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A LOADING STUDY OF OLDER HIGHWAY BRIDGES IN VIRGINIA

Part 3

A Concrete Tee-Beam Bridge in Nelson County

by

H. L. Kinnier
Faculty Research Engineer

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

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SUMMARY

A 40-foot clear span, standard reinforced tee-beam bridge over the Tye River in Nelson County was tested with a 23-ton, tandem axle test vehicle in July 1976. The test span was one of four identical spans making a total bridge length of 170 feet. Strain gages were placed on the bottom face of each of the four stems, on the sides of the four stems, and on the underside of the bridge deck between the stems. In addition to the strains in 22 gages, midspan deflections were measured in gages on the four tee-beams for each position of the test vehicle. The purpose of this phase of the three-part study was to measure the live load response of this structure which is typical of a large number of tee-beam bridges built in Virginia and throughout the United States in the early twentieth century.

The test results indicated that the experimental strains and deflections were much smaller than the live load stresses and deflections one would expect from the applied loads and calculations by conventional elastic structural theory.

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BACKGROUND

The purpose of this study was to determine experimentally the live load stresses 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 tee-beam spans. The Part 1 report,[1] issued in February 1976, presented test results from a steel truss bridge tested in July 1974. The Part 2 report,[2] issued in November 1976, presented test results from a concrete slab and steel beam bridge tested in July 1975. This, the final report of the series, Part 3, reports on a concrete tee-beam bridge span in Nelson County tested in July 1976.

DESCRIPTION OF THE TEST STRUCTURE

The span selected for testing was a standard 40-foot, clear span concrete tee-beam with a 24-foot roadway. This standard plan, now obsolete, was prepared in 1931. The test span was one of four identical spans of a bridge on Route 56 over the Tye River, 1 mile east of Massies Mill. (See Figures 1, 2, 3, and 4.*) The structure was constructed in 1936 and the plans are available from the Bridge Office of the Virginia Department of Highways and Transportation under the designation of LXIV-19 dated June 27, 1936. The test span was constructed from the standard plan designated C-24-40 prepared in 1931 as mentioned above. See Figure 5 for transverse sections and Figures

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*All figures and tables are attached.

6 and 7 for longitudinal sections. The structure was basically in an excellent state of repair at the time of testing. However, the end diaphragms showed considerable spalling at their lower corners where salts from melting compounds had dripped through the joints at the ends of the spans. Reinforcing bars in the diaphragms were bared where large chunks of the concrete had spalled off (see Figures 8 and 9). The same type of deterioration, apparently simply from weathering, had developed on the concrete bridge railings as shown in Figure 10. The main tee-beams, the roadway slab, the abutments, and the piers were in excellent condition.

There is no question, based on the measured strains and deflections, the apparent good physical condition of the bridge, and the results of destructive tests on bridges of similar design in other states,[3,4] that this structure could continue to carry traffic for a number of years as well as successfully support an infrequent overload of considerable magnitude.

The bridge was designed and constructed in accordance with the Virginia Department of Highways Bridge Specifications, 1932,[5] for an H-15 Standard AASHTO[6] Loading.

INSTRUMENTATION

Twenty-two SR-4 type A-9-3 strain gages were placed on the faces of the concrete surface on the stems of the four tee-beams and the underside of the concrete deck as shown in Figure 11. Engineer's scales with 20 divisions to the inch were attached to the four concrete stems as shown in the photograph of Figure 12. The scales were read with a precise N-3 Wild Level (Figure 13) with a least reading of 0.001 inch. All of the strain and deflection gages were placed at midspan.

The 22 strain gages were wired into two 10-channel Model SB-1 switch and balance units manufactured by Vishay Instruments, Inc. A battery powered Model P-350 portable digital strain indicator was used to read the strains. Both of these pieces of equipment are shown in Figure 14.

TRUCK LOADING

A privately owned dump truck, which is typical of those frequently contracted for by the Virginia Department of Highways and Transportation for hauling, was rented for use in the test. The truck was loaded with crushed stone and the front and rear axle weights, as well as the total weight, were measured and recorded at a nearby rock quarry. The truck axle dimensions and average axle loads are shown in Figure 15.

The truck loading was nearly the same as the type 3 unit loading designated in Plate 15, p. 59 of the 1974 edition of the Manual for Maintenance Inspection of Bridges.(7) The type 3 unit loading has a total weight of 23 tons, while the truck used in this study had a total weight of 22.85 tons. Figure 16 shows the axle dimensions and loads of the type 3 truck. The resulting bending moments from the test loading and standard type 3 vehicle for this test span were practically identical.

For the 41.25 foot effective span length for the test span, the maximum live load bending moment for the test vehicle was 356 foot kips at midspan where the strain gages were located, whereas for the type 3 truck loading the maximum midspan bending moment would be 338 foot kips. That is, the test loading produced a midspan bending moment approximately 5% in excess of that from the type 3 legal load limit.

TEST PROCEDURE

The test vehicle was placed at five lateral midspan positions as shown in Figure 17 to determine the distribution of strains and deflections to each of the four tee-beams for each of the load positions. The first rear axle was placed at the midspan strain gage positions to provide maximum flexural stresses at that location where the 22 type A-9-3 strain gages and 4 deflection gages were placed.

With the test vehicle placed in each of the five midspan positions, the 22 strain gages and 4 deflection gages were read and recorded. The procedure was repeated three times to provide four complete sets of strains at 22 positions and midspan deflections of the four tee-beams. Two sets of readings were made with the test vehicle heading east and two with it headed west.

TEST RESULTS

The test results were all in the form of strain readings from the 22 type A-9-3 SR-4 wire strain gages and vertical deflections from the scales placed on the sides of the stems of the four concrete tee-beams.

Average values of all of the strain readings are shown in Table 1. All of the strains were extremely small in magnitude. The strains in the 6 S gages which were oriented in the direction of the bridge span and placed on the underside of the concrete deck were essentially unresponsive. The values varied from 0 to 13 microinches/inch (0 to 39 psi), reversed erratically from compression to tension in the four different test loadings, and the results were obviously affected by either cracks in the slab or poor adherence of the gages to the concrete surface. These data are not included in this report. The results from the 8 W gages 12 inches up on the sides of the webs were also small but somewhat larger than the S gage results. See Figure 18 for the locations of the strain and deflection gages. The strain readings from the W gages were less consistent and reproducible than those from the L gages on the lower flange, but in general were indicative of a linear variation in magnitude of flexural strains in proportion to the distance from the neutral axis.

The average lower flange strains and midspan deflections are tabulated in Table 1, and the lateral distribution to the four girders on the basis of strain and deflections are plotted for the five test load positions in Figures 19 and 20. The deflection measurements indicated a high degree of reliability, both from the symmetry of the deflection readings as the test vehicle was moved across the structure in its five lateral positions and the fact that these same deflections were closely reproduced when an additional set of readings were made with the test vehicle headed in the opposite direction.

Table 2 lists the unit strain in microinches per inch on the lower surface of the stems of the four tee-beams. These strains are listed for each tee-beam for each of the five lateral positions of the test vehicle. These strains, as mentioned earlier, are extremely low but do show a pattern which reflects the application of the truck loading on the bridge deck. This distribution of the loading is plotted in Figure 19.

Table 3 lists the beam deflections at midspan in inches for each of the five lateral positions of the test vehicle. These data produced symmetrical results for the four beams with the five positions of the test vehicle and the measurements were reproducible for the four sets of test data. These deflections showed the structure to be well in the lower limits of the elastic stress range, even with this heavy type 3 test loading. The distributions of the loading from the deflection measurements are plotted in Figure 20 and are similar to the corresponding results from the strain readings.

Table 4 lists the effective moments of inertia for each of the four tee-beams as calculated from the strain readings with the test vehicle in each of the five positions. The modulus of elasticity was assumed to be 3×10^6 psi and the common elastic flexural stress formula $I = \frac{Mc}{f}$ was used to calculate the moments of inertia. These values are approximately three times the corresponding values computed from the beam deflection readings and five times the cracked section theory values. The reliability of the strain readings on the concrete stems is somewhat questionable because of their low magnitude.

There was some apprehension concerning the proper adherence between the gages and the 40 year old weathered concrete. The value of 514,480 inches^4 is believed to be high for the moment of inertia of an interior tee-beam.

Table 5 lists the effective moments of inertia for each of the four tee-beams as calculated from the midspan deflection readings with the test vehicle in each of the five positions. The calculations of these values are shown in the Appendix. These values are two and a half to three times the theoretical values and are believed to reliably predict the additional strength of this type structure over the capacity as calculated by cracked section elastic theory. The ultimate capacity of the total section is calculated to be 19 times the applied load on page A-5 of the Appendix. Even with a substantial load factor of three, there remains a considerable reserve live load capacity after deducting the dead load moment.

Table 6 summarizes the effective moments of inertia as computed by elastic theory and the average values as determined by the two experimental procedures.

Table 7 summarizes the theoretical dead load stresses, the theoretical live load stresses, and the experimental live load stresses by simulating the test vehicle in passing lanes simultaneously. The ratios listed in the bottom two lines show the reserve strength of this structure over that calculated by elastic theory.

Table 8 lists the live load stresses in the four beams from placing the test vehicle in two passing lanes simultaneously.

Table 9 lists the live load deflections in the four beams from placing the test vehicle in two passing lanes simultaneously.

CONCLUSIONS

1. The 22.85 ton test vehicle in this experiment developed extremely low live load strains and deflections in this concrete tee-beam bridge designed for an H-15 (15 ton) live loading. The live load strains were measured with the test vehicle applied in a static condition, i.e., no impact strains were included in the test measurements.

2. This structure has served its intended purpose for over 40 years (1936 - 1977) with little need for maintenance and no apparent diminishing of its live load carrying capacity.

3. This structure has a substantial overload capacity, as can be noted from the calculations of its theoretical ultimate strength as well as from the small experimental strain and deflection measurements. A test to failure of a similar concrete tee-beam bridge in Tennessee in 1970[3] also attested to the sturdiness and high live load capacity of this type highway bridge.

4. The effective moments of inertia of the concrete tee-beams are substantially larger than those calculated by the conventional elastic cracked section theory that was used exclusively by concrete designers until recent years when elastic theory has been largely replaced with ultimate strength design in many engineering design offices.

ACKNOWLEDGMENTS

The experimental phase of this study required the assistance and cooperation of many persons in the employ of the Virginia Department of Highways and Transportation. The author gives particular recognition to James French, Steven Leftwich, Richard Hankins, and Preston Hubble for their assistance in both the installation of the strain and deflection gages and the recording of the data during the loading process. The author also thanks Wayne Schwartz, district bridge engineer, for his assistance in locating a suitable test structure in the vicinity of Charlottesville.

The research project was conducted under the general supervision of Jack H. Dillard, head, Virginia Highway and Transportation Research Council, and Harry E. Brown, assistant head. 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. Kinnier, H. L., "A Loading Study of Older Highway Bridges in Virginia, Part 1, A Steel Truss Bridge in Alleghany County," Virginia Highway & Transportation Research Council, VHTRC 76-R42, February 1976.

2. Kinnier, H. L., and L. L. Ichter, "A Loading Study of Older Highway Bridges in Virginia, Part 2, A Concrete Slab and Steel Beam Bridge in Clarke County," Virginia Highway & Transportation Research Council, VHTRC 77-R26, November 1976.

3. Burdette, E. G., and D. W. Goodpasture, "Full-Scale Bridge Testing; An Evaluation of Bridge Design Criteria," Department of Civil Engineering, University of Tennessee, December 31, 1971.

4. Jorgenson, J. L., and Wayne Larson, "Field Testing a Reinforced Concrete Highway Bridge," College of Engineering, North Dakota State University, December 1974.

5. Virginia Department of Highways Bridge Specifications — Design, 1932.

6. AASHTO, Standard Specifications for Highway Bridges, 11th Ed., 1973.

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

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Figure 1. Bridge showing test vehicle on span.

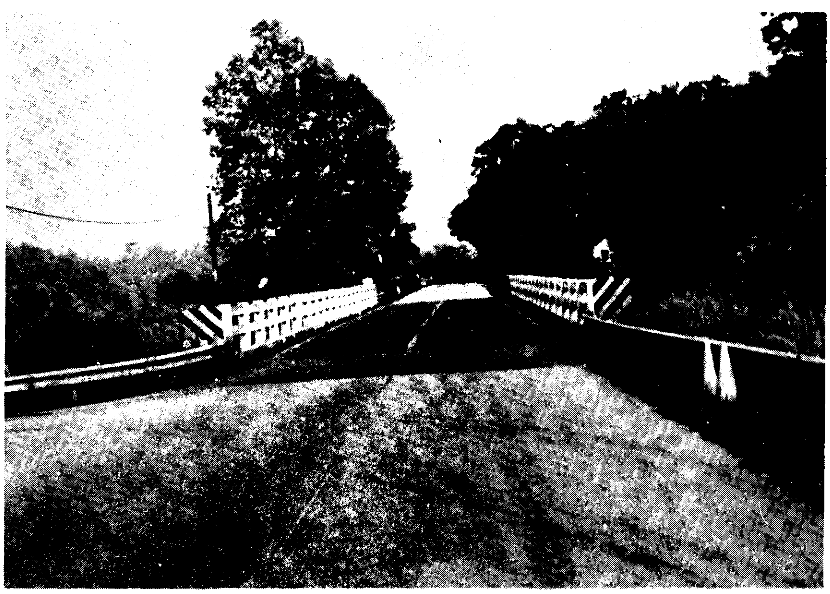

Figure 2. Bridge deck looking west.

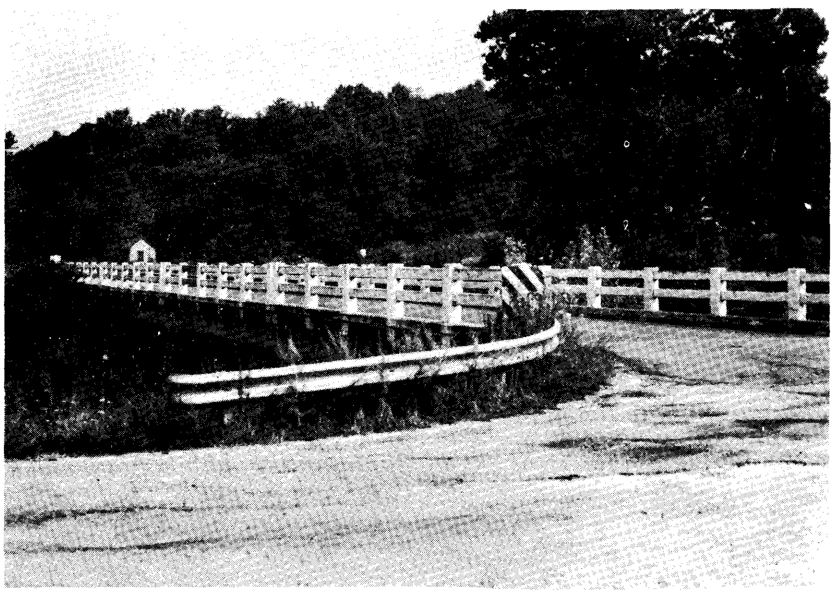

Figure 3. Bridge deck looking east.

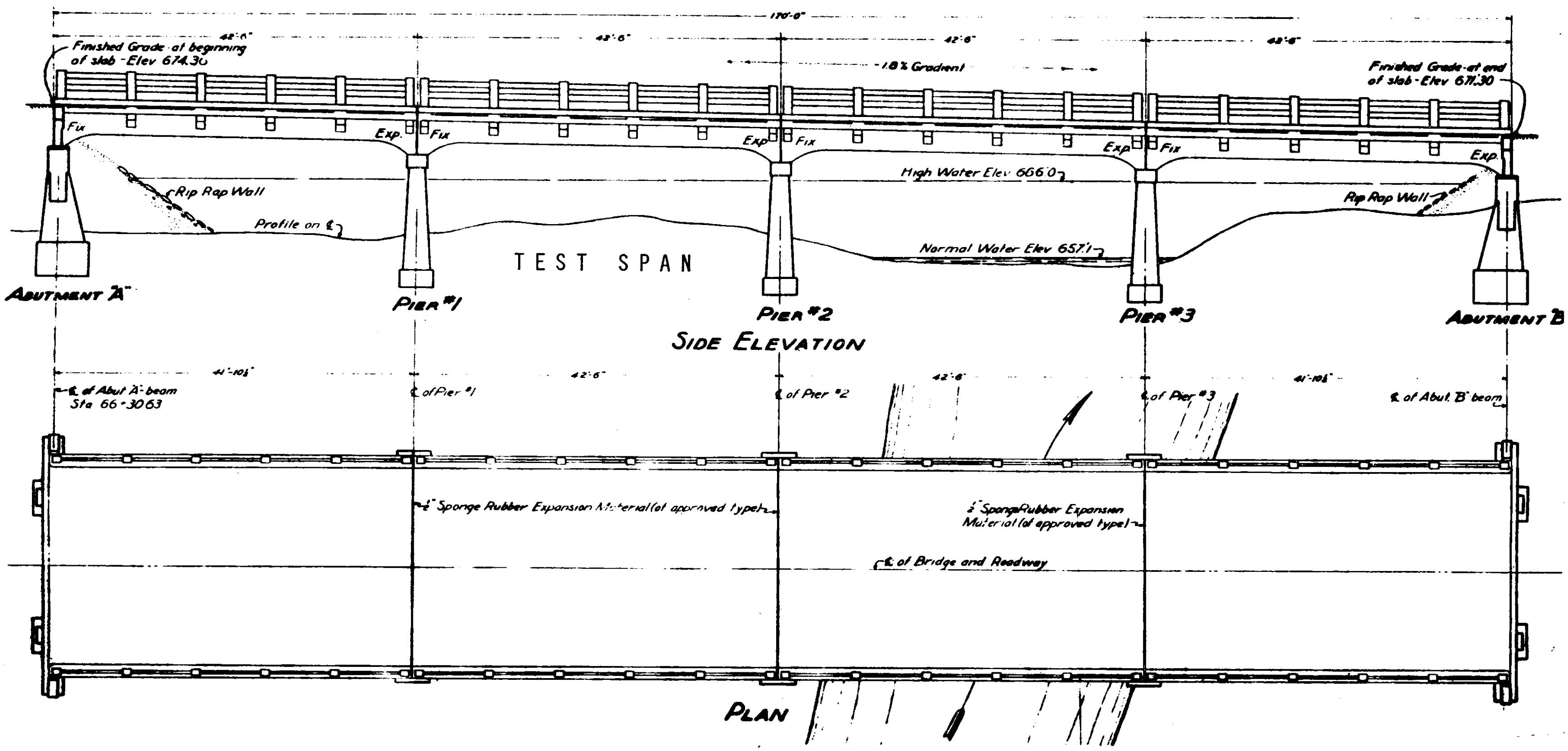

Figure 4. Plan and side elevation of bridge showing test span.

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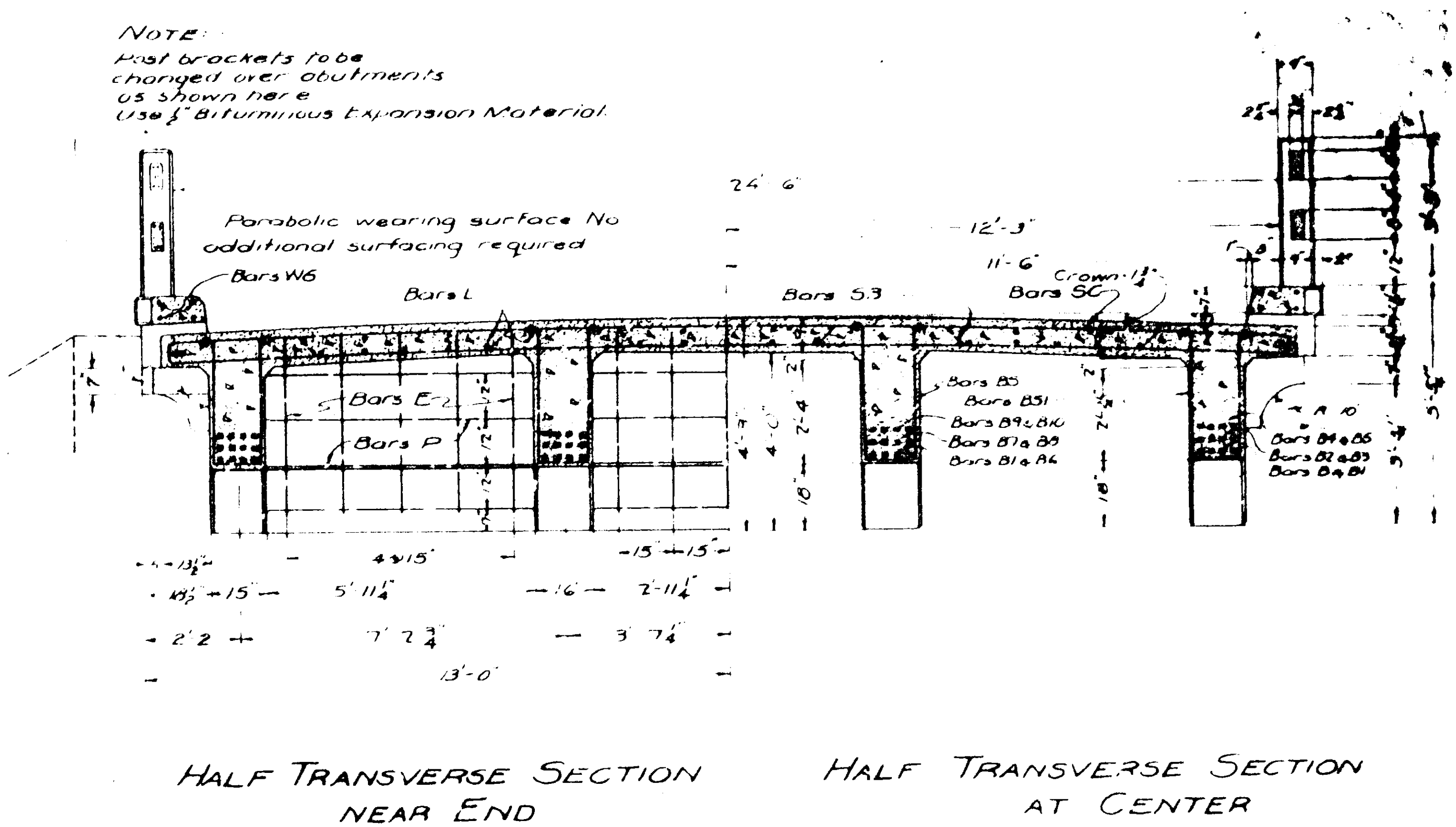

Figure 5. Half transverse sections.

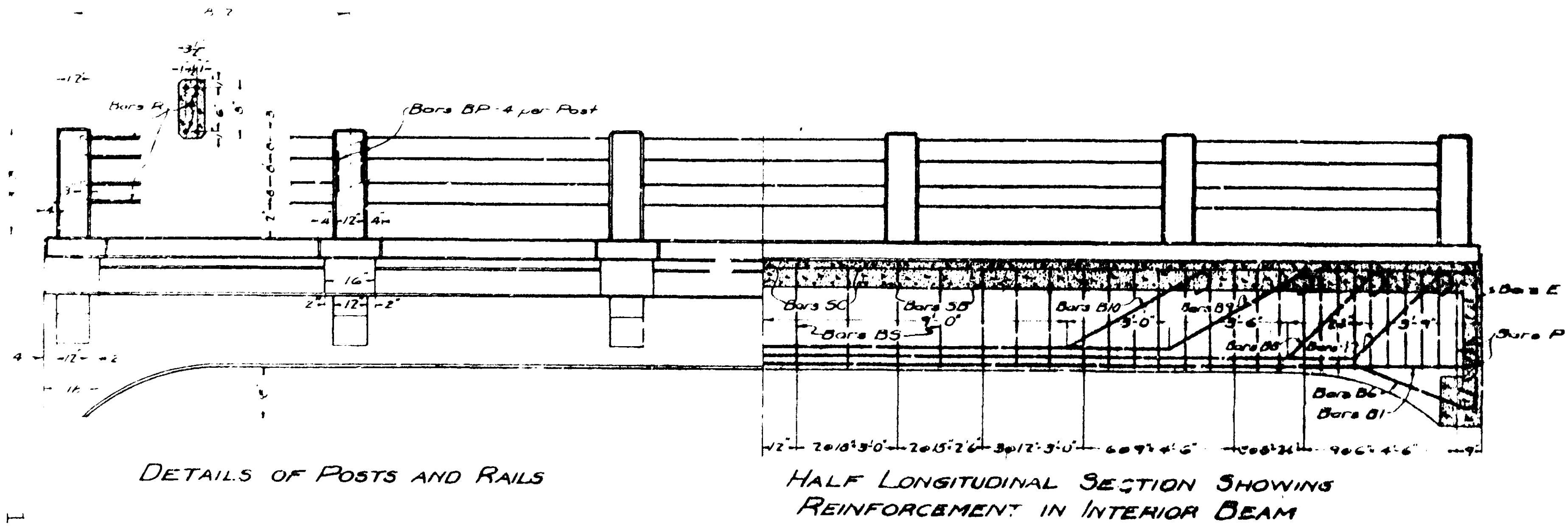

Figure 6. Half longitudinal section of interior beam.

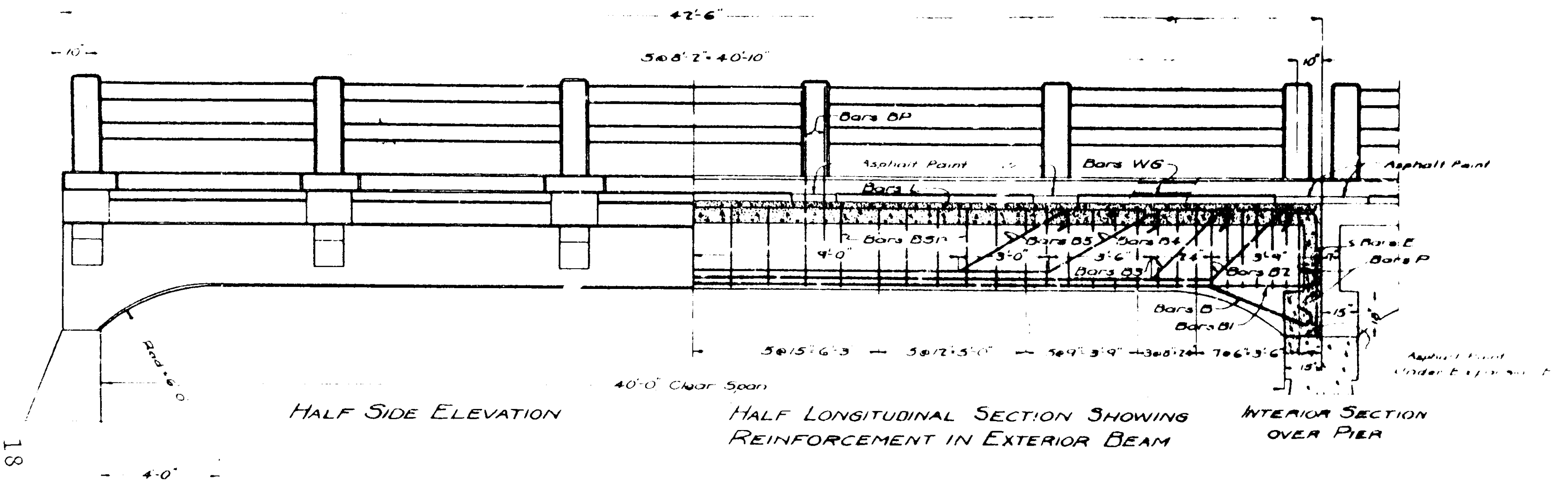

Figure 7. Half longitudinal section of exterior beam.

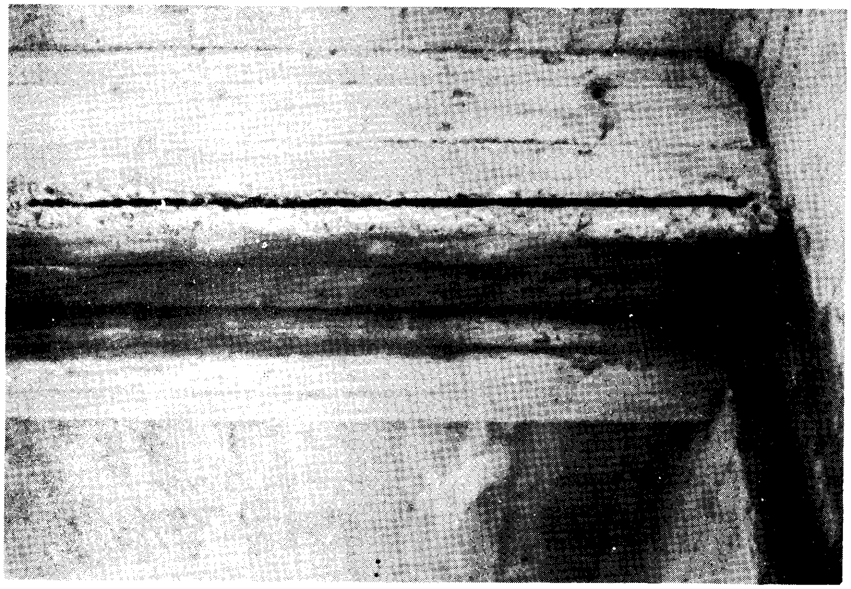

Figure 8. Spalling of concrete on the end diaphragms baring reinforcing steel.

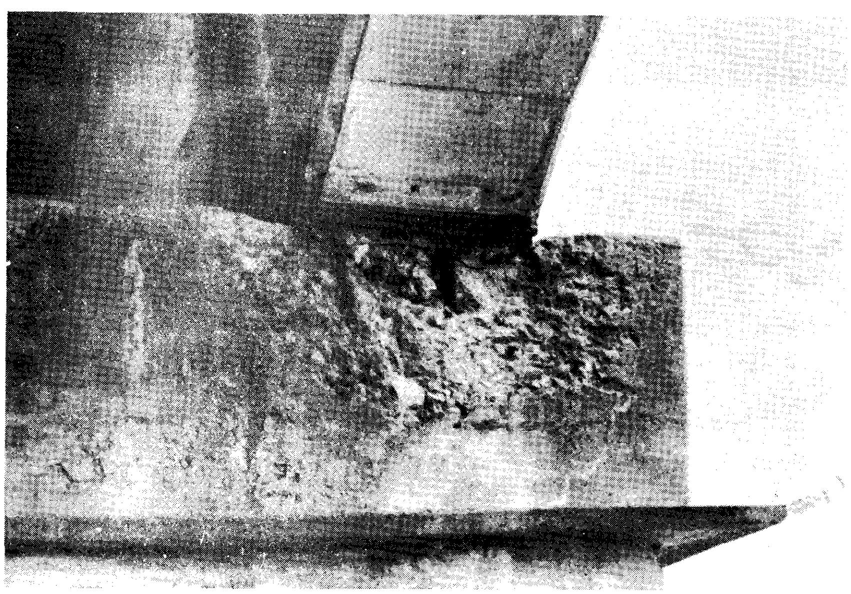

Figure 9. Spalling of concrete on the pier cap baring reinforcing steel.

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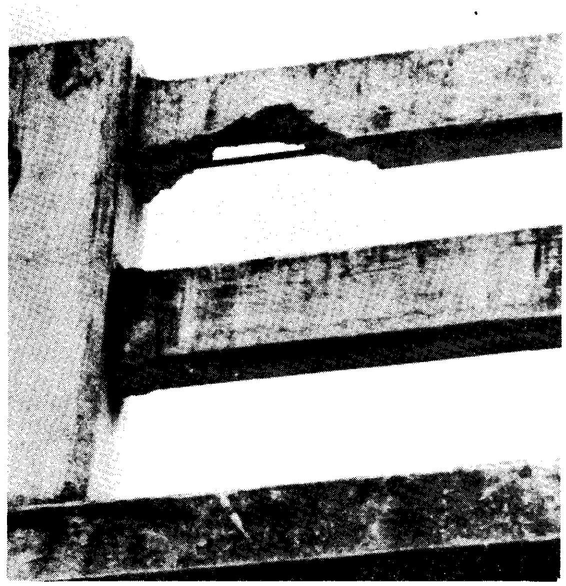


Figure 10. Deterioration of concrete railing baring reinforcing steel.

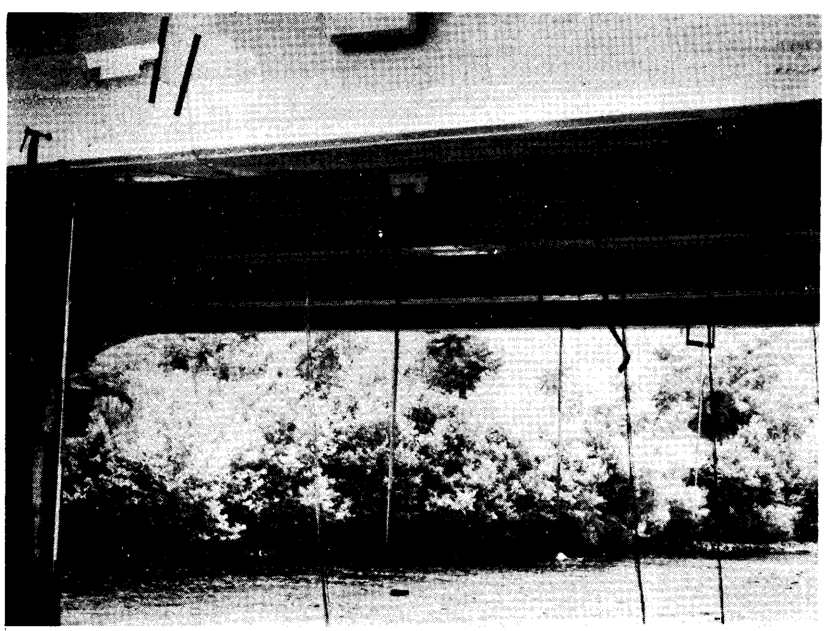


Figure 11. Deflection and strain gages mounted at midspan.

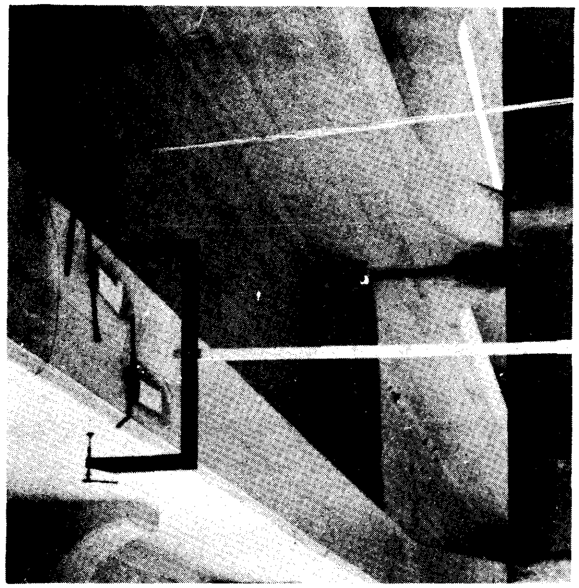


Figure 12. One of the engineer's scales mounted for measuring deflections.

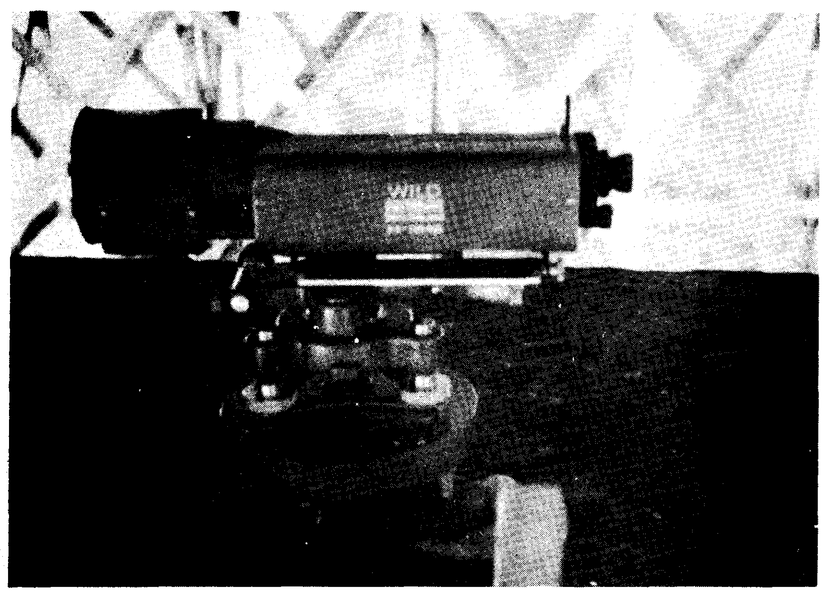


Figure 13. N-3 Wild precise level used for measuring deflections.

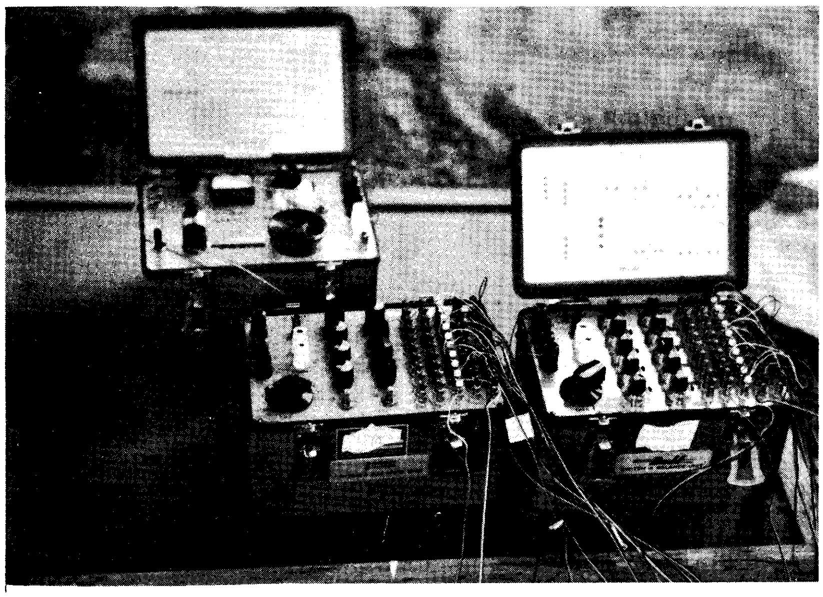


Figure 14. Two portable digital strain indicators and switch and balance unit.

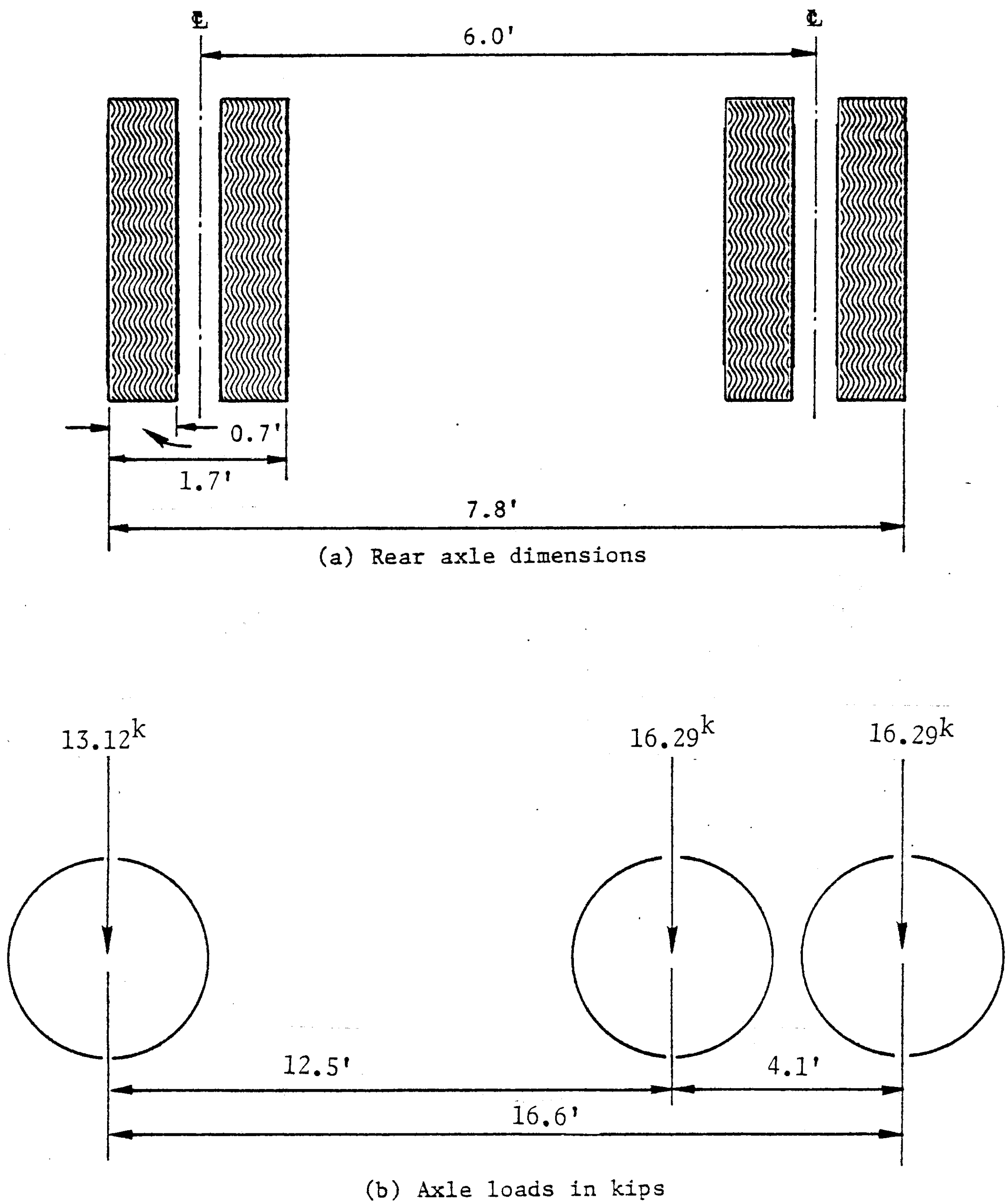
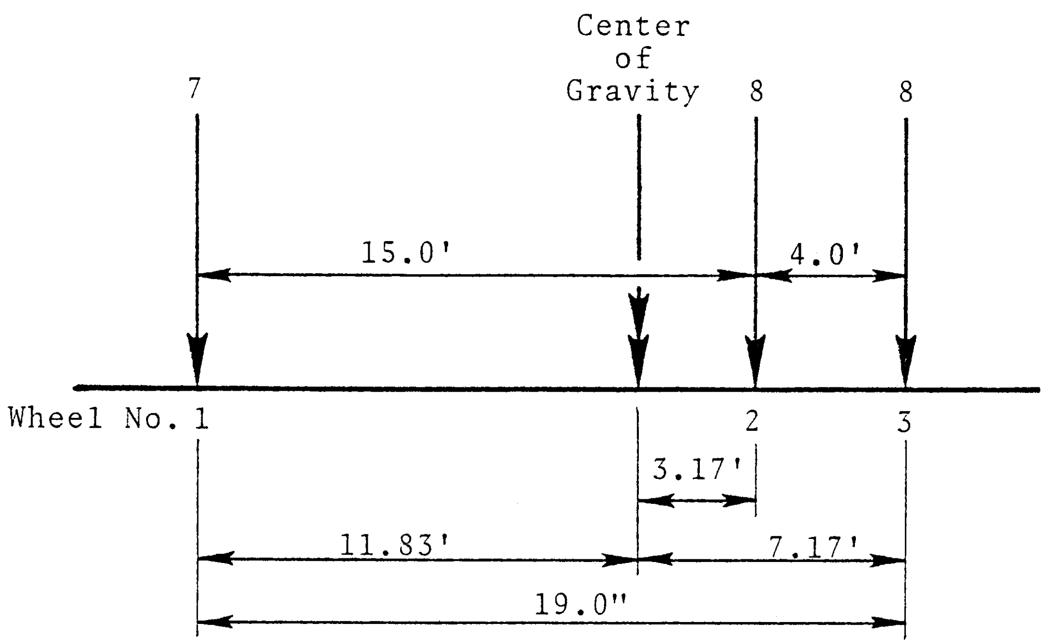


Figure 15. Axle dimensions and axle loads of the test vehicle.



NOTE:

Indicated concentrations are wheel loads in kips or axle loads in tons.

Figure 16. Axle dimensions and axle loads of the standard Type 3 vehicle.

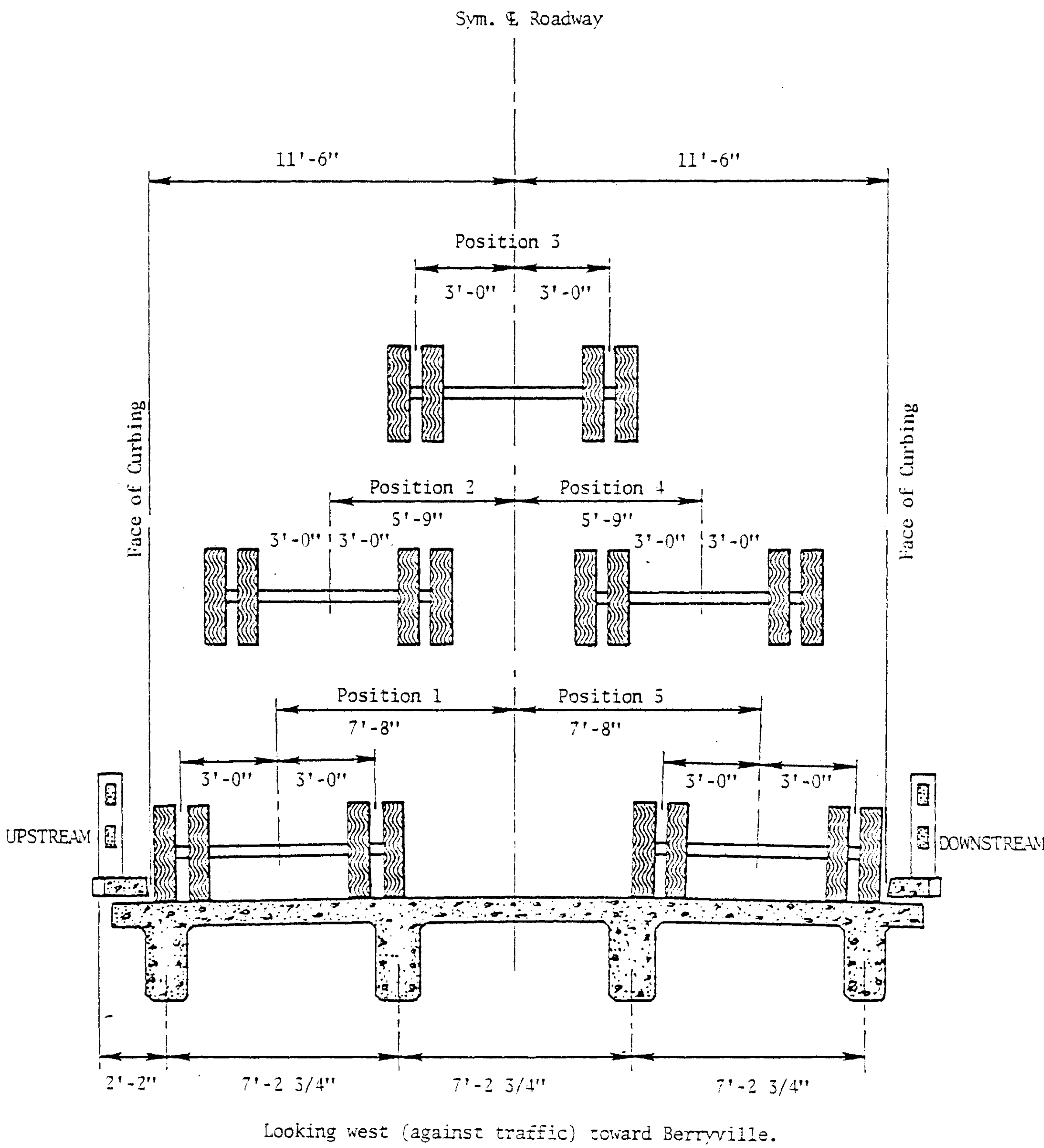


Figure 17. Lane positions.

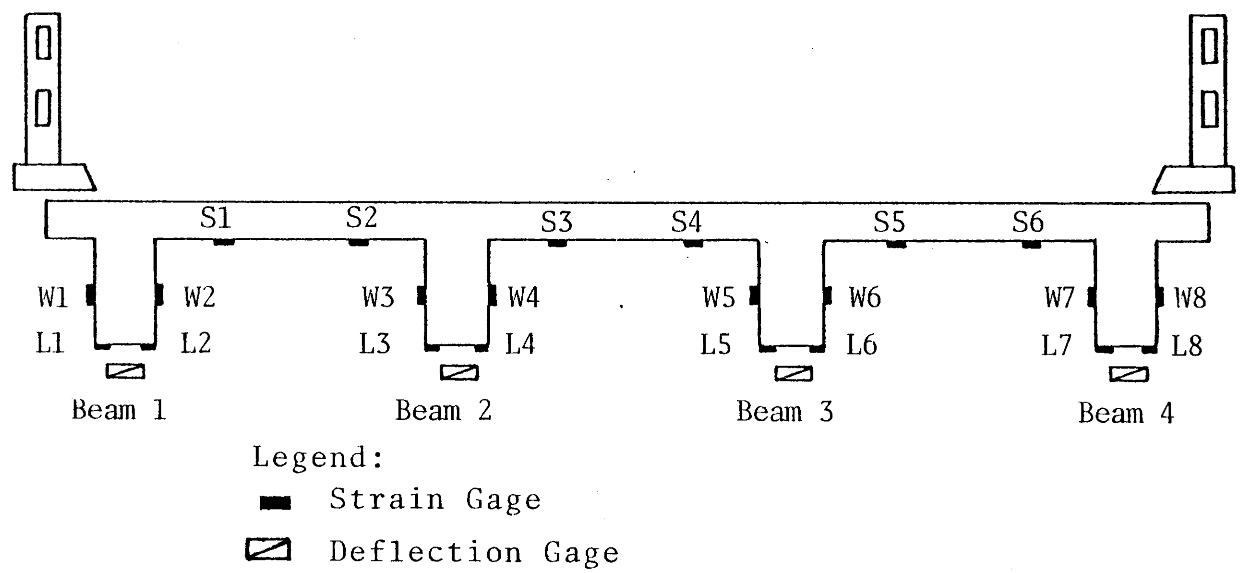


Figure 18. Location of strain and deflection gages.

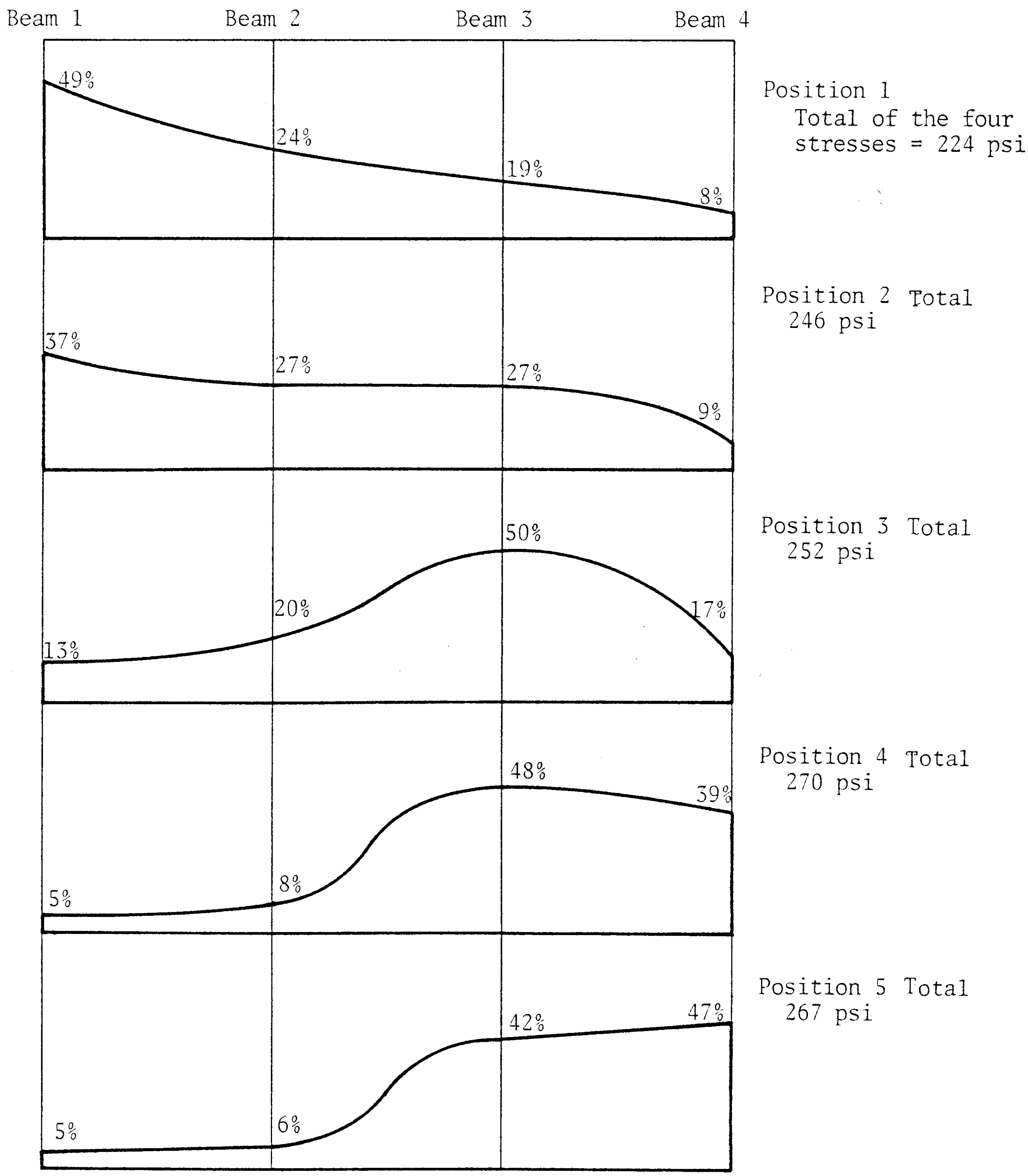


Figure 19. Percentage of live load distributed to concrete girders on basis of stresses.

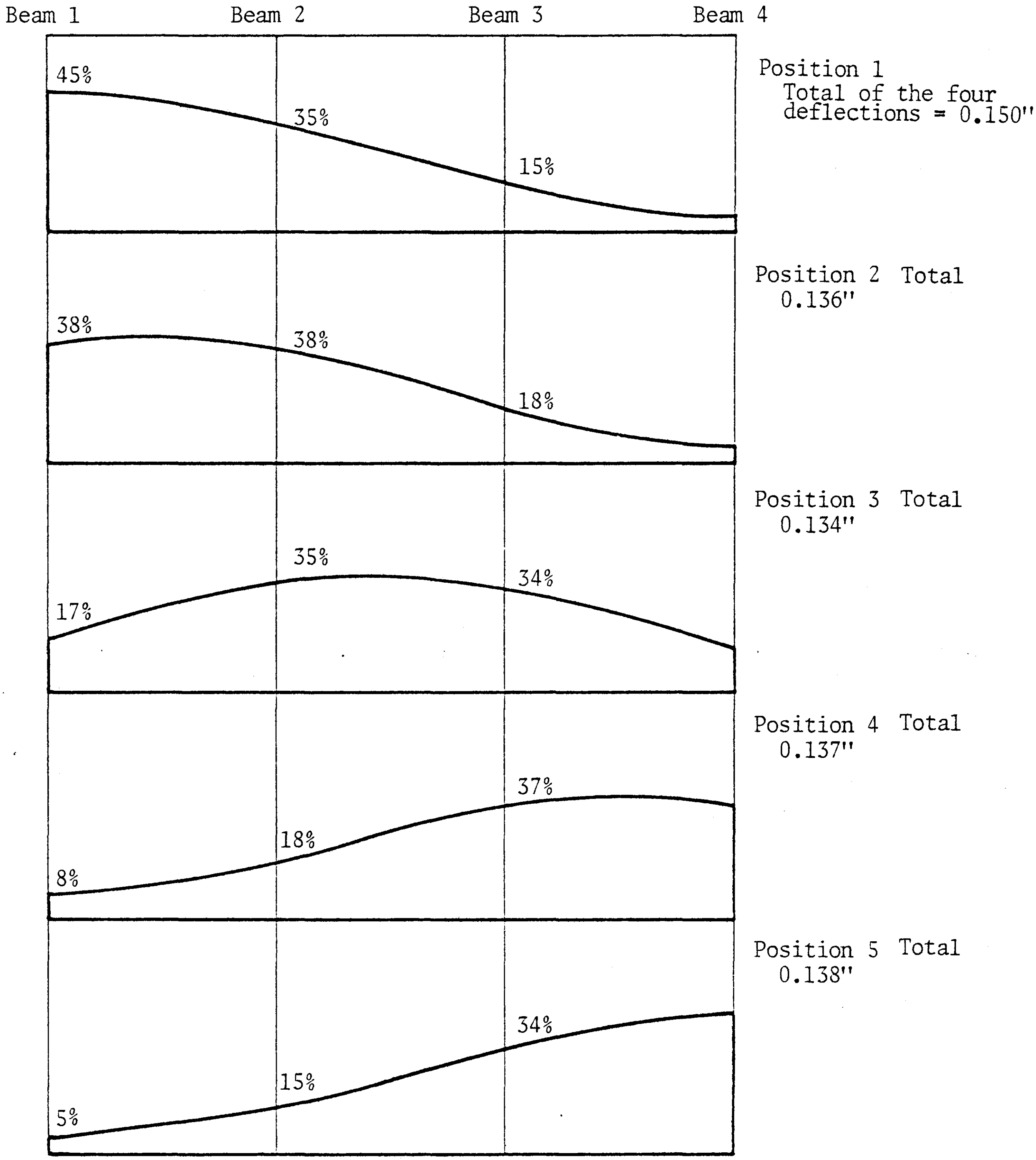


Figure 20. Percentage of live load distributed to concrete girders on basis of deflections.

TABLE 1

AVERAGE UNIT STRAINS, STRESSES AND DEFLECTIONS AT MIDSPAN

Position of Test Vehicle	Beam	Lower Surface of Stems L Gages		Sides of Stems W Gages		Deflections inches	Live Load Steel Stress at Centroid of Group from Test Load psi
		Strain μ in/in	Stress psi	Strain μ in/in	Stress psi		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	1	36	108	13	39	0.067	885
	2	18	54	26	78	0.053	450
	3	14	42	5	15	0.023	350
	4	6	18	4	12	0.007	150
2	1	30	90	9	27	0.052	740
	2	22	66	15	45	0.051	550
	3	22	66	5	15	0.025	550
	4	7	21	3	9	0.008	170
3	1	11	33	3	9	0.023	270
	2	17	51	12	36	0.047	425
	3	42	126	15	45	0.045	1045
	4	14	42	5	15	0.019	345
4	1	5	15	1	3	0.010	125
	2	7	21	7	21	0.025	175
	3	43	129	17	51	0.051	1070
	4	35	105	11	33	0.051	860
5	1	5	15	2	6	0.007	125
	2	5	15	4	12	0.021	125
	3	37	111	14	42	0.047	920
	4	42	126	14	42	0.063	1035

EXPLANATION OF COLUMNS:

Column 1—Position of test vehicle. See Figure 17.
Column 2—Identification of beam. See Figure 18.
Column 3—Average of measured strains of L gages on lower surface of stems. See Figure 18.
Column 4—Strains in column 3 multiplied by the assumed modulus of elasticity of concrete, E = 3,000,000 psi.
Column 5—Average of measured strains of W gages on sides of stems, located 12" above lower surface of stem.
Column 6—Strains in column 5 multiplied by the assumed modulus of elasticity of concrete, E = 3,000,000 psi.
Column 7—Average of measured midspan deflections, inches.
Column 8—Straight-line variation in strain was assumed and the strain at the centroid of the reinforcing steel was calculated from the theoretical position of the neutral axis as calculated on page A1 for the exterior tee-beam and on page A3 for the interior tee-beam. The stresses tabulated were calculated by multiplying these strains by the assumed modulus of elasticity of steel, E = 30,000,000 psi.

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TABLE 2

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

	Beam 1	Beam 2	Beam 3	Beam 4	TOTAL
	Position 1				
Average Strain Percentage	36 48.7%	18 24.3%	14 18.9%	6 8.1%	74 100%
	Position 2				
Average Strain Percentage	30 37.0%	22 27.2%	22 27.2%	7 8.6%	81 100%
	Position 3				
Average Strain Percentage	11 13.1%	17 20.2%	42 50.0%	14 16.7%	84 100%
	Position 4				
Average Strain Percentage	5 5.5%	7 7.8%	43 47.8%	35 38.9%	90 100%
	Position 5				
Average Strain Percentage	5 5.6%	5 5.6%	37 41.6%	42 47.2%	89 100%
	Average Total Strain in the Four Beams - 84 μ in/in				

NOTE: These values are plotted in Figure 19.

TABLE 3

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

	Beam 1	Beam 2	Beam 3	Beam 4	TOTAL
	Position 1				
Avg. Defl.	0.065	0.053	0.023	0.007	0.150
Percentage	44.7%	35.3%	15.3%	4.7%	100%
	Position 2				
Avg. Defl.	0.052	0.051	0.025	0.008	0.136
Percentage	38.2%	37.5%	18.4%	5.9%	100%
	Position 3				
Avg. Defl.	0.023	0.047	0.045	0.019	0.134
Percentage	17.2%	35.1%	33.6%	14.1%	100%
	Position 4				
Avg. Defl.	0.010	0.025	0.051	0.051	0.137
Percentage	7.3%	18.3%	37.2%	37.2%	100%
	Position 5				
Avg. Defl.	0.007	0.021	0.047	0.063	0.138
Percentage	5.1%	15.2%	34.1%	45.6%	100%
	Average Total Deflection in the Four Beams - 0.139 inch				

NOTE: These values are plotted in Figure 20.

TABLE 4

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

	Exterior Beams Positions				
	1	2	3	4	5
Beam 1	535,410	439,329	471,340	435,360	443,280
Beam 4	534,310	486,250	472,110	439,880	444,780
	Average for Exterior Beams - 470,200				
	Interior Beams Positions				
	1	2	3	4	5
Beam 2	577,740	529,110	508,520	476,870	479,310
Beam 3	577,740	529,110	509,470	475,730	481,160
	Average for Interior Beams - 514,480				

Example Calculation for 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 2

$$I = \frac{355.92 \times 12 \times 27.2\% \times 30.06}{22 \times 3} = 529{,}110 \text{ in.}^4$$

Strains were measured at bottom face of stems. y = 30.06" for interior beams, and 27.80" for exterior beams.

TABLE 5

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

	Exterior Beams Positions				
	1	2	3	4	5
Beam 1	219,360	241,540	245,890	240,020	239,550
Beam 4	220,770	242,490	244,000	239,830	237,990
	Average for Exterior Beams - 237,140 in.^{4}				
	Interior Beams Positions				
	1	2	3	4	5
Beam 2	218,990	241,760	245,550	240,680	237,990
Beam 3	218,720	242,000	245,500	239,830	238,550
	Average for Interior Beams - 236,960 in.^{4}				

TABLE 6

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

	Exterior Beam	Interior Beam
Theoretical values	83,170	105,870
Average of experimental values from strain measurements	470,200	514,480
Average of experimental values from deflection measurements	237,140	236,960

TABLE 7

MIDSPAN LOWER FLANGE STRESS
Reinforcing Steel (psi)

	Ext. Beam	Int. Beam
Dead Load Stress	8800	9600
Experimental L.L. Stress in Steel Reinforcing (Max. from Table 1) Simulated Ext. Beam 4 stress with Test Vehicle in Positions 1 and 5 Simulated Int. Beam 3 stress with Test Vehicle in Positions 2 and 4	1185	1620
Theoretical Live Load Stress	4740	5985
Impact (Exp. L.L. 30.00%)*	355	485
Impact (Theor. L.L. 30.00%)	1420	1795
Total Using Exp. L.L. Stress	10340	11705
Total Using Theor. L.L. Stress	14960	17380
Ratio $\frac{\text{Exp. L.L. Stress}}{\text{Theor. L.L. Stress}}$	0.25	0.27
Ratio $\frac{\text{Total Stress (Exp. L.L.)}}{\text{Total Stress (Theor. L.L.)}}$	0.691	0.673

*Impact factor from reference 6.

TABLE 8

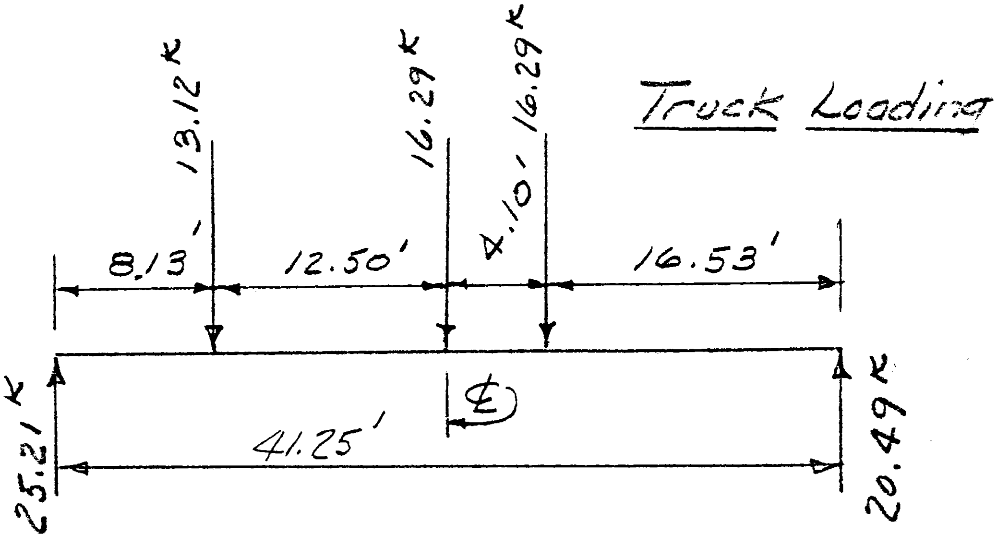
AVERAGE EXPERIMENTAL MIDSPAN LOWER FLANGE LIVE LOAD STEEL STRESSES (psi) FROM SIMULATED TWO-LANE LOADING (See Table 1)

Beam	Stress Test Vehicle in Lane 2	Stress Test Vehicle in Lane 4	Sum
1	740	125	865
2	550	175	725
3	550	1070	1620
4	170	860	1030

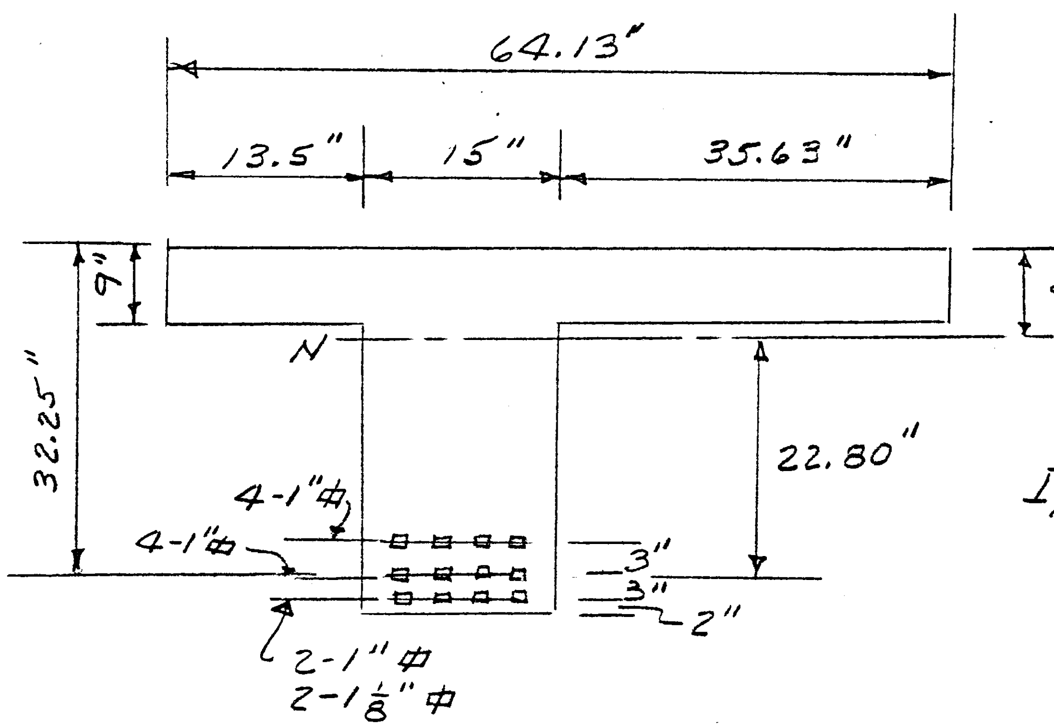
TABLE 9

AVERAGE EXPERIMENTAL MIDSPAN LIVE LOAD DEFLECTIONS (inches) FROM SIMULATED TWO-LANE LOADING (See Table 1)

Beam	Deflection Test Vehicle in Lane 2	Deflection Test Vehicle in Lane 4	Sum
1	0.052	0.010	0.062
2	0.051	0.025	0.076
3	0.025	0.051	0.076
4	0.008	0.051	0.059

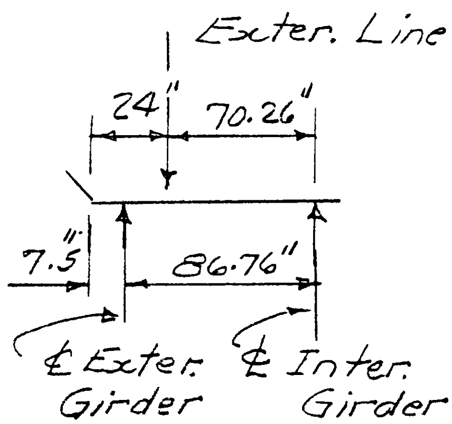


Max. Mom. at ℄ = 356 ft. kips for test vehicle.
= 178 ft. kips for line of wheels.



$I_{NA} = 83{,}170 \text{ in}^4$
$n = 10$

EXTERIOR GIRDER



Live Load Distribution to Exterior Girder $\frac{70.26}{86.76} = 0.81$

L.L. Moment to Exterior Tee-Beam =

$0.81 \times 177.96 = 144.15 \text{ fk} - 1730 \text{ iK}$

Live load flexural steel stress =

$$\frac{10 \times 1{,}730{,}000 \times 22.8}{83{,}175} = \underline{4{,}740 \text{ psi}}$$

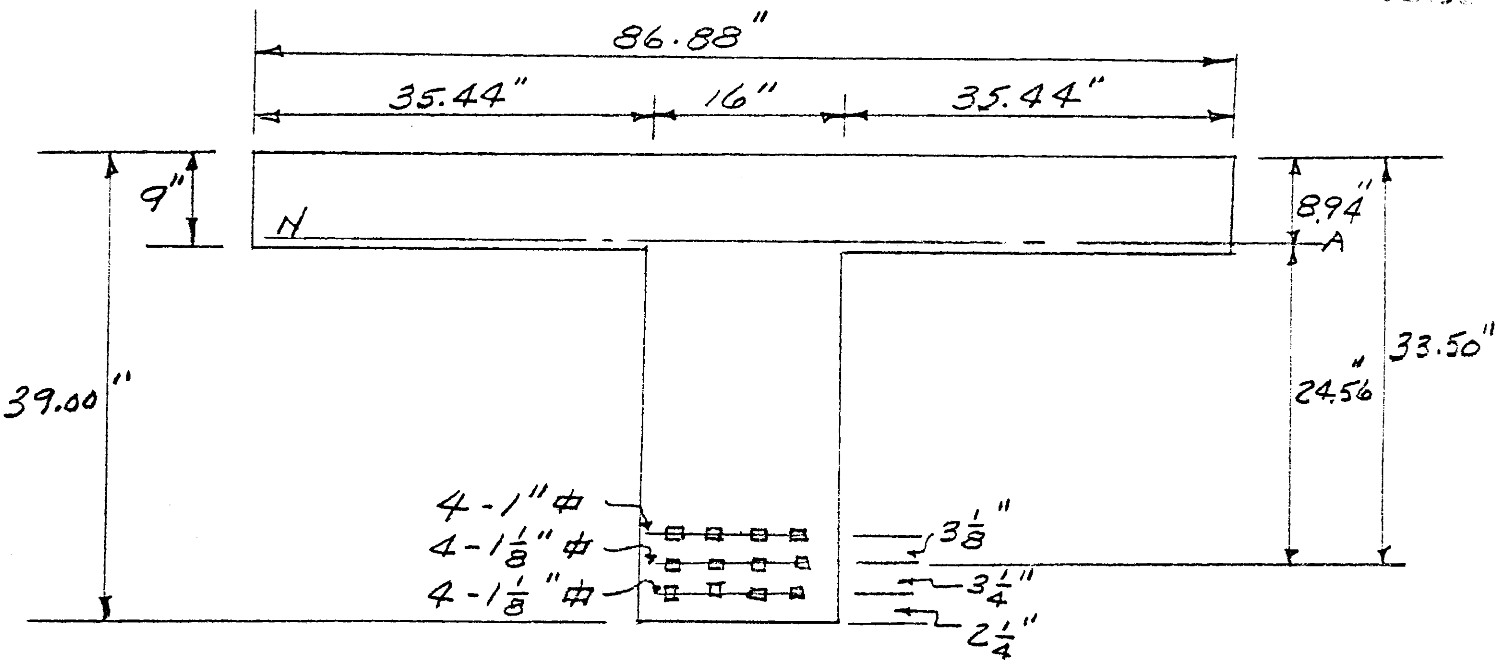
D.L. Moment to Exterior Tee-Beam

Curbs, rails, posts, brackets, etc divided equally between the four beams	130 plf.
Slab	600
Stem	440
Future Wear. Surf.	100
Uniform dead load	1270 plf

$$DL.\ Moment = \frac{wl^2}{8} = \frac{1270 \times 41.25^2}{8}$$
$$= 270 \text{ ft. Kips}$$
$$= 3240 \text{ in. Kips}$$

D.L. flexural steel stress =

$$\frac{10 \times 3{,}240{,}000 \times 22.8}{83{,}170} - \underline{8{,}800 \text{ psi}}$$



INTERIOR GIRDER

Live load moment to Interior Tee-Beam

Distribution Factor $= \frac{S}{6} = \frac{7.24}{6} = 1.21$

L.L.M. $= 177.96 \times 1.21 = 214.74$ f.k. $= 2580$ i.k.

$I_{N.A} = 105,870 \text{ in}^4$

Live load flexural stress $= \dfrac{10 \times 2,580,000 \times 24.56}{105,865}$

$= 5,985$ psi

D.L. Moment to Interior Tee-Beam

Curbs, rails, etc. divided equally between

the four beams	130 plf
Slab	815
Stem	500
Future Wear. Surf.	175
Uniform dead load	1,620 plf

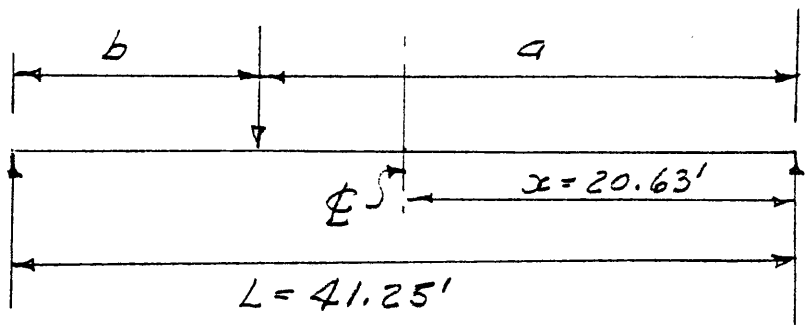
Dead Load Moment $= \dfrac{wL^2}{8} = \dfrac{1620 \times 41.25^2}{8}$

$= 345$ fk $= 4,140$ ik.

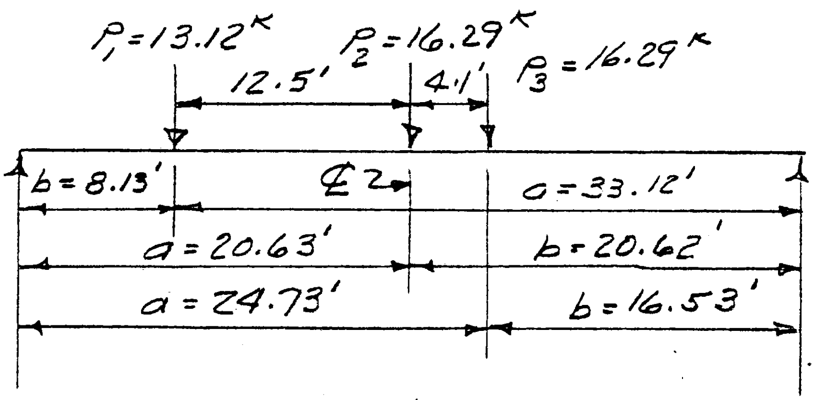
D.L. flexural steel stress $=$

$\dfrac{10 \times 4140000 \times 24.56}{105,865} = 9,600$ psi

996



$$\Delta_x = \frac{Pbx}{6EIL}\left(L^2 - b^2 - x^2\right) \quad \text{for } x < a$$



CALCULATION OF ℄ DEFLECTIONS FROM LOADS P_1, P_2 AND P_3

$$\Delta_1 = \frac{13.12 \times 8.13 \times 20.63}{6 \times 3 \times 10^3 I \; 41.25}\left(41.25^2 - 8.13^2 - 20.63^2\right)1728 = \frac{6200}{I}$$

$$\Delta_2 = \frac{PL^3}{48EI} = \frac{16.29 \times 41.25^3}{48 \times 3 \times 10^3 I} \times 1728 = \frac{13{,}720}{I}$$

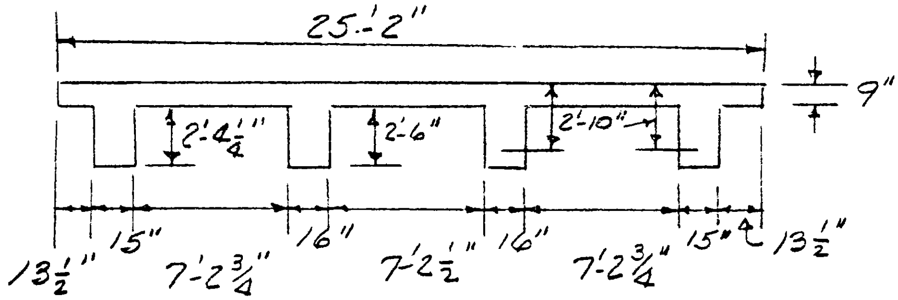
$$\Delta_3 = \frac{16.29 \times 16.53 \times 20.63}{6 \times 3 \times 10^3 I \; 41.25}\left(41.25^2 - 16.53^2 - 20.63^2\right) = \frac{12{,}960}{I}$$

$$\text{TOTAL} \quad \frac{32{,}880}{I}$$

For Beam 3 and Test Vehicle in Position 2

$$I = \frac{32880}{0.025} \times 18.4\% = 242{,}000 \text{ in.}^4$$

ULTIMATE STRENGTH OF THE FOUR TEE-BEAMS ACTING AS A UNIT



$\Sigma A_s = 53.3$ s.i.

Area of steel in exterior tee-beams 12.53 s.i.
" " " " interior " " 14.12 s.i

Calculate depth of compression block "y".

$$(25.17 \times 12)\,y \times 4500\ \text{psi} = 53.3\ \text{s.i.} \times 40{,}000\ \text{psi}$$

$$y = 1.57\ \text{in.}$$

$$\text{Lever Arm} = 39 - \frac{1.57}{2} = 38.215''$$

$$\text{Ultimate Moment} = 38.215 \times 40{,}000 \times 53.3$$
$$= 81{,}474\ \text{i.k.}$$
$$= 6{,}790\ \text{f.k.}$$

Max. Applied ℄ Moment from test vehicle = 355.92 f.k.

$$\text{Ratio} \quad \frac{\text{Ultimate Moment}}{\text{Applied Moment}} = \frac{6790}{355.92} = \underline{19.1}$$