A LOADING STUDY OF OLDER HIGHWAY BRIDGES IN VIRGINIA

Part 2

A Concrete Slab and Steel Beam Bridge in Clarke County

by

H. L. Kinnier
Faculty Research Engineer

and

L. L. Ichter
Graduate Assistant

(The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the sponsoring agencies.)

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SUMMARY

A 60-foot non-composite steel beam and concrete deck highway bridge span over the Shenandoah River on Route 7 in Clarke County was tested with a 23-ton, tandem axle test vehicle in July 1975. Strain gages were placed near midspan on the lower flanges, the webs and the upper flanges of the steel stringers (W36 x 150) and in one position on the underside of the concrete deck. Midspan deflections were measured for each of the five stringers. The purpose of the study was to determine the present capacity of this typical design that was used extensively in the 1920s and 1930s when no shear connectors were employed to provide composite action.

The results indicated that the experimental stresses from static live loading were well below the calculated stresses based on conventional design theory and distribution factors. The distribution of the test load to each of the five stringers for five lateral positions of the test vehicle was similar to the distributions found in earlier studies, (see references 3 and 4), for composite spans.

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BACKGROUND

A large number of highway bridges designed and constructed in the 1920-45 period for the AASHTO H-15 standard loading remain in use on both the Virginia primary and secondary systems. Although truck loadings have generally increased since that period, there is some reluctance to post these bridges for limited live loading unless deterioration or deadweight overloads from excess asphalt wearing surfaces have developed. Periodically, proposals are made in the state legislature to increase the legal loads allowed on one or more of the categories of Virginia highways, generally the interstate system or the primary system. It is recognized, however, that an increase in the allowed loading on one highway system has a spillover effect on the other, lower rated systems. Consequently, a proven method of accurately appraising the live load capacity of some of the older bridge types remaining in use would be of value to those technical personnel responsible for recommendations when legal highway live load increases are being considered or when decisions are made on the granting of overload permits.

OBJECTIVE

The purpose of this study was to determine experimentally the live load stresses that are developed from standard design loadings in key members or critical locations of the three older bridge categories that exist in large numbers throughout the primary and secondary systems of Virginia; namely, (1) steel truss spans,

(2) steel beam spans, and (3) concrete beam spans. The Part 1 Report, issued in February 1976, presented test results from a steel truss bridge tested in July 1974. Part 2 reports on a steel beam span tested in July 1975.

DESCRIPTION OF THE TEST STRUCTURE

The structure selected for testing (a 60'-0" steel beam span) is one of 25 spans of the bridge on the eastbound roadway of Route 7 over the Shenandoah River in Clarke County, 5 miles east of Berryville. The bridge is made up of fifteen 40'-0" reinforced concrete beam spans, four 60'-0" and four 90'-0" steel beam spans, and two 200'-0" steel truss spans. See Figures 1 and 2*. The structure was constructed in 1939 and plans are available from the Bridge Office of the Virginia Department of Highways and Transportation under the designation of LXXIV-25 dated October 20, 1938. The test span was constructed from the standard plan designated SM-24-60 dated July 1932. Figures 3 and 4 show a half side elevation, a quarter plan and a half transverse section of the superstructure.

The bridge was designed and constructed in accordance with the Virginia Department of Highways Bridge Specifications, 1932 for an H-15 Standard AASHTO loading.

INSTRUMENTATION

Eighteen SR-4 type A3-56 strain gages were placed on the five W36 x 150 steel girders (see Figure 5) and two SR-4 type A-9-3 strain gages were placed on the underside of the concrete deck slab (see Figure 6). Engineer's scales with 20 divisions to an inch were attached to the lower flanges of the steel girders for measuring the deflections (Figure 7). The scales were read with a precise N-3 Wild Level (Figure 8) with a least reading of 0.001". Figure 9 shows the location of the strain and deflection gages. Figure 10 shows the scaffolding erected under the test span for installation of the gages, and Figure 11 shows technicians grinding the steel surfaces for placement of the strain gages.

The 20 strain gages were wired into two 10-channel Model SB-1 Switch and Balance Units manufactured by Vishay Instruments, Inc.

*All figures and tables are attached.

A battery powered Model P-350 portable digital strain indicator was used to read the strains. See Figure 12.

TEST LOADING

A Mack MB-400 gasoline truck with tandem rear axles was borrowed from the Staunton District Equipment Depot of the Virginia Department of Highways and Transportation for use as the test vehicle. The truck axle dimensions and loads are detailed in Figure 13 and a photograph of the truck is shown in Figure 14. The truck weights and dimensions closely approximated the Type 3 unit loading designated in the Manual for Maintenance Inspection of Bridges, 1974.(1) The bending moment from the test vehicle at the gage positions was 311.7 ft. kips for a line of wheels, which is about 10% in excess of the maximum bending moment of 284.5 ft. kips for a Type 3 loading on the same effective span length of 61.25'. The absolute maximum bending moment calculated for this loading on this span is 312.3 ft. kips at a position 0.22' from midspan. The strain gages on the steel girders were placed 1' east of midspan to avoid interference with the diaphragms located at midspan. It may be noted that the maximum legal load limit of 36 tons for a Type 3S2 loading on this span length develops a bending moment of 323.1 ft. kips. This bending moment is only 3.5% in excess of the bending moment from the test loading.

TEST PROCEDURE

The test vehicle was placed at five lateral midspan positions (see Figure 15) to determine the distribution of strains and deflections to each of the five girders for each of the load positions. The first rear axle was placed at the strain gage position to provide maximum bending moment there. Two type A-9-3 strain gages were placed 7'-7" east of midspan on the underside of the 8" concrete deck (Figures 6 and 9) in a position to be directly under the dual wheels when the test vehicle was located in position 6.

With the test vehicle placed in each of the five midspan positions, the 20 strain gages and 5 deflection scales were read and recorded. The procedure was repeated in its entirety for a second set of readings, and repeated for positions 2 and 4 for a third set. The test vehicle was placed in position 6 for two sets of strain readings on the underside of the concrete deck slab.

TEST RESULTS

The test results were all in the form of strain readings from the 20 SR-4 gages and vertical deflections from the scales placed on each of the five steel beams. The average unit stresses and deflections for positions of the test vehicle are presented in Tables 1 and 2.

Below, based on the strains and/or deflections, calculations are made and comments presented on the –

1. distribution of the static live load to the five steel beams;

2. effective moments of inertia of the exterior and interior beams;

3. location of the neutral axis of the beams;

4. experimental midspan lower flange live load stresses in the steel beams;

5. experimental static live load stresses and deflections from simulation of the live load in the two passing lanes of the bridge deck; and

6. flexure stresses in the concrete deck slab.

1. Distribution of the Static Live Load to the Five Steel Beams

The distribution of the truck load to the five beams for each of the five lateral positions is shown in Table 3 based on midspan lower flange strains and Table 2 based on midspan deflections. The results in the two sets of data agreed closely.

The greatest percentage of the truck load distributed to an interior beam (beams 2, 3, and 4) was computed to be 31.5% for beam 2 with the test vehicle in position 2 and 31.3% in beams 2 and 4 with the test vehicle in positions 1 and 5, respectively. These values compare with

$$\frac{S}{5.5} = \frac{5.583}{5.5} = 101.5\%$$

of a line of wheels, or 50.8% for the total truck from the AASHTO Standard Design Specifications for an interior girder for this type structure. The disparity between experimental data and design specifications is consistent with findings from previous experimental studies conducted by the Research Council.[3,4] For example, a study of the Hazel River Bridge[3] in 1962 showed the maximum experimental distribution to an interior girder to be 35.7% while the AASHTO Standard Design Specifications required 69.8%. The Hazel River Bridge consisted of 66'-5" composite spans and 7'-8" beam spacing.

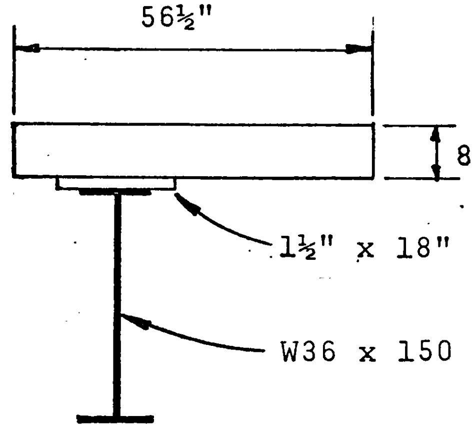
The greatest percentage of the truck load carried by an external beam (beams 1 and 5) was determined to be 43.5% (Table 3) for beam 5 with the test vehicle in position 5. From Section 5.2.2, Manual for Maintenance Inspection of Bridges[1] 1974, 85.1% of a line of wheels (42.6% of a truck) is specified in rating a bridge structure on the basis of the exterior beam. See Figure 16. Unlike the interior girder comparison, there is a very close correlation between the experimental and theoretical distributions of live load to the exterior beam. Similar results were obtained in the Hazel River Bridge test; 44.0% of a truck load to the exterior beam experimentally and 47.8% of the load from the Inspection Manual Requirements.

2. Effective Moments of Inertia of the Exterior and Interior Beams

Based on conventional elastic beam theory, the moments of inertia of the exterior and interior beams were calculated from the measured strains and also from the measured deflections. These values of I are tabulated and averaged in Table 4 (from strains) and Table 5 (from deflections). The experimental values exceeded the theoretically calculated values somewhat, but they were remarkably close — particularly for the experimental results for interior beams based on deflections, which were about 6% above the corresponding theoretical values.

Calculations for the moments of inertia of the exterior and interior beams follow. Full composite action and a value of "n" (ratio of modulus of elasticity of steel to that of concrete) of 8 are assumed. Table 6 summarizes the corresponding data.

Exterior Beam



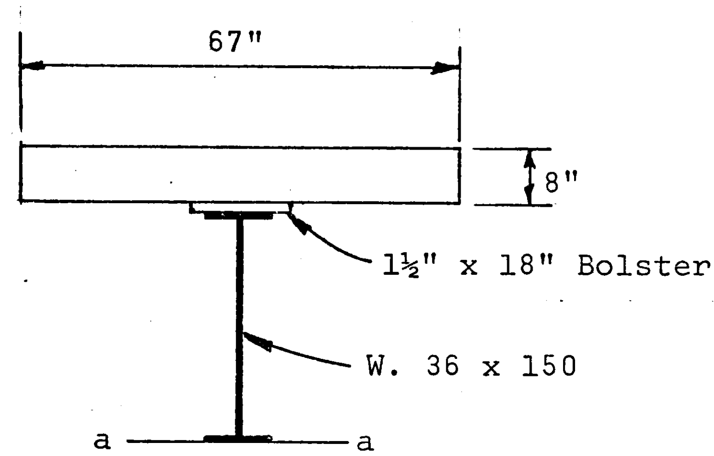
Part	Area in.^{2}		Lever Arm (a-a) in.		Moment in.^{3}
W36 x 150	44.2	x	17.92	=	792.1
Bolster	$\frac{1.5 \times 18}{8} = 3.36$	x	36.59	=	123.7
Slab	$\frac{56.5 \times 8}{8} = 56.5$	x	41.34	=	2335.7
	104.1				Σ 3251.5

$$\bar{y} = \frac{3251.5}{104.1} = 31.23''$$

Moment of Inertia

W36 x 150	$9030 + 44.2(31.23 - 17.92)^2$	$= 16{,}860 \text{ in.}^4$
Bolster	$3.38(36.59 - 31.23)^2$	$= 100$
Slab	$\frac{56.5}{8} \times \frac{8^3}{12} + 56.5(41.34 - 31.23)^2$	$= 6{,}080$
		$I = 23{,}040 \text{ in.}^4$

Interior Beam



Part	Area in.^{2}		Lever Arm (a-a) in.		Moment in.^{3}
W36 x 150	44.2	x	17.92	=	792.1
Bolster	$\frac{1.5 \times 18}{8} = 3.38$	x	36.59	=	123.7
Slab	$\frac{67 \times 8}{8} = 67$	x	41.34	=	2769.8
	114.6				Σ 3685.6

$$\bar{y} = \frac{3685.6}{114.6} = 32.16 \text{ in.}$$

Moment of Inertia

W36 x 150	$9030 + 44.2(32.16 - 17.92)^2$	$= 18{,}000 \text{ in.}^4$
Bolster	$3.38(36.59 - 32.16)^2$	$= 70$
Slab	$\frac{67}{8} \times \frac{8^3}{12} + 67(41.34 - 32.16)^2$	$= 6{,}000$
		$I = 24{,}070 \text{ in.}^4$

3. Location of the Neutral Axis

The neutral axis positions above the lower flange for an exterior and an interior beam are calculated in the previous section of the test results based on conventional elastic theory and assuming full composite action between the concrete deck and the wide flange steel beams. The theoretical values of 31.23" and 32.16" for an exterior and interior beam, respectively, agree very closely with the averages of the experimental values shown in Table 7.

The experimental values were calculated by assuming a linear variation in strain using three sets of strain readings, namely: Method 1 — Using lower flange gages and web gages; Method 2 — Using lower flange gages and upper flange gages; and Method 3 — Using web gages and upper flange gages. There was only small disparity between the results of the three methods, and their closeness to theoretical values indicates the existence of complete composite action as assumed and the correctness of elastic theory for this type of bridge structure.

4. Experimental Midspan Lower Flange Live Load Stresses in the Steel Beams

The experimental lower flange stresses from the placement of the static live load at the five midspan positions were low compared to the corresponding design stresses. The maximum average experimental values were 3.83 ksi in the exterior beam 1 for the test vehicle in position 1, and 2.76 ksi in the interior beam 2 for the test vehicle in positions 1 and 2. The same average stress developed in beam 4 for the test vehicle in Position 5. Table 8 lists these experimental stresses with dead load and live load stresses calculated from conventional design theory and using the AASHTO[2] recommendations for distributing dead and live loads to the exterior and interior beams. Supporting calculations for values listed in Table 8 are shown in the Appendix.

5. Experimental Static Live Load Stresses and Deflections from Simulation of the Live Load in the Two Passing Lanes of the Bridge Deck

Table 9 lists the summation of the midspan live load stresses from simulating the test vehicle in the two passing lanes of the bridge deck, namely, positions 2 and 4. The maximum simulated stress in an exterior beam was 3,230 psi compared to 3,830 psi shown in Table 8 for the test vehicle in position 1. For an interior beam, the maximum simulated stress from two positions of the test vehicle was 3,690 psi, compared to 2,760 psi for a measured stress from a single position of the test vehicle.

The stresses developed in the beams from the placement of known live loads on the bridge deck can be reasonably and conservatively predicted by conventional elastic beam theory and the AASHTO Standard Specifications for live load distribution factors. The experimental live load stresses listed in both Tables 8 and 9 are less than the theoretically calculated live load stresses.

The response of the span to dynamic loading was not evaluated in this study. However, a number of previous bridge loading field studies have shown the AASHTO Standard Specifications(2) for impact factors to liberally predict the increase in live load stresses from moving vehicles. The only exceptions where the impact factors exceeded the specification values were in the inconsequential cases of very low live load stresses, and in the cases where the experiment was designed to develop large impact factors by running the test vehicle over small ramps. Even in these cases the specification impact factor was not greatly exceeded.

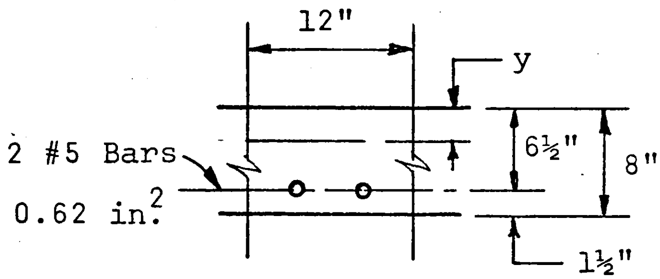
6. Flexure Stresses in the Concrete Deck Slab

Two SR-4 type A-9-3 strain gages (S1 and S2) were placed on the underside of the deck midway between beams 4 and 5 as shown in Figure 9. Position 6 of the test vehicle placed a wheel of the first of the tandem axles directly over gages S1 and S2. The center to center wheel spacing (6'-0") is essentially the same as the spacing of the gages (5'-7"). Figure 16 shows the location of position 6 as well as that of the two gages.

The average measured strains are listed in Table 1 as 65 and 44 inches/inch for gages S1 and S2, respectively, which values correspond to tensile stresses of 200 and 130 psi for a modulus of elasticity for concrete of 3×10^6 psi. No meaningful theoretical stress calculations can be made for this loading condition for a number of reasons including, (1) the four rather wide rubber tire wheels applying varying pressure to the slab at the gage position, (2) the additional four rubber tire wheels on the other tandem axle 4.2' away, (3) the questionable fixity condition of the concrete slab over the exterior beam, (4) the distribution of the loading over the length of the deck slab, and (5) the extent of cracking and nonhomogeneity of the concrete in flexure.

It is interesting to note by comparison that the AASHTO Standard Specification live load design calculations which follow show the steel stress for a Type 3 loading (8,000 lb. wheel load) to be 5,550 psi. The corresponding concrete stress at a distance 1½" farther from the neutral axis would be 750 psi.

$$M = \frac{0.8(S+2)P}{32} = \frac{0.8(5.25+2)}{32} \times 8 = 1.45\ f_k$$



$$n = \frac{29}{3} = 9.7$$

$$\frac{12y^2}{2} = 9.7 \times 0.62\ (6.5-y)$$

$$y = 1.74"$$

Calculation of I_t

$$\frac{12 \text{x} 1.74^3}{3} = 21.1$$

$$9.7 \times 0.62(6.50-1.74)^2 = \underline{123.7}$$

$$I_t = 144.8 \text{ in.}^4$$

$$f_s = 9.7\ \frac{1.45 \text{x} 12 \text{x} 4.76}{144.8} = 5{,}550 \text{ psi}$$

$$f_c = \frac{1.45 \text{x} 12 \text{x} 6.26}{144.8} = 750 \text{ psi}$$

CONCLUSIONS

1. The percentage distributions of the static load to the five steel beams for the several lateral positions of the test vehicle were somewhat less than that prescribed by the design specifications. The percentage distributions based on strain and deflection measurements agreed closely.

2. The effective moments of inertia calculated from the measured strains and deflections agreed closely with the theoretical values based on composite action between the beams and the concrete deck, although composite action was not provided in the design.

3. The locations of the neutral axis of the "non-composite" beams as determined by the measured strains agreed closely with the theoretically calculated locations based on elastic theory for composite beams.

4. The experimental midspan lower flange live load stresses in the steel beams were less than, but not exceedingly so, the corresponding design stresses.

5. The stresses developed in the beams of concrete deck and steel beam bridges of short to moderate length can be reasonably and conservatively predicted, whether they are constructed with shear connectors or not, by conventional elastic theory and the AASHTO Design Specifications.

6. The tensile flexure stresses on the bottom surface of the concrete deck slab were consistent with the tensile reinforcement steel stresses as calculated by conventional cracked concrete theory and the semiempirical rules for the distribution of the loading as specified by the AASHTO Specifications.

ACKNOWLEDGMENTS

The experimental phase of this study required the assistance and cooperation of many persons in the Virginia Department of Highways and Transportation. The authors particularly thank District Bridge Engineer L. L. Misenheimer for his consultation and suggestions as well as for locating an appropriate test structure; Resident Engineer C. L. Ellington for his general assistance and suggestions; and Bernard Lohr, Gary Cooper, Eddie Sours, Larry Kenney, and Donald Kipps for their fine assistance in constructing the necessary scaffolding, instrumenting the structure with strain gages, reading the strains and deflections, and managing the traffic while loading the structure. Further, the authors thank John Jacob Rohrer of the Staunton District who skillfully operated the test vehicle during the test.

The research project was conducted under the general supervision of Jack H. Dillard, head, Virginia Highway and Transportation Research Council, and Harry E. Brown, research engineer. W. T. McKeel, Jr., research engineer, offered valuable suggestions and criticisms during both the field phase of the study and the preparation of the report.

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1. AASHTO Operating Subcommittee on Bridge and Structures, Manual for Maintenance Inspection of Bridges, 1974.

2. AASHTO, Standard Specifications for Highway Bridges, 11th ed., 1973.

3. Kinnier, H. L., and W. T. McKeel, Jr., "A Dynamic Stress Study of the Hazel River Bridge," Virginia Council of Highway Investigation and Research, 1964.

4. Kinnier, H. L., "A Dynamic Stress Study of the Weyer's Cave Bridge", Virginia Council of Highway Investigation and Research, Charlottesville, Virginia, 1963.

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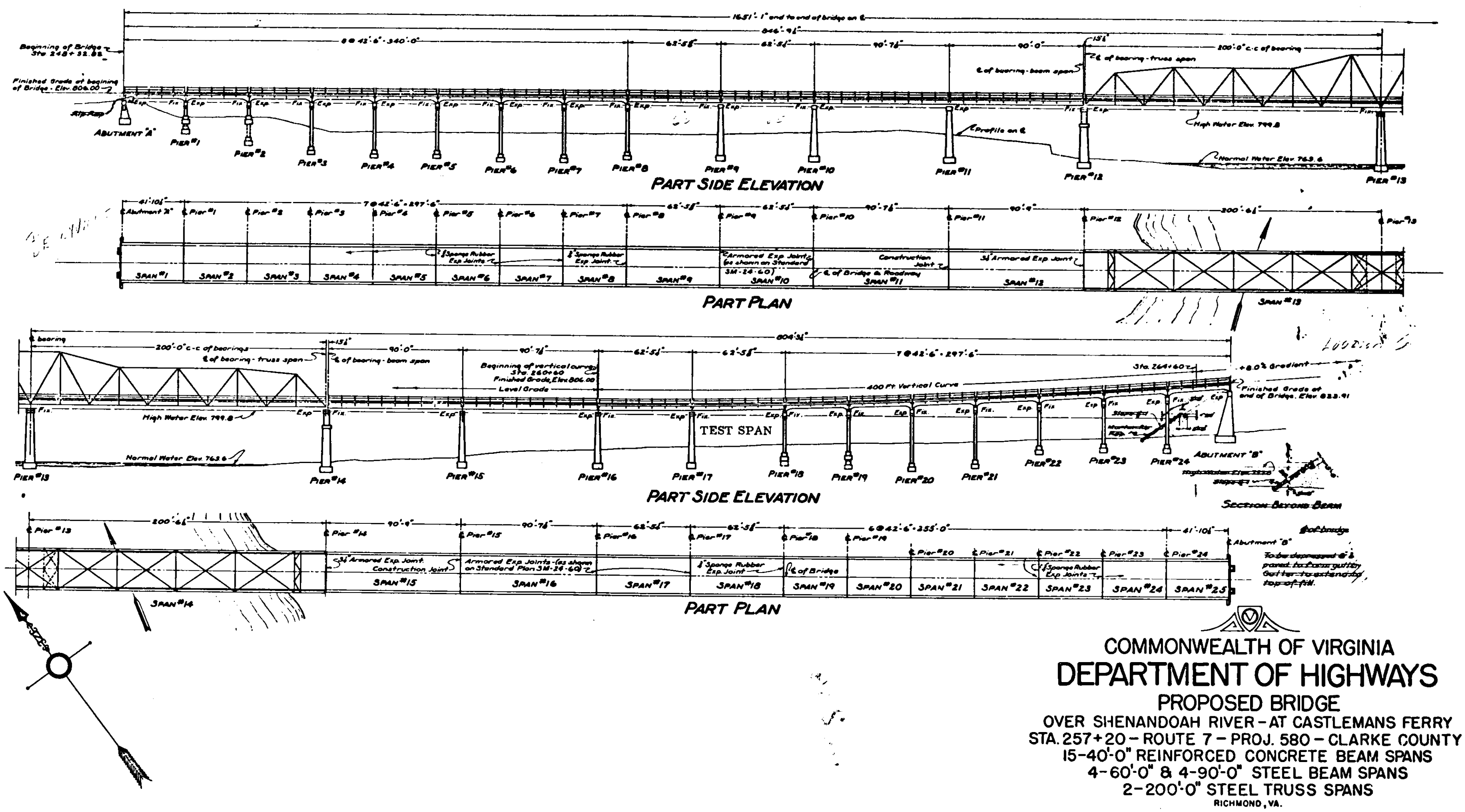


Figure 1. Side elevation of bridge showing test span.

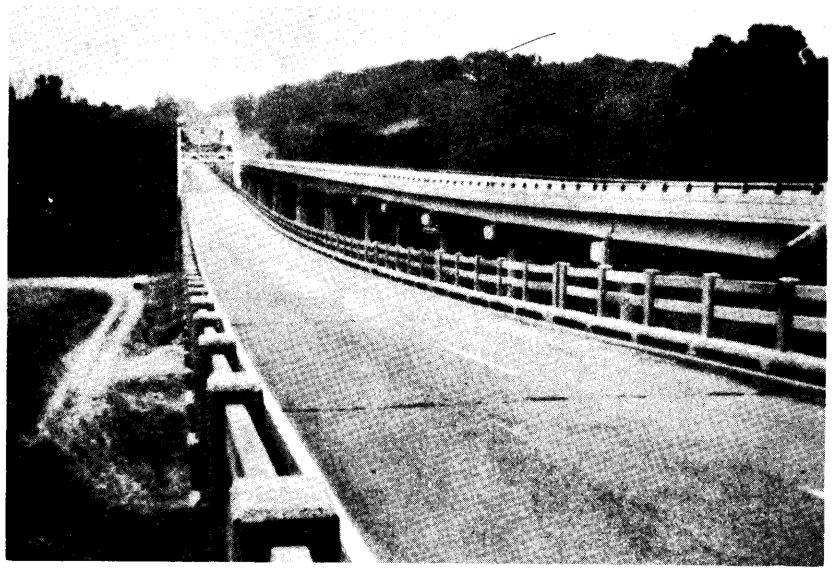


Figure 2. Eastbound roadway of bridge looking west.

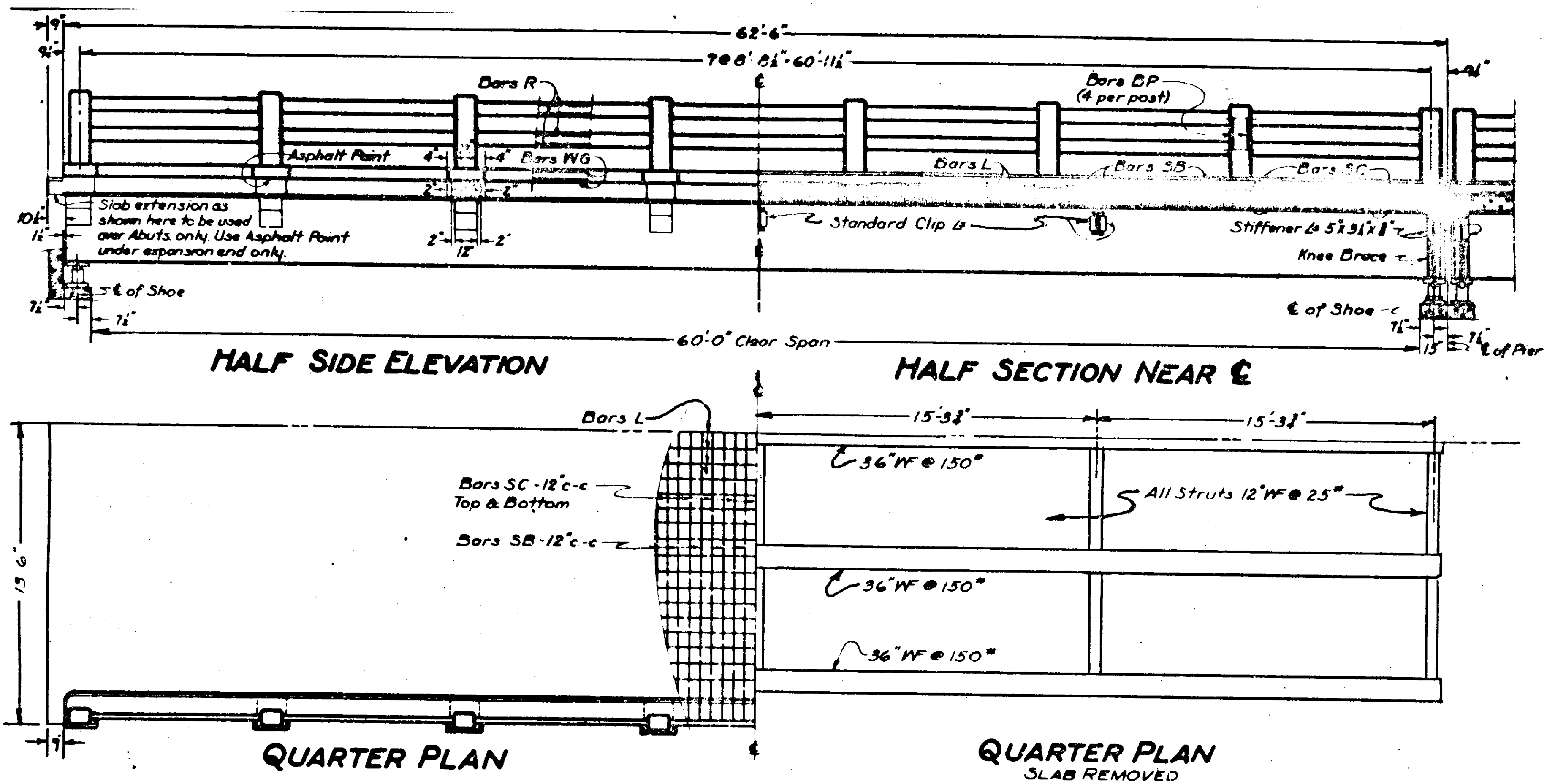


Figure 3. Half side elevation and quarter plan.

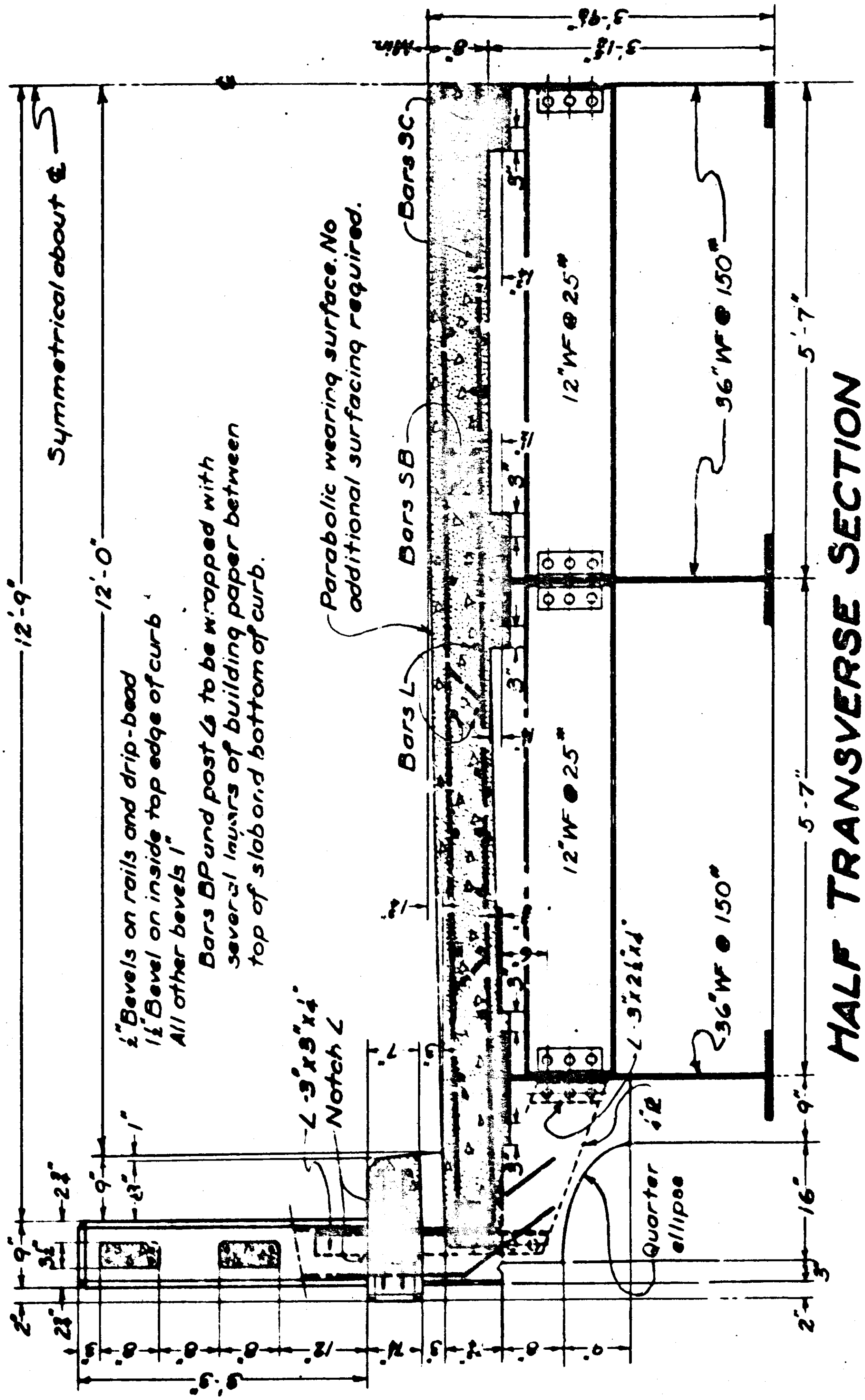


Figure 4. Half transverse section.

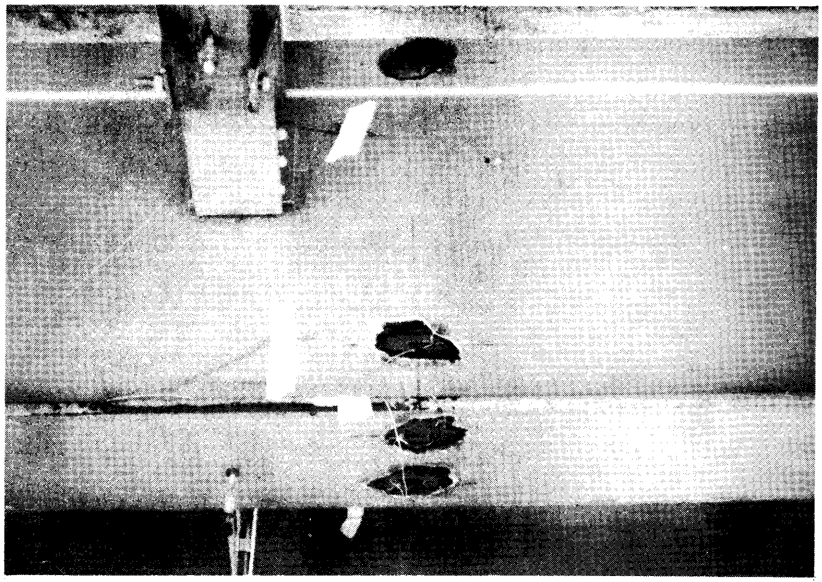


Figure 5. Strain gages on lower flange, web, and upper flange of a typical girder.

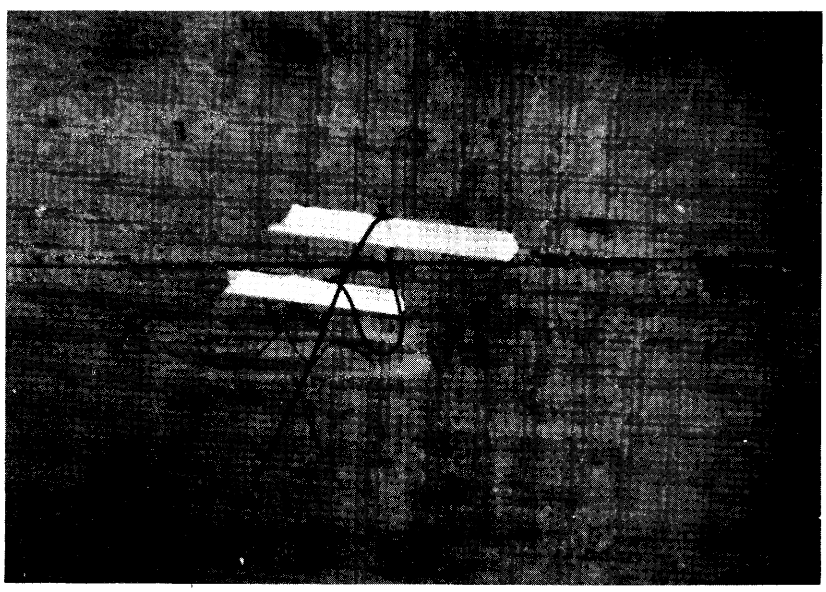


Figure 6. Strain gages on underside of concrete slab.

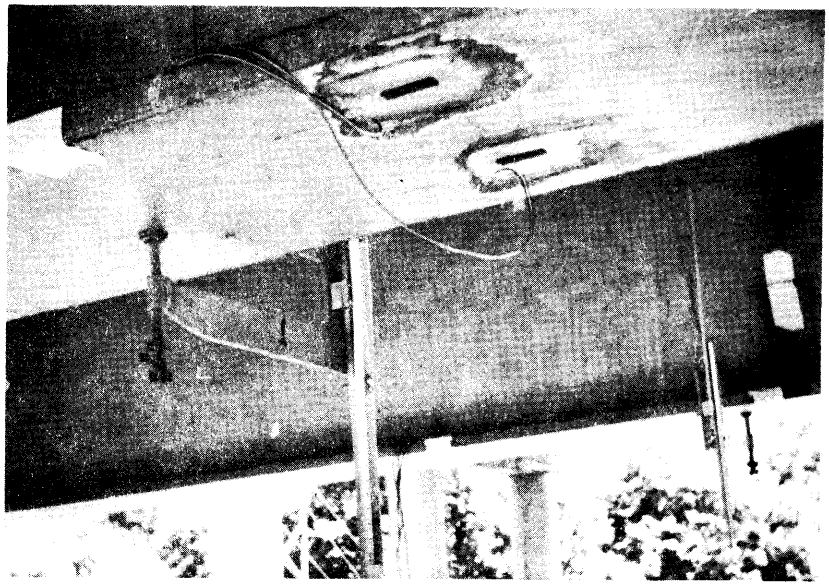


Figure 7. Deflection gages placed on a typical girder.

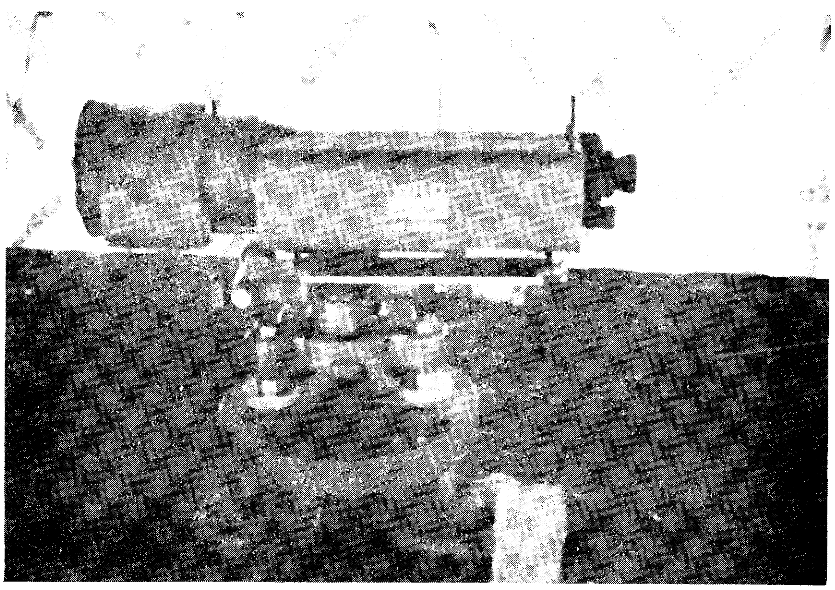
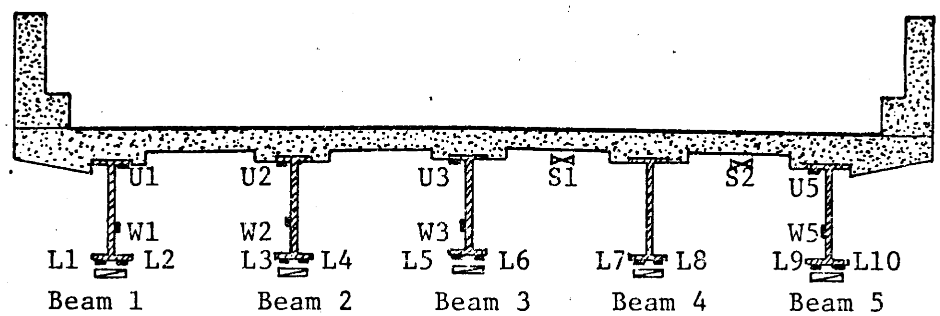


Figure 8. N-3 Wild Precise Level used for measuring deflections.

Legend:
- strain gages 12" east of midspan.
- deflection gages.
- gages S1 and S2 are on concrete 7'-7" east of midspan.



Transverse Section

Figure 9. Location of strain and deflection gages.

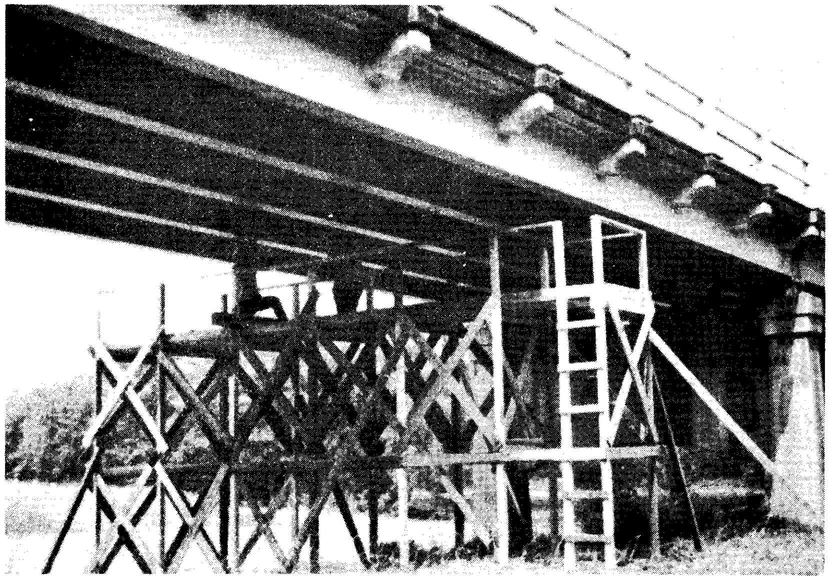
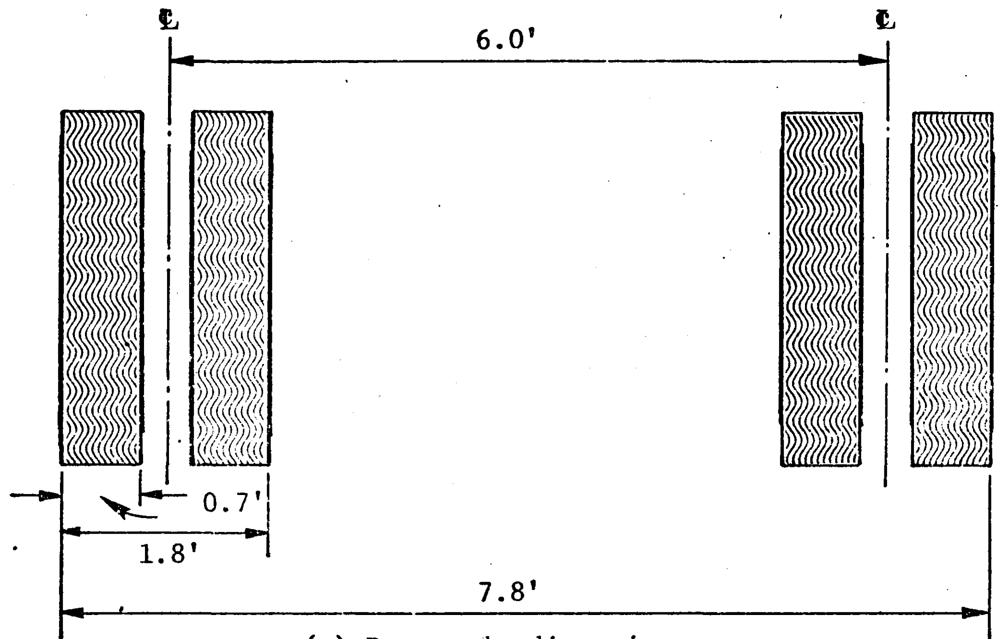


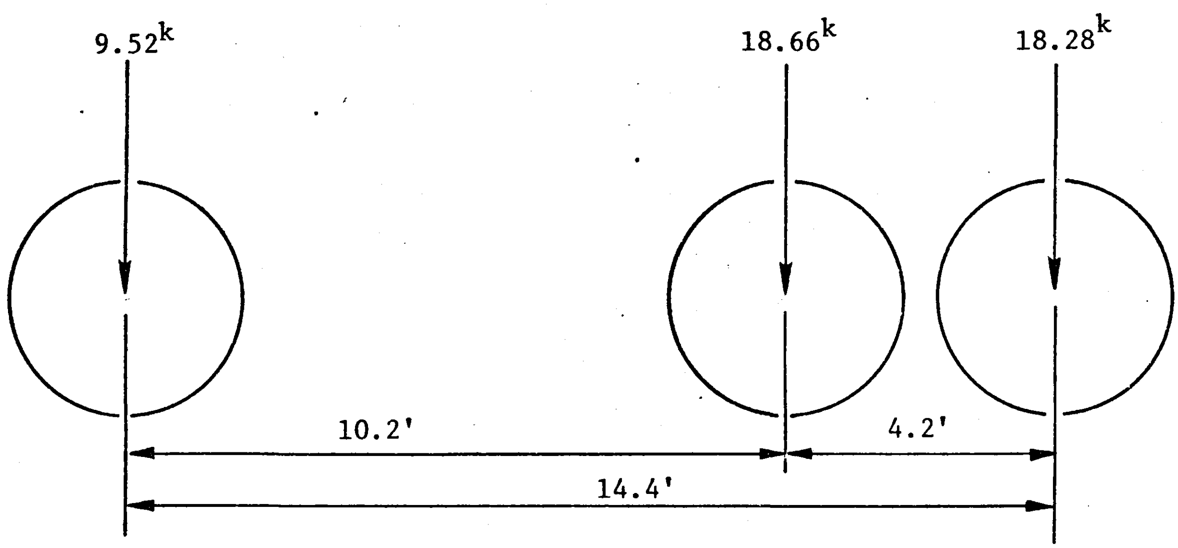
Figure 10. Test span with scaffolding erected for installation of gages.

Figure 11. Grinding of surfaces of steel girders in preparation for placing strain gages.

Figure 12. Portable digital strain indicator and switch and balance unit.



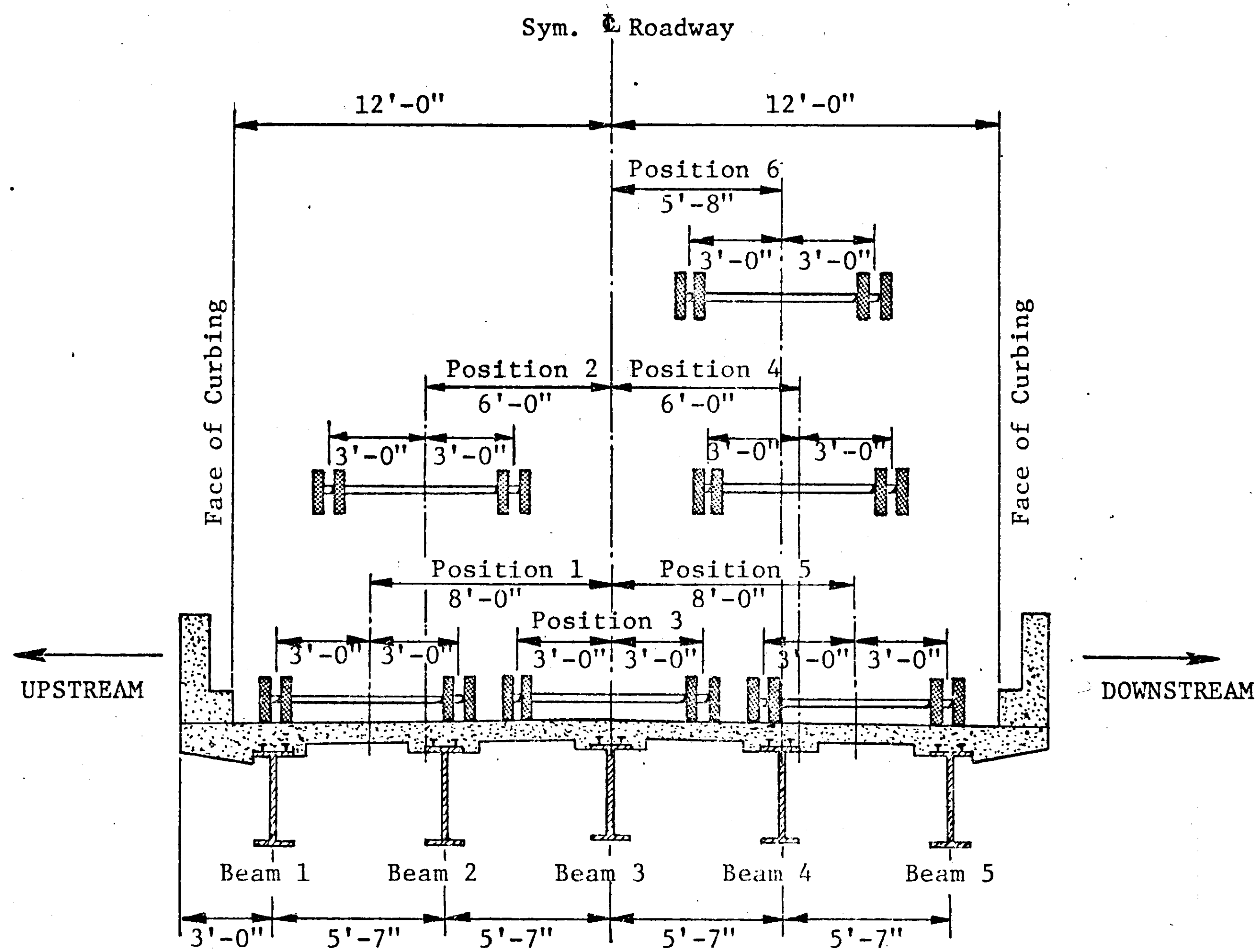
(a) Rear axle dimensions



(b) Axle loads in kips

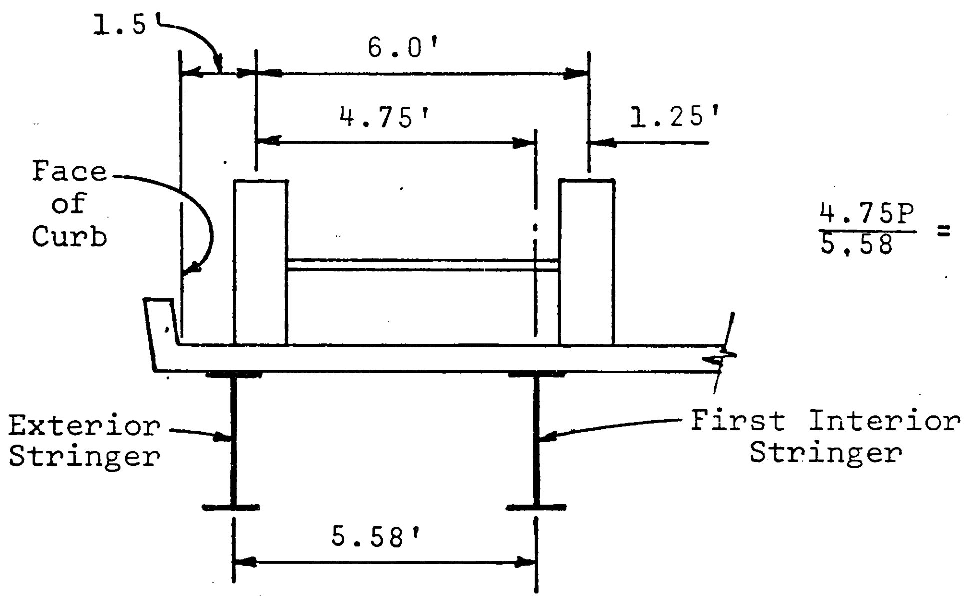
Figure 13. Axle dimensions and axle loads of the test vehicle.

Figure 14. Gasoline truck test vehicle.



Looking West (against traffic) toward Berryville.

Figure 15. Lane positions.



$$\frac{4.75P}{5.58} = 0.851P$$

Figure 16. Transverse position of test vehicle for calculation of live load distribution factor for exterior beam.

TABLE 1

AVERAGE UNIT STRAINS AND STRESSES

Position	Beam	Lower Flange L Gages		Web W Gages		Upper Flange U Gages		Deck Slab S Gages	
		Strain µin/in	Stress psi	Strain µin/in	Stress psi	Strain µin/in	Stress psi	Strain µin/in	Stress psi
1	1	132	3830	92	2670	-7	-200		
	2	95	2760	67	1940	-4	-120		
	3	49	1420	35	1020	-7	-200		
	4	23	670						
	5	5	150	1	30	0	0		
2	1	101	2930	70	2030	-5	-150		
	2	95	2760	69	2000	-4	-120		
	3	62	1800	45	1310	0	0		
	4	32	930						
	5	12	350	7	200	-1	-30		
3	1	38	1100	22	640	-5	-150		
	2	66	1910	50	1450	-3	-90		
	3	80	2320	61	1770	6	-170		
	4	70	2030						
	5	42	1220	31	900	-3	-90		
4	1	10	290	5	150	-1	-30		
	2	32	930	23	670	-1	-30		
	3	65	1890	47	1360	-4	-120		
	4	93	2700						
	5	99	2870	73	2120	-1	-30		
5	1	2	60	1	30	-3	-90		
	2	22	640	15	440	-2	-60		
	3	52	1510	38	1100	-2	-60		
	4	95	2760						
	5	132	3830	97	2810	-2	-60		
6	S1							65	200
	S2							44	130

NOTES:
1. See Figure 9 for location of gages.
2. See Figures 15 and 16 for location of positions.
3. Modulus of elasticity of steel assumed to be 29×10^6 psi and that of concrete assumed to be 3×10^6 psi.
4. No gages were placed on web or upper flange of Beam 4.

TABLE 2

DISTRIBUTION OF TRUCK LOAD TO BEAMS BASED ON
AVERAGE EXPERIMENTAL MIDSPAN DEFLECTIONS (inches)

	Beam 1	Beam 2	Beam 3	Beam 4	Beam 5	TOTAL
	Position 1					
Avg. Defl. Percentage	0.194 40.7%	0.147 30.9%	0.088 18.5%	0.039 8.2%	0.008 1.7%	0.476 100%
	Position 2					
Avg. Defl. Percentage	0.158 32.5%	0.144 29.6%	0.101 20.7%	0.057 11.7%	0.027 5.5%	0.487 100%
	Position 3					
Avg. Defl. Percentage	0.072 14.4%	0.110 22.0%	0.124 24.8%	0.109 21.8%	0.085 17.0%	0.500 100%
	Position 4					
Avg. Defl. Percentage	0.021 4.2%	0.067 13.3%	0.111 22.0%	0.146 29.0%	0.159 31.5%	0.504 100%
	Position 5					
Avg. Defl. Percentage	0.005 1.00%	0.047 9.4%	0.100 19.9%	0.158 31.5%	0.192 38.2%	0.502 100%
	Average Total Deflection - 0.494 inch					

TABLE 3

DISTRIBUTION OF TRUCK LOAD TO BEAMS BASED ON EXPERIMENTAL MIDSPAN LOWER FLANGE STRAINS (μin/in)

	Beam 1	Beam 2	Beam 3	Beam 4	Beam 5	TOTAL
	Position 1					
Avg. Strain Percentage	132 43.4%	95 31.3%	49 16.1%	23 7.6%	5 1.6%	304 100%
	Position 2					
Avg. Strain Percentage	101 33.4%	95 31.5%	62 20.5%	32 10.6%	12 4.0%	302 100%
	Position 3					
Avg. Strain Percentage	38 12.8%	66 22.3%	80 27.0%	70 23.7%	42 14.2%	296 100%
	Position 4					
Avg. Strain Percentage	10 3.4%	32 10.7%	65 21.7%	93 31.1%	99 33.1%	299 100%
	Position 5					
Avg. Strain Percentage	2 0.7%	22 7.3%	52 17.2%	95 31.3%	132 43.5%	303 100%
	Average Total Strain - 301 μin/in					

TABLE 4

EFFECTIVE MOMENTS OF INERTIA FROM EXPERIMENTAL STRAIN MEASUREMENTS (inches^4)

	Exterior Beams Positions				
	1	2	3	4	5
Beam 1	26,490	26,640	27,140	27,390	28,200
Beam 5	25,780	26,850	27,240	26,930	26,550
	Average for Exterior Beams = 26,920				
	Interior Beams Positions				
	1	2	3	4	5
Beam 2	27,330	27,510	28,030	27,740	27,530
Beam 3	27,260	27,430	28,000	27,700	27,440
Beam 4	27,410	27,480	28,090	27,740	27,330
	Average for Interior Beams = 27,600				

Example Calculation for above Table 4:

$$f = \frac{Mc}{I} \qquad\qquad I = \frac{Mc}{f}$$

M = Bending moment from test vehicle on span multiplied by the distribution factor for the particular beam and particular lane of test vehicle.

c = Distance from neutral axis to extreme fiber

f = Experimental strain x E

For beam 3 and test vehicle in position 3

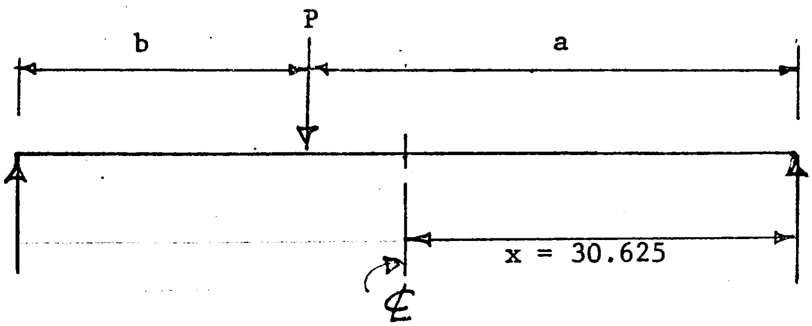
$$I = \frac{623.38 \times 12 \times 27\% \times 32.16}{80 \times 29 \times 10^3} = 28,000 \text{ in.}^4$$

TABLE 5

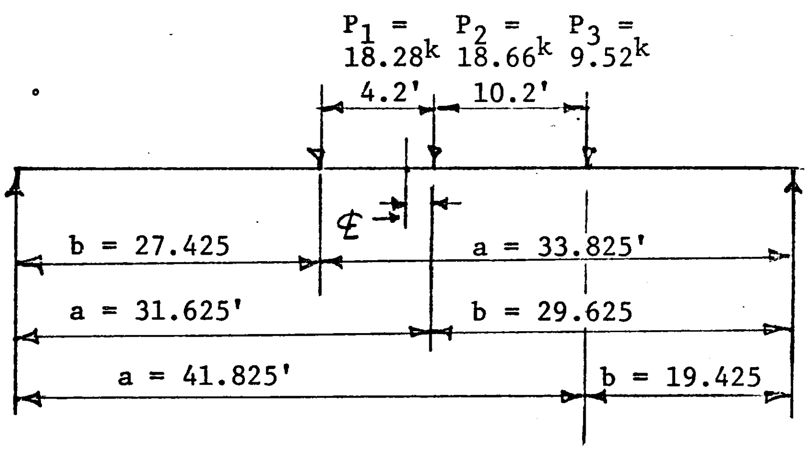
EFFECTIVE MOMENTS OF INERTIA FROM
EXPERIMENTAL DEFLECTION MEASUREMENTS (inches^4)

	Exterior Beams Positions				
	1	2	3	4	5
Beam 1	26,600	26,080	25,360	25,360	25,360
Beam 5	26,950	25,830	25,360	25,120	25,230
	Average for Exterior Beams - 25,730 in.^{4}				
	Interior Beams Positions				
	1	2	3	4	5
Beam 2	26,650	26,060	25,360	25,360	25,360
Beam 3	26,660	25,990	25,360	25,360	25,360
Beam 4	26,650	26,030	25,360	25,190	25,280
	Average for Interior Beams - 25,730 in.^{4}				

Example Calculations for Table 5, Moments of Inertia from Experimental Deflections



$$\Delta_x = \frac{Pbx}{6EIl}(l^2 - b^2 - x^2) \quad \text{for } x<a$$



$$\Delta_1 = \frac{18.28}{EI} \times \frac{27.425 \times 30.625}{6 \times 61.25}(61.25^2 - 27.425^2 - 30.625^2)$$

$$= \frac{P_1}{EI} \times \frac{b}{12}(2813.7 - b^2) = P_1\, 4711.5 \times \frac{1728}{29\times10^6 I} = \frac{5{,}130}{I}$$

$$\Delta_2 = \frac{P_2}{EI} \times \frac{29.625}{12}(2813.7 - 29.625^2) = P_2\, 4779.6 \times \frac{1728}{29\times10^6 I} = \frac{5{,}310}{I}$$

$$\Delta_3 = \frac{P_3}{EI} \times \frac{19.425}{12}(2813.7 - 19.425^2) = P_3\, 3943.8 \times \frac{1728}{29\times10^6 I} = \frac{2{,}240}{I}$$

$$\Sigma\Delta = \frac{12{,}680}{I}$$

$$I = \frac{12.680 \times 24.8\%}{0.124} = 25{,}360 \text{ in.}^4$$

For Beam 3 and test vehicle in Position 3.

TABLE 6

SUMMARY OF THEORETICAL AND EXPERIMENTAL VALUES OF MOMENTS OF INERTIA (inches^4)

	Exterior Beam	Interior Beam
Theoretical values	23,040	24,070
Average of experimental values from strain measurements	26,920	27,600
Average of experimental values from deflection measurements	25,730	25,730

TABLE 7

LOCATION OF NEUTRAL AXIS ABOVE LOWER
SURFACE OF BOTTOM FLANGE (inches)

	Method 1	Method 2	Method 3	Average
Exterior Beams				
1	29.51	32.24	31.73	31.16
5	34.19	33.72	33.63	33.85 32.51
Interior Beams				
2	32.17	32,24	32.20	32.20
3	33.62	33.93	33.94	33.83 33.02

NOTES: Interior Beam 4 was not instrumented with web and upper flange gages.

Method 1 - extrapolated between experimental strains from lower flange and web gages.

Method 2 - interpolated between experimental strains from lower flange and upper flange gages.

Method 3 - interpolated between experimental strains from web and upper flange gages.

TABLE 8

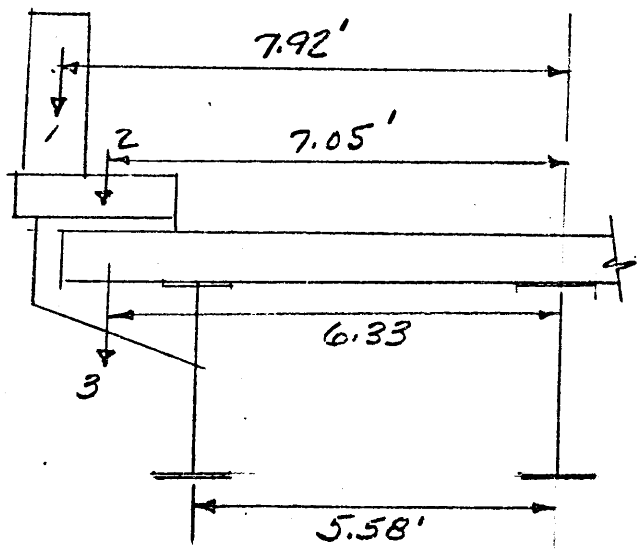
MIDSPAN LOWER FLANGE STRESS COMPARISONS (ksi)

	Exterior Beam	Interior Beam
Dead Load Stress	9.16 (1,2,3)	10.53 (1,2,3)
Experimental Live Load Stress (Max. From Table 1) Simulated Beam 5 stress with Test Vehicle in Positions 1 and 5 Simulated Beam 2 stress with Test Vehicle in Positions 2 and 4	3.98	3.69
Theoretical Live Load Stress	4.31 (4,5)	5.07 (4,5)
Impact (Exp. LL. 26.85%) 6	1.06 (6)	0.99
Impact (Theor. LL. 26.85%)	1.16	1.36
Total Using Exper. LL stress	14.20	15.21
Total Using Theor. LL stress	14.63	16.93
Ratio $\frac{\text{Exp. LL. Stress}}{\text{Theor. LL. Stress}}$	92.3%	72.8%
Ratio $\frac{\text{Total Stress (Exp. LL.)}}{\text{Total Stress (Theor. LL.)}}$	97.1%	89.8%

NOTES:
1. Dead load stresses based on weights of curbs, posts and railings being equally distributed to all beams.
2. Dead load stresses are 12.17 ksi and 9.35 ksi for exterior and interior beams, respectively, based on exterior beam carrying all of the curbs, posts, and railings and simple beam reactions from first interior beam.
3. Dead load stresses based on beams not being shored during placement of concrete deck. The bridge was designed and constructed before shear connectors were generally in use.
4. Live load stresses based on complete composite action, which more closely agrees with experimental results.
5. Theoretical live load stresses are 6.32 ksi and 7.53 ksi for exterior and interior beams, respectively, based on non-composite action.
6. Impact Factor = $\frac{50}{61.25+125}$ = 26.85%.

CALCULATION OF DEAD LOAD FLEXURAL STRESSES IN EXTERIOR AND INTERIOR BEAMS
(Listed in Table 8)

Exterior Beams



Assume posts, railings, curbing and brackets applied to exterior beams based on simple beam moments about first interior beam.

1.	Posts & railings	100.4 lbs	×	7.92'	=	795.2 fp.
2.	Curbing	109.4	×	7.05'	=	771.3
3.	Brackets	54	×	6.33	=	341.8
	Bolster	28	×	5.58	=	156.2
	Asphalt	115	×	3.19	=	366.9
	Slab	750	×	3.75	=	2812.5
	Beam	150	×	5.58	=	837.0
						6080.9 fp.

$$R = \frac{6080.9}{5.58} = 1089.8 \text{ plf}$$

$$M = \frac{1089.8 \times 61.25^2}{8} = 511.0 \text{ fk.}$$

$$f = \frac{511.0 \times 12}{504} = 12.17 \text{ ksi}$$

Interior Beams

Slab	558 plf
Beam	150
Bolster	28
Asphalt	101
	837 plf

$$M = \frac{837 \times 61.25^2}{8} = 392.5 \text{ fk}$$

$$f = \frac{392.5 \times 12}{504} = 9.35 \text{ ksi}$$

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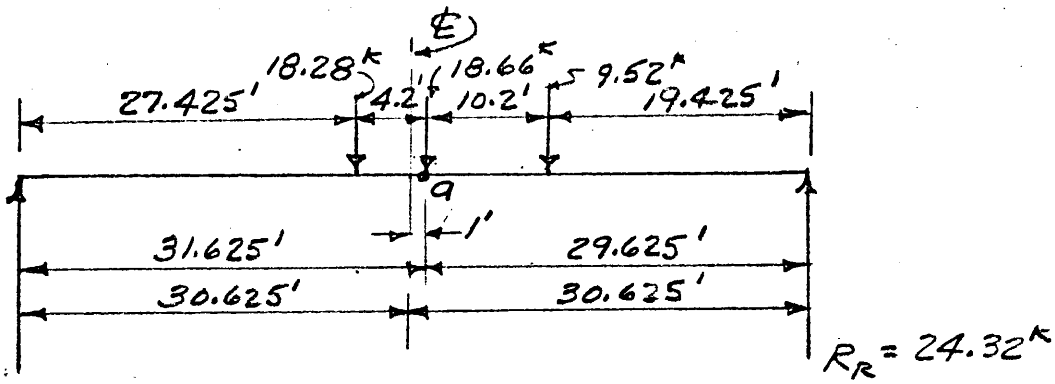
CALCULATION OF LIVE LOAD FLEXURAL STRESSES IN EXTERIOR AND INTERIOR BEAMS (Listed in Table 8)



$R_L = 22.14^k$

$$M_a = 24.32 \times 29.625 - 9.52 \times 10.2$$
$$= 623.38 \text{ fk}^{*}$$

Exterior Beams:

$$M = 623.38 \times 12 \times \tfrac{1}{2} \times 0.851 = 3183.0 \text{ ik}$$

$$\text{Section Modulus} = \frac{I}{c} = \frac{23{,}040}{31.23} = 737.8 \text{ in}^3$$

$$f_{LL} = \frac{3183.0}{737.8} = 4.31 \text{ ksi}$$

Interior Beams:

$$M = 623.38 \times 12 \times \tfrac{1}{2} \times 1.015 = 3796.4 \text{ ik}$$

$$\text{Section Modulus} = \frac{I}{c} = \frac{24{,}070}{32.16} = 748.4 \text{ in}^3$$

$$f_{LL} = \frac{3796.4}{748.4} = 5.07 \text{ ksi}$$

* Absolute max. bending moment for this loading on this span is 624.52 fk at a point 0.22 ft. from centerline. Max. bending moment for a Type 3 Truck Loading for this effective span is 569.0 fk. See Plate 2 of the <u>Manual for Maintenance Inspection of Bridges</u>, 1974.

CALCULATION OF DEAD LOAD FLEXURAL STRESSES IN EXTERIOR AND INTERIOR BEAMS (Cont'd)
(Listed in Table B)

Exterior Beams

Assume posts, railings, curbing, and brackets applied equally to the five exterior and interior beams.

Slab	$\frac{56\frac{1}{2} \times 8}{144} \times 150$	$= 471$ plf
Beam WF		$= 150$
Bolster	$\frac{1\frac{1}{2} \times 18}{144} \times 150$	$= 28$
Asphalt	$\frac{43.5 \times 1\frac{1}{2} \times 144}{144}$	$= 65$
Posts, etc.	$\left(\frac{100.4 + 109.4 + 54}{5}\right) 2$	$= 106$
		Σ 820 plf

$$M = \frac{820 \times 61.25^2}{8} = 384.5 \text{ fk}$$

$$f = \frac{384.5 \times 12}{504} = 9.16 \text{ ksi}$$

Interior Beams

Slab	$\frac{67 \times 8}{144} \times 150$	$= 558$ plf
Beam WF		$= 150$
Bolster		$= 28$
Asphalt	$\frac{67 \times 1\frac{1}{2} \times 144}{144}$	$= 101$
Posts, etc.		$= 106$
		Σ 943 plf

$$M = \frac{943 \times 61.25^2}{8} = 442.2 \text{ fk}$$

$$f = \frac{442.2 \times 12}{504} = 10.53 \text{ ksi}$$

TABLE 9

AVERAGE EXPERIMENTAL MIDSPAN LOWER FLANGE LIVE LOAD STRESSES (psi) FROM SIMULATED TWO-LANE LOADING

Beam	Stress Test Vehicle in Lane 2	Stress Test Vehicle in Lane 4	Sum
1	2930	290	3220
2	2760	930	3690
3	1800	1890	3690
4	930	2700	3630
5	350	2870	3220

TABLE 10

AVERAGE EXPERIMENTAL MIDSPAN LIVE LOAD DEFLECTIONS (inches) FROM SIMULATED TWO-LANE LOADING

Beam	Deflection Test Vehicle in Lane 2	Deflection Test Vehicle in Lane 4	Sum
1	0.158	0.021	0.179
2	0.144	0.067	0.211
3	0.101	0.111	0.212
4	0.057	0.146	0.203
5	0.027	0.159	0.186