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A LOADING STUDY OF OLDER HIGHWAY BRIDGES IN VIRGINIA

Part 1. A Steel Truss Bridge In Allegheny 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 comprehensive field test was conducted on a highway truss bridge in Allegheny County, Virginia, in July 1974. All typical truss members as well as structural members of the bridge floor system were instrumented and unit strains measured when the structure was loaded with a 3-axle tractor trailer weighting 78,000 lb. placed in incremental positions throughout its length. The purpose of the study was to determine the present capacity of this representative older type design that was used extensively in the 1920's and 1930's and is represented in many bridges which remain in use today. Although the structure was designed for two 30,000 lb. vehicles passing, it was found that the static loading of the 78,000 lb. test vehicle did not cause serious stresses.

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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 the 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 dead weight 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, an accurate appraisal of 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. Part 1 reports on a steel truss bridge tested in July 1974.

DESCRIPTION OF THE TEST STRUCTURE

The structure selected for testing was constructed in the summer of 1934 and is similar to many Virginia steel truss bridges constructed in the early 1900's and remaining in use today. This particular truss (see Figures 1 and 2) is located in Allegheny County west of Covington on a stretch of U. S. 60 which has since been relocated. This section of the highway, which is about 1/4 mile (402 m) in length, now serves only several residences; consequently, the test site proved highly satisfactory with the rather sparse traffic seldom interrupting the testing procedure.

The 150 ft. (45.7 m) steel truss span plans are available from the Bridge Office of the Virginia Department of Highways & Transportation under the designation XXXIV-5 dated May 9, 1934. The truss was constructed in accordance with the Standard Plan in use at that time (SC-24-150) with the modification of W18 x 50 and W21 x 62 replacing the I18 x 49 and I20 x 60 for exterior and interior stringers, respectively. Sketches showing truss dimensions and details of the truss members and floor system framing are shown in Figures 3, 4, and 5.

The structure was designed in accordance with the Virginia Department of Highways. Bridge Specifications, 1932, for the H-15 standard AASHTO loading.

INSTRUMENTATION

A total of twenty-eight SR-4 type A3-S6 strain gages were placed on the structure as shown on Figures 6 and 7. Nine gages were placed on typical members for each of the two trusses, i.e., an upper chord member, a lower chord member, a diagonal, a vertical, and an end post. In addition, one gage was placed on the transverse portal bracing and nine gages were placed on the floor system framing. Five gages were placed on three stringers and four gages on two floor beams.

The twenty-eight gages were wired into two ten-channel Model SB-1 Switch and Balance Units manufactured by Vishay Instruments, Inc. One of the switching units accommodated two sets of ten channels. This arrangement required changing several leads for each set of readings. A battery powered Model P-350 Portable Digital Strain Indicator was used to read the strains. See Figure 8.

TEST PROCEDURE

The test vehicle, furnished by the Federal Highway Administration and operated by their personnel, was a three-axle diesel tractor semitrailer loaded with crushed stone to simulate an HS-20 AASHTO loading or a VDH&T type 3S2 truck. A photograph of the truck and sketches giving dimensions between wheels and axles as well as wheel loadings are shown in Figures 9 and 10. As mentioned earlier, the truss was designed for an H-15 loading, much lighter than the test loading.

The deck of the test structure was marked off to designate three driving lanes; namely, an eastbound lane, a westbound lane, and a centerline lane. See Figure 11. For each of these three lanes, the test vehicle was positioned so that the drive axle (the second axle) was at each of the seven interior panel points of the truss indicated L1 through L7 in Figure 12 and also at position 1A, which was located halfway between L1 and L2.

Corresponding members of each type on each truss were instrumented. The members selected were the center panel upper chord U_3U_5, the lower chord in the panel adjacent to midspan L_3L_4, a diagonal adjacent to midspan U_3L_4, the vertical U_1L_1, and the end post L_0U_1. These instrumented members as well as those gages on the floor system framing are shown in Figures 3, 6, and 7.

For each position of the test vehicle the static live load strain was read for the twenty-eight gages on the truss members and the floor system. The total procedure was repeated one time to provide two sets of duplicate data.

Testing began on the afternoon of July 24, 1974, and was completed on the afternoon of July 25, 1974. Preparation of the steel surfaces and installation of the gages required the time of two undergraduate engineering students for the equivalent of five work days.

A 9" x 3" (.229m x .076m) coupon was burned from a lower chord bottom plate. A 2" (.0508m) length in the center portion was machined to 1.500" x 0.318" (0.381m x .00808m) (Area = 0.477 sq. in. (3.078×10^{-4} m^2)) and tested in the Council's 300,000 lb. (136,000 kg) Universal testing machine on June 16, 1975. The results were as follows: Yield stress = 37.30 ksi (257.2×10^6 Pa); ultimate stress = 59.04 ksi (407.1×10^6 Pa); modulus of elasticity = 29.16×10^3 ksi (201.1×10^9 Pa).

To convert experimental strain values to stresses, 29×10^3 ksi (200×10^9 Pa) was used throughout the study.

TEST RESULTS

The test results were all in the form of strain readings from the twenty-eight SR4-Type A3-S6 strain gages placed on typical truss members and key positions of the floor framing and are presented in the nineteen appended tables. Following the nineteen tables is a section explaining in detail the column headings for each table. The results are considered reliable and gererally satisfactory, i.e., for the most part the respective strains were reproduced when the loading positions were repeated and consistent strains were developed in the east lane truss compared to the west lane truss for the test vehicle placed in comparable lanes. Although most of the strains were in expected ranges of magnitude, some of the strain readings were obviously erratic and unusable. This is not completely unexpected with the use of instrumentation as delicate as strain gages on steel surfaces with some variation in thickness (due to corrosion) found on a forty-year old steel structure exposed to the elements. Further, there were several brief rains during both the period of installing the gages and the actual testing. Waterproofing procedures were used on the gages but 100% effectiveness cannot always be obtained.

Only those data which are considered reliable are used in the appended tables. Where data are missing in the tables, it is the result of the strain readings being discarded because of their suspected invalidity.

It may be noted in the appended tables that the stresses developed in the structural elements tested under these carefully controlled static conditions were found to be relatively low. Hurried conclusions should not be drawn, however. The reader is cautioned that practical live load capacities of bridge structures in general should be established only after carefully considering many additional factors. Other considerations include:

1) Normal impact stresses resulting from the condition of the approach pavements as well as the bridge roadway itself.

2) The degree of deterioration of both main and secondary structural elements including riveted, bolted and welded connections. Particular attention should be paid to connections with critical areas obscured by fabrication.

3) Fatigue stresses. Connections and weldments are particularly vulnerable and recent experiences in other highway systems prove the dangers that can result from fatigue failures.

4) Availability of funds and personnel for regular and effective continued maintenance.

5) State of repair of bearing assemblies with particular attention to expansion bearings.

6) Substructure condition with attention to damage resulting from frost heaval or poor drainage.

7) Potential traffic damage to structural elements lessening the live load capacity. The portal bracing and end posts are particularly susceptible to collision.

The foregoing as well as several other important items are brought out and discussed in some detail in paragraphs "Inspection Procedure" and "Inspection Items" of the AASHTO Bridge Inspection Manual. (1)

Truss Members

In Tables 1 through 5 the live load forces in the truss members are calculated from the experimental strains and comparisons are made with the live load forces as calculated from the influence diagram with the known external loading and an assumed lateral distribution to each truss based on simple beam reactions. There is good correlation between the experimentally determined forces and the theoretically determined forces except for the lower chord member L_3L_4. For this member on both trusses, for all three lateral positions of the test vehicle, the force in the member from the experimental strain reading was consistently much less than that calculated theoretically from conventional truss analysis. The experimentally determined live load force was about 65% of the theoretically determined force as can be noted from column 5 of Table 2. These lower chord experimental data seem to indicate that other elements of the floor system are working with the lower chord member to resist the tension forces developed by moment. The floor beams frame into the truss well above the lower chord members as shown in Figure 5. The lower flange of the 30" (.762m) deep floor beam is above the top flange of the 12" (.305m) deep channels making up the lower chord. It appears that portions of the floor system framing including the 7-1/2" (.191m) concrete slab act with the lower chord in the structural action to support the live load. The lower chord members will very probably not be the truss elements controlling the live load. Tensile members are generally not critical in any event.

Column 7 of Tables 1-5 adds the theoretical dead load force to the experimentally determined live load force plus impact and column 8 tabulates the corresponding values including theoretically determined live load forces. Column 9 shows the total calculated forces to be very close to those forces including the experimental live load results. The dead load forces are relatively large compared to the live load forces and since these same values are added to the live load forces computed both ways, it follows that the comparisons in column 9 could be expected to be close.

In Table 6 live load plus impact forces are simulated from superimposing results from the test vehicle being in the east lane with those results with the test vehicle being in the same relative position in the west lane. These simulated forces are compared with the forces in each member available for live load plus impact as specified by AASHTO. Column 7 shows that the applied live load forces are less than 50% of the available capacity with only a few exceptions. Column 8, entitled "rating", is simply the reciprocal of column 7 and lists a factor which could be applied to the live load and still be within the allowable capacity of the member. This column also points out the critical member of those instrumented as it is the one with the lowest rating, U_3U_5. Theoretically calculated forces show diagonal L_2U_3 to be the most critical truss member as listed in Table 18.

Table 7 is similar to Table 6 but includes the theoretically calculated dead load force in each member in addition to the experimentally determined live load forces.

One strain gage was placed on a double angle element of a knee brace on the west portal bracing. In this secondary member, the static live load stresses were low in magnitude with irregular flucuations between tension and compression for the several longitudinal positions and transverse paths of the test vehicle. The resulting stress varied irregularly between 250 psi (1.72×10^6 Pa) compression and 250 psi (1.72×10^6 Pa) tension. Secondary bracing members play a more important role in resisting dynamic loading than the static loading conditions in this experiment.

Floor Framing Members

The floor framing of this highway bridge truss is typical. It consists of a 7-1/2" (.191m) concrete slab supported on five wide flange stringers spaced 5'-6" (1.68m) on center with the stringers supported by 30" (.762m) deep wide flange floor beams that frame into the trusses at the panel points. A photograph of the floor framing from under the bridge is shown in Figure 13. Although shear connectors were not in use when this structure was designed in the early 1930's, some composite action does exist because of the bond between the concrete slab and the upper flanges of the steel beams. In the case of the floor beams, the upper flanges are fully encased in the concrete.

The concrete floor slab, the stringers and the floor beams comprise a strong, interactive complex structural system. Unit flexural and shear live load stresses would be difficult to calculate but it is clear from both theoretical considerations and the experimental results that the live load stresses are low. It is believed the critical capacity of a bridge will not be limited by stresses in the main members of this type of floor system. The degree of deterioration of the floor slab itself and the condition of the riveted or bolted connections between steel members could, however, be critical and should be closely inspected when this type bridge is rated.

In Tables 9 through 13, experimentally determined live load stresses are added to theoretically determined dead load stresses. The beams are considered simply supported and non-composite in these tables.

Table 14 superimposes live load stresses resulting from the test vehicle in the eastbound lane and the westbound lane and makes a comparison between the live load stress developed and that available. The superimposed measured live load stresses with impact are 54% or less of that available for live load.

Tables 15, 16, and 17 compare experimentally determined live load flexure stresses with theoretically determined stresses on the basis of both composite and non-composite beam sections.

Bridge Ratings

Allowable unit stresses, theoretically applied dead load and live load forces, and theoretical inventory ratings are calculated for all typical truss members and listed in Table 18. These calculations are based on a 3S2 type truck 36 tons (320 N) for inventory ratings (0.55 Fy). The 3S2 type truck is a hypothetical loading simulated by the test vehicle. This structure was designed for the standard H-15 AASHTO live loading using A-7 steel with 33,000 psi (227.5×10^6 Pa) as yield stress.

Table 19 compares the theoretical ratings of live load as limited by both truss members and floor system elements with the ratings as determined by the experimental results of those members instrumented. The critical truss member was found to be a compressive member, the second diagonal, L_2U_3, rated at 35.1 tons (312 N). The critical floor system element was determined theoretically to be the interior floor beam, which is rated at 31.0 tons (276 N).

Based on the experimental strains, the critical truss member was found to be the upper chord member, U_3U_5, rated at 50.2 tons (447 N) and the critical floor system element was again found to be the interior floor beam rated at 62.1 tons (552 N).

The theoretical ratings for the five instrumented truss members range from 63% to 91% of the experimental ratings of corresponding members. However, the theoretical ratings of the floor system elements are substantially smaller than the ratings based on the experimental strains. These relative ratings are listed in column 5 of Table 19. The theoretical capacities based on flexural stresses and assumed non-composite action are recognized as conservative. As mentioned earlier, the experimental stresses are substantially below the computed values. The considerable difference lies, perhaps, in the conservative AASHTO live load lateral distribution factor ($\frac{S}{5}$), the moment restraint offered by the end connections of stringers to the floor beam and the complex plate action of the concrete floor slab supported by five parallel stringers and the transverse floor beams. The actual bending moment applied to steel flexural members in the floor system appears to be less than that theoretically computed. Although theoretical calculations for rating this structure are included in this report (Table 19), this field test was conducted primarily to establish the correlation between theoretical stresses and experimental stresses resulting from static live loads. The actual rating of a bridge structure requires many additional examinations not within the scope of this project.

CONCLUSIONS

1. For the typical highway truss span in this study, which was constructed in 1934, the experimentally determined live load stresses developed from a standard truck loading were conservative when compared with theoretically calculated live load stresses. The experimental stresses were lower than the theoretical values for both the truss members, which were analyzed on the assumption of pin connected joints; and the floor framing members, which were analyzed on the recognized liberal assumptions of simple beam end supports and non-composite action with the concrete slab.

2. The current analytical methods and procedures for rating the live load capacities for older highway truss bridges are adequate and appropriately conservative.

3. The highway bridge engineer should be constantly aware that the theoretical rating is only one phase of the rating procedure. The appraisal of the degree of deterioration of existing bridge elements is equally important and requires a high level of mature engineering judgment.

ACKNOWLEDGEMENTS

The experimental phase of this study required the cooperation of many persons within both the Virginia Department of Highways & Transportation and the Federal Highway Administration. In particular, the authors express their appreciation to L. L. Misenheimer, Staunton District bridge engineer, and L. G. Ferris, resident engineer, of the Virginia Department of Highways & Transportation; and C. F. Galambos and Harry Laatz of the Federal Highway Administration.

The research project was conducted under the general supervision of Jack H. Dillard, Head, Virginia Highway & Transportation Research Council. W. T. McKeel, Jr., Head of the Council's Structures Section, not only actively participated in the experimental phase of the project but offered valuable suggestions both during the theoretical study and the preparation of the report. In addition, L. L. Ichter, an undergraduate civil engineering student at the University of Virginia, contributed significantly to the study in the obtaining of field data, the theoretical analysis, and the preparation of tabular data given in the results.

REFERENCES

1. Truss Bridge Inspection Instructions, Virginia Department of Highways, 1973.

2. Manual for Maintenance Inspection of Bridges, Operating Committee on Bridges and Structures, AASHO, 1970.

3. Steel Construction Manual, Fifth Edition, AISC, 1948.

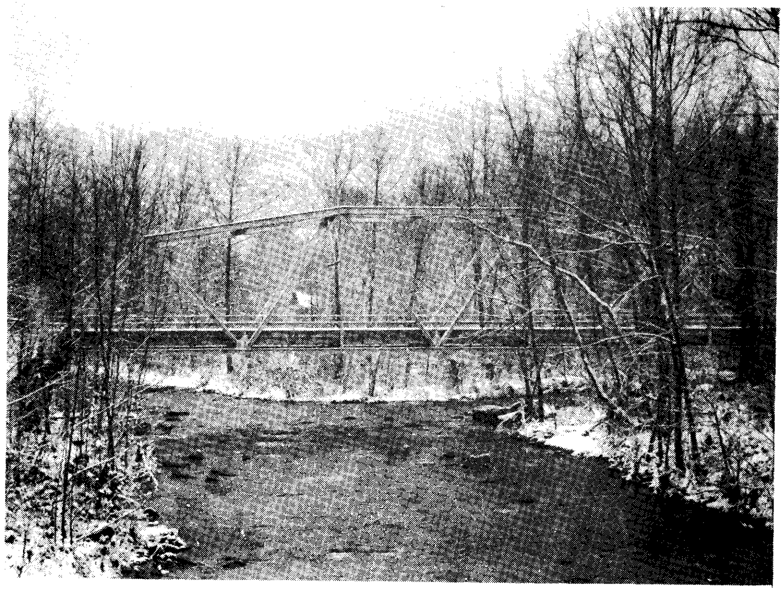

Figure 1. Test bridge.

Figure 2. End view of truss.

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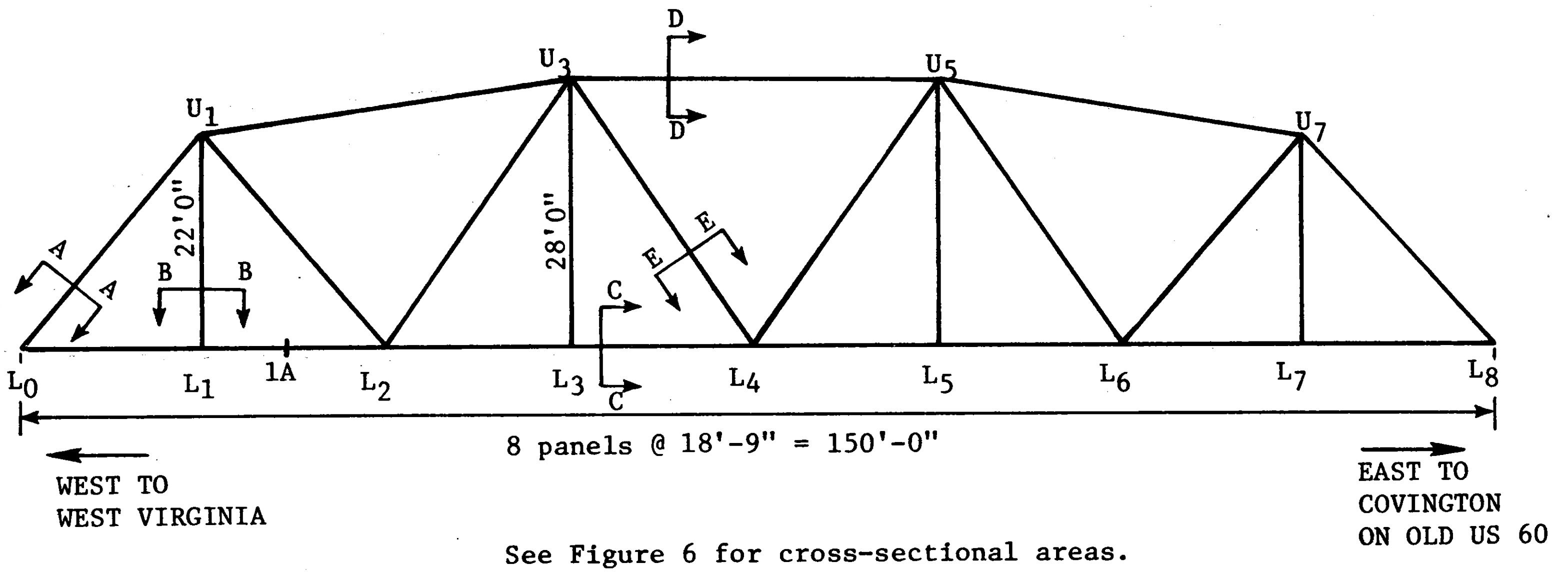


Figure 3. Elevation of upstream truss. (Near truss if standing upstream.)

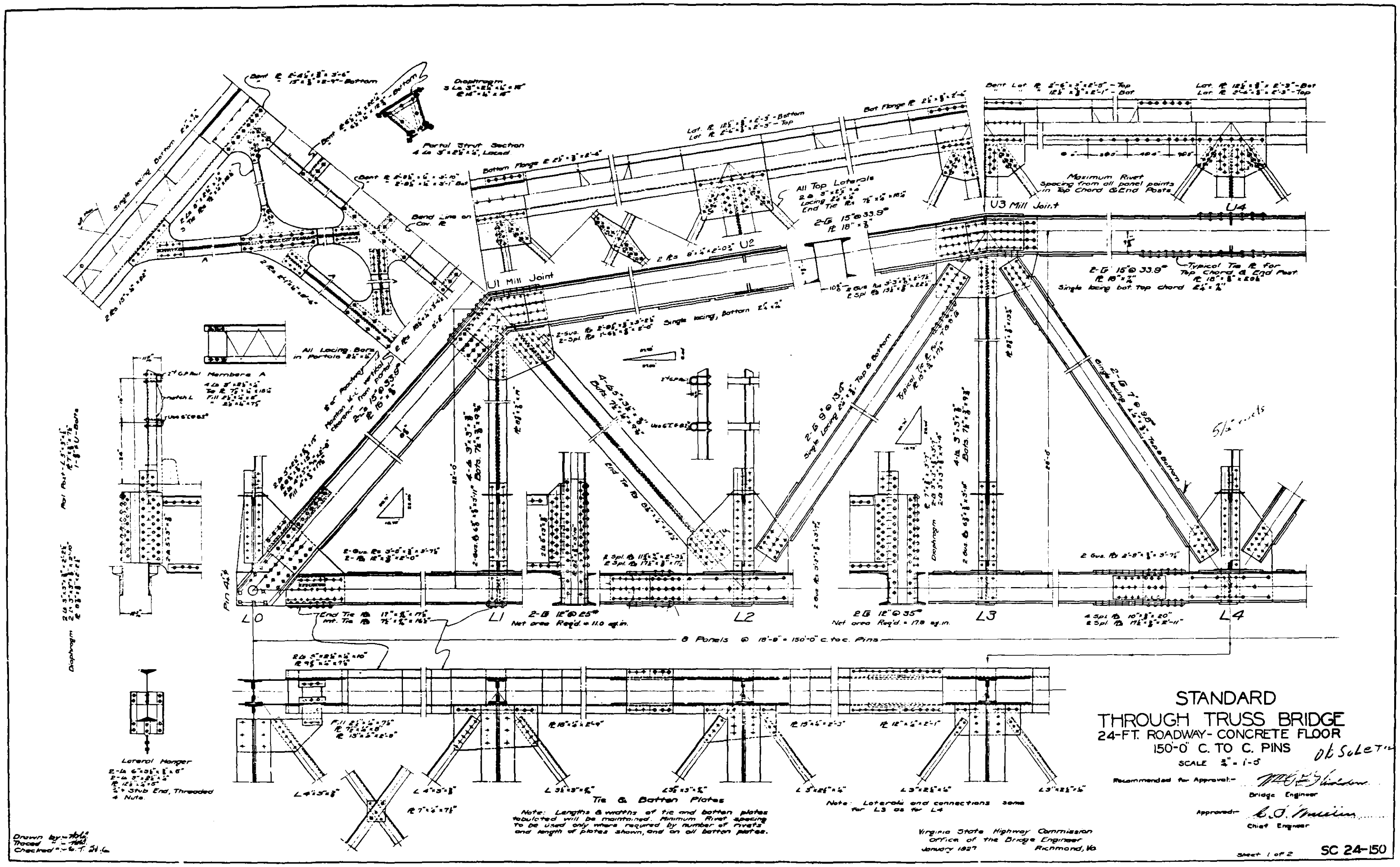


Figure 4. Elevation of truss.

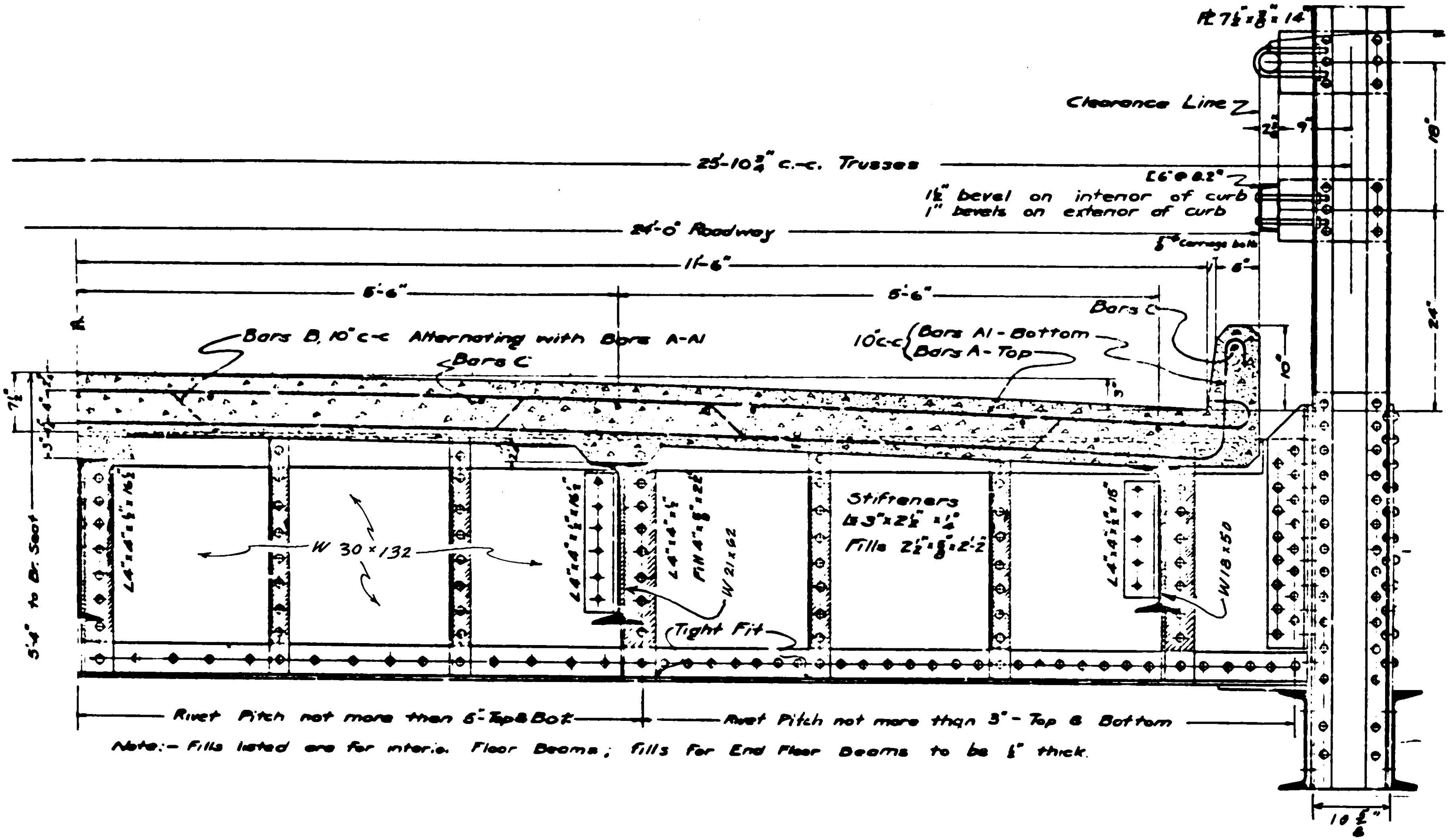
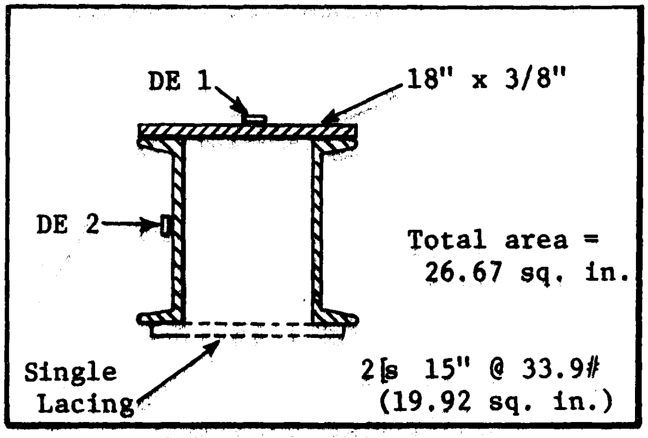
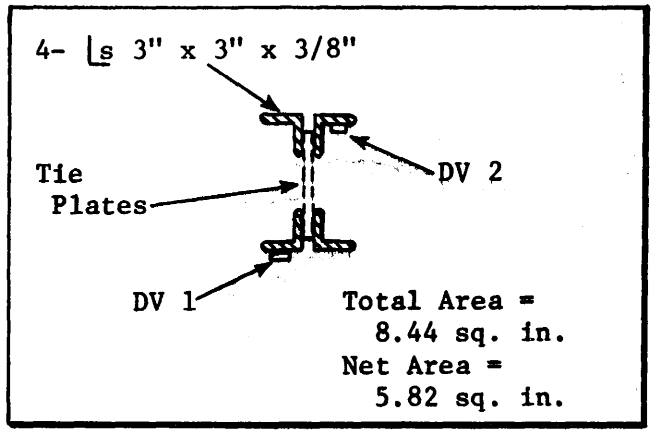


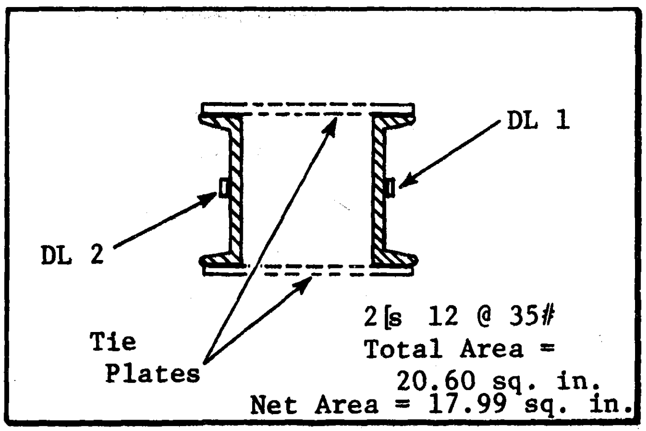
Figure 5. Half transverse section near L_3.



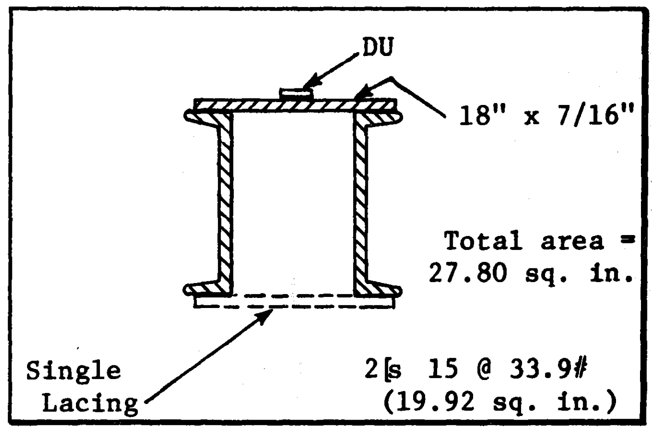
Section A-A
Endpost



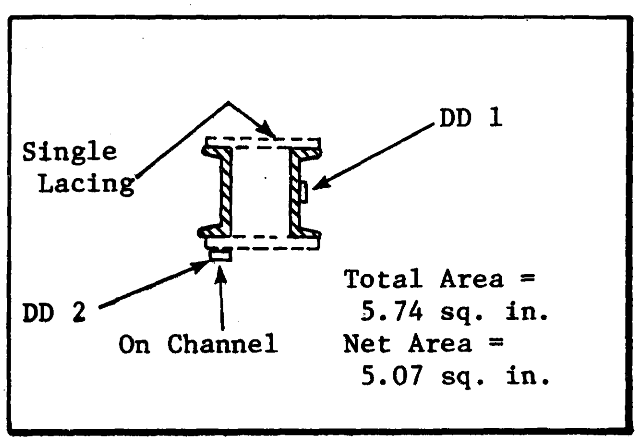
Section B-B
Vertical



Section C-C
Lower Chord



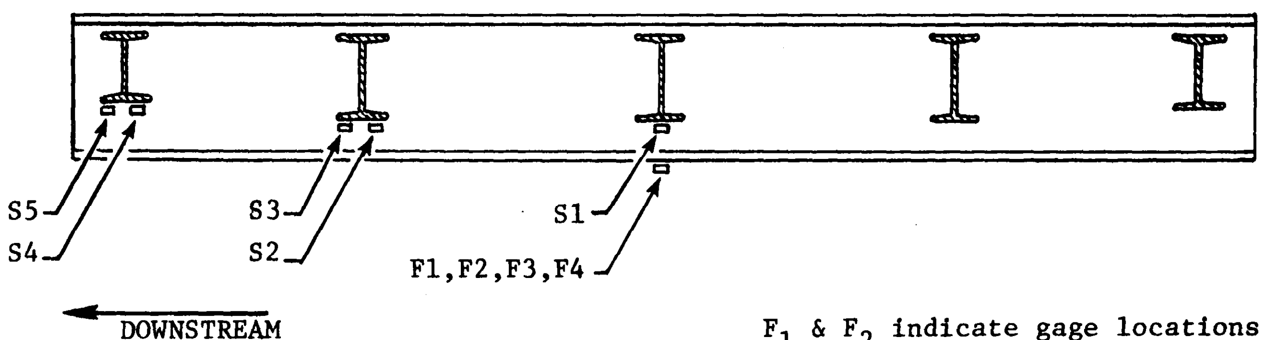
Section D-D
Upper Chord



Section E-E
Diagonal

The Fifth Edition (1948) of the AISC Steel Manual was used to determine cross-sectional areas for the standard steel shapes.
1 inch = 0.0254 m
1 sq. in. = 6.452 x 10^{-4} m^2

Figure 6. Truss sections.



F_1 & F_2 indicate gage locations on floorbeam at L_1
F_3 & F_4 indicate gate locations on floorbeam at L_2
S_2, S_3, S_4 & S_5 indicate midspan gages on stringers between L_1 & L_2
S_1 is 7" off midspan toward pier.
See Figure 3 for position of L_2

Figure 7. Floor beam at L_2.

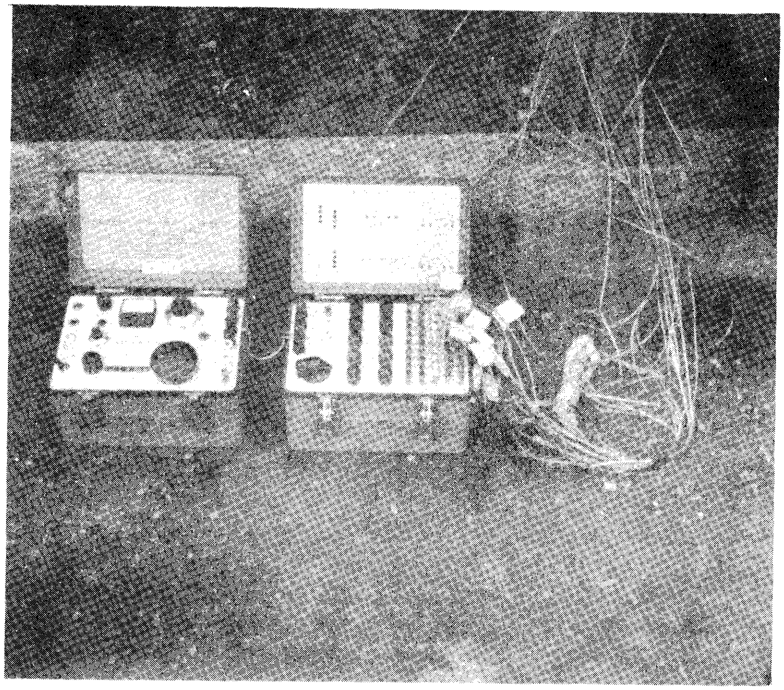
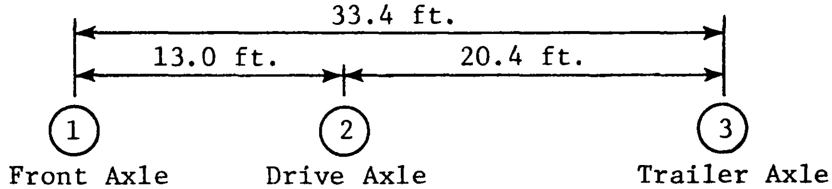


Figure 8. Strain reading equipment.

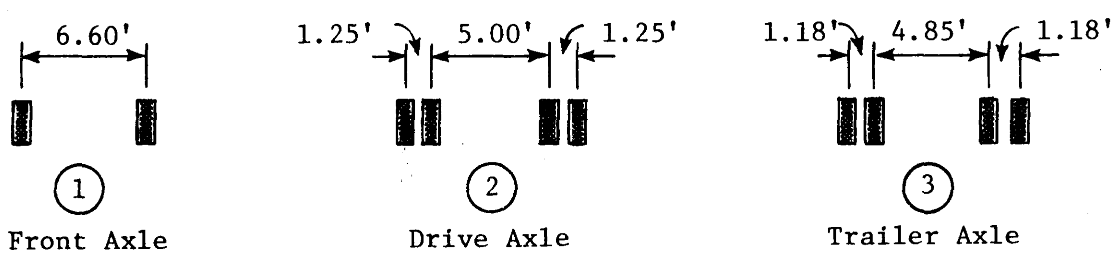
Figure 9. Test vehicle.



Axle Spacing

Axle Loading in kips
1 kip = 4.448×10^3 N

1	10.25	13%
2	34.41	44%
3	33.32	43%
	77.98	



Transverse Wheel Spacing

Figure 10. Simulated HS loading used in test.

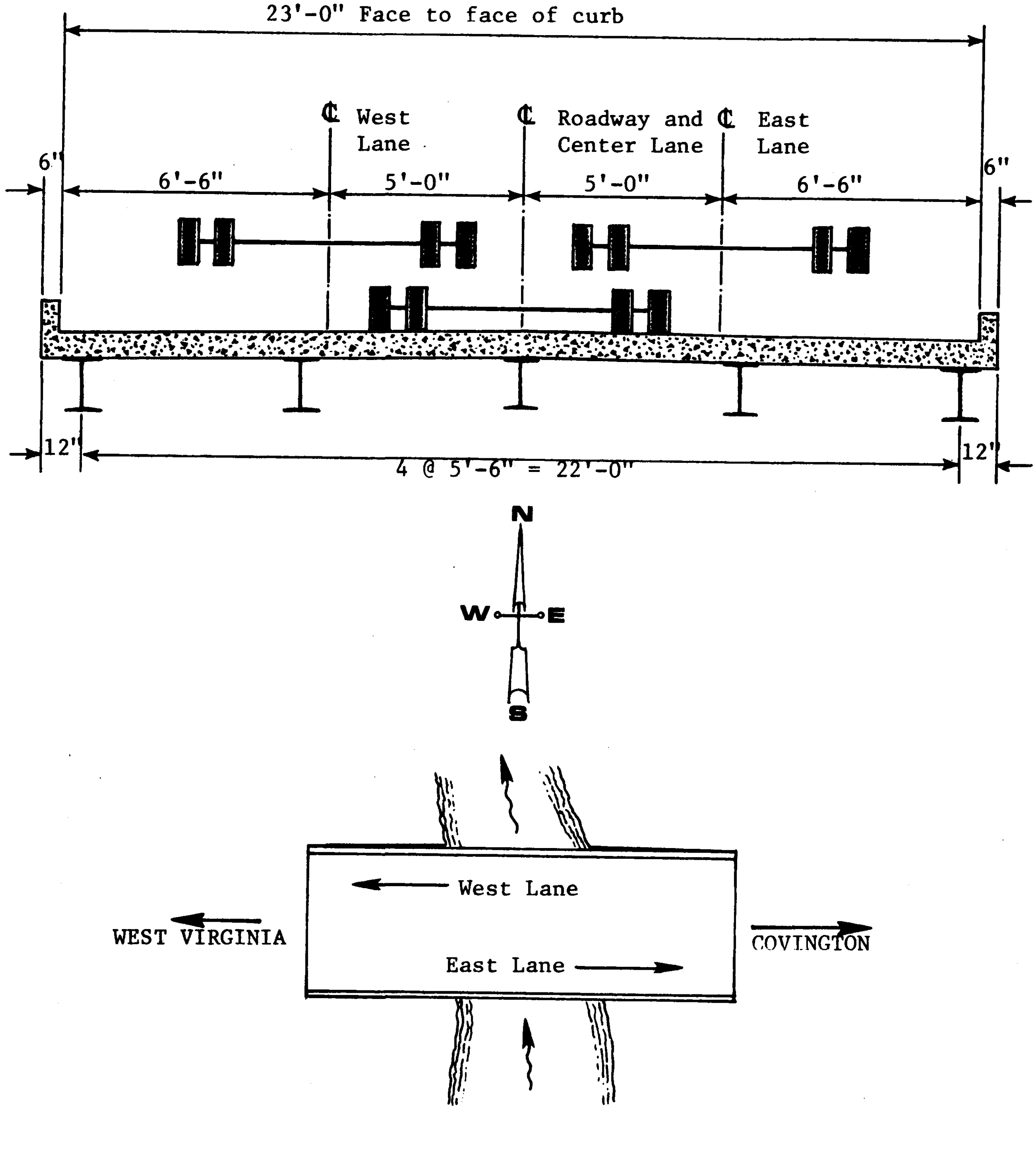
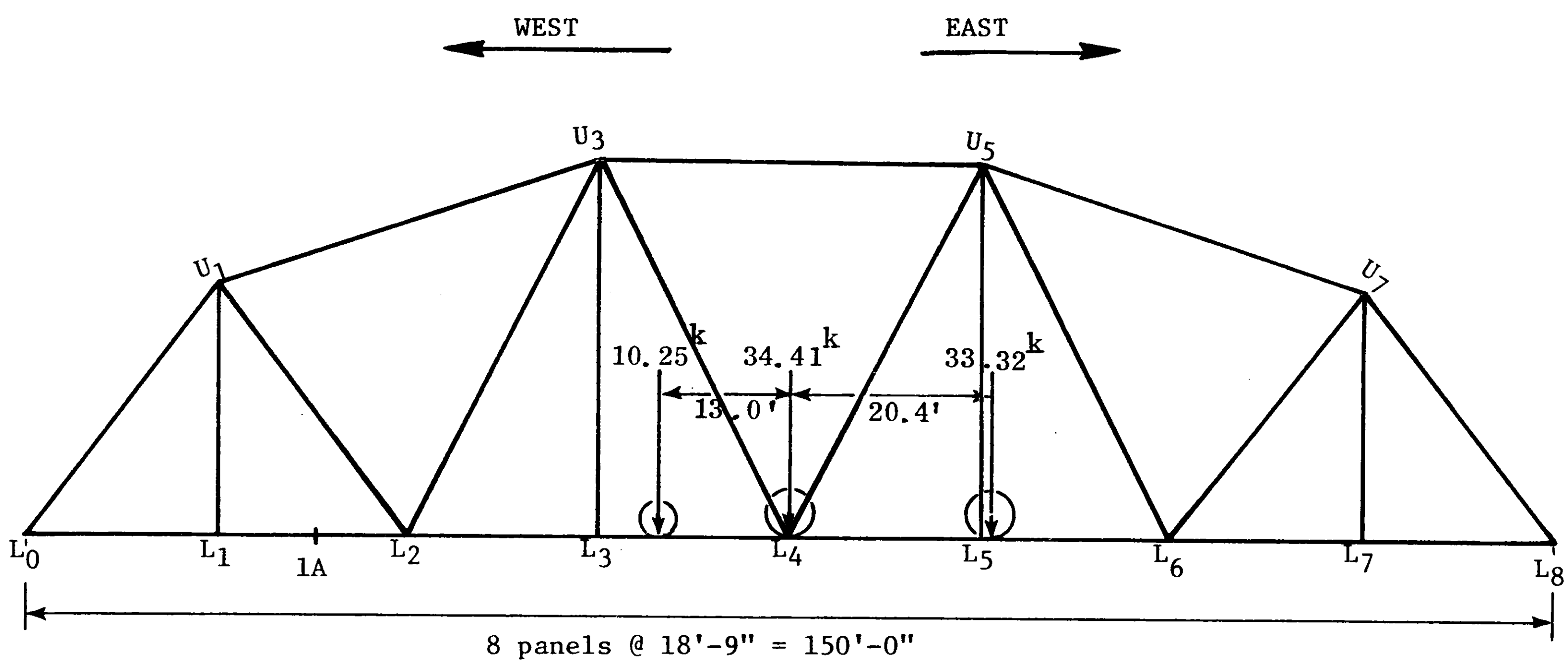


Figure 11. Lane positions and bridge orientation.



Vehicle always faced to the west as shown. Position numbers in following tables refer to the joint number where the middle (drive) axle was placed.
Illustration above shows test vehicle in position 4.

Figure 12. Vehicle positions.

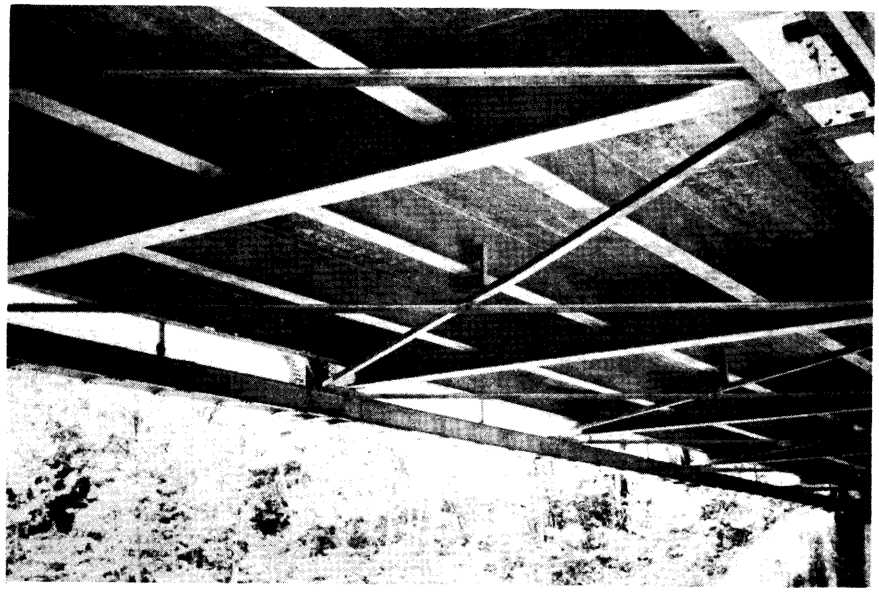


Figure 13. Underside of floor system.

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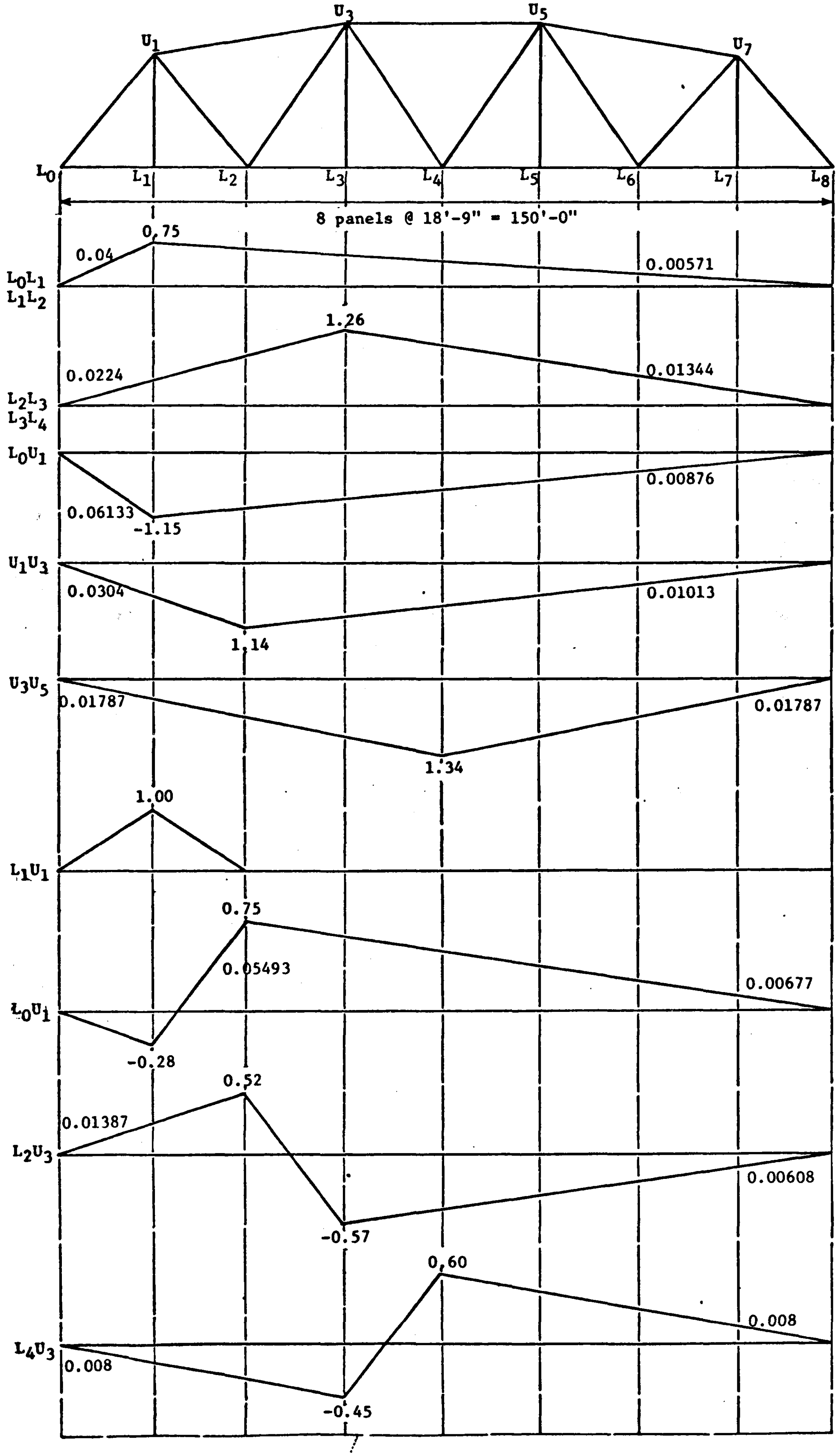

Figure 14. Influence lines.

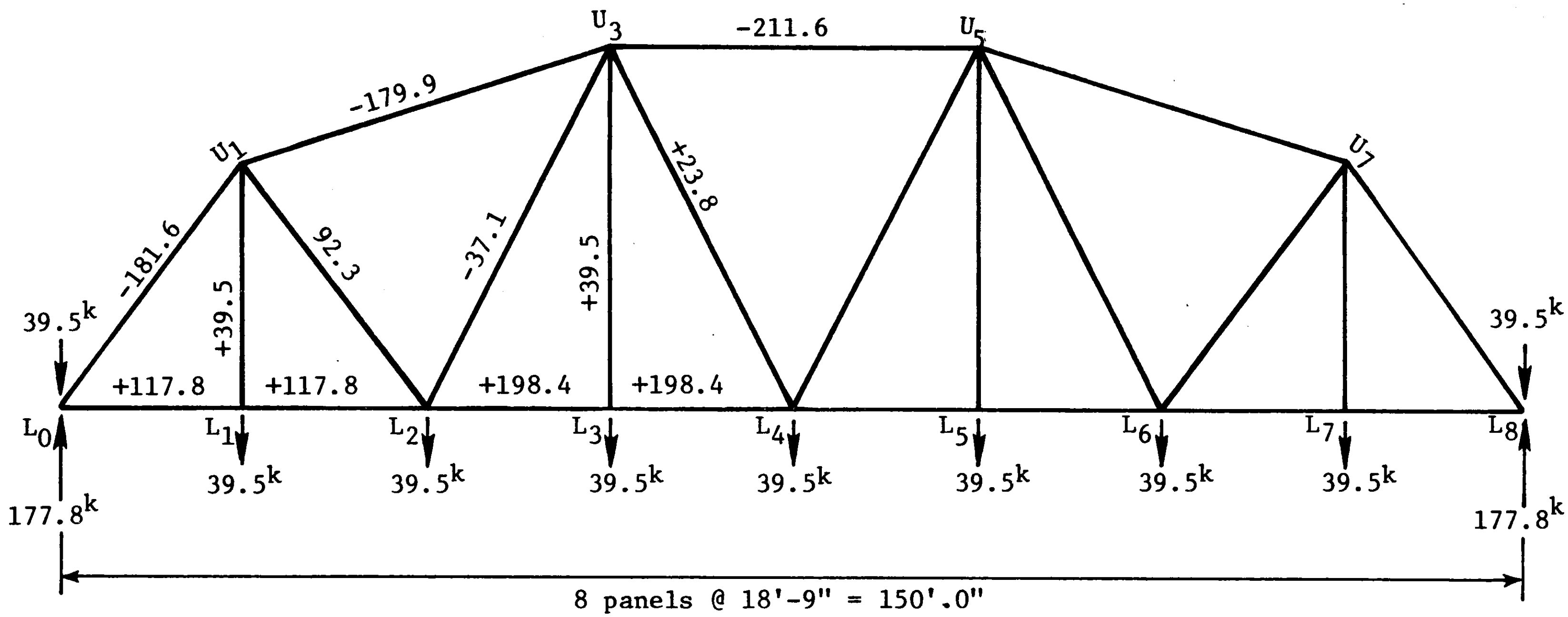

Figure 15. Dead load forces.

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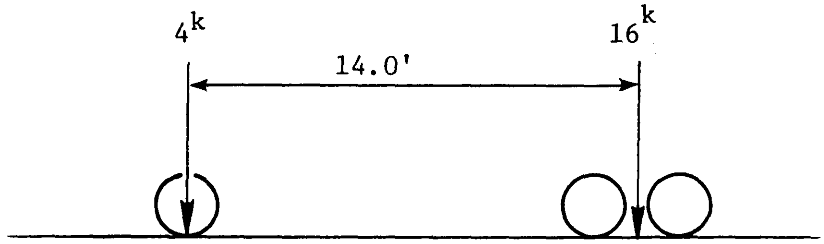
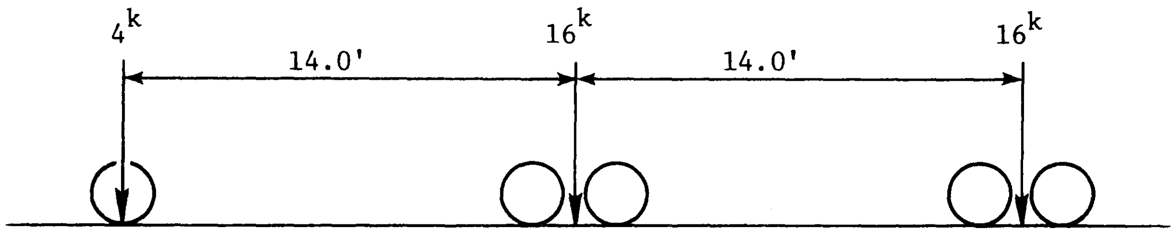
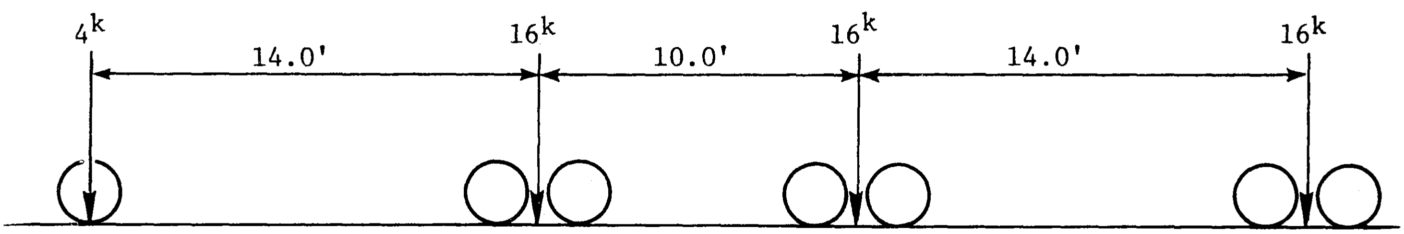


Type 3
3 Axle Single Truck
Total Weight - 20 Tons



Type 3S2
3 Axle Semi-Trailer with 2 Axle Trailer
Total Weight - 36 Tons



Type 3-4
3 Axle Truck with 4 Axle Trailer
Total Weight - 52 Tons

The three hypothetical vehicles shown above are used in Virginia as a basis for analyzing certain of the older existing bridges. The tandem axles, on 4' centers, are considered as a single concentrated load as indicated for calculations.

Forces shown are wheel loads in kips.

Figure 16. Truck loading types.

TABLE 1
UNIT STRESSES AND AXIAL LOADS IN TRUSS MEMBERS
Upper Chord U_3U_5

(1) Position of Test Vehicle	(2) Live Load Experimental Stress From Strain Gage (psi)	(3) Live Load Force From Strain Gage Stress (kips)	(4) Live Load Force From Infl. Diagram (kips)	(5) Col. 3/Col. 4 (percentage)	(6) Dead Load Force (D.L. at Panel Points (kips))	(7) Col. 3 + Impact + Col. 6 Design Force using Exper. L.L. Stresses (kips)	(8) Col. 4 + Impact + Col. 6 Design Force Using Infl. Diagram (kips)	(9) Col. 7/Col. 8 (percentage)
Upper Chord U_3U_5 (West Lane Truss) Area = 27.80 in.^{2} f_a = 13.2 ksi								
West Lane								
Pos. 5	2160	60.0	47.2	127.1	211.6	282.5	267.4	105.6
Pos. 4	1670	46.4	61.9	75.0	211.6	266.4	284.8	93.5
Pos. 3	1870	52.0	59.2	87.8	211.6	273.1	281.6	97.0
Center								
Pos. 5								
Pos. 4	1510	42.0	44.9	93.5	211.6	261.2	264.7	98.7
Pos. 3								
Upper Chord U_3U_5 (East Lane Truss)								
West Lane								
Pos. 5	725	20.2	22.4	90.2	211.6	235.5	238.1	98.9
Pos. 4	985	27.4	29.5	92.9	211.6	244.0	246.5	99.0
Pos. 3	1100	30.6	28.2	108.5	211.6	247.8	244.9	101.2
Center								
Pos. 5								
Pos. 4	1740	48.4	45.1	107.3	211.6	268.8	264.9	101.5
Pos. 3								
East Lane								
Pos. 5	1510	42.0	47.4	88.6	211.6	261.2	267.6	97.6
Pos. 4	2160	60.0	62.2	96.5	211.6	282.5	285.1	99.1
Pos. 3	2290	63.7	59.6	106.9	211.6	286.9	282.0	101.7

West Lane truss readings with test vehicle in east lane position were erratic and are omitted from this table.

TABLE 2
UNIT STRESSES AND AXIAL LOADS IN TRUSS MEMBERS
Lower Chord L_3L_4

Position of Test Vehicle (1)	Live Load Experimental Stress From Strain Gage (psi) (2)	Live Load Force From Strain Gage Stress (kips) (3)	Live Load Force From Infl. Diagram (kips) (4)	Col. 3/Col. 4 (percentage) (5)	Dead Load Force (D.L. at Panel Points (kips)) (6)	Col. 3 + Impact + Col. 6 Design Force using Exper. L.L. Stresses (kips) (7)	Col. 4 + Impact + Col. 6 Design Force Using Infl. Diagram (kips) (8)	Col. 7/Col. 8 (percentage) (9)
Lower Chord L_3L_4 (West Lane Truss)								
Area = 20.60 in.^{2} (net section 17.99 in.^{2}) f_a=18.0ks								
West Lane								
Pos. 4	1380	28.4	49.0	58.0	198.4	232.0	256.3	90.5
Pos. 3	1785	36.8	59.2	62.2	198.4	241.9	268.4	90.1
Pos. 2	1720	35.4	52.1	67.9	198.4	240.2	260.0	92.4
Center								
Pos. 4	1045	21.2	35.5	59.7	198.4	223.5	240.4	93.0
Pos. 3	1420	29.3	42.9	68.3	198.4	233.0	249.1	93.5
Pos. 2	1100	22.7	37.7	60.2	198.4	225.2	243.0	92.7
East Lane								
Pos. 4	680	14.0	22.1	63.3	198.4	214.9	224.5	95.7
Pos. 3	900	18.5	26.6	69.5	198.4	220.3	229.8	95.9
Pos. 2	855	17.6	23.4	75.2	198.4	219.2	226.1	96.9
Lower Chord L_3L_4 (East Lane Truss)								
West Lane								
Pos. 4	785	16.2	23.3	69.5	198.4	217.5	225.9	96.3
Pos. 3	1190	24.5	28.3	86.6	198.4	227.4	231.8	98.1
Pos. 2	885	18.2	24.8	73.4	198.4	219.9	227.7	96.6
Center								
Pos. 4	1095	22.6	35.7	63.3	198.4	225.1	240.6	93.6
Pos. 3	1405	28.9	43.2	69.9	198.4	232.6	249.5	93.2
Pos. 2	1180	24.3	38.0	63.9	198.4	227.1	243.3	93.3
East Lane								
Pos. 4	1385	28.5	49.2	57.9	198.4	232.1	256.5	90.5
Pos. 3	1790	36.9	59.5	62.0	198.4	242.0	271.7	89.1
Pos. 2	1550	32.0	52.4	61.1	198.4	236.2	270.6	87.3

TABLE 3
UNIT STRESSES AND AXIAL LOADS IN TRUSS MEMBERS
End Post L_oU_1

Position of Test Vehicle (1)	Live Load Experimental Stress From Strain Gage (psi) (2)	Live Load Force From Strain Gage Stress (kips) (3)	Live Load Force From Infl. Diagram (kips) (4)	Col. 3/Col. 4 (percentage) (5)	Dead Load Force (D.L. at Panel Points (kips)) (6)	Col. 3 + Impact + Col. 6 Design Force using Exper. L.L. Stresses (kips) (7)	Col. 4 + Impact + Col. 6 Design Force Using Infl. Diagram (kips) (8)	Col. 7/Col. 8 (percentage) (9)
Endpost L_0U_1 (West Lane Truss) Area = 26.67 in.^{2} F_a = 13.8 ksi								
West Lane								
Pos. 2	2040	54.4	49.6	109.7	181.6	245.9	240.2	102.4
Pos. 1A	2090	55.7	52.2	106.7	181.6	247.4	243.3	101.7
Pos. 1	1980	52.8	51.9	101.7	181.6	244.0	242.9	100.5
Center								
Pos. 2	1335	35.6	35.9	99.2	181.6	223.7	224.0	99.9
Pos. 1A	1305	34.8	37.8	92.1	181.6	222.7	226.3	98.4
Pos. 1	1240	33.1	37.6	88.0	181.6	220.7	226.0	97.7
East Lane								
Pos. 2	930	24.8	22.3	111.2	181.6	210.9	208.0	101.4
Pos. 1A	900	24.0	23.5	102.1	181.6	210.0	209.4	100.3
Pos. 1	840	22.4	23.4	95.7	181.6	208.1	209.3	99.4
Endpost L_0U_1 (East Lane Truss)								
West Lane								
Pos. 2	950	25.3	23.6	107.2	181.6	211.5	209.5	101.0
Pos. 1A	940	25.1	24.9	100.8	181.6	211.3	211.0	101.1
Pos. 1	870	23.2	24.9	93.2	181.6	209.0	211.0	99.1
Center								
Pos. 2	1355	36.1	36.1	100.0	181.6	224.3	224.3	100.0
Pos. 1A	1430	38.1	38.1	100.0	181.6	226.6	226.6	100.0
Pos. 1	1565	41.7	37.9	110.0	181.6	230.9	226.4	102.0
East Lane								
Pos. 2	1890	50.4	49.8	101.2	181.6	241.2	240.5	100.3
Pos. 1A	1960	52.2	52.4	99.6	181.6	243.3	243.5	99.9
Pos. 1	1880	50.1	52.1	96.2	181.6	240.8	243.2	99.0

TABLE 4
UNIT STRESSES AND AXIAL LOADS IN TRUSS MEMBERS
Vertical L_1U_1

(1) Position of Test Vehicle	(2) Live Load Experimental Stress From Strain Gage (psi)	(3) Live Load Force From Strain Gage Stress (kips)	(4) Live Load Force From Infl. Diagram (kips)	(5) Col. 3/Col. 4 (percentage)	(6) Dead Load Force (D.L. at Panel Points (kips))	(7) Col. 3 + Impact + Col. 6 Design Force using Exper. L.L. Stresses (kips)	(8) Col. 4 + Impact + Col. 6 Design Force Using Infl. Diagram (kips)	(9) Col. 7/Col. 8 (percentage)
Vertical L_1U_1 (West Lane Truss) Area = 8.44 in.^{2} (net section 5.82 in.^{2}) f_a=18.0 ksi								
West Lane								
Pos. 2	710	6.0	4.9	122.4	39.5	46.6	54.3	102.9
Pos. 1A	2050	17.3	17.5	98.9	39.5	59.9	60.2	99.5
Pos. 1	2790	23.6	25.8	91.5	39.5	67.4	70.0	96.3
Center								
Pos. 2	615	5.2	3.6	144.4	39.5	45.6	43.8	104.1
Pos. 1A	1470	12.4	12.7	97.6	39.5	54.2	54.5	99.4
Pos. 1	1940	16.4	18.7	87.7	39.5	58.9	61.6	95.6
East Lane								
Pos. 2	450	3.8	2.2	172.7	39.5	44.0	42.1	104.5
Pos. 1A	990	8.4	7.9	106.3	39.5	49.4	48.8	101.2
Pos. 1	1320	11.1	11.6	95.7	39.5	52.6	53.2	98.9
Vertical L_1U_1 (East Lane Truss)								
West Lane								
Pos. 2	350	3.0	2.2	136.4	39.5	43.0	42.1	102.1
Pos. 1A	855	7.2	8.3	86.7	39.5	48.0	49.3	97.4
Pos. 1	1180	10.0	12.4	80.6	39.5	51.3	54.2	94.6
Center								
Pos. 2	454	4.6	3.6	127.8	39.5	44.9	43.8	102.5
Pos. 1A	1325	11.2	12.8	87.5	39.5	52.7	54.6	96.5
Pos. 1	1915	16.2	18.8	86.2	39.5	58.6	61.7	95.0
East Lane								
Pos. 2	655	5.5	4.9	112.2	39.5	46.0	45.3	101.5
Pos. 1A	1885	15.9	17.6	90.3	39.5	58.3	60.3	96.7
Pos. 1	2620	22.1	25.9	85.3	39.5	65.6	70.1	93.6

TABLE 5
UNIT STRESSES AND AXIAL LOADS IN TRUSS MEMBERS
Diagonal L_4U_3

Position of Test Vehicle (1)	Live Load Experimental Stress From Strain Gage (psi) (2)	Live Load Force From Strain Gage Stress (kips) (3)	Live Load Force From Infl. Diagram (kips) (4)	Col. 3/Col. 4 (percentage) (5)	Dead Load Force (D.L. at Panel Points (kips)) (6)	Col. 3 + Impact + Col. 6 Design Force using Exper. L.L. Stresses (kips) (7)	Col. 4 + Impact + Col. 6 Design Force Using Infl. Diagram (kips) (8)	Col. 7/Col. 8 (percentage) (9)
Diagonal L_4U_3 (West Lane Truss)								
Area = 5.74 in.^{2} (net section 5.07 in.^{2}) f_a=18.0 ksi								
West Lane								
Pos. 4	3515	20.2	23.3	86.7	23.8	47.7	51.3	93.0
Pos. 3	530	3.1	0.3	--	23.8	27.5	24.2	113.6
Pos. 2	-2210	-12.7	-16.7	76.0	23.8	8.8	4.1	214.6
Center								
Pos. 4	2605	15.0	16.9	88.8	23.8	41.5	43.8	94.7
Pos. 3	480	2.8	0.2	--	23.8	27.1	24.0	112.9
Pos. 2	-1510	-8.7	-12.1	71.9	23.8	13.5	9.5	142.1
East Lane								
Pos. 4	1515	8.7	10.5	82.9	23.8	34.1	36.2	94.2
Pos. 3	270	1.6	0.1	--	23.8	25.7	23.9	107.5
Pos. 2	-935	-5.4	-7.5	72.0	23.8	17.4	14.9	116.8
Diagonal L_4U_3 (East Lane Truss)								
West Lane								
Pos. 4	1515	8.7	11.2	77.7	23.8	34.1	37.0	92.2
Pos. 3	175	1.0	0.1	--	23.8	25.0	23.9	104.6
Pos. 2	-970	-5.6	-7.9	70.9	23.8	17.2	14.5	118.6
Center								
Pos. 4	2550	14.6	17.0	85.9	23.8	41.1	43.9	93.6
Pos. 3	415	2.4	0.3	--	23.8	26.6	24.0	110.8
Pos. 2	-1675	-9.6	-12.2	78.7	23.8	12.5	9.4	133.0
East Lane								
Pos. 4	3465	19.9	23.4	85.0	23.8	47.3	51.5	91.8
Pos. 3	535	3.1	0.4	--	23.8	27.7	24.3	114.0
Pos. 2	-2285	-13.1	-16.8	78.0	23.8	8.3	3.9	212.8

TABLE 6
SIMULATED TWO LANE LOADING
Live Load + Impact

Member (1)	Simulated Position of Test Vehicles (2)	West Lane Live Load Force From Gages (kips) (3)	East Lane Live Load Force From Gages (kips) (4)	Live Load + Impact Force 18.2%. Simulated two lane load (kips) (5)	Available for Live Load + Impact (kips) (6)	Col. 5/Col. 6 Percentage (7)	L.L. Rating Col. 6/Col. 5 (8)
Upper Chord U_3U_5 West Truss	5 4 3	60.0 46.4 52.0	9.7 20.2 --	82.4 78.7 --	155.4 155.4 155.4	53.0 50.6	1.9 2.0 --
U_3U_5 East Truss	5 4 3	20.2 27.4 30.6	42.0 60.0 63.7	73.5 103.3 111.4	155.4 155.4 155.4	47.3 66.5 71.7	2.1 1.5 1.4
Lower Chord L_3L_4 West Truss	4 3 2	28.4 36.8 35.4	14.0 18.5 17.6	50.1 65.4 62.6	125.4 125.4 125.4	40.0 52.2 49.9	2.5 1.9 2.0
L_3L_4 East Truss	4 3 2	16.2 24.5 18.2	28.5 36.9 32.0	52.8 72.6 59.3	125.4 125.4 125.4	42.1 57.9 47.3	2.4 1.7 2.1
Endpost L_0U_1 West Truss	2 1A 1	54.4 55.7 52.8	24.8 24.0 22.4	93.6 94.2 88.9	186.4 186.4 186.4	50.2 50.5 47.7	2.0 2.0 2.1
L_0U_1 East Truss	2 1A 1	25.3 25.1 23.2	50.4 52.2 50.1	89.5 91.4 86.6	186.4 186.4 186.4	48.0 49.0 46.5	2.1 2.0 2.2
Vertical L_1U_1 West Truss	2 1A 1	6.0 17.3 23.6	3.8 8.4 11.1	11.6 30.4 41.0	65.3 65.3 65.3	17.8 46.6 62.8	5.6 2.1 1.6
L_1U_1 East Truss	2 1A 1	3.0 7.2 10.0	5.5 15.9 22.1	10.0 27.3 37.9	65.3 65.3 65.3	15.3 41.8 58.0	6.5 2.4 1.7
Diagonal L_4U_3 West Truss	4 3 2	20.2 3.1 -12.7	8.7 1.6 -5.4	34.2 5.6 -21.4	67.5 67.5	50.7 8.3	2.0 12.0
L_4U_3 East Truss	4 3 2	8.7 1.0 -5.6	19.9 3.1 -13.1	33.8 4.8 -22.1	67.5 67.5	50.1 7.1	2.0 14.1

TABLE 7
SIMULATED TWO LANE LOADING
Dead Load + Live Load + Impact

Member (1)	Simulated Position of Test Vehicles (2)	West Lane Live Load Force From Gages (kips) (3)	East Lane Live Load Force From Gages (kips) (4)	Dead Load Force D.L. at Panel Points (kips) (5)	Simulated Two Lane Loading (DL+LL+I) (kips) (6)	Allowable Design Force = Allow. Stress x Area (kips) (7)	Col. 6/Col. 7 (percentage) (8)
Upper Chord U_3U_5 West Truss	5	60.0	9.7	211.6	294.0	367.0	80.1
	4	46.4	20.2	211.6	290.3	367.0	79.1
	3	52.0	--	211.6	--	367.0	
U_3U_5 East Truss	5	20.2	42.0	211.6	285.1	367.0	77.7
	4	27.4	60.0	211.6	314.9	367.0	85.8
	3	30.6	63.7	211.6	323.0	367.0	88.0
Lower Chord L_3L_4 West Truss	4	28.4	14.0	198.4	248.5	323.8	76.7
	3	36.8	18.5	198.4	263.8	323.8	81.5
	2	35.4	17.6	198.4	261.0	323.8	80.6
L_3L_4 East Truss	4	16.2	28.5	198.4	251.2	323.8	77.6
	3	24.5	36.9	198.4	271.0	323.8	83.7
	2	18.2	32.0	198.4	257.7	323.8	79.6
Endpost L_0U_1 West Truss	2	54.4	24.8	181.6	275.2	368.0	74.8
	1A	55.7	24.0	181.6	275.8	368.0	74.9
	1	52.8	22.4	181.6	270.5	368.0	73.5
L_0U_1 East Truss	2	25.3	50.4	181.6	271.1	368.0	73.7
	1A	25.1	52.2	181.6	273.0	368.0	74.2
	1	23.2	50.1	181.6	268.2	368.0	72.9
Vertical L_1U_1 West Truss	2	6.0	3.8	39.5	51.1	104.8	48.8
	1A	17.3	8.4	39.5	69.9	104.8	66.7
	1	23.6	11.1	39.5	80.5	104.8	76.8
L_1U_1 East Truss	2	3.0	5.5	39.5	49.5	104.8	47.2
	1A	7.2	15.9	39.5	66.8	104.8	63.7
	1	10.0	22.1	39.5	77.4	104.8	73.9
Diagonal L_4U_3 West Truss	4	20.2	8.7	23.8	58.0	91.3	63.5
	3	3.1	1.6	23.8	29.4	91.3	32.2
	2	-12.7	-5.4	23.8	2.4	91.3	2.6
L_4U_3 East Truss	4	8.7	19.9	23.8	57.6	91.3	63.1
	3	1.0	3.1	23.8	28.6	91.3	31.3
	2	-5.6	-13.1	23.8	1.7	91.3	1.9

TABLE 8
MAXIMUM TRUSS FORCES FROM CENTER LINE POSITION OF TEST VEHICLE

Member	Live Load Exper. Stress From Strain Gages (psi)	Live Load Force Exper. Measured (kips)	Live Load Force From Influence Diagram (kips)	Col. 3/Col. 4 (percentage)
Drive Axle positioned at L_1 (See Figure 3)				
Endpost - U_0U_1				
(W. Truss)	1380	36.8	37.8	97.4
(E. Truss)	1390	37.1	38.1	97.4
Resultant of 2 heavy axles at L_4 (midspan) (See Figure 3)				
Upper Chord - U_3U_5				
(W. Truss)	1510	42.0	44.9	93.5
(E. Truss)	1740	48.4	45.1	107.3
Lower Chord - L_3L_4				
(W. Truss)	1100	22.7	42.9	52.9
(E. Truss)	1130	23.3	43.2	53.9

TABLE 9
FLOOR SYSTEM
FLOOR BEAM AT L_1

(1) Position of Vehicle	(2) Experimental Live Load Stress (ksi) 1 ksi = 6.895 x 10^6Pa	(3) Theoretical Dead Load Stress (ksi) 1 ksi = 6.895 x 10^6Pa	(4) Col. 2 + Impact + Col 3 Total Stress (ksi) 1 ksi = 6.895 x 10^6Pa	(5) Allowable Flexural Stress (ksi) 1 ksi = 6.895 x 10^6Pa	(6) Col. 4/Col. 5 (percentage)
West Lane Pos. 2	0.55	6.35	7.07	18.0	39.3
Pos. 1A	1.68	6.35	8.53	18.0	47.4
Pos. 1	2.45	6.35	9.55	18.0	53.1
Center Pos. 2	0.72	6.35	7.29	18.0	40.5
Pos. 1A	2.12	6.35	9.11	18.0	50.6
Pos. 1	3.34	6.35	10.69	18.0	59.4
East Lane Pos. 2	0.55	6.35	7.07	18.0	39.3
Pos. 1A	1.71	6.35	8.57	18.0	47.6
Pos. 1	2.46	6.35	9.55	18.0	53.1

TABLE 10
FLOOR SYSTEM
FLOOR BEAM AT L_2

Position of Vehicle (1)	Experimental Live Load Stress (ksi) 1 ksi = 6.895×10^6Pa (2)	Theoretical Dead Load Stress (ksi) 1 ksi = 6.895×10^6Pa (3)	Col. 2 + Impact + Col 3 Total Stress (ksi) 1 ksi = 6.895×10^6Pa (4)	Allowable Flexural Stress (ksi) 1 ksi = 6.895×10^6Pa (5)	Col. 4/Col. 5 (percentage) (6)
West Lane					
Pos. 3	0.67	6.35	7.22	18.0	40.1
Pos. 2	2.62	6.35	9.76	18.0	54.2
Pos. 1A	2.37	6.35	9.43	18.0	52.4
Pos. 1	2.33	6.35	9.38	18.0	52.1
Center					
Pos. 3	0.78	6.35	7.36	18.0	40.9
Pos. 2	3.34	6.35	10.69	18.0	59.4
Pos. 1A	2.95	6.35	10.19	18.0	56.6
Pos. 1	3.15	6.35	10.45	18.0	58.1
East Lane					
Pos. 3	0.69	6.35	7.25	18.0	40.3
Pos. 2	2.57	6.35	9.69	18.0	53.8
Pos. 1A	2.33	6.35	9.38	18.0	52.1
Pos. 1	2.31	6.35	9.35	18.0	51.9

TABLE 11
FLOOR SYSTEM
EXTERIOR STRINGER

Position of Vehicle (1)	Experimental Live Load Stress (ksi) 1 ksi = 6.895 x 10^6Pa (2)	Theoretical Dead Load Stress (ksi) 1 ksi = 6.895 x 10^6Pa (3)	Col. 2 + Impact + Col 3 Total Stress (ksi) 1 ksi = 6.895 x 10^6Pa (4)	Allowable Flexural Stress (ksi) 1 ksi = 6.895 x 10^6Pa (5)	Col. 4/Col. 5 (percentage) (6)
West Lane Pos. 2	0.71	2.98	3.90	18.0	21.7
Pos. 1A	2.23	2.98	5.88	18.0	32.7
Pos. 1	0.71	2.98	3.90	18.0	21.7
Center Pos. 2	0.36	2.98	3.45	18.0	19.2
Pos. 1A	0.78	2.98	3.99	18.0	22.2
Pos. 1	0.62	2.98	3.79	18.0	21.1
East Lane Pos. 2	0.18	2.98	3.21	18.0	17.8
Pos. 1A	0.41	2.98	3.51	18.0	19.5
Pos. 1	0.45	2.98	3.57	18.0	19.8

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TABLE 12
FLOOR SYSTEM
INTERIOR STRINGER

Position of Vehicle (1)	Experimental Live Load Stress (ksi) 1 ksi = 6.895 x 10^6Pa (2)	Theoretical Dead Load Stress (ksi) 1 ksi = 6.895 x 10^6Pa (3)	Col. 2 + Impact + Col 3 Total Stress (ksi) 1 ksi = 6.895 x 10^6Pa (4)	Allowable Flexural Stress (ksi) 1 ksi = 6.895 x 10^6Pa (5)	Col. 4/Col. 5 (percentage) (6)
West Lane					
Pos. 2	0.55	2.72	3.44	18.0	19.1
Pos. 1A	2.52	2.72	6.00	18.0	33.3
Pos. 1	0.11	2.72	2.86	18.0	15.9
Center					
Pos. 2	0.50	2.72	3.37	18.0	18.7
Pos. 1A	1.88	2.72	5.16	18.0	28.7
Pos. 1	0.18	2.72	2.97	18.0	16.5
East Lane					
Pos. 2	0.28	2.72	3.08	18.0	17.1
Pos. 1A	0.72	2.72	3.66	18.0	20.3
Pos. 1	0.21	2.72	2.99	18.0	16.6

TABLE 13
FLOOR SYSTEM
MIDDLE STRINGER

Position of Vehicle (1)	Experimental Live Load Stress (ksi) 1 ksi = 6.895 x 10^6Pa (2)	Theoretical Dead Load Stress (ksi) 1 ksi = 6.895 x 10^6Pa (3)	Col. 2 + Impact + Col 3 Total Stress (ksi) 1 ksi = 6.895 x 10^6Pa (4)	Allowable Flexural Stress (ksi) 1 ksi = 6.895 x 10^6Pa (5)	Col. 4/Col. 5 (percentage) (6)
West Lane					
Pos. 2	0.36	2.72	3.19	18.0	17.7
Pos. 1A	2.05	2.72	5.39	18.0	29.9
Pos. 1	0.29	2.72	3.10	18.0	17.2
Center					
Pos. 2	0.58	2.72	3.47	18.0	19.3
Pos. 1A	2.83	2.72	6.40	18.0	35.6
Pos. 1	0.38	2.72	3.21	18.0	17.8
East Lane					
Pos. 2	0.41	2.72	3.25	18.0	18.1
Pos. 1A	1.94	2.72	5.24	18.0	29.1
Pos. 1	0.35	2.72	3.18	18.0	17.7

TABLE 14
SIMULATED TWO LANE LOADING ON FLOOR SYSTEM

Member (1)	Position of Vehicle (2)	West Lane Live Load Stress from Gages (ksi) (3)	East Lane Live Load Stress from Gages (ksi) (4)	Simulated Two Lane Loading with 30% Impact (ksi) (5)	Available for Live Load + Impact (ksi) (6)	Col. 5/Col. 6 (percentage) (7)	Rating Col. 6/Col. 5 (# Vehicles) (8)
Floor Beam L_2	2	2.62	2.57	6.75	11.65	57.9	1.73
	1A	2.37	2.33	6.11	11.65	52.4	1.91
	1	2.33	2.31	6.03	11.65	51.8	1.93
Floor Beam L_1	1A	1.68	1.71	4.41	11.65	37.9	2.64
	1	2.46	2.46	6.40	11.65	54.9	1.82
Middle Stringer	1A	2.05	1.94	5.19	15.28	34.0	2.94
Interior Stringer	1A	2.52	0.72	4.21	15.28	27.6	3.63
Exterior Stringer	1A	2.23	0.41	3.43	15.02	22.8	4.38

TABLE 15
LIVE LOAD STRESSES IN STRINGERS
Middle Stringer

(1) Position of Vehicle	(2) Experimental Live Load Stress (ksi)	(3) Theoretical Live Load Stress non-composite (ksi)	(4) Col. 2/Col. 3 (percentage)	(5) Theoretical Live Load Stress Composite Section (ksi)	(6) Col 2./Col. 5 (percentage)
West Lane Pos. 2	0.36	1.40	25.7	0.87	41.4
Pos. 1A	2.05	7.66	26.8	4.75	43.2
Pos. 1	0.29	0.00	--	0.00	--
Center Pos. 2	0.58	1.40	41.4	0.87	66.7
Pos. 1A	2.83	7.66	36.9	4.75	59.6
Pos. 1	0.38	0.00	--	0.00	--
East Lane Pos. 2	0.41	1.40	29.3	0.87	47.1
Pos. 1A	1.94	7.66	25.3	4.75	40.8
Pos. 1	0.35	0.00	--	0.00	--

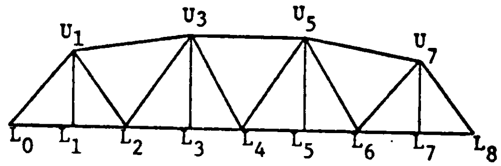
TABLE 16
LIVE LOAD STRESSES IN STRINGERS
Interior Stringer

Position of Vehicle (1)	Experimental Live Load Stress (ksi) (2)	Theoretical Live Load Stress non-composite (ksi) (3)	Col. 2/Col. 3 (percentage) (4)	Theoretical Live Load Stress Composite Section (ksi) (5)	Col 2./Col. 5 (percentage) (6)
West Lane					
Pos. 2	0.55	1.40	39.3	0.87	63.2
Pos. 1A	2.52	7.66	32.9	4.75	53.1
Pos. 1	0.11	0.00	--	0.00	--
Center					
Pos. 2	0.50	1.40	35.7	0.87	57.5
Pos. 1A	1.88	7.66	24.5	4.75	39.6
Pos. 1	0.18	0.00	--	0.00	--
East Lane					
Pos. 2	0.28	1.40	20.0	0.87	32.2
Pos. 1A	0.72	7.66	9.4	4.75	15.2
Pos. 1	0.21	0.00	--	0.00	--

TABLE 17
LIVE LOAD STRESSES IN STRINGERS
Exterior Stringer

Position of Vehicle (1)	Experimental Live Load Stress (ksi) (2)	Theoretical Live Load Stress non-composite (ksi) (3)	Col. 2/Col. 3 (percentage) (4)	Theoretical Live Load Stress Composite Section (ksi) (5)	Col 2./Col. 5 (percentage) (6)
West Lane					
Pos. 2	0.71	1.99	35.7	0.97	73.2
Pos. 1A	2.23	10.87	20.5	5.29	42.2
Pos. 1	0.71	0.00	--	0.00	--
Center					
Pos. 2	0.36	1.99	18.1	0.97	37.1
Pos. 1A	0.78	10.87	7.2	5.29	14.7
Pos. 1	0.62	0.00	--	0.00	--
East Lane					
Pos. 2	0.18	1.99	9.0	0.97	18.6
Pos. 1A	0.41	10.87	3.8	5.29	7.8
Pos. 1	0.45	0.00	--	0.00	--

TABLE 18
TRUSS STRESS COMPUTATION SUMMARY
RTE. 710, ALLEGHENY COUNTY OVER DUNLAP CREEK



(1) Member	(2) L.L. Computations(Influence Line Ordinates)x(wheel loads)x (Lateral Distr. Factor 2.39)x(Impact Factor,1.18)	(3) Net Area or No. of Rivets	(4) Allow. Axial Stress or Allow. Shear in Rivet	(5) Total Allow. Load	(6) Dead Load P=39.5k	(7) Avail. for LL+I	(8) LL+I Type 3S2	(9) Total DL+LL+I Type 3S2	(10) Total Stress Type 3S2	(11) Under Stress	(12) Inventory Rating
		$in.^2$	ksi kips	kips	kips	kips	kips	kips	ksi	ksi	tons
L_0L_2	[16(.75x.75-14x.00571)+4(.75-28x.00571)]2.82=+70.7	12.67 42	+18.0 5.96	+228.1 250.3	+117.8	+110.3	+70.7	+188.5	+14.9	-3.1	56.2
L_2L_4	[16(1.26+1.26-14x.01344)+4(1.26-14x.0224)]2.82=+115.9	17.99 60	+18.0 5.96	+323.8 357.6	+198.4	+125.4	+115.9	+314.3	+17.5	-0.5	39.0
L_0U_1	[16(1.15+1.15-14x.00876)+4(1.15-28x.00876)]2.82=-108.5	26.67 66	(a) -13.8 5.96	-368.0 393.4	-181.6	-186.4	-108.5	-290.1	-10.9	-2.9	61.8
U_1U_3	[16(1.14+1.14-14x.01013)+4(1.14-28x.01013)]2.82=-106.1	26.67	(b) -13.2	-352.0	-179.9	-172.1	-106.1	-286.0	-10.7	-2.5	58.4
U_3U_5	[16(1.34+1.34-14x.01787)+4(1.34-14x.01787)]2.82=-121.9	27.80	(b) -13.2	-367.0	-211.6	-155.4	-121.9	-333.5	-12.0	-1.2	45.9
L_1U_1 + L_3U_3	[16(1.0+1.0-14x.05333)+4(1.0-14x.05333)]2.82=+59.4	5.82 32	+18.0 5.96	+104.8 190.7	+39.5	+65.3	+59.4	+98.9	+17.0	-1.0	39.6
U_1L_2	[16(.75+.75-14x.00677)+4(.75-28x.00677)]2.82=+69.7	10.89 36	+18.0 5.96	+196.0 190.7	+92.3	+103.7	+69.7	+162.0	+14.9	-3.1	53.6
L_2U_3	[16(.57+.57-14x.00608)+4(.57-28x.00608)]2.82=-52.1	7.78 22	(c) -11.3 5.96	-87.9 131.1	-37.1	-50.8	-52.1	-89.2	-11.5	+0.2	(d) 35.1
U_3L_4	[16(.6+.6-14x.008)+4(.6-28x.008)]2.82=+53.3	5.07 24	+18.0 5.96	+91.3 143.0	+23.8	+67.5	+53.3	+77.1	+15.2	-2.8	45.6

(a) See page 56 for calculation of this value.
(b) See page 57 for calculation of this value.
(c) See page 58 for calculation of this value.
(d) Critical member.

1 $in.^2 = 6.452 \times 10^{-4} m^2$
1 kip = 4.448×10^3N
1 ksi = 6.895×10^6Pa
1 ton = 8.896×10^3N

TABLE 19
COMPARISON OF TRUSS BRIDGE RATINGS
BASED ON THEORETICAL COMPUTATIONS AND EXPERIMENTAL RESULTS

Inventory Ratings (0.55 Fy) for Type 3S2 Trucks - A7 Steel				
Member (1)	Theoretical Rating (tons) 1 ton = 8.896×10^3N (2)	Experimental Rating (tons) 1 ton = 8.896×10^3N (3)	Position of Drive Axle (4)	Ratio Col. 2/Col. 3 Percentage (5)
Lower Chord				
L_0L_2	$\frac{110.3}{70.7} \times 36 = 56.2$	(not instrumented)		
L_2L_4	$\frac{125.4}{115.9} \times 36 = 39.0$	$\frac{125.4}{72.6} \times 36 = 62.2$	3	63%
End Post L_0U_1	$\frac{186.4}{108.5} \times 36 = 61.8$	$\frac{186.4}{94.2} \times 36 = 71.2$	1A	87%
Upper Chord				
U_1U_3	$\frac{172.1}{106.1} \times 36 = 58.4$	(not instrumented)		
U_3U_5	$\frac{155.4}{121.9} \times 36 = 45.9$	$\frac{155.4}{111.4} \times 36 = 50.2$*	3	91%
Verticals L_1U_1	$\frac{65.3}{59.4} \times 36 = 39.6$	$\frac{65.3}{41.0} \times 36 = 57.3$	1	69%
L_3U_3	" " "	(not instrumented)		
Diagonals				
U_1L_2	$\frac{103.7}{69.7} \times 36 = 53.6$	(not instrumented)		
L_2U_3	$\frac{50.8}{52.1} \times 36 = 35.1$*	(not instrumented)		
U_3L_4	$\frac{67.5}{53.3} \times 36 = 45.6$	$\frac{67.5}{34.2} \times 36 = 71.1$	4	64%
Floor System				
End Bm.	$\frac{18.8}{20.04} \times 36 = 33.8$	(not instrumented)		
Int. Bm.	$\frac{18.1}{21.05} \times 36 = 31.0$*	$\frac{11.65}{6.75} \times 36 = 62.1$*	2	50%
Ext. Str.	$\frac{85.3}{75} \times 36 = 40.9$	$\frac{15.02}{3.43} \times 36 = 157.6$	1A	26%
Int. Str.	$\frac{123.4}{75} \times 36 = 59.2$	$\frac{15.28}{4.21} \times 36 = 130.7$	1A	45%
Mid. Str.	" " = "	$\frac{15.28}{5.19} \times 36 = 106.0$	1A	56%

*Indicates critical members in the truss and in the floor framing as determined theoretically and experimentally.

EXPLANATION OF COLUMNS IN TABLES 1 THROUGH 5
Summary of Unit Stresses and Axial Loads in Truss Members

Column 1 - Lateral and longitudinal position of test vehicle. See Figures 11 and 12.

Column 2 - Live load experimental stress from strain gage. See Figure 6 for positions of strain gages.

Column 3 - Live load forces in truss members calculated by multiplying the stresses in column 2 by the cross-sectional areas of the members.

Column 4 - Live load forces in truss members calculated from the wheel loads in Figure 10 and the influence diagram ordinates in Figure 14. The two lines of wheel loads were proportioned to the two trusses by calculating simple beam reactions of the floor beams.

Column 5 - A ratio of column 3 to column 4 in percentages. Provides a comparison between experimentally determined live load forces and theoretically calculated live load forces.

Column 6 - Theoretically calculated dead load forces in truss members from dead loads applied at panel points as shown in Figure 15.

Column 7 - Truss member design load including experimentally determined live load.

Column 8 - Truss member design load including theoretically determined live load.

Column 9 - A ratio of column 7 to column 8 in percentages. Provides a comparison between total design loads from experimentally measured live loads and theoretically calculated live loads.

EXPLANATION OF COLUMNS IN TABLE 6
Simulated Two Lane Loading Live Load Plus Impact

Column 1 - Truss member designation. See Figure 3.

Column 2 - Longitudinal position of the two simulated test vehicles. East lane loading plus west lane loading. See Figures 11 and 12.

Column 3 - Live load forces in truss members calculated by multiplying the stresses from strain gages by the cross sectional areas of the members. Test vehicle in the west lane.

Column 4 - Same as column 3 except the test vehicle is in the east lane.

Column 5 - Live load forces in truss members from test vehicle in both east and west lanes simultaneously plus 18.2% impact.

Column 6 - The total design capacity of the truss member (according to AASHTO Spec.) less the force from dead load. Listed in Table 18.

Column 7 - The percentage of the total force available for live load that is developed from the test load in the east and west lanes.

Column 8 - The member rating for the several positions defined as the ratio of the live load capacity available to that which is developed from this test procedure.

EXPLANATION OF COLUMNS IN TABLE 7
Simulated Two Lane Loading Dead Load Plus Live Load Plus Impact

Column 1 - Truss member designation. See Figure 3.

Column 2 - Longitudinal position of the two simulated test vehicles. East lane loading plus west lane loading. See Figures 11 and 15.

Column 3 - Live load forces in truss members calculated by multiplying the stresses from strain gages by the cross sectional areas of the members. Test vehicle in the west lane.

Column 4 - Same as column 3 except the test vehicle is in the east lane.

Column 5 - Theoretically calculated dead load forces in truss members from dead loads applied at panel points as shown in Figure 15.

Column 6 - Total calculated load in truss members including live load from test vehicle in both east and west lanes plus 18.2% impact plus dead load. (column 3 plus column 4) 1.182 + column 5.

Column 7 - The total design capacity of the truss member according to AASHTO Spec. (listed in Table 18).

Column 8 - A ratio of column 6 to column 7 in percentages. Provides a comparison between the total forces including those from test vehicle, impact and dead load and the total design capacity of the truss member.

EXPLANATION OF COLUMNS IN TABLE 8
Maximum Forces From Center Line Position of Test Vehicle

Column 1 - Truss member designation. See Figure 3.

Column 2 - Live load stress determined by multiplying the experimental strain reading by the approximate modulus of elasticity of 30×10^6 psi (1 psi = 6.895×10^3 Pa).

Column 3 - Live load force in truss member calculated by multiplying the experimental stress (column 2) by the cross sectional area of the member.

Column 4 - Live load force in truss members calculated from the wheel loads in Figure 10 and the influence diagram ordinates in Figure 14. The two lines of wheel loads were proportioned to the two trusses by calculating simple beam reactions of the floor beams.

Column 5 - A ratio of column 3 to column 4 expressed in percentages. Provides a comparison between experimentally determined live load forces and theoretically calculated live load forces.

EXPLANATION OF COLUMNS IN TABLES 9 THROUGH 13
Floor System Flexural Stresses

Column 1 - Lateral and longitudinal position of test vehicle. See Figures 11 and 12.

Column 2 - Live load experimental stress from strain gage. See Figure 7 for positions of floor system strain gages.

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Column 3 - Dead load flexural stress. Non-composite section.

Column 4 - Live load experimental stress from strain gage plus 30% impact factor plus theoretical dead load stress.

Column 5 - Allowable tensile flexural stress (18,000 psi) by Virginia Department of Highways & Transportation rating practice.

Column 6 - A ratio of column 4 to column 5 in percentages. Provides a comparison between the total stresses including those from the test vehicle, impact and dead load and the VDH&T rating stress.

EXPLANATION OF COLUMNS IN TABLE 14
Simulated Two Lane Loading on Floor System

Column 1 - Structural element of floor system.

Column 2 - Longitudinal position of test vehicle. See Figure 12.

Column 3 - Live load experimental stress with test vehicle in west lane. See Figure 11.

Column 4 - Live load experimental stress with test vehicle in east lane. See Figure 11.

Column 5 - Sum of live load stresses from test vehicle positioned in both the west lane and the east lane plus 30% impact factor.

Column 6 - Stress available for live load plus impact. The allowable flexure tensile stress of 18 ksi less dead load stress. See Tables 9-13 for dead load stresses.

Column 7 - A ratio of column 5 to column 6 in percentages. Provides a comparison between the experimental two lane live load stresses plus impact with the value available for live load stress plus impact.

Column 8 - The reciprocal of column 7 expressed in decimals. This is a factor which could be applied to the standard live load applied and the stress developed in the member would still be within the allowable.

EXPLANATION OF COLUMNS IN TABLES 15 THROUGH 17
Live Load Stresses in Middle, Interior and Exterior Stringers

Column 1 - Lateral and longitudinal position of test vehicle. See Figures 11 and 12.

Column 2 - Live load experimental stress from strain gage. See Figure 7 for position of stringer strain gages.

Column 3 - Live load stringer stress assuming non-composite beam action ($F = \frac{M}{Z}$), and AASHTO lateral distribution factors ($\frac{S}{5.5}$).

Column 4 - A ratio of column 2 to column 3 in percentages. Provides a comparison between experimentally determined stresses and stresses calculated by theory used at the time of design (1934).

Column 5 - Live load stringer stress assuming composite beam action, n = 10, $F = \frac{M}{Z_{Transformed}}$ and AASHTO lateral distribution factors.

Column 6 - A ratio of column 2 to column 5 in percentages. Provides a comparison between experimentally determined stresses and stresses calculated on the basis of unbroken bond between the concrete slab and the top flange of the steel stringer.

EXPLANATION OF COLUMNS IN TABLE 18
Summary of Theoretical Live Load Capacities of Truss Members

Column 1 - Truss members.

Column 2 - Calculation of live load forces in truss members. Wheel loads times influence diagram ordinates times lateral distribution factor (2.39) times impact factor (1.18). See calculations following.

Column 3 - Net cross sectional area on top line and number of rivets on second line.

Column 4 - Allowable axial stress in kips per square inch on top line and allowable shear in rivet in kips on second line. See calculation sheets following.

Column 5 - Total allowable load in member in kips as controlled by axial stress or by number of rivets.

Column 6 - Dead load force in member in kips from a panel load of 39.5 kips.

Column 7 - Total allowable force in member less force from dead load. Net force available for live load. Column 5 less column 6.

Column 8 - Force in member from live load plus impact. Column 2 forces repeated.

Column 9 - Force in members from dead load plus force in member from live load. Column 6 plus column 8.

Column 10 - Total design stresses. Member forces from column 9 divided by cross sectional areas in column 3.

Column 11 - Under stresses. Allowable stresses from column 4 less design stresses in column 10.

Column 12 - Live load capacity of the structure in tons applied at the standard 3S2 axle spacing. Calculated by multiplying 36 tons (3S2 type) times the ratio of the available live load force to the design live load force. The ratings listed are based on theoretical considerations only. The most critical member theoretically, the diagonal L_2U_3, was not instrumented in the field study.

EXPLANATION OF COLUMNS IN TABLE 19
Comparison of Truss Bridge Ratings Based on
Theoretical Computations and Experimental Results

Column 1 - Component member of bridge.

TRUSS DEAD LOAD

(Quantities from Va. Dept. of Highways Standard Plan SC-24-150)

Concrete	91.9 cu. yds. x 27 x 145 lbs./c.f. =	360,000
Reinf. Steel		18,000
Structural Steel		202,000
Asphalt wearing surface	15 psf x 23' x 150' =	52,000
	TOTAL	632,000 lbs.

$$\frac{632^k}{2} = 316 \text{ kips per truss}$$

$$\frac{316}{8} = 39.5 \text{ kips per panel point}$$

See Figure 15 for member dead loads.

TRUSS LIVE LOAD

(See Ref. 1, VDH&T, "Truss Bridge Inspection Instructions", Plate 4, Rev. 1/27/71)

Wheel Lines per truss = $(1 + \frac{W-18}{C})\ 2$

W = Width of roadway = 23 ft.

C = Center to center of trusses = 25.9 ft.

Wheel lines per truss = $(1 + \frac{23-18}{25.9})2 = 2.39$

Impact = $\frac{50}{L+125} = \frac{50}{150+125} = 0.18$

Wheel lines plus impact factor = 2.39 x 1.18 = 2.82

CALCULATION OF RATINGS OF THE CRITICAL TRUSS MEMBER, L_2U_3

Ratings of other truss members shown in column 12 of Table 18 calculated in a similar manner.

Inventory rating 0.55 fy = 18 ksi

f_a = 11.3 ksi in compression

Total allowable load (TAL) = 11.3 x 7.78 = 87.9^k

Dead load (See Figure 15) = 37.1^k

Available for live load plus impact = 50.8^k

Rating for Type 3 truck = $\frac{50.8}{31.2} \times 20 = 32.6$ tons

Rating for Type 3S2 truck = $\frac{50.8}{52.1} \times 36 = 35.1$ tons

Rating for Type 3-4 truck = $\frac{50.8}{70.6} \times 52 = 37.4$ tons

See Figure 13 for truck types

The values 31.2, 52.1, and 70.6 kips were computed from the influence line for L_2U_3 (See Figure 14) and the wheel weights (See Figure 16) for the respective truck types. Lateral distribution factor and impact factor are included. See Table 18 for calculation of the 52.1^k value.

Column 2 - Theoretical capacity in tons for a Type 3S2 truck as limited by the several component members of the bridge.

Truss members: $\left[\frac{\text{Force available for LL+I}}{\text{Theoretically applied force from LL+I}}\right]36$. See Table 18.

Floor beam: $\frac{\text{Available Stinger reactions}}{\text{Applied Stringer Reactions}} \times 36T$, See Plate 4. Ref. 1

Stringers: $\frac{\text{Available Bending Moment (LL+I)}}{\text{Applied Bending Moment (LL+I)}} \times 36T$. (See calculation sheets following for floor beam and stringer sample calculations.

Column 3 - Capacity in tons as determined experimentally by applying a simulated 3S2 type truck. See Figure 10

Truss members: $\left[\frac{\text{Force Available for LL+I}}{\text{Experimentally Applied Force from LL+I}}\right]36$. See Table 6

Floor Beams: $\left[\frac{\text{Available stress for LL+I}}{\text{Experimentally applied stress from LL+I}}\right]36$. See Table 14.

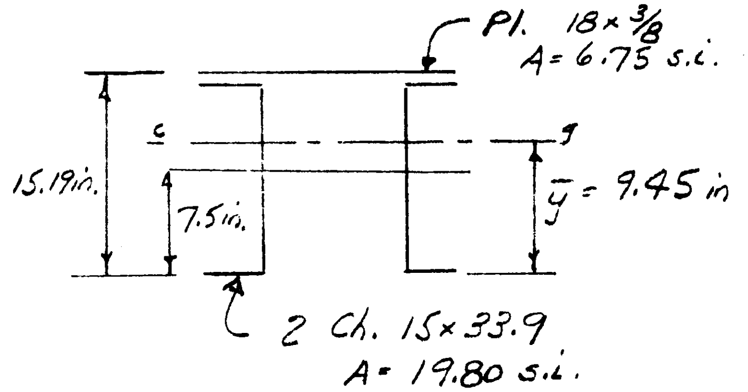
Stringers: $\left[\frac{\text{Available stress for LL+I}}{\text{Experimentally applied stress from LL+I}}\right]36$. See Table 14.

Column 4 - Position of drive axle for forces and stresses shown. See Figure 12.

Column 5 - A ratio of theoretical rating to experimental rating expressed in percentages.

ALLOWABLE STRESSES IN COMPRESSIVE MEMBERS

L_0U_1 End Post



Element	Area	Moment Arm	Moment	Moment Arm	Moment of Inertia
Channels					625.2 $in.^4$
"	19.80	7.5	148.50	7.5	1113.8
Plate	6.75	15.19	102.53	15.19	1557.5
Totals	26.67		251.93		3296.5

$$\bar{y} = \frac{251.93}{26.67} = 9.45 \text{ in.} \quad \times 251.93 = -2380.7$$

$$I_{cg} = 915.8 \text{ in.}^4$$

$$r = \sqrt{\frac{915.8}{26.67}} = 5.86 \text{ in.}$$

$$\frac{l}{r} = \frac{28.91 \times 12}{5.86} \times 0.75 = 44 \qquad \frac{ec}{r^2} = 0 \quad \alpha = +1$$

F_a = 13.8 ksi   from secant formula column graphs for $f_{y.p}$ = 33,000 psi   A7 steel Inventory Rating

Note: I_{cg} about 'y' axis = 935.2 > I_{cg} x-axis

ALLOWABLE STRESSES IN COMPRESSIVE MEMBERS (CONT'D)

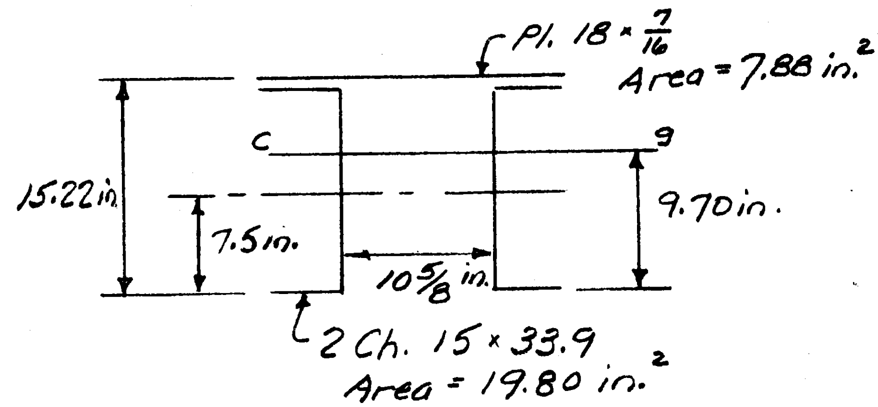
U_1U_3 Same section as L_0U_1

$\bar{I}_x = 920.6 \text{ in.}^4$ $\quad \bar{r} = 5.88 \text{ in.}$ $\quad \frac{ec}{r^2} = 0$ $\quad \alpha = +1$

$$\frac{l}{r} = \frac{37.98 \times 12}{5.88} \times 0.75 = 58.1$$

$F_a = 13.2$ ksi

U_3U_5



Element	Area	Moment Arm	Moment	Moment Arm	Moment of Inertia
Channels					625.2
"	19.80	7.5	148.50	7.5	1113.8
Plate	7.88	15.22	119.92	15.22	1825.24
	27.68		268.42		3564.24

$$\bar{y} = \frac{268.42}{27.68} = 9.70 \text{ in.} \quad \times 268.42 = -2602.94$$

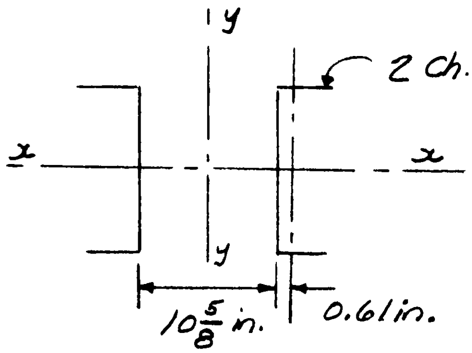
$$I_{cg.} = 961.30 \text{ in.}^4$$

$$\bar{r} = \sqrt{\frac{961.30}{27.68}} = 5.89 \text{ in} \qquad \frac{l}{r} = \frac{37.5 \times 12}{5.89} \times 0.75 = 57.3$$

$$\frac{ec}{r^2} = 0 \qquad \alpha = +1 \qquad F_a = 13.2 \text{ ksi}$$

ALLOWABLE STRESSES IN COMPRESSIVE MEMBERS (CONT'D)

L_2U_3



2 Ch. 9×13.4
Area = 7.78 in.^{2}
$\bar{I}_x = 94.6$ in.^{4}
$\bar{I}_y = 1.8$ in.^{4}

$$I_{yy} = 2\left[1.8 + 3.89(5.31 + 0.61)^2\right] = 276.3 \text{ in.}^4$$

$$I_x = 94.6 \text{ in}^4$$

$$r = \sqrt{\frac{94.6}{7.78}} = 3.49 \text{ in.}$$

$$\frac{l}{r} = \frac{33.70 \times 12}{3.49} \times 0.75 = 87$$

$$\frac{ec}{r^2} = 0 \qquad \alpha = +1 \qquad F_a = 11.3 \text{ ksi}$$

ALLOWABLE STRESS IN TENSILE MEMBERS

A-7 STEEL   Minimum Yield = 33,000 psi

$0.55\,F_y = 18{,}000$ psi

ALLOWABLE SINGLE SHEAR IN RIVETS

3/4" DIAM.   AREA = 0.442 s.i.

0.442×13.5 ksi = 5.96 kips/rivet

END FLOOR BEAM

Stringer reactions from one side only

End Floor Beam W 30 × 116 $S = 327.9 \text{ in.}^3$

Dead Load Reaction $= \frac{1}{2}\left(9.7 + 12.5 + \frac{12.5}{2}\right) + \frac{0.116 \times 25}{2}$

$= 15.7$ kips

Max. D.L. Moment $= 15.7 \times 12.5 - 4.85 \times 11 - 6.25 \times 5.5 - \frac{0.116 \times 12.5}{2}$

$= 99.5$ f.k.

Total Available Moment $= Sf = 327.9 \times \frac{18}{12} = 491.9$ fk

Available Moment for L.L. + Impact $= 491.9 - 99.5 = 392.4$ fk

Available Moment for L.L. $= \frac{392.4}{1.3} = 301.8$ fk

$$\text{Allowable } R = \frac{\text{Available Moment}}{L - 9 + \frac{2.25}{L}}$$

$$= \frac{301.8}{25 - 9 + 0.09} = 18.8 \text{ Kips}$$

Inventory Rating for 3S2 Type Truck (36 Tons)

$$\frac{18.8}{20.04} \times 36 = 33.8 \text{ Tons}$$

20.04 Kips is the applied stringer reaction from a line of wheels for a 36 Ton Type 3S2 Truck for 18.75 ft. stringers supported by an end floor beam.

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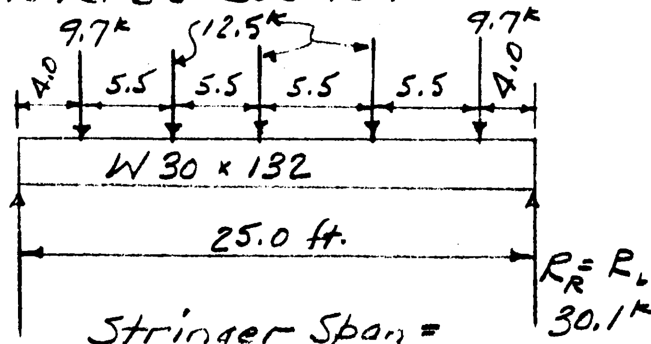
Interior Floor Beam

See Figure 5 for transverse section



Dead Load

Interior Stringer D.L.

Reaction $= 664.3 \times 18.75$
$= 12.5^k$

Stringer Span = 18.75 ft

Exter. Str. D.L. Reaction =
$515 \times 18.75 = 9.7^k$

Max. D.L. Moment $= 30.1 \times 12.5 - 9.7 \times 11 - 12.5 \times 5.5 - 0.132 \times \frac{12.5^2}{2} = 190.5$ fk.

Total Allowable Moment (TAM) $= Sf = 379.7 \times \frac{18}{12} = 569.6$ f.k.

Available for L.L.M. + Impact $= 569.6 - 190.5 = 379.1$ f.k

Available for LL.M. only $= \frac{379.1}{1.3} = 291.6$ fk

Max. impact factor = 30 o/o

Allowable R $= \dfrac{\text{Available Moment}}{L - 9 + \frac{2.25}{L}} = \dfrac{291.6}{25 - 9 + \frac{2.25}{25}}$ See Plate 4 Ref. 1 or p.49 Ref. 2

$= 18.1^k$

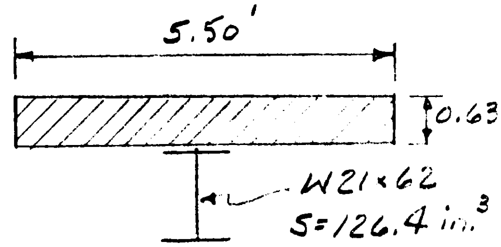
Inventory Rating for 3S2 Type Truck

$\frac{18.1}{21.05} \times 36 = 31$ Tons

21.05 kips is the applied stringer reaction from a line of wheels for a 36 ton Type 3S2 truck for 18.75 ft. stringers supported by an intermediate floor beam.

INTERIOR STRINGER

See Figure 5 for transverse section



Dead Load Weight

Wearing Surface - $5.5 \times 15 = 82.5$

Concrete Deck - $5.5 \times 0.63 \times 150 = 519.8$

Steel Beam $\quad 62.0$

$\quad 664.3$ lbs/ft

$$\text{Dead Load Moment} = \frac{664.3 \times 18.75^2}{8} \times 12 = 350.3 \text{ ik}$$

Total Available Moment (TAM): $fS = 18 \times 126.4 = 2275.2$ ik

Available Moment for L.L. + Impact $= 2275.2 - 350.3$
$= 1924.9$ ik

$$\text{Lateral Distribution Factor} = \frac{5.5}{5.5} = 1$$

$$I = \frac{50}{18.75 + 125} = 0.35 \quad \text{Use } 30\% \text{ max.}$$

$$\text{Available moment per wheels line} = \frac{1924.9}{1.3 \times 1} = 1480.7 \text{ ik}$$
$$= 123.4 \text{ fk}$$

Safe Load Capacity - Inventory Rating

$$\frac{123.4}{75.0} \times 20 = 32.9 \text{ Tons for Type 3 Trucks}$$

75.0 fk is max. moment for a Type 3 - 20 ton truck on an 18.75 ft. span. From VDH and T Tables of Equivalents. Ref. 1 - Capacity of 59.2 Tons for Type 3S2 Trucks and a capacity of 79.8 Tons for Type 3-4 Trucks. Note:- Operating Ratings are based on flex. stresses of $0.75 F_{y.p}$. Inventory Rat. on $0.55 F_{yp}$

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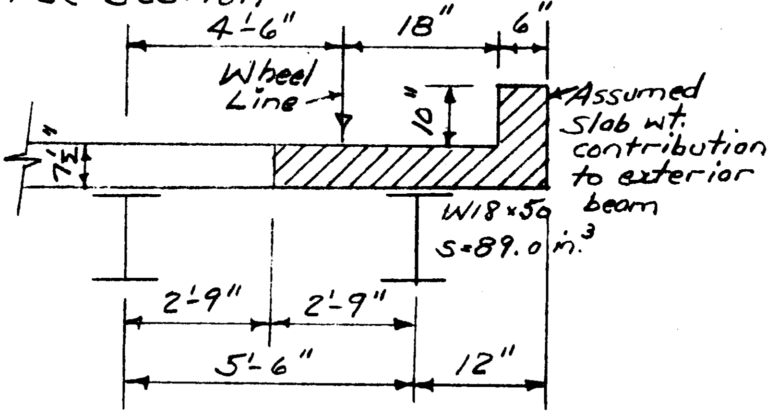
See Figure 5 for transverse section



Dead Load

Wearing Surface-3.25×15 = 49

Conc. Deck- 0.63×3.75×150 = 354

Curb - 0.50×0.83×150 = 62

Steel Beam = 50

515 lbs/ft.

Dead Load Moment = $\frac{515 \times 18.75^2}{8} \times 12 = 271.6$ ik

Total Available Moment = fS = 18×89 = 1602 ik

Available Moment for L.L. + Impact = 1602 − 271.6
= 1330.4 ik

L.L. Lateral Distribution Factor = $\frac{5.5}{5.5} = 1$

(AASHTO Bridge Spec. - Simple beam reaction but S/5.5 as a minimum)

Max. Impact Factor = 30%

Available L.L. Moment per wheel line = $\frac{1330.4}{1.3 \times 1}$
= 1023.4 ik = 85.3 fk

Safe Load Capacity - Inventory Rating

$\frac{85.3}{75} \times 20 = 22.7$ Tons for Type 3 Trucks

($M = \frac{PL}{4} = \frac{16 \times 18.75}{4} = 75$ fk for 20 Ton Truck)

From VDH&T Tables of Equivalents - Ref. 1

Capacity of 40.9 Tons for Type 3S2 Trucks

" " 54.9 " " " 3-4 "

EXTERIOR STRINGER, continued . . .

The foregoing computations are based on 0.55 $f_{y \cdot p}$ = 18 ksi for inventory rating. Capacities for operating rating may be made in a similar manner using

0.75 $f_{y \cdot p}$ = 24.75 ksi

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