{\sqrt{8750}} = 0.68$$

Without Rail:

$A_{c.s.}$ = 1034.4 in.^{2}/girder

Y_t = 31.87 in.

$I_{c.s.}$ = 496,700/girder

Inclusion of Rail:

$A_{rail} = (10)(14)(0.68)(2) = 190.4 \text{ in.}^2$

$\overline{Y} \approx [(31.87)(1034.4)(8) + (190.4)(-23.5)]/((8)(1034.4) + 190.4)$

$= 30.62 \text{ in.}$

$I_{bridge} \approx [(496{,}700)(8) + (1034.4)(8)(31.87 - 30.62)^2)$

$+ ((10)(14)^3)/12 + (30.62 + 23.5)^2 (190.4)]$

$\therefore I_{bridge} \approx 4{,}546{,}493 \text{ in.}^4$

Percent Increase in Stiffness
(for Inclusion of Barrier Rail Only)

$4{,}546{,}493 - (8)(496{,}700) = 572{,}893$

$572{,}893/((8)(496{,}700) = 0.1442$

$(1 - 1/1.1442) \times 100\% = \underline{\underline{12.6\%}}$

d) Effect of Increased Modulus of Elasticity (Simplified)

If $f_c = 8750$, then $E_c = 57\sqrt{8750} = 5332$ psi

$332/5000 = 0.0664$

(assumed in Δ calculations)

$(1 - 1/1.0664) \times 100\% = \underline{\underline{6.23\%}}$ increase in stiffness due to modulus only

e) Effect of c) and d) Combined

$(1 - (1/(1.0664)(1.1442)) \times 100\% = \underline{\underline{18\%}}$ total increase in stiffness